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Prediction of Liquefied Strata from the 1987 Edgecumbe Earthquake

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ABSTRACT: On March 2 1987, an earthquake of magnitude (ML) 6.3 occurred near the town of Edgecumbe, in the North Island of New Zealand. Extensive ground damage in the form of liquefaction and surface faults resulted from the shock, especially in the recent sediments of the Rangitaiki Plains surrounding Edgecumbe. This paper focuses on a major lateral spreading site, at the Landings Road Bridge, which lies on the eastern side of the Rangitaiki Plains. The liquefaction beside the bridge produced both sand boils and lateral spreading of the overburden towards the river. Data from the probes and the use of empirical and semi-empirical models enabled the prediction of which strata liquefied during the earthquake. A three-dimensional model of the lateral spreading in this area was established, and a verification of lateral spreading models was accomplished.

1 INTRODUCTION

This paper provides background information on the events of the 1987 Edgecumbe earthquake and then looks at the effects at the Landing Road Bridge site in detail. Firstly, a general overview of the geology of the area is given, which is then followed by a summary of the earthquake events and resulting damage. Details of the effects that the earthquake had on the Landing Road Bridge site are then examined.

1.1 Background information and geology

The subduction boundary between the Pacific Plate and the Australian Plate, see inset on Figure 1, makes the central North Island of New Zealand a seismicity active region. Within this region, the Taupo Volcanic Zone is an area of marked high seismic and volcanic activity. The Edgecumbe earthquake sequence occurred in the Whakatane Graben, which lies to the northern end of the Taupo Volcanic Zone. The Whakatane Graben is composed of the alluvial Rangitaiki Plains surrounded by hills and is crossed by three major rivers the Whakatane, Rangitaiki and the Tarawera Rivers. At the time of the earthquake in excess of 25 000 people lived throughout the plains, concentrated mainly in the towns of Whakatane, Edgecumbe and Kawerau (Pender and Robertson, 1987).

The hills to the west of the Rangitaiki Plains are composed of late Quaternary rhyolitic volcanics while the hills to the east are composed of mesozoic greywackes. Within the graben mesozoic greywackes have been down-faulted to a depth of two kilometres below the current sea level (Nairn and Beanland, 1989). Infilling of the basin has occurred with Quaternary volcanics and sediments from the catchments to the south. Subsidence along the graben axis has been occurring at a rate of 2-3 mm/year for the last 5 500 years, whereas the hills to the sides of the graben have been rising by in excess of 0.5 mm/year for the last 120 000 years and the graben is widening at 7 mm/year. Ground water logs across the plains indicate lithological variability both vertically and horizontally, with deep bores indicating alternating layers of pumice derived alluvial sand and gravel, tephra, marine silt and sand.

A mixture of dextral and sinistral faulting implies either local reversals of the stress field or that the zones widens via a complex system of mainly dextral faults in the west, sinistral in the east and normal faults in the middle (Pender and Robertson, 1987). The basement structure dips north-east from Kawerau into the Whakatane graben. The Edgecumbe fault which pre-existed before the 1987 earthquake (but was unrecognised), dips to the west at $45^{\circ} \pm 10^{\circ}$ and swings upward from 100 m deep to become vertical at the surface. Fault traces are controlled by surface geology, which is indicated by their non-linearity as seen in Figure 1.

The present surface of the Rangitaiki Plains was formed around 700 years ago when the sea attained its present level. The shoreline has since prograded about 10 km with major sedimentation episodes following the Whakatane (c.1850 years BP) and Kaharoa (c.800 years BP) pyroclastic eruptions. Heavy forest covered the Rangitaiki Plains 150 years ago, but the land is now used for pastoral farming.

