6. PERFORMANCE OF STOPBANKS

Shortly after their arrival in the Canterbury area in the mid-nineteenth century, the western settlers started constructing drainage systems and levees along rivers (Larned et al., 2008). Particularly, flooding of the Waimakariri River and tributaries posed a constant threat to the Christchurch and Kaiapoi areas. The current levee system is a culmination of several coordinated efforts that started in earnest in the 1930s and is comprised of both primary and secondary levee systems. The primary levee system is designed for the 450 year flood. Damage estimates for scenarios where the flood protection system is breached have been assessed at approximately NZ$5 billion (van Kalken et al., 2007). As a result, the performance of the levee system during seismic events is of critical importance to the flood hazard in Christchurch and surrounding areas.

During the 2010 Darfield and 2011 Christchurch earthquakes, stretches of levees were subjected to motions having peak horizontal ground accelerations (PGAs) of approximately 0.32g and 0.20g, respectively. Consequently, in areas where the levees were founded on loose, saturated fluvial sandy deposits, liquefaction related damage occurred (i.e., lateral spreading, slumping, and settlement). The performance summary presented herein is the result of field observations and analysis of aerial images (NZAM 2010, 2011), with particular focus on the performance of the levees along the eastern reach of the Waimakariri River and along the Kaiapoi River.

In the sections that follow, background information about the levee system is first presented. This is followed by an overview of the performance of the levees during the Darfield and Christchurch earthquakes. Next, the relationship between the severity of damage to the levees along the downtown stretch of the Kaiapoi River and the response of the foundation soils are discussed.

Background of Stopbank System

The Waimakariri River flows from the Southern Alps, across the Canterbury Plains between Christchurch, to the south, and Kaiapoi, to the north, and empties into Pegasus Bay in the East (Figure 6-1). The river drains a mountainous catchment area of 3566 km^2 and poses the most significant flood hazard in New Zealand (van Kalken et al., 2007). Early efforts by western settlers to realign and contain the river within its banks were piecemeal and only partially successful (e.g., Wotherspoon et al., 2011). To better coordinate the efforts and to ensure equal flood protection to both Christchurch and Kaiapoi, the Waimakariri River Trust was established in 1923 (Griffiths, 1979). In response to the 1926 floods (Figure 6-2), the Trust implemented a major river improvement scheme in 1930, known as the Hays No. 2 Scheme. Among other things, the scheme entailed an overall improvement of the levee system along the Waimakiriri River. However, these improvements were unable to prevent the major floods in 1940, 1950, and 1957. These floods prompted a further river improvement scheme in 1960, which entailed benching existing levees and construction of new levees.

The mean annual flow of the Waimakariri River is 120 m^3/s. However, in 1957 the largest flood on record occurred, with an estimated peak discharge of 4248 m^3/s (Griffiths, 1979). This flood was initially estimated to have a 100 year return period (Griffiths, 1979) but was later revised to an approximately 450 year return period (Heslop, 2010), and essentially served as the design basis flood for the levee improvement scheme implemented in the 1960s. Flood protection now includes approximately 100 km of levees.

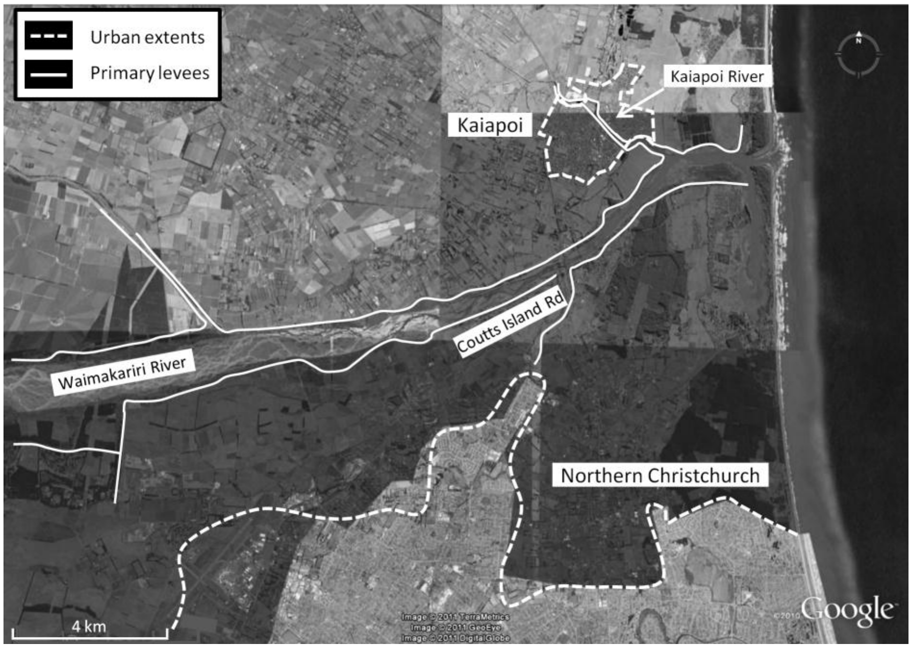


Figure 6-1. Canterbury region of New Zealand.


