GROUND MOTIONS AND DAMAGE OBSERVATIONS IN THE MARLBOROUGH REGION FROM THE 2013 LAKE GRASSMERE EARTHQUAKE

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SUMMARY

This paper presents various aspects of the preliminary damage observations caused by ground motions in the Marlborough region following the M_w6.6 Lake Grassmere earthquake on 16 August 2013. To emphasize the severity of the ground shaking, the observed pseudo-acceleration response spectra are compared to those from the 21 July 2013 M_w6.5 Cook Strait earthquake and the NZS1170.5:2004 design spectrum. The near-source damage to State Highway 1 roads, bridges and buildings is presented within. Stainless steel wine storage tanks showed various damage states that were consistent with observations from previous earthquake events. The performance of wine tanks and other winemaking infrastructure are discussed with future design considerations. Eleven water storage dams within 12 kilometres of the earthquake source were inspected and preliminary observations are discussed. A 250,000 cubic metre dam located 10 kilometres southwest of Seddon suffered moderate damage following the 21 July event while significant further damage was sustained following the 16 August event and emergency earthworks were undertaken to reduce the risk of dam failure (to those living downstream). The performance of residential housing in rural townships of Seddon and Ward was satisfactory with respect to preserving life safety however there was moderate levels of damage which are presented within. Post-earthquake business disruption was minimal as commercial buildings in the Blenheim central business district sustained either minor or no damage.

INTRODUCTION

On 16 August 2013 at 2.32pm local time, a moment magnitude M_w6.6 earthquake occurred beneath Lake Grassmere, approximately 30 kilometres south-east of Blenheim. The rural communities of Seddon and Ward, both within 10 kilometres of the earthquake rupture source, represented the worst hit regions. Prior to the Lake Grassmere earthquake, the M_w6.5 Cook Strait earthquake on 21 July 2013 occurred approximately 25 kilometres east of Seddon. The ground motions from the 16 August event were of higher amplitude than those from the 21 July event at these locations, principally as a result of the 21 July event occurring offshore.

No casualties were reported (following either event) and few cases of moderate non-critical injuries were sustained. While ground shaking in the Marlborough region was relatively severe, the population density is relatively low and the distribution of structural forms is quite different to larger urban areas such as Christchurch and Wellington. A large proportion of the region's non-residential structures are used for the purposes of agriculture and viticulture and consequently numerous water storage dams and wine storage tanks suffered various levels of damage. In the near-source region, damage to residential housing, State Highway 1, rural roads and bridge structures was found to be moderate.

The first part of this paper examines the ground motions recorded from this event in comparison to the ground motions in the prior 21 July 2013 event as well as the design response spectra according to NZS1170.5:2004. Subsequently, the extent of damage observed to various different engineered facilities in the Marlborough region is presented. Brief discussion is finally given for various lessons to be learnt in the wake of these events.

TECTONIC AND GEOLOGIC SETTING

New Zealand resides on the boundary of the Pacific and Australian plates and its active tectonics are dominated by [1]: (i) oblique subduction of the Pacific plate beneath the Australian plate along the Hikurangi trough in the North Island; (ii) oblique subduction of the Australian plate beneath the Pacific plate along the Puysegur trench in the south west of the South Island; and (iii) oblique, right lateral slip along numerous crustal faults in the axial tectonic belt, of which the 650-km long Alpine Fault is inferred to accommodate approximately 70-75% of the approximately 40 mm/yr plate motion [2], [3].

North-east of the Alpine Fault, and south of the Hikurangi subduction trench, is the Marlborough Fault Zone (MFZ) [4]. The MFZ is principally comprised of the four nearly-parallel

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Wairau, Awatere, Clarence, and Hope Faults. Two major fault ruptures have occurred in the Marlborough region during historical times: (i) the Hope River section of the Hope Fault in the M_w7.0-7.3 1888 Amuri earthquake [5]; and (ii) the eastern segment of the Awatere Fault in the M_w~7.5 1848 Marlborough earthquake [6]. Other, smaller, more recent events also include the M_L5.8 1966 Seddon and M_L6.0 1977 Cape Campbell earthquakes.

OBSERVED STRONG GROUND MOTIONS

Volume 1 ground motion records were obtained from GeoNet (www.geonet.org.nz/) and processed on a record-by-record basis. The overall processing methodology adopted is elaborated in [7]. All ground motions were processed with a low-pass causal Butterworth filter of 50 Hz, and while the corner frequency of the high-pass filter was record-specific, a frequency of less than 0.05 Hz provided physically realistic Fourier spectra amplitudes and integrated displacement histories for all the near-source ground motions. Owing to the digital nature of all of the instruments, baseline corrections were found to be unnecessary following the above filtering. As a result, the processed ground motions can be considered to provide reliable estimates of peak ground accelerations (*PGA*) and spectral ordinates over the range 0.01-10 seconds [8], which are typically of engineering interest.

Figure 1 illustrates the horizontal and vertical ground motions that were recorded in the Marlborough region in the 16 August 2013 Lake Grassmere earthquake. The BWRS station produced a clipped record for this event and is therefore not shown. The MGCS, WDFS, and KEKS stations are permanent instruments while the RCS1 and RCS2 are temporary instruments that were installed in the region following the 21 July 2013 event. It can be seen that the strongest shaking was observed in Seddon and Ward, with the strong portion of the shaking lasting on the order of 10-15 seconds. Notably weaker shaking was observed in Blenheim, Wairau Valley and Kekerengu.

Table 1 provides a summary of the recorded ground motions. It can be seen that geometric mean horizontal peak ground accelerations of 0.74 and 0.56g and peak ground velocities of 39.6 and 18.3 cm/s were recorded at RCS2 (i.e. Seddon) and WDFS (i.e. Ward), respectively. At these two locations the vertical ground motion amplitude was also appreciable with vertical PGA's of 0.24 and 0.28g, respectively.

Table 1: Summary of observed ground motions at strong motion stations in the 16 August 2013 Lake Grassmere earthquake.

Station Name	Code	Site class[1]	R_{rup}[2] (km)	PGA[3] (g)	PGV[4] (cm/s)	D_{s5-95}[5] (s)	PGA_v[6] (g)
Kekerengu Valley Road	KEKS	B	20.7	0.06	4.8	15.9	0.04
Blenheim Marlborough Girls College	MGCS	D	27.8	0.12	7.9	14.2	0.04
Ward Fire Station	WDFS	C	6.0	0.56	18.3	10.4	0.28
Response Cook Strait 1	RCS1	C	42.4	0.04	2.1	15.9	0.02
Response Cook Strait 2	RCS2	C	6.5	0.74	39.6	9.2	0.24

[1]As defined by the New Zealand Loadings Standard, NZS1170.5:2004 based on information at https://magma.geonet.org.nz/delta/app; [2]Closest distance from fault plane to site; [3]Peak ground acceleration; [4]Peak ground velocity; [5]Significant duration (5-95%); [6]Peak vertical ground acceleration. Note that with the exception of PGA_v, ground motion parameters are geometric mean horizontal definition.

Comparison with design spectra for NZS1170.5:2004

Figure 2(a) illustrates the larger component response spectra of the observed ground motions in the Marlborough region in comparison to the NZS1170.5:2004 design response spectrum for Z= 0.4 (the representative value for Seddon and Ward, while Blenheim has Z= 0.33). It can be seen that shaking in Seddon (i.e. RCS2) and Ward (i.e. WDFS) significantly exceeded the site class C design spectrum for vibration periods less than 0.3 seconds. For periods greater than T= 0.3 s the spectral amplitudes in Ward rapidly reduce (due to very shallow soils overlying rock), while the spectrum in Seddon exceeds the design spectrum for periods beyond T= 1.0 s. As previously noted, the ground motion amplitudes in Blenheim, Wairau Valley and Kekerengu can be seen to be significantly smaller than those in Seddon and Ward, and well below design levels.

Figure 2(b) illustrates, for comparative purposes, the response spectra of the observed motions in the Marlborough region during the M_w6.6 21 July 2013 Cook Strait earthquake. As noted in Figure 1, this event was located off-shore relative to the 16 August 2013 Lake Grassmere event. Thus while both events have the same moment magnitude (M_w6.6), the larger source-to-site distances is the principal reason for the significantly reduced response spectral amplitudes observed in Ward during this event, while the SA amplitudes are similar in Blenheim (i.e. MGCS and BWRS) for both the 16 August and 21 July 2013 events. As previously noted, the RSC1 and RCS2 instruments were installed following the 21 July event, so the ground shaking in Seddon for this event is unknown.