1.2 1987 Edgecumbe earthquake sequence

Seismic activity started in the region one week prior to the main event and culminated in a foreshock of magnitude (ML) 5.2 seven minutes before the main event. The main event of magnitude (ML) 6.3 occurred at 01h 42m 34s UT (1.42 pm local time), with a focal depth of 12 ± 1 km (Pender and Robertson, 1987). Strong ground motion (>0.05g) lasted 8-9 seconds, with a maximum of 0.33g being recorded at Matahina Dam, which was the closest accelograph to the epicentre (within 15 km of the epicentre). This is the strongest ground motion ever recorded in New Zealand. Response spectra of the

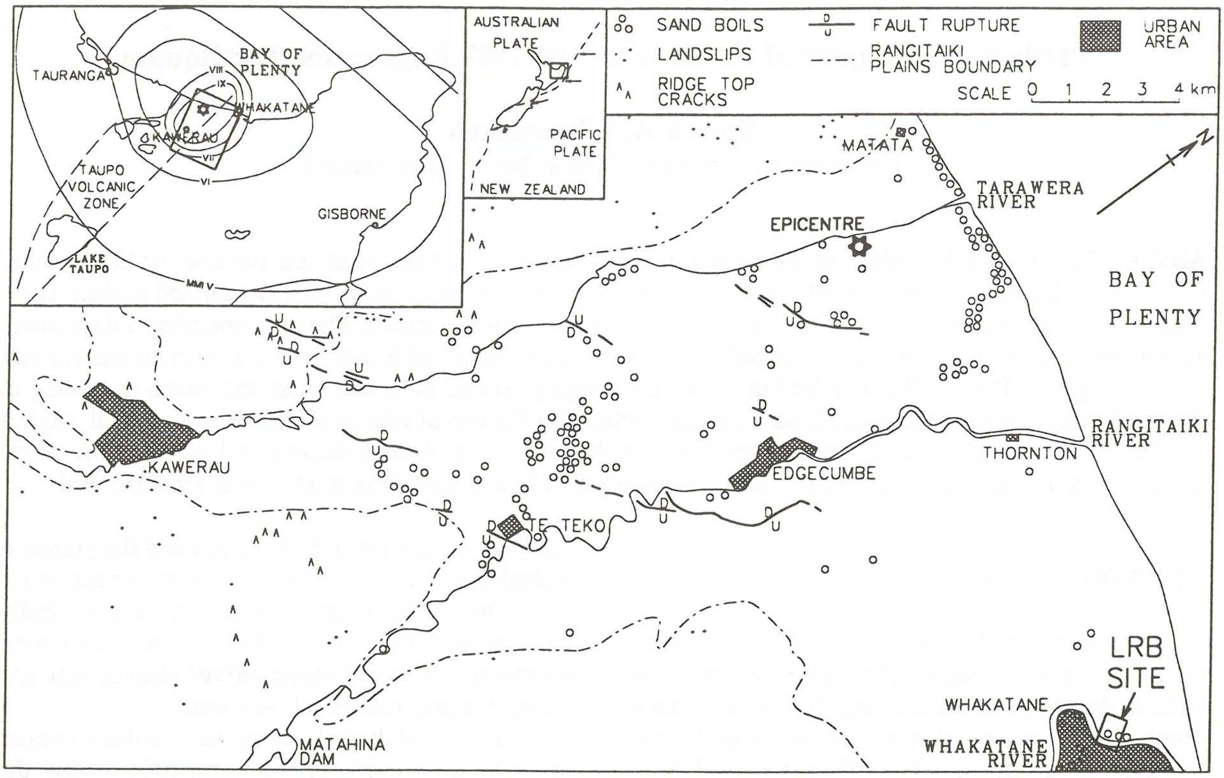

Figure 1: Map of the Rangitaiki Plains showing the ground damage caused by the 1987 Edgecumbe earthquake. Large inset showing Bay of Plenty region and isoseismal from the earthquake.

recording at Matahina Dam, are similar to that of El Centro NS 1940 in periods less than 0.5 s and greater than 1.2 s. In the hours following the earthquake four aftershocks of magnitudes between (ML) 5.1 and 5.6 were recorded. In total 600 shocks of magnitude (ML)$>$3.0, in an area of 70 x 50 km, were noted throughout the earthquake sequence.

This earthquake produced Modified Mercalli Intensities of up to MMI X in the Edgecumbe area, this high epicentral intensity being due to the shallow focal depth. Rapid diminution of intensity with distance was due to high attenuation in the near surface sediments (Pender and Robertson, 1987).