Figure 6-2. 1926 photograph of the Waimakariri River overflowing its banks in Christchurch. (Te Ara Encyclopedia of New Zealand, 2010)

A typical levee cross section in the Canterbury region has 3:1 horizontal to vertical slopes on both the river and land sides (Figure 6-3). They range in height from 3-5 m above the subgrade and have a 4-m wide top, which also serves as an access road. A flood event originating in the headwaters of the Waimakariri River takes approximately 1.5 days to travel downstream before it reaches the levee system that protects Christchurch and surrounding areas. At its crest, the 450 year event would leave 90 cm of freeboard, but may only last for four hours (Heslop, 2010).

The levees were often constructed by pushing up river gravels and silts. A typical cross section is made up of a gravel core with 1-m thick silt cap, which extends from the river side across the top (Figure 6-3). The levees typically sit on sandy soils at or near the ground water level. A toe filter was also constructed on the land side of the levee to prevent piping of sand during a high water event. During the 1960 river improvement scheme, some new levees were constructed and benches were added to some of the existing levees, both of which were compacted using vibrating rollers (Boyle, 2010). However, no compaction control or foundation analysis was conducted (Heslop, 2010).

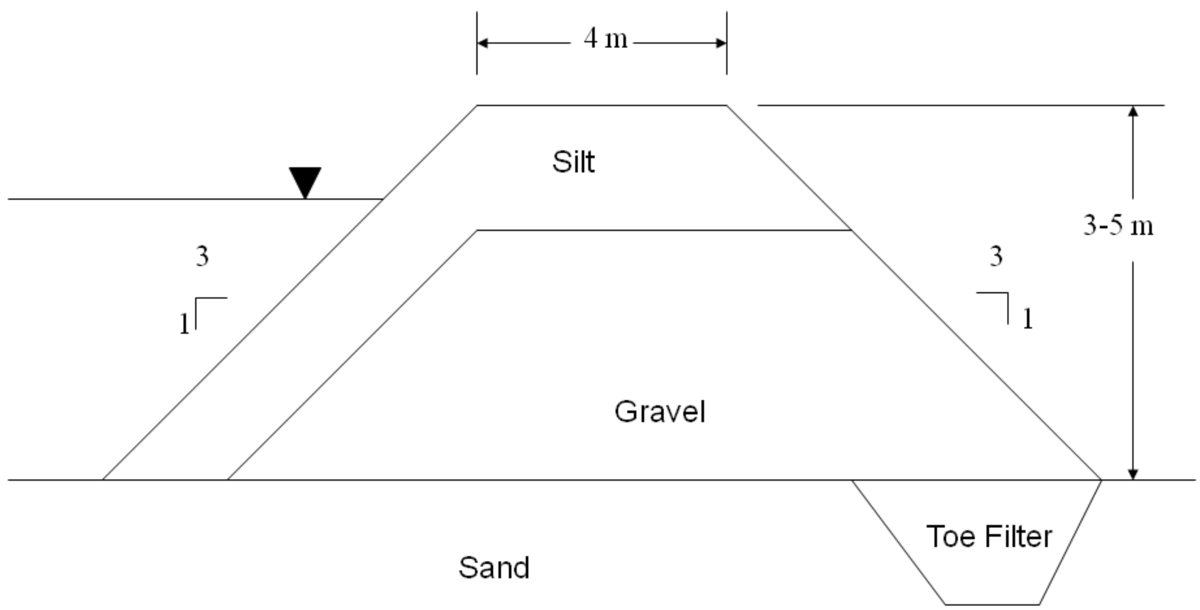


Figure 6-3. Typical geometry and soil composition of levee cross section.