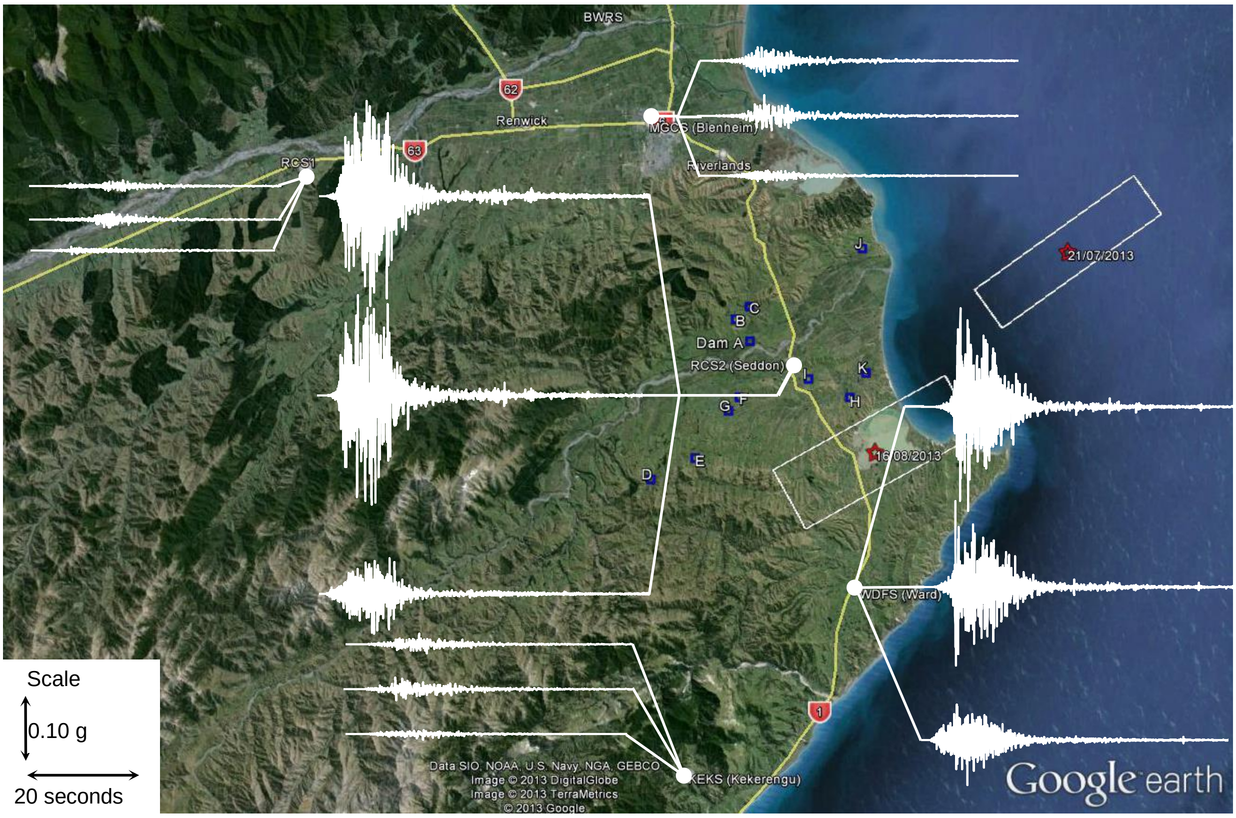


***Figure 1:** Ground motions observed in the Marlborough region from the 16 August 2013 M_w6.6 Lake Grassmere earthquake. Fault normal (top), fault parallel (middle), and vertical (bottom) components are provided at each recording station. The inferred causative fault planes from the 16 August and 21 July earthquakes are also illustrated.*

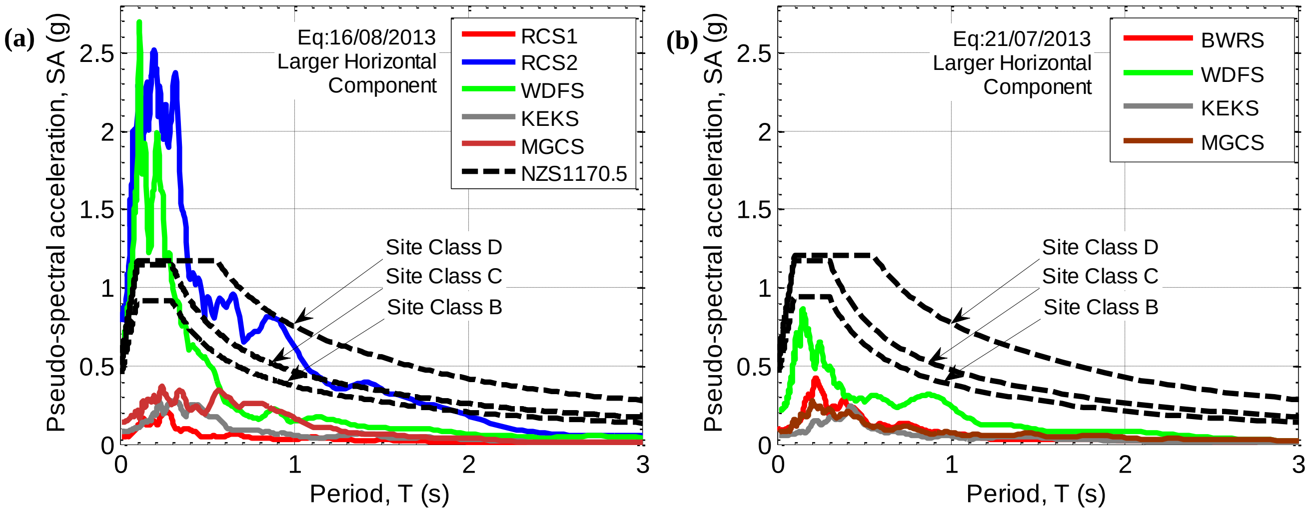


***Figure 2:** Larger horizontal response spectra recorded at strong motion stations in Marlborough during: (a) the 16 August 2013 Lake Grassmere earthquake and; (b) the 21 July 2013 Cook Strait earthquake in comparison to design response spectra from NZS1170.5:2004 for Z= 0.4 (Seddon/Ward).*

OBSERVED DAMAGE TO TRANSPORTATION INFRASTRUCTURE

State Highway 1 roads

The major difficulty in constructing state highways and local roads is the constraint imposed by New Zealand's natural topography. State Highway 1 (SH1) negotiates rolling terrain between Seddon and Ward such that a significant number of man-made cuts and fills permit the widened carriageways. Following the 16 August event the main damage observations illustrated in Figure 3(a)-(h) includes:

- Slope instability on cut sections producing fallen debris on the road.
- Cracks extending through the pavement and engineered fill volumes.
- Settlement of carriageways.
- Settlement and lateral spreading of adjacent shoulders.

Figure 3(a) shows an example of slopes adjacent to the road shoulder that are restrained by gabion baskets. Following the 16 August event large cracks were observed at the interface between poorly compacted materials on the slopes themselves and adjacent to competent material beneath the pavement. In general crack widths of approximately 50 mm were observed at sections with steep slopes and cambered corners with large depth of fill and near the embankments of bridges and culverts. The largest observed pavement cracks were approximately 150 mm in width as shown in Figure 3(c). Vertical settlements on SH1 generally ranged between 250-300 mm, with the largest observed settlement up to 500 mm at a Highway intersection within 1 kilometre of the earthquake source [W. Oldfield, pers. comm.].

Immediately following the 16 August event, earthworks were undertaken to clear fallen debris from the pavement to permit regular volumes of SH1 traffic. Filling and repaving was regularly required for settlements at bridge approaches to smooth over rough pavement cracks and vertical offsets as shown in Figure 5(a). No raw photographic observations were obtained for slips on SH1; however Figure 3(g) and (h) show slips on two local roads within 1 kilometre of the earthquake source.

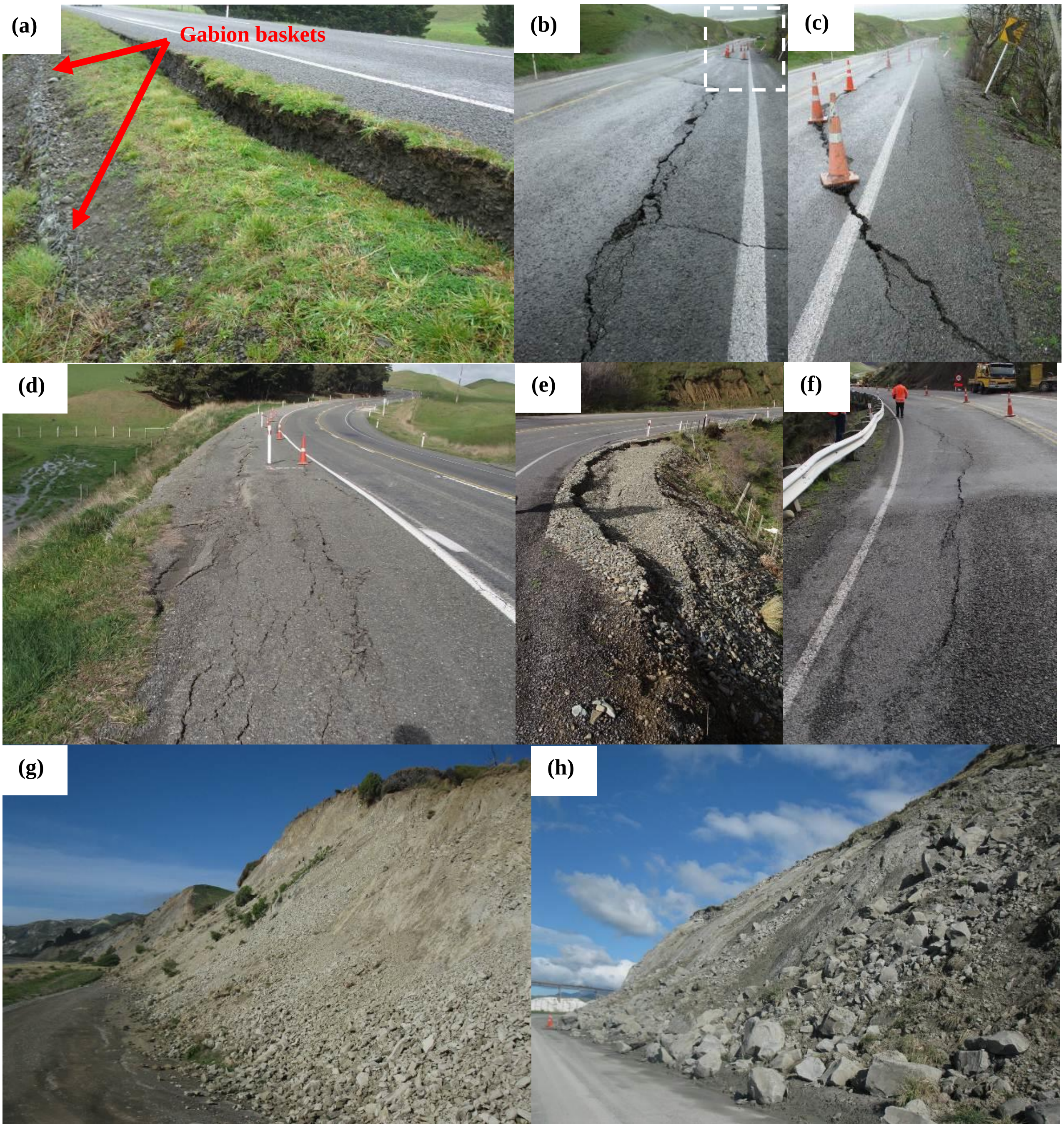


Figure 3: ***(a)-(d) Damage to State Highway 1 indicates instability of engineered fill materials and shoulders; (e)-(f) similar damage on Weld Pass; (g)-(h) slips on local roads within 1 kilometres of the earthquake source (Photos (d-f) courtesy of NZTA).***