1.3 Observed damage of the earthquake

Heavy structural damage resulted in the large dairy factory in Edgecumbe, while paper and paper board mills at Kawerau and Whakatane suffered damage to a lesser extent. Other structural damage included: compression buckling of railway tracks and pavements; shell failures and tops pushed off liquid containers; a locomotive and trucks overturned; intensive breakage of pipes in Edgecumbe; destruction of a shopping mall in Edgecumbe; toppling of unreinforced concrete block walls; cranes and electricity transformers pushed off rails; chimneys were toppled and a plastic hinge formed in one of the pier-pile interfaces on the Edgecumbe rail bridge. Damage to structures in Whakatane was mainly restricted to areas adjacent to the Whakatane River.

Ground damage was mainly confined to the recent soft sediments of the Rangitaiki Plains. Ten fault traces, a mixture of pre-existing and new, accounted for total surface fractures of 18.4 km. The largest fault, the Edgecumbe fault was 7 km long with a average vertical displacement of 1.5 m extending to an estimated 2.1 m at the maximum. Along with the normal faults and compressional rolls, there was regional subsidence of up to 2 m at Edgecumbe, which varied gradually to a few centimetres uplift in Whakatane. Level ground liquefaction was widespread throughout the plains and lateral spreading often occurred next to the rivers. In excess of 26 km of stopbanks required repairs and 13 (6 serious) of 39 drainage schemes were damaged. Stopbanks adjacent to the Whakatane River, made of well graded material, suffered only longitudinal cracks, whereas Tarawera River stopbanks (made of coarse pumice sand, 90 years old) were damaged due to slumping (Franks et al.). Repairs for the extensive damage to drainage works due to lateral spreading, fault traces and general subsidence cost in excess $NZ 10M. Other ground damage included ridge top cracks up to 150 mm wide and slope stability problems due to slumping of cohesionless lapilli in the hills.

Liquefied ejecta ranged from medium sands to fine sands, in general carrying pumice lapilli up to 20 mm in diameter; in some areas coarse silts were ejected. Ejecta were quite stratified with coarse sands on the bottom and fine sands on top or vise versa. The liquefaction which occurred followed generally accepted patterns of behaviour (Jennings et al., 1988).

2 LANDING ROAD BRIDGE SITE

This site is of particular interest due to the intensive degree of liquefaction and lateral spreading that occurred in result of the 1987 Edgecumbe earthquake. This site, labled LRB in Figure 1 and shown in detail in Figure 2, lies beside State Highway 2 (SH2) on the approach to the town of Whakatane. SH2 crosses the Landing Road Bridge which traverses the Whakatane River on the southern side of the site.

The Landing Road Bridge site lies beside a very active river area. At this location beside the river, the water table depends on the tide which can vary by 2 m at the bridge. This site was on the edge of the Whakatane River estuary at the time of the Taupo Pumice eruptions (c.1800 years BP). But more recently the river has prograded towards the south 300 metres in the last 100 years. The site comprises medium to coarse pumiceous sands, overlain by approximately one metre of sandy silt. In general the sands are loosely compacted to a depth of six metres below the ground surface, and from this point they are some what denser. Insitu saturated density of the soil averages 1.1 t/m^3 (Jennings and Smith, 1991). The average yearly rainfall for the area is approximately one metre and during the dry summer months, the water table often drops to 1.1 metre below the ground surface. The site is used as pastoral farming land.

The Landing Road Bridge was constructed in 1962 and consists of 13 x 18.3 m non-continuous spans which support two concrete traffic lanes plus two footpaths. The superstructure comprises 5 precast post-tensioned concrete I-beams which are interlinked with linkage bolts through diaphragms over the piers and through the abutment backwalls. The bearings consist of 16 mm rubber pads under the beams which are tied down using holding bolts at all piers and abutments. The substructure is made up of 12 concrete slab piers on 8 x 406 mm square raked prestressed concrete piles. The abutments are supported by 406 mm square raked prestressed concrete piles (2 rows of 5 and 3 piles). The abutment backwall is tight packed and bolted to the beam diaphragm. There are no approach slabs. Five river piers were additionally underpinned with two extra 1.1 m diameter concrete cylinders around 1985 after flooding had undermined one of the piers.

2.1 Observed damage at the Landing Road Bridge site

Modified Mercalli Intensities in the area were estimated at between MMI VII & VIII.