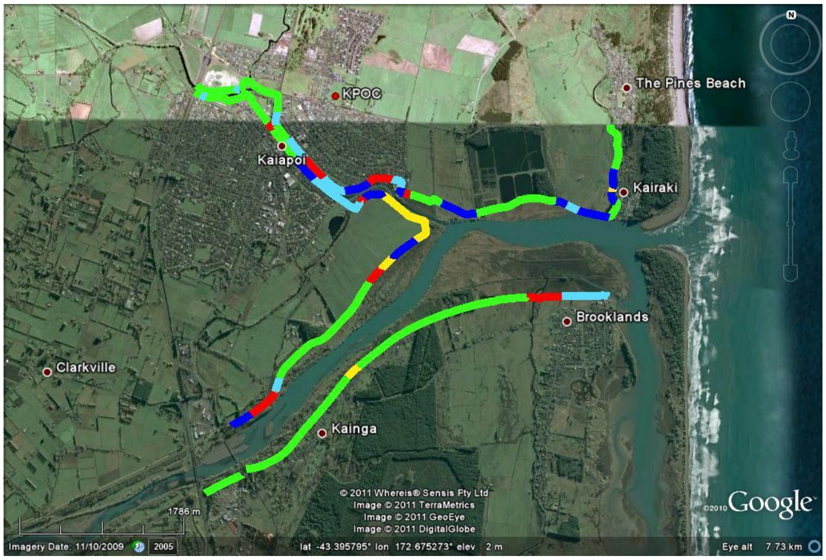
Seismic Performance of Stopbanks

The GEER team performed damage surveys along stretches of the primary and secondary levees for the Waimakariri River and along the primary levees for the Kaiapoi River following both the 2010 Darfield and 2011 Christchurch earthquakes. The surveys were performed on foot, in an automobile, and from a helicopter. Additionally, the GEER team used high resolution aerial images to aid in the damage survey and corresponded with the Environment Canterbury (ECan) personnel (Ian Heslop and Tony Boyle). Heslop and Boyle oversaw the post-earthquake damage assessments performed by a local consulting firm and continue to oversee the repairs to the sections of the damaged levees. Below is a summary of the levee performance along the eastern reach of the Waimakariri River and along the Kaiapoi River.

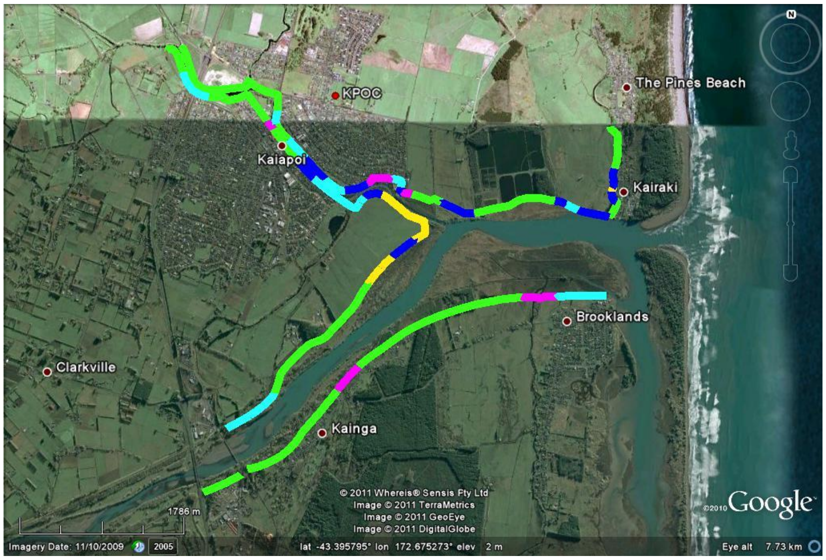
As a result of the damage caused by Darfield Earthquake, ECan estimated that the flood capacity of the Waimakariri levee system had been reduced from a 450 year event (4730 m^3/s) to approximately a 15 year event (1500 m^3/s). Subsequently, concerns were raised when river flows rose to approximately 1000 m^3/s in the days following the Darfield earthquake (Boyle, 2010, Heslop 2010). ECan proceeded with repairs to the most severely damaged sections of levees within weeks of the Darfield earthquake. Repairs progressed from severely damaged areas to those with only minor damage. By December 2010, the reconstruction had increased the flood protection capacity of the system to 2500 m^3/s or 1 in 20 year return flood event (Heslop, 2011). These reconstruction costs, as a result of the Darfield earthquake, were approximately $NZ 4 million, which was at the upper bound of the estimate provided by ECan shortly after the earthquake (Heslop 2010).

Damage repair to the levee system on the Waimakariri River was nearly complete at the time of the February 2011 Christchurch earthquake, with the system having been returned to a 3000 m^3/s or 1 in 30 year event in December of 2010. The Christchurch earthquake reduced the flood protection capacity back to 2500 m^3/s or a 1 in 20 year return flood event. As of July 2011, the restoration work has nearly been completed, increasing the capacity to 4000 m^3/s or a 100 year flood event. ECan estimates that an additional $NZ 2 million in damages to the levees were caused by the Christchurch earthquake (Heslop, 2011). Total restoration to pre-Darfield earthquake flood capacity is expected by end of 2011. There was minor damage to the levee system caused by the 13 June 2011, M_w6.0 aftershock, but it did not result in a reduction in flood capacity.

The majority of the damage to the levees resulting from both the Darfield and Christchurch earthquakes occurred east of SH1 as depicted in Figure 6-4 (note, SH1 is shown in Figure 6-7). In Figure 6-4, damage severity is categorized using the scale developed by Riley Consultants (2010, 2011). The scale has five grades that range from No Damage to Severe Damage, as summarized in Table 6-1. As may be observed from Figure 6-4, the damage patterns to the levees following both earthquakes are very similar, but are in general less severe for the Christchurch earthquake as compared to the Darfield earthquake. Note that some portions of the levees were already under repair by the time the GEER team were able to inspect them following the Christchurch earthquake. In these cases, the GEER team supplemented their field observations, to the extent possible, with both observations from high resolution aerial images taken the day after the Christchurch earthquake and field observations made by ECan consultants (Riley Consultants, 2011).