Awatere Bridge

The Awatere Bridge, located 2 kilometres to the north of the Seddon township, comprises ten spans with a total length of 275 m and a 10 m carriageway. Three prestressed concrete U-shaped beams support the prestressed deck. A notable feature of the deck design is the absence of intermediate expansion joints. To maintain diaphragm action the U-beams are tied together both between spans and to the abutments using un-bonded Reid™ bars passing through the beams bottom flange and terminated at flange openings. The linkage ties pass through the abutment to anchor at the back face as illustrated in Figure 4(b). Neoprene rubber provides flexibility and bolts are intended to be snug tightened such that the bars have zero pre-strain.

Reported damage after the 21 July 2013 Cook Strait earthquake

Inspections of the north abutment showed failure of the concrete adjacent to the anchorage of the linkage bars in the beam flange. Under large seismic demands the desired behaviour is for bar yielding, however beneath the ducts cast into the flange the area of concrete to resist the imposed bearing stresses is relatively small. Following the concrete break out the link bars were projected downward as indicated by Figure 4(c). Also at the north abutment, the outside corners of the U-beams sustained minor concrete crushing which is due to transverse displacements. The fill material at the north approach was estimated to have settled by approximately 20 mm [12]. No damage at the south abutment or bridge piers was reported.

Damage observed after 16 August 2013

As illustrated in the previous section on observed ground motions, the Awatere bridge was likely subjected to significantly stronger ground shaking during the 13 August event than that on 21 July. Figure 4(d) shows the rubber bearings between the U-beams and the south abutment indicating residual displacement in the south direction. A 30-40 mm offset was observed for the south abutment relative to the embankment and gabion walls had moved up to 30 mm away from both abutments towards the river. No further damage was observed to the linkage bars from that already seen after the 21 July event. Figure 4(f) illustrates concrete spalling at central piers which suggests significant transverse deflections [13]. On the carriageway itself, the north approach settlement was approximately 15 mm while 5 mm settlement was observed at the south approach. Minor hairline cracks were observed to the north and south abutments.

At the west side of the south abutment a series of lateral spreading cracks between 50-150 mm propagated approximately 30 m in the upstream direction. Cracks spread parallel to the Highway from the north abutment approximately 10 m on the north-west side. Given that the shaking in Seddon exceeded 500 year design levels for short to moderate vibration periods, the observed moderate embankment damage, which did not impact the bridge's functionality, and can be considered a good performance.

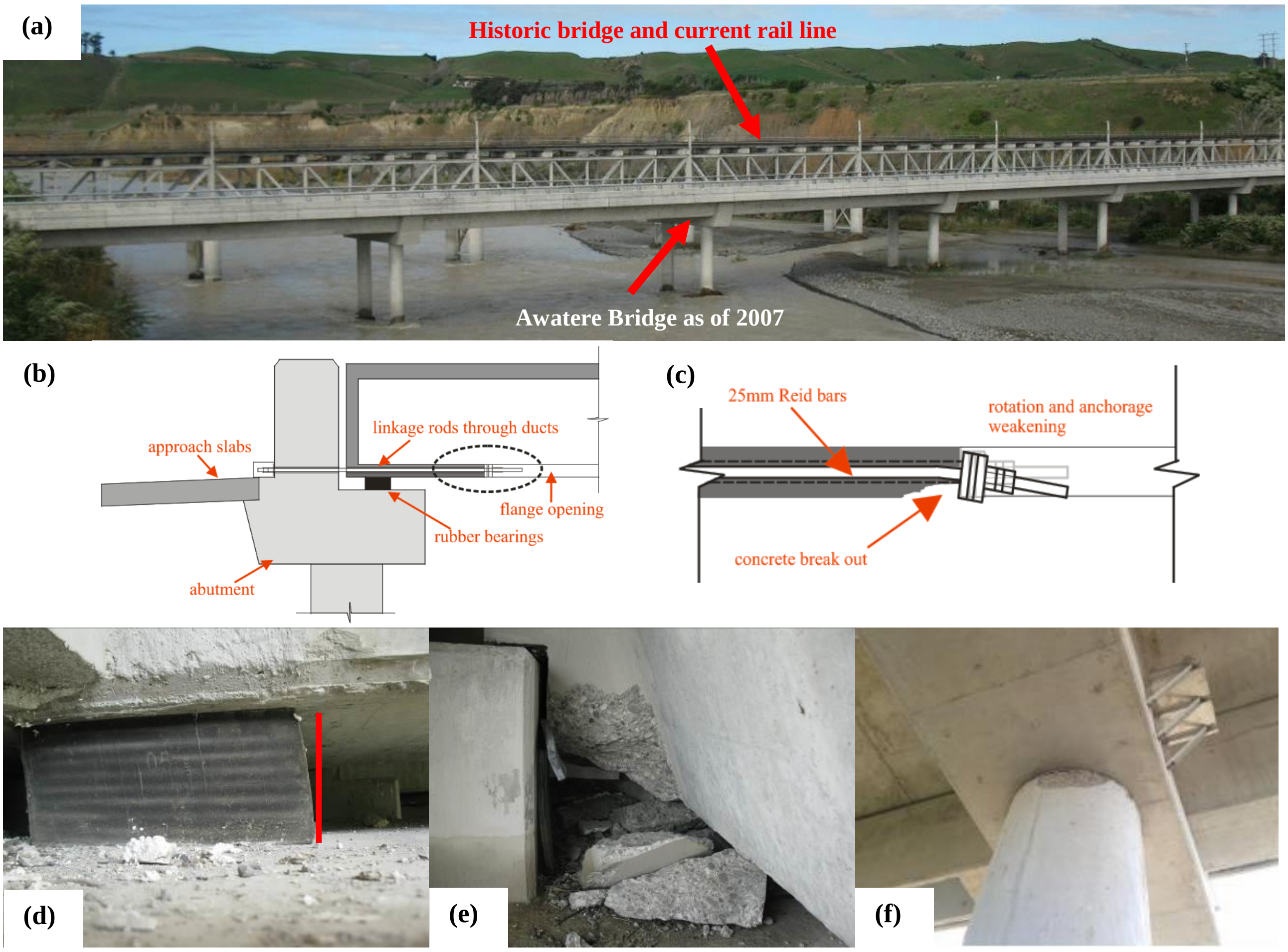


Figure 4: (a) The 275 m Awatere Bridge; (b) details of the abutment-diaphragm connection; (c) post- 21 July damage at north abutment. Post-16 August damage; (d) rubber bearings indicate residual longitudinal displacement; (e) spalling of cover concrete at edge of U-bream girders and; (f) spalling at central piers (last image courtesy of NZTA [13]).

Impacts on other bridges

Lateral spreading and approach settlements were observed at several smaller bridges on SH1 and local roads. Figure 5(a) illustrates the resurfaced north approach at the Needles Creek Bridge (located North of Ward), which settled approximately 100 mm and required re-filling and re-surfacing. Figure 5(b) shows the largest reported [13] settlement of 400 mm at Merrifield's Bridge on Blind River Loop Road (within 5 kilometres of the fault plane).

Structural damage observed at other bridges included minor spalling at girder-pier supports and the spalling of relatively small volumes of concrete adjacent to bolts providing shear transfer. Figure 5(c) and (e) show cracks up to 5 mm to the abutment wing-walls on the State Highway 1 road and rail bridges over the Starborough Creek in Seddon. Repair work on the rail bridge abutments commenced within a few days after 16 August.

Figure 5: Approach settlements observed at: (a) 100 mm Needles Creek Bridge (rapid repair); (b) 400 mm Merrifields Bridge (courtesy of NZTA). Damage to road and rail bridges over the Starborough Creek on State Highway 1 included approach settlements, cracked abutment wingwalls and slope stability issues.

OBSERVED PERFORMANCE OF WINEMAKING FACILITIES

Marlborough is an internationally recognised wine growing region with an annual production on the order of 250 million litres from approximately 150 wineries which contributes 70-80% of New Zealand's total active wine production [14]. Other high seismicity regions with large volumes of wine production include California, Italy, Chile and Argentina. The production of wine requires a range of infrastructure including buildings, storage tanks, catwalks, grape presses, barrels and various plant and services. The production facilities are distributed across the Marlborough region, though clusters of larger facilities are located near the Riverlands and Cloudy Bay business parks and many smaller facilities around Renwick (refer to Figure 1). Much like other buildings in the Blenheim area, winery buildings typically suffered little damage. Greater damage was however observed to wine storage tanks, and in some instances the associated infrastructure.

The damage states observed in Marlborough are not unique, as extensive damage to thin walled metallic steel tanks has been reported [15] from previous events such as: 1989 Loma Prieta, US, 1992 Landers, US and 2010 Maule, Chile earthquakes. In New Zealand, recent observations of damage to wine tanks followed the 2007 Gisborne earthquake and 2010-2011 Canterbury earthquakes. The authors are aware of several cases of pullout of epoxied anchors in Gisborne, and minor damage to frames of leg mounted tanks at a particular winery located in Belfast following the Canterbury earthquakes.