Cracks and sand boils associated with lateral spreading occurred as indicated in Figure 2 and continued in the same manner a few hundreds of metres further downstream of this site. The cracks tended to be parallel with the river's edge at all places except immediately beside the bridge, where they swung around to approximately 45° to the bridge. In total there was about five major cracks sequences, which were in excess of 200 mm wide, in the 300 m beside the true left bank of the river. There was no observed evidence of liquefaction on the true right bank of the river. Particle size distributions of 13 samples of ejecta taken from the left bank (both upstream and downstream) beside the bridge, D_{50}'s from these samples ranged from 0.2 to 0.5 mm. One sample of ejecta obtained from approximately 100 m downstream of SH2 and 100 m on the land side of the stopbank had a D_{50} of 0.1 mm. Some of the ejecta contained pumice particles while others did not contain any pumice. The stopbank contained longitudinal cracks which were in excess of 100 m in length in places. Intensive cracking and subsidence of the north-west approach to the bridge from the abutment to past the intersection of SH2 with Keepa Road. An eye witness report indicates that this approach was passable immediately after the main earthquake, but on returning one hour later it was no longer passable in a vehicle. Gaps between the piers and the soil of up to 600 mm appeared on the river side of the piers on the berm of the left bank of the river. Soil had also piled up behind the piers that were located on the berm of the river. At the time of the earthquake the tide was approaching low tide, and the river level was particularly low due to the preceding dry months.

The bridge superstructure did not undergo any significant distress. Deck joints over the berm on the left bank generally showed tension while the joints over the river piers varied between tension and compression (H. Chapman, pers. comm.). Opening in the joints over the piers may have induced tension in the linkage bolts. Excavation to about one metre, at the northern abutment showed that the front raked piles were cracked on the river side. These cracks extended through 75% of the width of the piles, with there being no indication of cracking on the bridge approach side of the piles. The cracks were repaired with epoxy resin at the time of excavation. Soil at the northern abutment settled 300 to 500 mm exposing the piles. The northern abutment appeared to have rotated (pers. comm. L. McCallin). The southern abutment was not inspected after the earthquake. The tops of the first two piers from the northern abutment were leaning towards the river by about 1°. The rest of the piers appear vertical. Horizontal cracks in piers H & J (which are the first two piers additionally underpinned from the left bank of the river) were not noticed until some years after the earthquake, but were considered to have occurred as a result of the earthquake in 1987. These cracks were repaired with epoxy resin in 1992.

3 COLLECTION AND EVALUATION OF DATA

Soil profiling using a truck mounted piezocone was undertaken across the Rangitaiki Plains in 1993. In total 55 probes were made at 15 sites, ranging from intensive investigation of the lateral spreading sites to investigation of sites with only a single sand boil. In addition 5 rotary borings with Standard Penetration Tests (SPT) were completed. At the Landing Road Bridge site 12 piezocone penetrometers (CPT) and one boring were completed.

Measurement of the extent of horizontal and vertical displacement at the Landing Road Bridge site was attempted using photogrammetry. The photogrammetry used photography from January 1982 (scale 1:26000) and photography flown one week after the earthquake (scale 1:11400). There was difficulty in gaining reliable measurements of the displacements due to the lack of fixed detail within the site.

3.1 Estimation of liquefied strata

Five well known methods were used to determine whether a soil is susceptible to liquefaction. These were then plotted along with the CPT data to both ascertain the viability of the method and to determine which soil liquefied during the earthquake. The methods used were: Shibata and Teparaksa, 1988; Zhou, 1980; Taiping et al., 1984; Law et al., 1990; and Davis and Berrill, 1982. The widely conflicting results of these prediction methods are shown in Figure 4. For these results, when the cone resistance, qc, is less than the predicted critical cone resistance, then this soil layer is deemed to have liquefied.

Due to the lack of consistency obtained by the prediction methods, other factors were introduced to help determine which layers liquefied during the Edgecumbe earthquake. One such important factor is the matching of the Particle Size Distribution (PSD) of the ejected soil with that of the retrieved soils from the SPT tests. Due to the lack of detail as to the location the sand boils from which the PSD of the ejecta were gained, a photograph showing a three layered sand boil was used to try and match ejecta with soil layers from the SPT. From this, the starting point of the liquefaction front was established, and using methods outlined below, the extent of the liquefaction throughout the CPT probe was established. Using this first probe as a guide to the liquefiable soils, the other CPT probes at this site were then analyzed to determine which soils liquefied during the 1987 Edgecumbe earthquake.