(a)



(b)

Figure 6-4. Observed damage to levees following the (a) 4 September 2010, M_w7.0 Darfield earthquake and (b) 22 February 2011, M_w6.1 Christchurch earthquake. (Adapted from Riley Consultants, 2010, 2011).

The majority of the damage to the levees was a consequence of liquefaction in the foundation soils that result in lateral spreading, slumping, and/or settlement. The damage mostly manifested as longitudinal cracks running along the crest of the levees (Figure 6-5a). Although not desirable, moderate crack widths for this mode of damage are not believed to be critical to the functionality of the levees because it does not provide a direct seepage path from one side of the levee to the other. However, there is always the potential for these longitudinal cracks to connect undetected transverse cracks or flaws that only penetrate partway through opposite sides of the levee. Such a tortuous seepage path could potentially enlarge rapidly due to internal erosion and piping at high river levels.

Transverse cracks in the levees were less commonly observed than longitudinal cracks and were often associated with sharp bends along the length of the levees and/or slumping of the embankment (Figure 6-5b). Because these cracks provide a direct seepage path from one side of the levee to the other, they can severely impact the functionality of the levees. Even transverse cracks having minor widths could potentially enlarge rapidly due to internal erosion and piping at high river levels and lead to the failure of that section of the levee.

Settlement of levee sections resulted from both post-liquefaction consolidation in the foundation soils and bearing capacity failures due to the reduced strength of the liquefied foundation soil. In addition to the degradation in levee functionality due to cracking associated with the settlement (similar to that discussed above), settlement also reduces the amount of freeboard at high river levels. The significance of this loss is dependent on magnitude of the settlement, but in general it is not thought to be a significant issue with the levee system.

Another liquefaction related mode of degradation to the levees' capacity is where liquefaction and/or lateral spreading formed on both sides of the levee. In these cases a potential flow path is formed down through the vertical feeder dike on the river side of the levee, laterally through the source stratum under the levee, and up through the vertical feeder dike on the other side of the levee. Extensive liquefaction was observed on both sides of the levee along an approximate 0.5 km stretch of the Waimakariri River on Coutts Island Rd (Figure 6-6). From interviews with local land owners and review of maps of the area from 1865, this area was part of an old river channel (Wotherspoon et al., 2011). Additionally, borings performed by the GEER team using a hand auger showed a deep sand deposit along this 0.5 km stretch of levee and buried sticks and logs on both ends, consistent with an old river channel and river channel banks.

Severity of Damage and Foundation Soils

To examine the relationship between the severity of the induced damage to the levees and the liquefaction response of the foundation soil, a stretch of levees along the Kaiapoi River was examined in more depth. As shown in Figure 6-4, these levees sustained damage ranging from No Damage to Severe Damage (Table 6-1). Following the Darfield earthquake, the New Zealand Earthquake Commission (EQC) contracted a local firm to perform a series of cone penetration tests (CPT), among other in-situ tests, throughout North and South Kaiapoi (Tonkin and Taylor, 2010). The locations of the CPT soundings performed on, or adjacent to, the levees along the Kaiapoi River are shown in Figure 6-7.

(a)

(b)

Figure 6-5. Cracks in levee: (a) Example of longitudinal cracks running along the crest of the levee; and (b) Example of transverse (or oblique) crack in levee.

(a)

(b)

Figure 6-6. (a) Large sand boil on landside of the levee on Coutts Island Rd. (b) Large sand boil on riverside of the same section of the levee.

Table 6-1. Damage severity categories (Riley Consultants, 2010, 2011)

Category	Description
No Damage	No observed damage
Minor Damage	Cracks up to 5 mm wide and/or 300 mm deep. Negligible settlement of crest.
Moderate Damage	Cracks up to 1 m deep. Some settlement of crest.
Major Damage	Cracks greater than 1 m deep. Evidence of deep seated movement and/or settlement.
Severe Damage	Severe damage or collapse. Gross lateral spread and/or settlement, cracks showing deformation of 500 mm or more.