The support structures for thin walled stainless steel tanks most commonly used in Marlborough can generally be separated into two categories; smaller 5,000 to 60,000 litre leg-mounted tanks and, larger 60,000 to 300,000 litre plinth-mounted tanks. With obvious differences in scale and construction, the different mounting of tanks has led to distinctly different damage states from seismic excitation. Even 500,000 litre tanks may be unanchored as seismic restraint is not required for very low aspect ratios (height/radius).

There are two predominant modes of tank response contributing to seismic demands that are transferred through the tank walls, connections and support structures (base mounts or steel legs). For tanks with high aspect ratios the *impulsive mode* is dominant as a larger portion of constrained fluid content provides a relatively large inertial mass. For tanks with small aspect ratios the *convective mode* (sloshing of tank contents) is dominant, which is more sensitive to long-period ground motion [17].

Seismic design recommendations for storage tanks

In 2009, a New Zealand Society for Earthquake Engineering (NZSEE) study group revised the 1986 recommendations for the *Seismic Design of Storage Tanks* [16] to provide information that is consistent with changes in legislation, the NZS1170.0:2002 targeted performance criteria [18], and the NZS1170.5:2004 seismic design actions [9]. The influence of damping of the seismic response depends on the participation of the impulsive and convections modes, hysteretic energy dissipation (for cases where ductility is permitted) and foundation stiffness. For the wine tanks typically used in Marlborough, the impulsive mode is expected to dominate the response, particularly for tanks that were full and sealed.

Seismic design actions

In the Marlborough region the design for wine tanks varies with the pre-determined targeted performance criteria and chosen ductility. The NZSEE recommendations state a 50 year design life should be adopted but the selection of *Importance Level* is open to the designer with some guidance on appropriate selection. From the third author's experience, many tanks in Marlborough have been designed to an Importance Level 1 in recent years. Ductility factors of up to μ = 2.0 are permitted in the NZSEE recommendations. Damping on the order of 3.0% is reasonable for tanks dominated by the impulsive mode with an aspect ratio H/R of 3.0. Additional equivalent viscous damping is considered for each ductility value.

Leg-mounted tanks

Leg mounted tanks are typically of the smaller tank variety with storage capacities ranging from 5,000-60,000 litres. A small size and limited connection to the floor slab offers wineries flexibility in layout. Wine within these tanks is often of high value per litre, as they contain the more boutique production, at least within larger facilities.

There are several variations in tank design dependent on the age of the tank, its purpose, and tank supplier. A typical leg-mounted tank consists of a stainless steel tank sitting on a mild steel support frame, with a layer of treated timber or high density insulation between the tank and frame. Stainless steel straps, concealed behind the cladding skirt, connect the tank to the frame and are welded to both at a number of locations around the perimeter of the assembly. The frame sits on adjustable feet that are typically bolted to the floor. Taller frames, particularly for tanks associated with red wine production, often have diagonal braces introduced into the framing arrangement. In order to resist lateral loads, the frames of these tanks either contain bracing elements, or need to act as moment resisting frames when no diagonal bracing is present. Consideration also needs to be given to the increased axial load on the legs and feet due to global overturning. The tank supported by the frame is designed following the principles outlined in the NZSEE guidelines [17] and the connections need to be appropriately detailed to transfer the shear and overturning loads into the frame.

As far as the authors are aware, the worst observed damage during the 16 August event was two tanks that overturned due to the failure of supporting legs, with one tank falling and damaging the walls of another nearby tank as illustrated in Figure 7(a). Various damage states typically observed to the majority of this type of tank following the 16 August event included:

- Buckling of legs, braces or adjustable feet, as shown in Figure 7(c)-(e).
- Fracture of straps connecting base frame to tank.
- Distortion of tank floor where connection straps were damaged. In some instances the distortion was extensive enough to cause tearing of the tank floor which resulted in some loss of product.
- Yielding and deformation of the base frame at joints, in some instances leading to stress on the tank floor and the rotation of the joints of the circular hollow section (CHS) legs and rectangular hollow section (RHS) frame beams through localised bending of the RHS flange.
- Movement of tank on the slab or rotation of the baseplate around a single bolt.

- One instance of large scale buckling of a tank wall was observed and this is believed to have been the result of the collapsing of the frame, resulting in large compressions on the tank wall as shown in Figure 7(b).
- Slight creasing or buckling of the tank skirt.
- Many unbolted tanks laterally slid across the winery floor up to approximately 100 mm.

Plinth-mounted Tanks

These stainless steel tanks sit on a reinforced concrete plinth with a thin layer of insulation between the plinth and the tank. The plinth is typically poured within a mould and tied to the concrete floor slab. As the plinth is constructed prior to the tank being lifted into place, there is usually a small (10-40 mm) gap between the plinth and the section of tank wall extending below the tank base. In most cases the tank wall does not extend down to the slab. Anchor chairs are welded to the tank skirt at regular centres around the perimeter and are used to fix in place anchors that are epoxied or grouted into the slab. The anchors for these tanks are often fabricated from stainless steel threaded bar and feature a length (approximately 150 mm) of reduced diameter through their mid-section. The intent behind reducing the anchor diameter is to promote yielding of the anchor with an even strain distribution such that there is a desirable hierarchy of strength that ensures the protection of other components based on a capacity design approach.

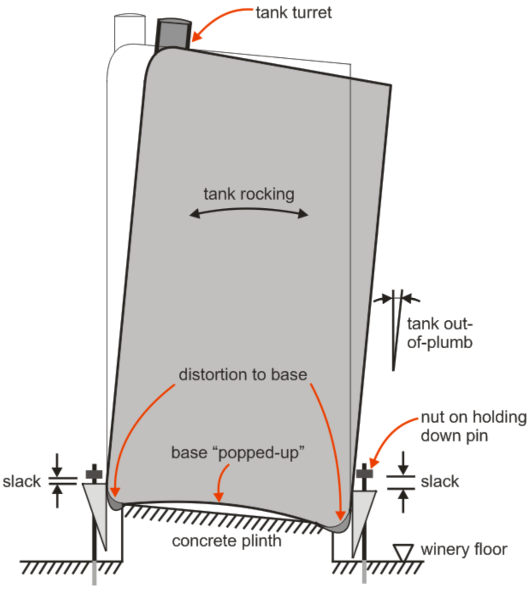


Figure 6: ***A characteristic damage state for plinth mounted tanks is tank settlement and "knuckle-squash" at the tank floor.***

Various damage states to plinth-mounted tanks in the 16 August earthquake included:

- Anchor failure in the form of anchor fracture, pullout through the epoxy, or concrete cone failure as shown in Figure 8(a)-(e).
- Lateral movement on the plinth, leading to bending of anchors, as shown in Figure 8(g).
- Settlement of tank around the plinth (illustrated in Figure 6) due to deformation of the tank floor across the gap between tank skirt and plinth. In rare instances tank deformation was sufficient to rupture the tank floor at the knuckle, resulting in loss of product.
- In extreme cases, buckling of tank walls, both in *elephant's foot* and *diamond shaped* buckling modes. The elephant's foot buckling appeared to be constrained by the coolant bands. Buckling appears to have occurred when the tank settled so far down the plinth that the skirt makes contact with the slab forcing the tank wall to resist excessive compressive loads.
- Localised stress to tank skirt associated with poor distribution of anchors around the tank.

Some older tanks were simply fixed by a number of bolts drilled horizontally into the plinth around the perimeter of the skirt. Shear failure of the majority of bolts fixed into the plinth had occurred at some facilities as shown in Figure 8(f).

In recent years (typically 2011 onwards), plinth-mounted tanks with poured in-situ plinths and a ductile, concealed holding-down system have been installed at some facilities. These tanks are positioned on the concrete slab, with holding-down bolts concealed behind the skirt and epoxy-fixed into the slab. The plinth concrete is then poured with the tank in-situ. No structural damage was observed to these tanks, although there was some spalled concrete around the base of some plinths. It is thought that this would have been excess concrete that had run under the skirt edge during plinth filling and had broken away due to flexing of the assembly.

A number of older tanks were not fixed to the slab with any form of anchor and these appear to have performed well. It is likely that the large mass of the wine provided a re-centring force to resist overturning and the relatively thick walls (compared to modern designs) were sufficient to resist the compressive forces. It is anticipated that the lack of anchorage led to larger than expected deflections and that while these tanks performed well in these events, they are exposed to a notably higher risk of failure in larger events.

Catwalks, stairs & services

In order to access the tops of the tanks for inspection and winemaking processes, a series of tanks are often fitted with lightweight steel catwalks and access man-ways. The catwalks are supported either from cleats welded to the tank wall or by a separately constructed frame. Some modern facilities use floating catwalks which slide over the tanks during ground motion excitation [D. Saunders, pers. comm.]. For the first catwalk type, the connection between the catwalk structure and the supporting elements are often slotted with the intention of providing some construction tolerance. In most cases, the underside of the catwalks is connected to services such as glycol and wine distribution pipes.

In the direction along a row of tanks, the cleat orientation is such that bending about the plate's weak axis results in cleat flexibility. Some distortion of plates was observed, however, there was no evidence of connection failure due to displacement in this direction. Across two rows of tanks, in the orthogonal direction, the connections are often stiffer and some displacement induced failures have been observed where the tolerance of the slot has been used up and bolts have failed in shear. Other examples of catwalk damage included the distortion of box-type cleats that are welded to the tank wall.