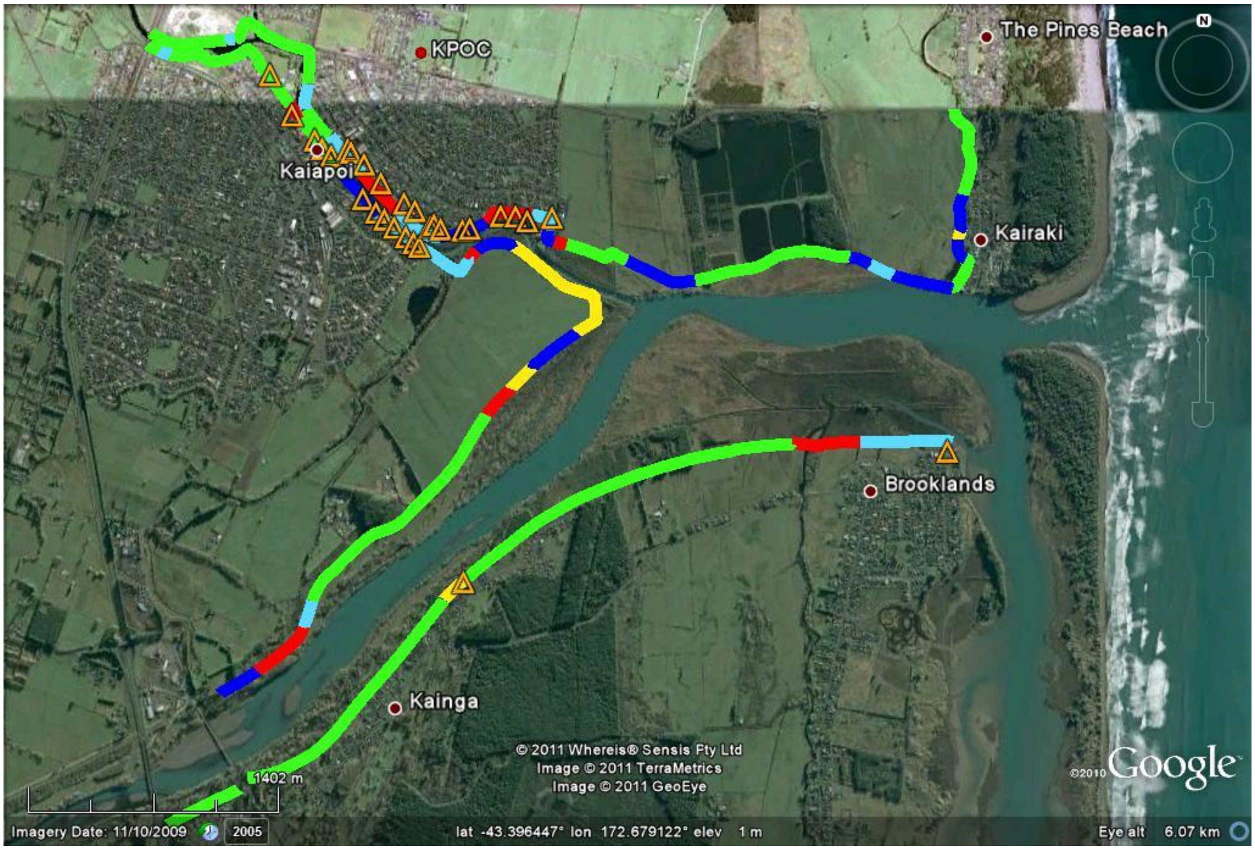


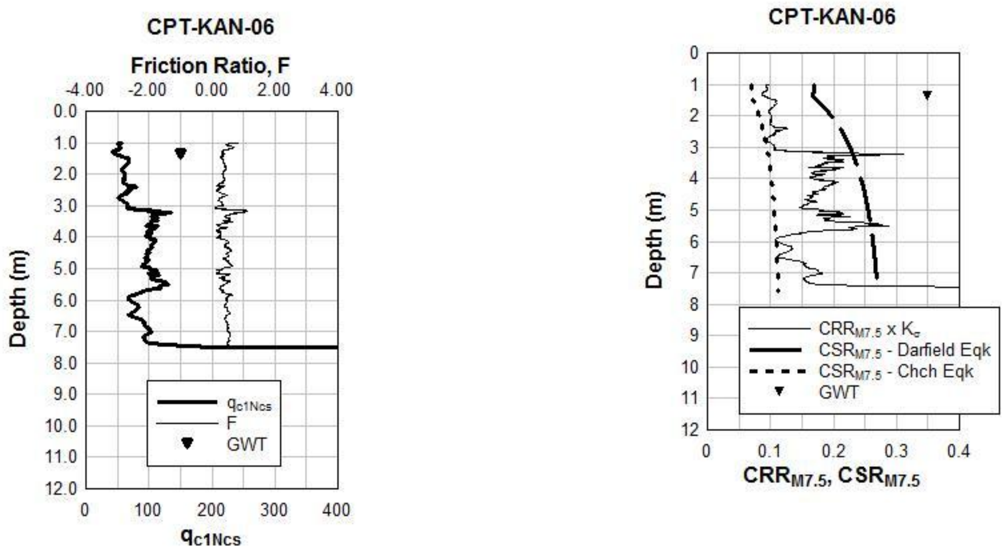
Figure 6-7. Locations of CPT soundings on or adjacent to levees.