Stacked Barrel Racks

Generally barrels used in storing and oaking of wine are stacked on top of each other with a small steel frame between tiers as shown in Figure 9(c). Alternative systems using stackable interlocking RHS frames have been recently introduced to the market. No collapses of the barrel stacks were reported, however these are also critical features of winemaking facilities that require careful consideration of the barrel falling hazard faced by people working in close proximity. Given the reliance on the mass of the barrels to prevent the stacks toppling, it is likely these systems will rock to a certain level of load before collapse.

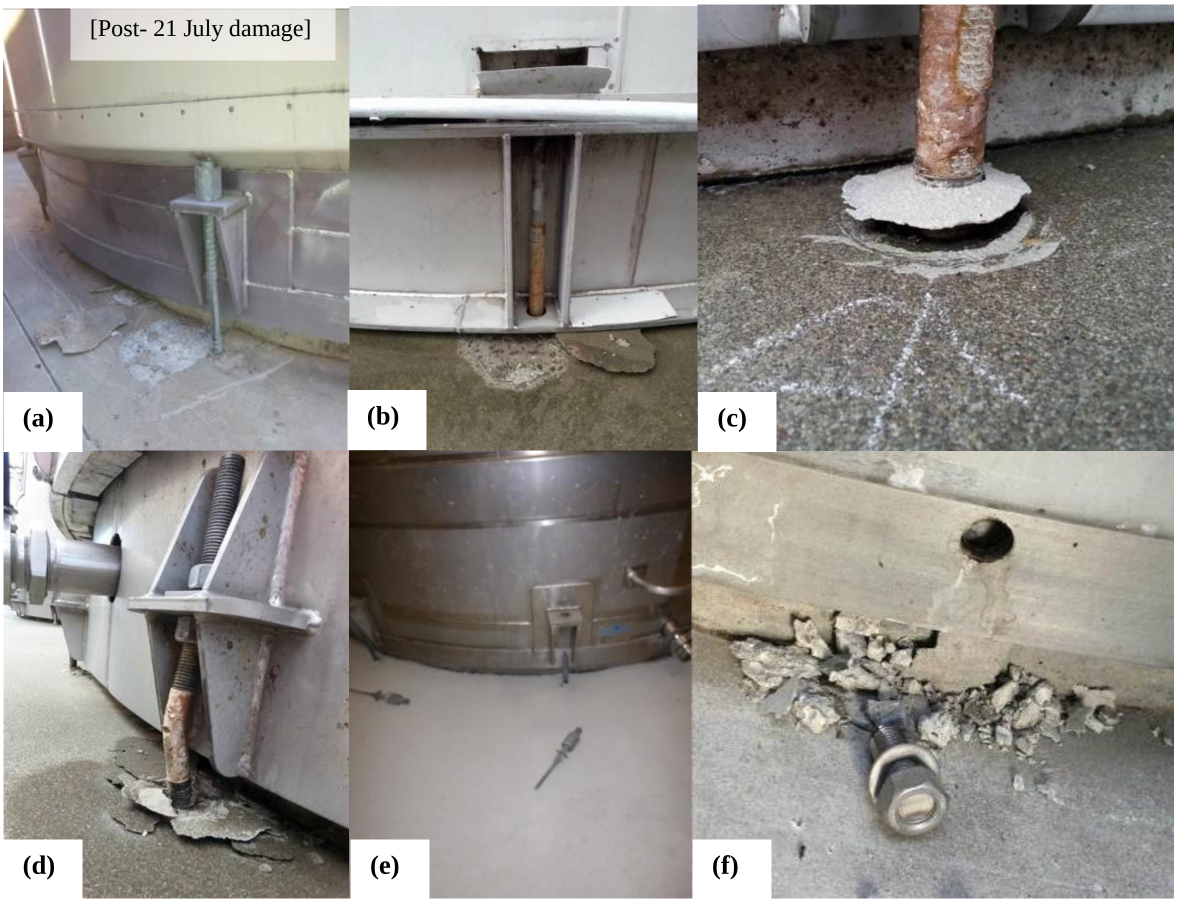
Damage to leg mounted tanks following buckling failure of tank legs

Skirt and base damage to red cellar leg mounted tank

Bending of frame

Buckling of brace

Figure 7: Various damage observations for leg mounted wine storage tanks.



Anchorage failure by pullout, bolt tensile fracture and bolt shear failure

Bending of anchor due to lateral tank movement relative to plinth

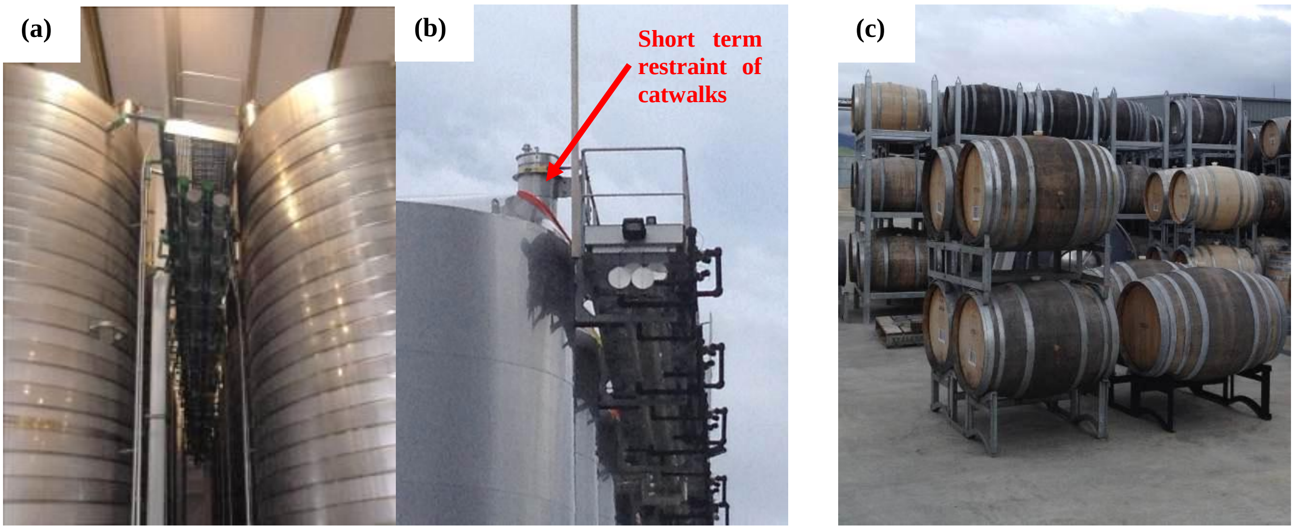
Buckling of tank skirt

Pullout failure on slab mounted anchor chair

Elephant's foot buckling of tank

Diamond shaped buckling

Figure 8: Various damage observations for plinth mounted wine storage tanks.



Loss of support to catwalk

Stacked barrel racks

Figure 9: (a) Catwalks were damaged at connections to tanks; (b) temporary catwalk restraint to tank turret; (c) stacked barrel racks.

Short Term Measures

Immediately following the 16 August event some wineries implemented short term measures to secure their infrastructure to reduce the risk faced by staff and to mitigate any further loss of product. Some of these measures included:

- Packing under anchor chairs in the form of steel tube or timber blocking to prevent further settlement of the tank down the plinth.
- Timber blocking under the frame rim of leg mounted tanks.
- Transfer of wine into undamaged, or better performing, tanks.
- Tightening of anchor nuts.
- Hanging damaged catwalks from roof or tank turret to allow temporary access.
- Condoning off areas of buildings, particularly barrel halls and un-stacking of barrel racks down to either single or two barrel tiers.

Future Design Considerations

The observations presented in this section should provide an increased awareness of the types of damage states for stainless steel tanks used for wine storage. Lessons learnt from these earthquake events may lead to future improvements for the seismic performance of winemaking facilities. Future considerations for new designs or the repairs of existing tanks and associated infrastructure may include:

- More reliable ductility provided by anchors may be achieved by using mild steel or the equivalent. Measures would also have to be taken to mitigate corrosion issues.
- Replaceability of anchors or other yielding elements.
- Eliminating potential tank settlement around the plinth by better consideration of load paths and improved awareness of potential increases in compressive loads in the tank walls.
- Improved design of leg mounted tanks including the distribution of legs to perimeter.
- Improved specification and monitoring of epoxy anchors. The occurrence of this failure both in Marlborough and previously in Gisborne has highlighted a potential area of improvement.
- Better consideration of likely displacements in catwalk design
- Improved storage methods for barrels.

OBSERVED PERFORMANCE OF WATER STORAGE DAMS

Marlborough is a relatively dry and windy region. In recent decades thousands of hectares of farmland has been developed into vineyards or other intensive agricultural uses with the use of irrigation. In many cases this development has required construction of dams and ponds to store water. Within 20 kilometres of the Lake Grassmere earthquake source there are 11 dams that were inspected, all less than 10 years in age. The storage capacity of the dams ranges from 20,000 to 350,000 cubic metres. There are also at least eight geomembrane (HDPE) lined ponds in this same geographical region. The locations of these dams are shown in Figure 1 and further details are summarised in Table 2. The dams are located within 2 and 12 kilometres of the earthquake source. Based on the Bradley 2010 [11] ground motion prediction equation (GMPE) the 16^{th}-84^{th} percentile range of predicted values PGA at these sites is between 0.20g and 0.60g. Without closer strong motion stations in the immediate vicinity of these dams, it is difficult to constrain the shaking distribution at these locations because important localized source, path, and site effects cannot be assessed. Inspections of the dams and some of the ponds were undertaken to assess their performance. Comments on their design and performance are provided in the following sections.

Table 2: Summary of water storage dams inspected following the 16 August 2013 Lake Grassmere earthquake.