Representative CPT soundings from the north and south banks of the Kaiapoi River are presented in Figures 6-8a and 6-9a. From interpreting twenty seven such CPT logs, as well as available borehole data (Tonkin and Taylor, 2011), the soil profile along the north bank of the

Kaiapoi River east of the Williams Street Bridge is characterized by approximately 4 m of very loose to loose sand overlying approximately 4 m of loose to medium dense gravelly sand. Below approximately 8 m, the sand and gravel layers tend to be significantly denser than the overlying layers. The depth to the ground water table varies, but is approximately 1.5 m deep. On the south bank of the Kaiapoi River east of the Williams Street Bridge, the soil profile is characterized by approximately 6 m of very loose to loose silty sand/sand overlying an approximately 2-m thick layer of loose to medium dense sand/gravelly sand. Samples of the liquefiable soils taken adjacent to the levees on the north bank had fines contents around 15%, with the fines being non-plastic. Below approximately 8 m the sand and gravel layers tend to be significantly denser than the overlying layers. The ground water table is approximately 2 m deep.

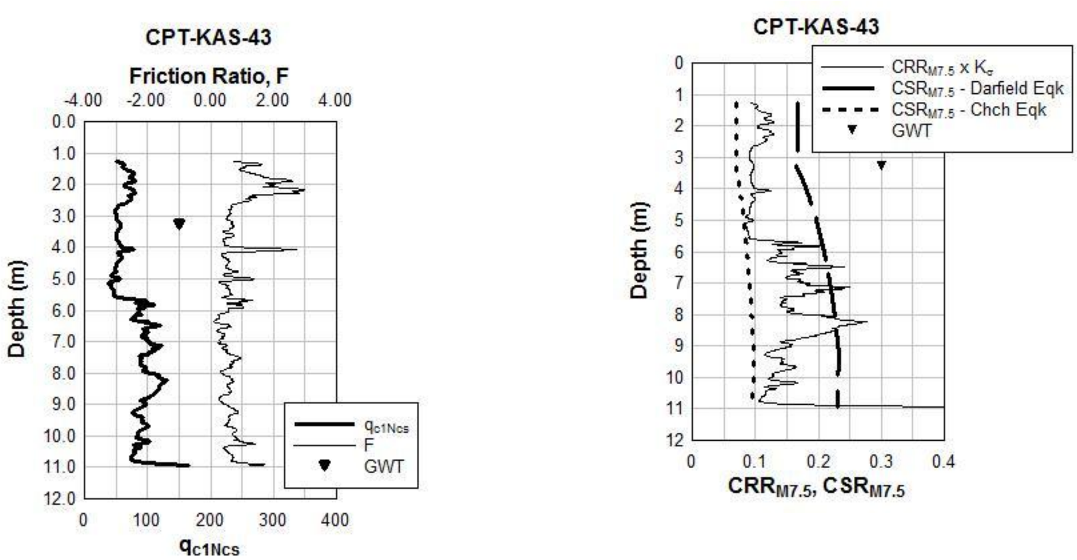
Using the twenty seven Kaiapoi levee CPT soundings and two additional soundings performed adjacent to the levees along the southern bank of the Waimakariri River in Kainga and Brooklands, the liquefaction of the foundation soils was analyzed following the procedures outlined in Youd et al. (2001). The strong motion seismograph station KPOC is located in North Kaiapoi (Figure 6-7) and recorded both earthquakes. The geometric mean of the horizontal peak ground accelerations (PGA) of the motions recorded during the Darfield and Christchurch earthquakes were 0.32g and 0.20g, respectively. The distance from the strong motion station to the CPT sounding locations ranges from approximately 0.7 to 3.7 km, with the majority of the soundings being located less than 1 km from the station. Because of this close proximity, 0.32g and 0.20g PGAs were used to compute the cyclic stress ratios (CSRs) imposed on the soil at all the sounding locations during the Darfield and Christchurch earthquakes, respectively.

Figures 6-8b and 6-9b show the results from the liquefaction evaluation for the two representative CPT soundings mentioned above. In these figures, the cyclic resistance ratio (CRR) for each profile and the CSRs for both events are plotted together, where both the CRR and CSR are adjusted to a M_w7.5 earthquake. For liquefiable soils (i.e., gravels, sands and cohesionless silts), liquefaction is predicted to have occurred at depths where the $CSR_{M7.5} > CRR_{M7.5}$. Accordingly, for both profiles, liquefaction is predicted to have occurred during the Darfield earthquake for almost the entire depth from the ground water table to the top of the dense gravel/sand layer (i.e., to ~7.5 m and ~11 m for the north and south river banks, respectively). However, during the Christchurch earthquake, marginal liquefaction is predicted to occur at a few isolated depths within both profiles.



(a) (b)

Figure 6-8. (a) Representative CPT sounding for the north bank of the Kaiapoi River; (b) Liquefaction evaluation of the site for both the Darfield and Christchurch earthquakes.