Dam ID	Approx. capacity (m^3)	Height (m)	R_{rup} (km)	Damage state
A	119,000	15	9.5	Some movement of riprap on upstream shoulder. Small crack in backfill above spillway outlet pipe.
B	165,000	16	11.4	Nil
C	350,000	13	11.8	Nil
D	314,000	18	9.4	Nil
E	250,000	17	6.0	Extensive damage, refer to text
F	45,000	10	6.0	Slumping of fill associated with road constructed above original dam, this fill extended over the upstream shoulder of the dam.
G	159,000	15	5.6	Nil
H	110,000	6	1.9	Nil
I	30,000	12	4.7	Slumping of upstream shoulder. longitudinal and transverse cracking through crest.
J	104,000	8	11.6	Nil
K	20,000	6	3.0	Nil

Typical design of dams

Dams in the area are typically zoned earth fill embankments with a central core comprised of compacted mudstone and shoulders comprised of outwash gravels (silty gravel or gravely silt) or loess (wind-blown soils, typically low plasticity silts). A typical cross section is shown in Figure 10. All the dams include a vertical chimney drain constructed from filter material.

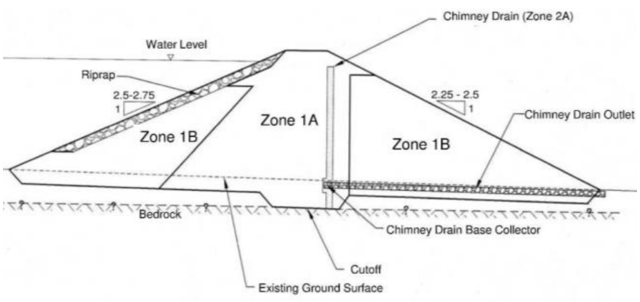


Figure 10: Typical embankment cross section.

Seismic design for water storage dams

The designs of the dams were undertaken in accordance with NZSOLD Dam Safety Guidelines [19]. All dams are considered as low potential impact classification (PIC) excluding Dam E, which was considered as medium PIC. There are two major active faults in the immediate vicinity with the two most prominent being the Awatere and Clarence Faults. The Awatere Fault is capable of M_w7.5 earthquakes and the Clarence M_w7.7 earthquakes. The "maximum design earthquake" (MDE) for most dams was assumed to be the median ground motion associated with either the Awatere or Clarence Faults. In most cases the Awatere Fault governs the design with PGAs ranging between 0.45g and 0.75g. Based on the recorded PGA at strong motion instrument (0.74g at RSC2) and estimated PGA using the Bradley (2010) GMPE, it is likely that the PGA associated with the MDE was equalled or exceeded by the 16 August ground motions at dams located close distances to the earthquake source. The PGA at dams located further away, and close to the Awatere Fault, (e.g. Dams A, B, and C) are likely to have been less than the PGA for the MDE.

Observed performance of dams

Reported damage after 21 July 2013

As far as the authors are aware, only Dam E was reported to have suffered damage after the 21 July 2013 earthquake. This consisted of some longitudinal cracking along the upstream shoulder near the embankment crest.

Damage observed after 16 August 2013

There was no damage reported for the majority of dams despite them being subjected to strong ground shaking that could be expected to result in some permanent deformations. Some damage was observed at Dams A, E, F and I. At a number of sites there was evidence of small scale instability around the reservoir margin. This was often associated with where the ground had been excavated to obtain fill for construction of the dam and the ground had been left over-steepened.

At Dam A damage was relatively minor and involved some movement of riprap on the upstream shoulder near the left abutment as shown in Figure 11(a). Riprap consisted of rounded river rock placed over a geotextile, so the frictional resistance between the riprap and geotextile was probably quite low. There was a small crack of limited length associated with backfill above the spillway outlet pipe.

Dam E suffered extensive deformation and longitudinal cracking. On the upstream shoulder slumping up to 0.75 m in the vertical direction was observed. Longitudinal cracks were observed over the full length of the upstream side of the dam crest as illustrated in Figure 11(c) and (d). There was also evidence of some deformation of the downstream shoulder. A 10 mm wide crack was observed along the centreline of the dam's crest (inferred to be above the chimney drain) and larger cracks on the upstream side of dam crest were on the order of 100-150 mm wide [C. Scott, pers. comm.]. A thin transverse crack was evident near the contact between the embankment and natural ground on the right abutment. Investigations are being undertaken to better understand the response of Dam E and to assist in determining appropriate remediation. The dam is curved in the downstream direction. It is not clear if this was a factor in the larger than average deformations or whether there were other contributing factors, such as localized amplification of ground motions or the strength of the fill. The water level in the reservoir was lowered by excavating a channel on the left abutment. This was done as a safety precaution for residents located downstream due to concern with seepage through the cracked core of the dam (inferred from a rising piezometric level) and due to a rising reservoir level associated with heavy rainfall that occurred soon after the earthquake.

Dam F also suffered some damage with evidence of slumping of the upstream shoulder and longitudinal cracks in the crest of the dam. However, it appears that much of the movement occurred in fill placed over the upstream shoulder and original crest of the dam to form an access road that was not properly conditioned or compacted to a standard normally required for water storage dams.

Dam I also suffered some damage with evidence of some slumping of the upstream shoulder and some longitudinal and transverse cracks in the crest as shown in Figure 11(b). The water levels in Dams E, F and I have been lowered to allow post-earthquake inspection and assessment of repairs.

Damage associated with HDPE Lined Ponds.

There are a number of HDPE lined water storage ponds in the area, up to about 125,000 m^3 capacity. Generally most of these dam types had minimal damage. However, the authors are aware of one pond where there is evidence of deformation in both upstream and downstream shoulders and longitudinal cracks are evident along sections of the crest.

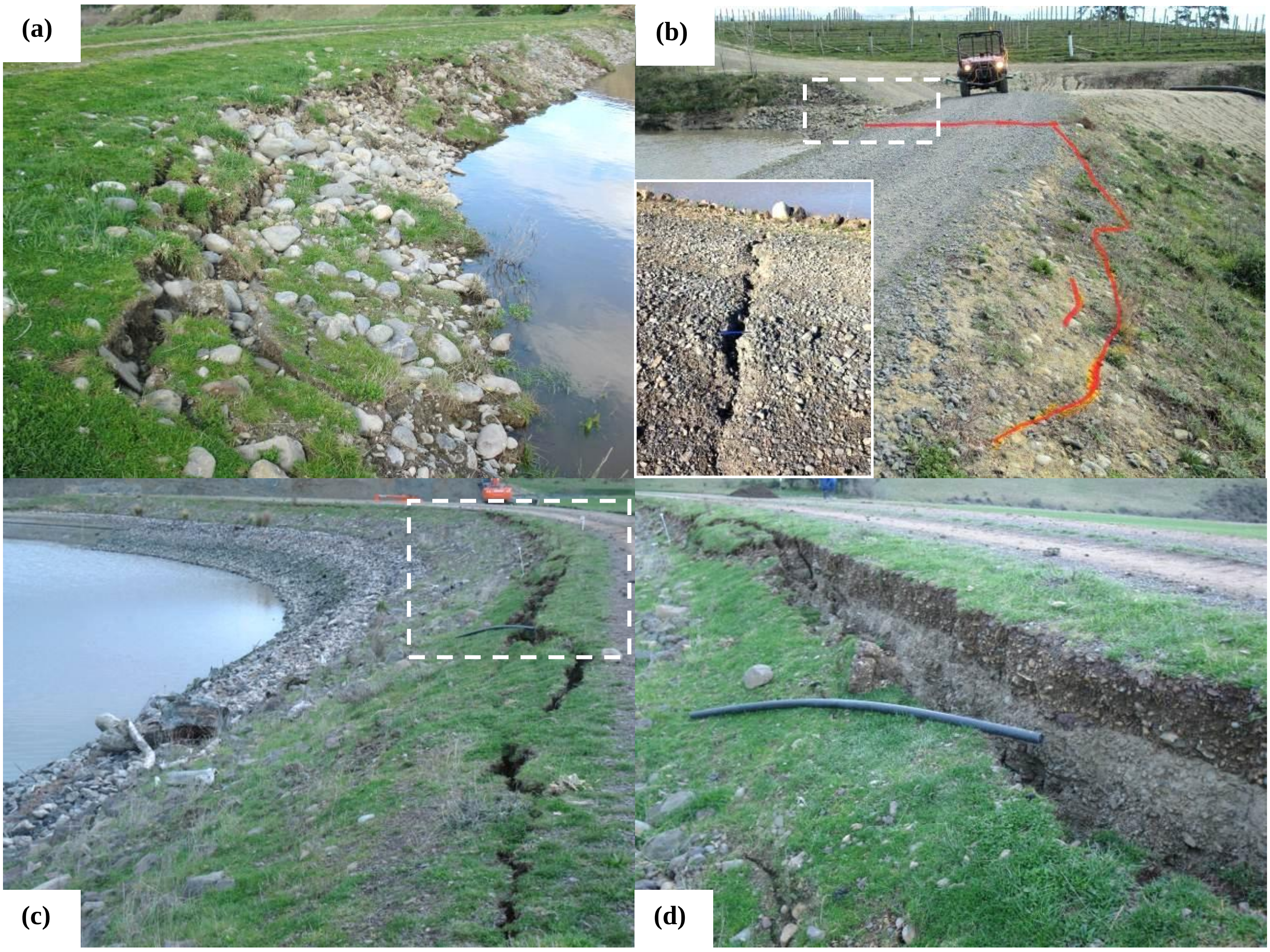


Figure 11: Various damage observations to water storage dams: (a) movement of riprap on the upstream shoulder of Dam A; (b) longitudinal and transverse crack (inset) to the crest of Dam I; (c)-(d) extensive damage at Dam E included longitudinal cracking to the full crest length.