(a) (b)

Figure 6-9. (a) Representative CPT sounding for the south bank of the Kaiapoi River; (b) Liquefaction evaluation of the site for both the Darfield and Christchurch earthquakes.

In an attempt to relate the severity of the observed levee damage to the liquefaction response of the foundation soil, plots of factor of safety against liquefaction versus damage index (i.e., FS_{Liq} vs. DI) and thickness of the liquefied layer versus damage index (i.e., T vs. DI) were made for the 29 CPT soundings analyzed. Note, damage index corresponds to the damage categories proposed by Riley Consultants (2010, 2011): 1 = No Damage, 2 = Minor Damage, 3 = Moderate Damage, 4 = Major Damage, and 5 = Severe Damage. Linear regressions were performed on the data, where first the data from the two earthquakes were kept separate (Figure 6-10) and then they were combined (Figure 6-11). In developing these plots, the sections of the levees that were under repair at the time of the GEER team's field inspections were assumed to have DI = 4. The basis for this is that the fact the sections were given high priority for repair implies that the sustained damage was significant. However, because the intensity of shaking during the Christchurch earthquake at these locations was significantly less than that during the Darfield earthquake, it is likely that the levels of damage induced by the Christchurch earthquake were less severe than that from the Darfield earthquake.

Expected trends can be identified in all plots (i.e., the damage index increases as the factor of safety against liquefaction decreases and as the thickness of the liquefied layer increases). However, the strength of the trends, as indicated by the correlation coefficients (r^2), varies between the two earthquakes when the data is treated separately. For example, for the Darfield earthquake, the lowest correlation coefficient (r^2 = 0.147) is for T vs. DI, but T vs. DI has the highest correlation coefficient (r^2 = 0.625) for the Christchurch earthquake. In contrast, the correlation coefficients for FS_{Liq} vs. DI are relatively consistent for both the Darfield and Christchurch earthquakes (i.e., r^2 = 0.562 and r^2 = 0.595, respectively). When the data from the two earthquakes are combined, r^2 = 0.348 and r^2 = 0.578 for T vs. DI and FS_{Liq} vs. DI, respectively.