OBSERVED DAMAGE TO BUILDINGS IN SEDDON

Residential housing

The predominant building construction in Seddon is one-storey light timber framed housing either with weatherboards or brick and masonry cladding, while some modern houses have an exterior plaster coating. Recent observations in the 2010-2011 Canterbury earthquakes showed that NZ timber framed housing generally performs well provided that foundations are robust [20]. The geotechnical impacts of the July/August events were less severe than those observed in the Canterbury earthquakes (where foundations were subjected to significant deformations due to liquefaction or lateral spreading). In Seddon, damage foundations was limited to minor perimeter cracking as shown in Figure 12(a). Some houses with timber piles were observed to displace relative to the ground.

Following the 21 July event, several measures were adopted to mitigate future damage to houses, such as removing chimneys that sustained partial damage and providing additional restraint or bracing brick veneer corners as illustrated in Figure 12(b). Minor damage was sustained to non-structural elements (i.e. internal linings) and contents following the 21 July event. In some cases there was improved fastening of contents which reduced damage from the 16 August event.

***Figure 12:** Various observations for residential buildings: (a) minor foundation cracking; (b) stabilisation of cladding after the 21 July event; (c) chimney collapse; (d)-(h) collapse of brick veneer at roof soffit, around chimneys, between openings; (i)-(k) cracking of plaster coating, and; (l)-(m) detachment and falling of roof tiles on churches and housing.*

Figure 13: Various states of interior damage was observed: (a)-(b) wall linings; (c) detached scotia beading; (d) brittle failure of kitchen splash-backs; (e)-(g) widespread contents damage at houses located within 10 km from the fault source.

Exterior damage

The 16 August event generally resulted in chimneys either collapsing or being in a state of incipient collapse (Figure 12(c)) and these were rapidly deconstructed in many cases. Figure 12(d)-(h) highlights the poor performance observed for brick and masonry cladding that had partially detached from the timber framing.

Typically brick veneer collapsed nearest the roof soffit and regions between windows and door openings. Large separations from the timber framing resulted in the cladding having a very low out-of-plane stiffness which was vulnerable to further damage. The most extensive cladding damage occurred at corner elements.

Plaster coat finishing of modern houses provides external insulation and improved aesthetics, however, in many cases the coatings had partially detached from the brick veneer. Figure 12(i)-(j) shows that damage to plaster coating was generally localised to cracks at corners of windows and door openings, as well as at corner elements and near foundations. Figure 12(k) shows plaster coat damage due to out-of-plane movement of the brick veneer. Moderate damage was sustained to brick and plaster fences of relatively large mass and will require additional repair.

The performance of weatherboard claddings agreed with observations from recent earthquakes [20] and only minor relative slipping of boards was observed and resulted in exterior paint cracking. Several older houses and church buildings showed detachment and falling of roof tiles as illustrated in Figure 12(l) and (m).

Interior damage

Moderate damage was observed to interior linings such as gypsum plasterboards. Observed cracking shown in Figure 13(a)-(c) were localised near window and door openings and damage to the edges of linings was indicated by paint cracking near ceilings, skirting boards, scotia beading and inside room corners. In some cases, deformations caused minor buckling of wall linings. Figure 13(d) illustrates brittle fracturing of kitchen splash-backs that were fastened to walls that suffered minor distortion. The seismic performance of various internal lining products (typically used in different construction eras) has previously been described in further detail elsewhere [20], [21]. Linings had separated from interior masonry (behind chimney flumes) which resulted in significant distortions in walls and ceilings. Figure 13(e)-(g) indicates the typical extent of contents damage to houses within 10 kilometres of the earthquake source.

Structural damage to non-residential buildings

Buildings located near the earthquake source that sustained moderate damage include a small number of commercial retail buildings (Seddon) and historic church buildings (Seddon and Ward). Figure 14 illustrates cracking of a precast concrete wall panel in a one-storey lightweight structure. The panels were cracked from the 21 July event and were repaired by epoxy injection. Further damage to the same wall panel was sustained in the 16 August event with crack widths of up to 2 mm observed. The crack pattern runs vertically for the full height near the centre of each wall panel, while inclined cracks were observed near the construction joints between panels.

Significant lessons have been learnt from recent major earthquakes events (e.g. 2009 L'Aquilla, 2010 Darfield, and 2011 Christchurch earthquakes [22]) on the seismic performance of unreinforced masonry (URM) buildings. Figure 15 shows the relatively short tower of an unreinforced masonry church located in Ward that sustained severe damage in both directions and was deemed vulnerable to collapsing in future strong ground motions. Cracks through the brick and mortar were observed near window and door openings. There was no reported church damage after the 21 July event.

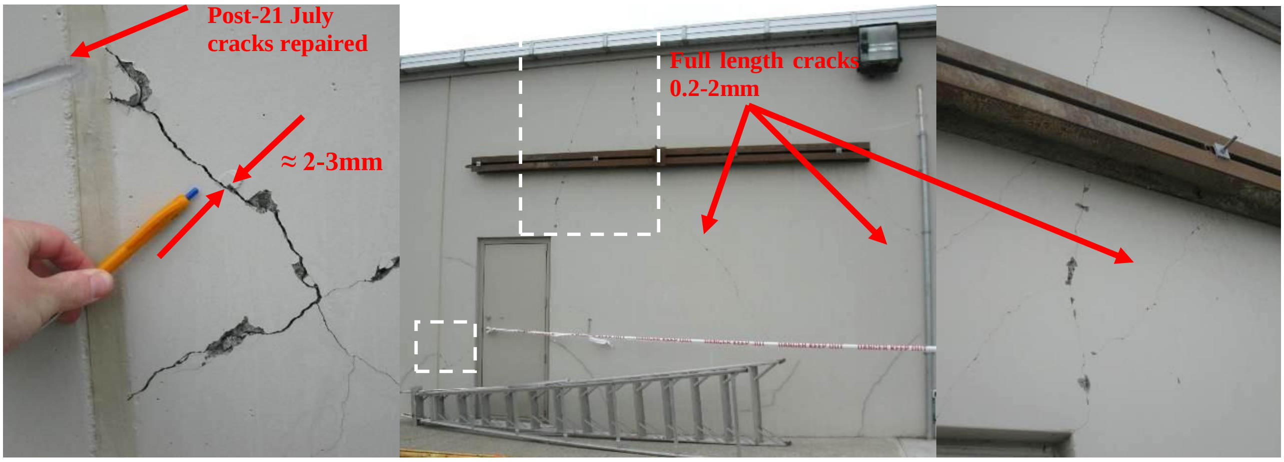


Figure 14: Precast concrete wall panels damaged following the 16 August earthquake. The panels had previously been repaired by epoxy injection of cracks following the 21 July earthquake.

Figure 15: Severe damage to a church building in Ward following the 16 August 2013 earthquake event.

PERFOMANCE OF COMMERICAL BUILDINGS IN BLENHEIM

Business disruption and rapid building evaluations

The ground motion intensity in Blenheim on 16 August was significantly less than the severe near-source recorded motions as illustrated in Figure 1 and Figure 2(a). For fundamental vibration periods between 0.5-1.0 s, the pseudo-acceleration response spectra for ground motion recorded at the MGCS station was 20-40% higher than the NZS1170.5:2004 spectra ($Z = 0.33$) for a serviceability level of ground motion (i.e. 25 years return period, $R = 0.25$). Very minor damage was reported [24] for the tallest buildings in the Blenheim central business district (CBD), e.g. Figure 16(a). For all other vibration periods the spectral demands are much lower and thus reflected by the minimal number of short period buildings observed to sustain either no or minor damage. Some businesses remained closed for up to seven days for either the reinstatement or replacement of contents, carrying out minor repairs, or to have rapid building evaluations completed by structural engineers. The latter action was adopted for several buildings previously identified as being earthquake prone; however there were no reports of buildings with any diminished capacity.

Non-structural damage

Damage to wall linings was generally found to be less extensive than observed in residential buildings in Seddon and is not discussed here. The following section presents evidence of damage to ceiling systems.

Ceiling systems

Interior damage to both non-residential buildings near Seddon and the Blenheim CBD was to components fixed to the roof system. Clear perspex and lighting brackets were partially or completely detached and fell from above. Fortunately there were no cases of falling tiles that resulted in serious impact or injury to building occupants. Distortions and edge deformations were observed for perimeter restrained suspended ceilings. The weight of ceiling systems is a likely factor contributing to the damage observed for grid members (T-bars) and the failure of rivet connections. The seismic performance of ceiling systems is understood to be influenced by a number of factors: large and heavier tiles, interaction between ceilings and mechanical services or equipment, and horizontal and vertical ground accelerations [23]. These factors would result in additional forces entering the main or secondary T-bars hence resulting in local buckling or connection failure of the grid system for carrying vertical loads. Figure 16(d) and (e) shows minor damage sustained to two systems containing relatively large heavy ceiling tiles. To prevent the connection failure in Figure 16(d), larger diameter or multiple rivets could be used.