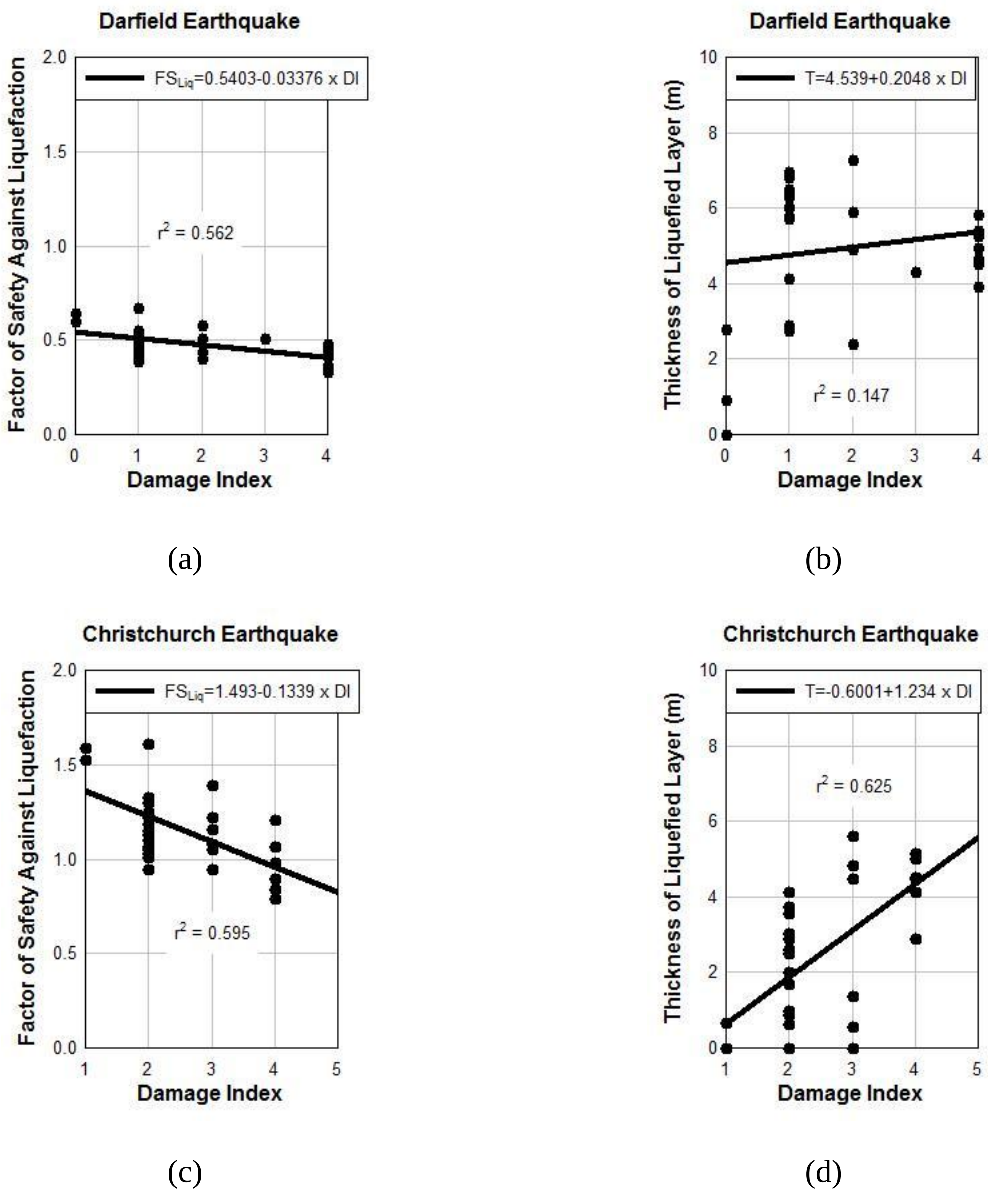
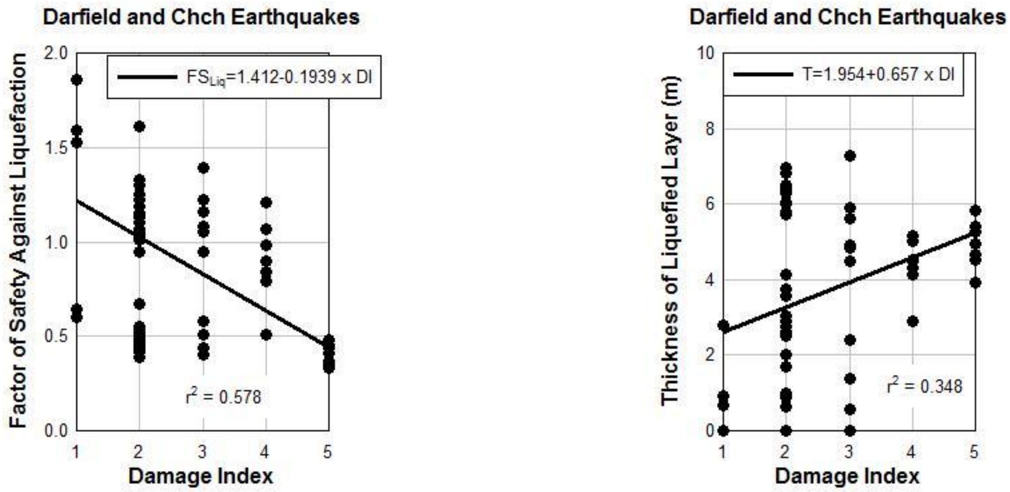


Figure 6-10. Correlations relating factor of safety against liquefaction and damage index and thickness of the liquefied layer for the data from the Darfield and Christchurch earthquakes regressed separately. (Note that the low r^2 value in (b) indicates an extremely weak correlation between the thickness of the liquefied layer and damage index for the Darfield earthquake; hence a dotted line is used to show the results of the regression).



(a) (b)

Figure 6-11. Correlations relating factor of safety against liquefaction and damage index and thickness of the liquefied layer for combined Darfield and Christchurch earthquake data.

From the correlation coefficients, the factor of safety against liquefaction appears to be a better index for damage severity than the thickness of the liquefied layer. This is not altogether surprising given that a lot of the damage to the levees resulted from lateral spreading, more so than deep seated slumping and settlement/bearing capacity failures. Of these three failure modes, lateral spreading can occur even if a relatively thin layer liquefies, while deep seated slumping and settlement/bearing capacity failures require a thicker layer to liquefy. This is likely the reason for the disparity between the r^2 values for the T vs. DI plots for the Darfield and Christchurch events. In the case of the Darfield earthquake, the levees were subjected to relatively intense shaking and the thickness of the liquefied layer was large. However, because lateral spreading can occur on even a thin liquefied layer, the r^2 value for the T vs. DI plot was very low (i.e., $r^2 = 0.147$). In contrast, the levees were subjected to less shaking during the Christchurch earthquake and the liquefied layers were relatively thin, where liquefaction occurred. However, even these relatively thin liquefied layers were thick enough for lateral spreading to occur, which resulted in damage to the levees and a relatively high value of r^2 for the T vs. DI plot (i.e., $r^2 = 0.625$). The implication of this is that liquefaction severity indices that account for both the factor of safety against liquefaction and thickness of the liquefied layer, such as the Liquefaction Potential Index (LPI) (Iwasaki et al., 1982), may not be appropriate for evaluating the risk of damage from liquefaction where lateral spreading is the primary failure mode.

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