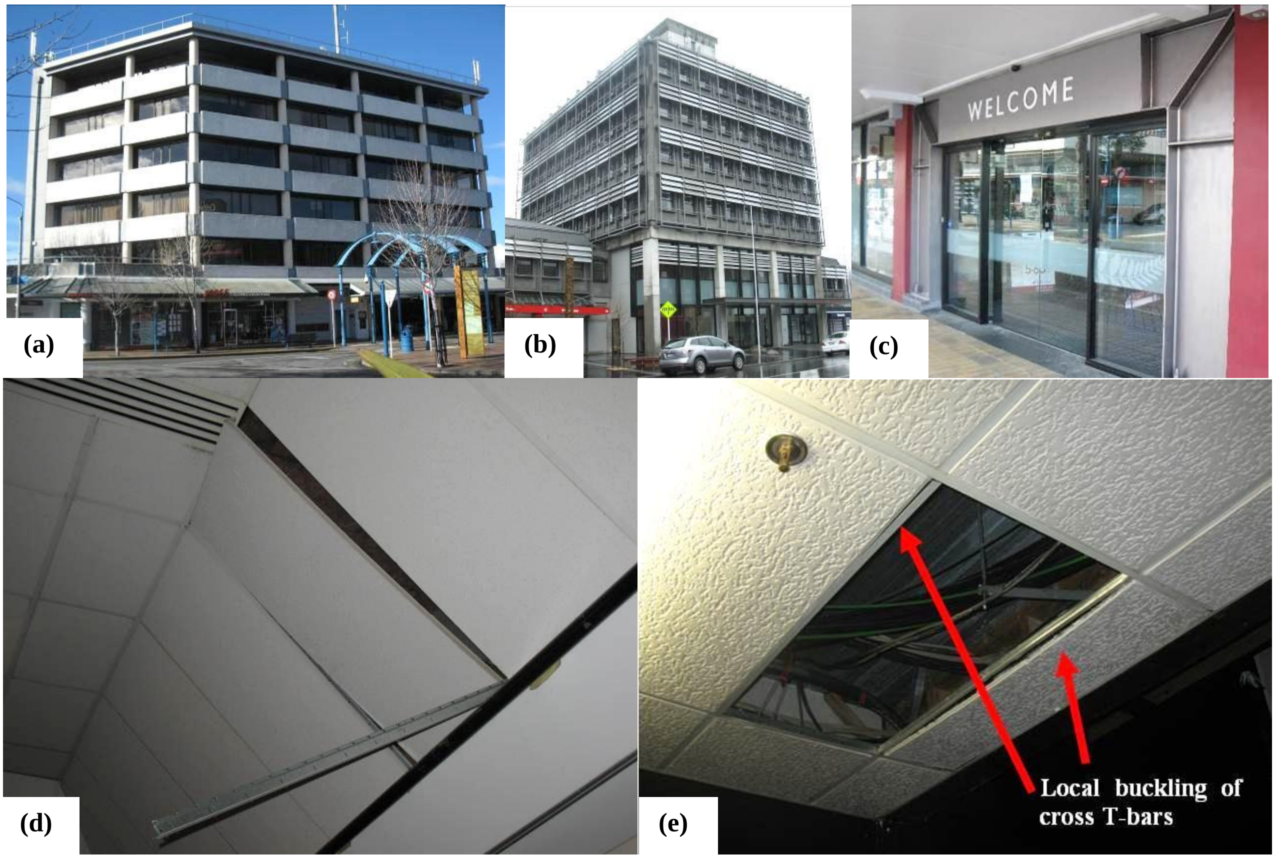


Figure 16: The largest commercial buildings in the Blenheim CBD: (a) Porse House; (b) Rangitane house. (c) One of several two-storey buildings retrofitted earlier in 2013. Observed failure of ceiling system grid members due to: (d) rivet connection failures; (e) local buckling of grid members resulting in falling tiles.

PRELIMINARY OUTCOMES AND CONCLUSIONS

This paper has presented the observed ground motions and various forms of damage in the Marlborough region from the 21 July and 16 August 2013 earthquakes. In near-source townships such as Seddon and Ward, the ground shaking was relatively severe however it is fortunate that the level of damage sustained to roading infrastructure, winemaking facilities, water retaining structures and building construction was generally moderate. Only a small proportion of the region's population experienced severe ground shaking and subsequent damage, as the ground shaking in Blenheim CBD was relatively low due to the earthquake source distance being approximately 30 kilometres. The aim of the following discussions is to reflect on the post-earthquake situation from this particular event and provide future considerations for various engineering aspects.

Earthquake prone buildings

Seismic assessment of New Zealand's existing building stock is in a state of constant engineering decision making. The Blenheim CBD is dominated by pre-1977 construction of two-storey buildings with open ground level retail spaces. A substantial portion of these buildings had to be addressed following the 2004 Building Act. At the end of 2007, the Marlborough District Council (MDC) commissioned the assessment of building seismic performance based on the Initial Evaluation Procedures (IEP) according to the 2006 NZSEE guidelines [25]. Approximately 55 buildings were subsequently identified as earthquake prone (assessed below 34 percent New Building Standard, *%NBS*). In 2011, the MDC gave notice to the owners of 22 buildings in Blenheim and Picton to undertake retrofit intervention or demolition within the next 5-10 years. While these earthquakes did not subject buildings in the Blenheim CBD to large seismic demands or cause noticeable damage, these events have provided a timely reminder for building owners to undertake action.

The authors are aware of five buildings that have been strengthened since 2011. Figure 16(c) shows large external steel frames that were installed at the ground level of a two storey building. The building in Figure 16(a) was not identified as being earthquake prone however a retrofit intervention is currently being undertaken to improve confidence levels of building occupants who had perceived the building as vulnerable. One building was demolished prior to the July-August events with planned reconstruction. The 16 August event was a catalyst for the owner of the neighbouring building to demolish their building (demolished at the time of writing) in advance of the MDC's notified 2018 deadline.

Public perception of earthquakes

Along with the 2010-2011 Canterbury earthquake sequence, these July/August 2013 earthquakes in central NZ have provided a unique survey of the public perception of earthquakes. While the potential post-earthquake consequences in Marlborough are a much lower priority than Wellington City, for example, this particular earthquake resulted in ground shaking that was much more severe in the Marlborough region than that in Wellington.

Initially after the 16 August event, managers and owners of wine companies had publicly announced that transporting their volumes of wine to another location in New Zealand was being considered as a reasonable future option to reduce the seismic risk the companies were exposed to [J. Stanton, media comm.]. Clearly such comments illustrate the need for improvements in the design and performance expectations of wine infrastructure following the learnings from this sequence of seismic events; but also that winery owners are informed of the seismic risks throughout New Zealand and the costs involved in mitigating these.

The general public would benefit from an increased awareness of the spatial and temporal distribution of seismic hazards in New Zealand. The trending public perception is that localised places such as Christchurch and Wellington are "*earthquake places"* thus demonstrating the lack of public awareness of the seismic hazards and risks throughout NZ. Future clarification of the challenges faced for design and construction in seismic regions will remain as an important responsibility for engineers.

Personal experiences from earthquake events will continue to be the major source of public "memory" or awareness for future events. At the present time in NZ there is a lack of sharing these past experiences and transferring the lessons learnt from old to young. For instance, older members of the Marlborough community previously experienced earthquake events between the 1960-1980s (discussed in an earlier section). However those living in the region between the 1990-2000s showed a general lack o*f* awareness that Marlborough was a known seismic region.

Ground motions and geotechnical impacts

The number and distribution of strong motion instruments in the Marlborough region is relatively low such that complex characteristics of source rupture, wave propagation and site response cannot be rigorously studied to the extent possible for other regions of NZ, such as for the Canterbury earthquake sequence [10].

The higher intensity ground motions generally occurred in places where geotechnical conditions were such that major ground deformation was not observed. This is one reason for the overall levels of damage being lower than expected. The foundations of buildings did not suffer notable settlements and liquefaction was not observed near the built environment while lateral spreading/slope stability was deemed as minor. The observed settlements at bridge approaches and deformations of embankment were generally found to be minor, but moderate in a few cases.

Transportation infrastructure

Rapid response works were effective in minimizing closure times on State Highway 1 as slips were cleared, pavement cracks were repaired and bridge approach settlements were filled in. Bridge structures generally performed very well. Minor damage to the Awatere Bridge will require minor repairs. Embankment stability continues to be an important factor for future roading construction, particular in areas of large fill volumes.

Winemaking facilities

The winemaking industry would benefit from improved communication between owners and structural designers on the design intent, the potential outcomes and the associated financial risk that is accepted with respect to the targeted performance criteria. Future design considerations for winemaking facilities were presented within an earlier section.

Water storage dams

A large number of small to medium sized water storage dams and HDPE lined ponds were located relatively close to the source of the 16 August Lake Grassmere earthquake. Most dams and ponds experienced no damage, despite the ground motions reaching a level where design analyses would have predicted some small permanent deformations. A small number of dams and at least one pond experienced some damage. There are current investigations being undertaken for the dams and at the pond that did experience damage. More lessons learnt from these events may become apparent with further investigation.

In the Marlborough area, a number of dams have used rounded cobbles from the local rivers as riprap. This material has a lower friction angle and is more prone to erosion and instability compared to quarried (angular) rock, particularly if a geotextile is used as an underlying separation/filter layer. This factor needs to be considered in design.

An interesting issue from this event was the fact that it is conventional to design water storage dams for seismic actions assuming that heavy rainfall will not be coincident. This was not the case when the 16 August earthquake occurred. Heavy volumes of rain fell after the earthquake which was a factor that led to excavation of a channel to lower the water level in Dam E. The effects of heavy rain following a large earthquake should be given consideration when preparing Operating and Emergency Action Plans.

Residential buildings

The performance of light timber framed housing is considered as very satisfactory with respect to the design objective of preserving life safety and particularly given the severity of ground shaking in Seddon and Ward. The lack of structural damage to housing may also be attributed to the lack of geotechnical impacts such as liquefaction and differential settlements compared to observations from the 2010-2011 Canterbury earthquakes.

A recommendation for future construction is to ensure that veneer ties are installed for brick cladding as a number of out-of-plane collapses were observed near the roof soffit, at exterior corner regions and around door and window openings. To some extent, external cladding restraint could be adopted to mitigate future damage to existing houses.

Commercial structures

In the Blenheim CBD the ground shaking was less severe than in Seddon or Ward and the only observed damage was to relatively fragile non-structural components such as interior wall linings and ceilings. The ceiling systems that sustained damaged generally contained relatively heavy ceiling tiles, which are not recommended for new designs. There were no reports for structural damage in the Blenheim CBD following this particular earthquake.

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