Scott and Zuckerman (1973) showed in laboratory tests, that an overlying relatively denser soil layer that would not liquefy under normal loading conditions, can be induced to liquefy from below by the behaviour of the underlying layer. This means that a soil layer that has a cone resistance in excess of the predicted critical cone resistance can liquefy under the correct conditions, if the underlying soil layer is liquefied.

Vreugdenhil (paper this conference) proposes that if a thin dense layer overlies a relatively softer layer then the cone resistance, qc, may be underdeveloped. This means that a thin soil layer under these conditions, will have a higher cone resistance than measured by the CPT test. This higher cone resistance may then exceed the critical cone resistance, as predicted by the five methods above, which would make the soil layer less likely to liquefy.

Combining the results of the estimated liquefied layers, for each of the CPT probes, gave a three dimensional insight into the mechanism of the liquefaction and lateral spreading at this site. These results are displayed in cross-sections AA and BB, Figures 3 and 5.

3.2 Estimation of displacements

Measurements of the settlement due to the liquefaction and lateral spreading was estimated by photogrammetry to be in the order of 400 mm, which was confirmed by the settlement measured at the northern abutment (300-500 mm) shortly after the earthquake in 1987.

Estimation of the horizontal displacements was not able to be accomplished with any accuracy using photogrammetry. But using a crude technique of adding estimated crack widths, an estimation of the horizontal displacement was made at 1.5 to 2.0 m. Using Hamada's method outlined in Bartlett and Youd (1992), gave a estimate of the horizontal displacement of between 1.2 and 2.0 m for the soils in Figure 5, soils in Figure 3 have both a smaller angle on which to move and movement is some what restrained by the bridge structure.

3.3 Liquefaction model for Landing Road Bridge

Soils that are estimated to have liquefied during the 1987 Edgecumbe earthquake are shown in Sections AA and BB, Figures 3 and 5 respectively.

Section BB liquefied soils have a greater angle of the bottom of the liquefied layer and hence gravity will induce larger displacements than that of Section AA. The bottom of the liquefied layer rises in Section BB close to the river, which would have reduced the overall angle of the bottom of the liquefied layer, and hence reduce the maximum possible horizontal displacement (some residual shear strength is assumed in the liquefied soil). There is still a net downwards

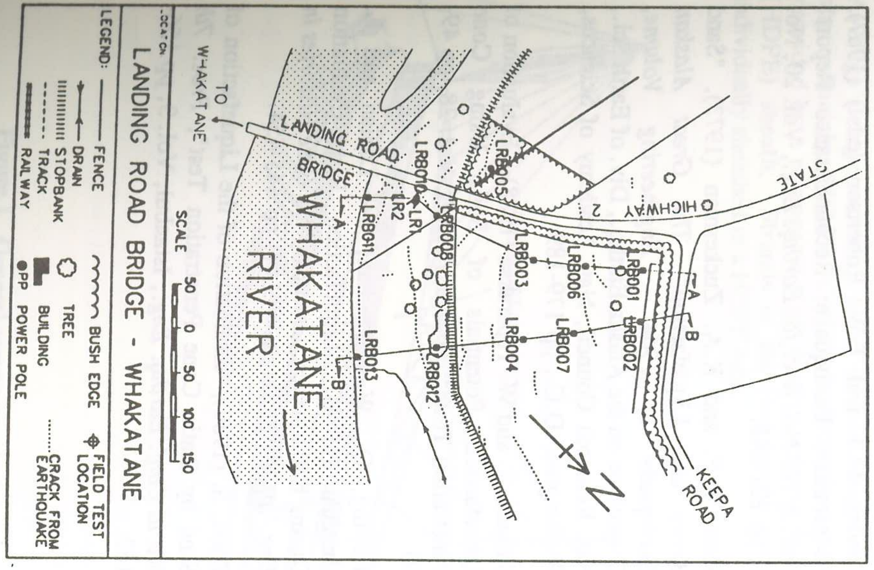

Figure 2: Map showing detail of Landing Road Bridge site.

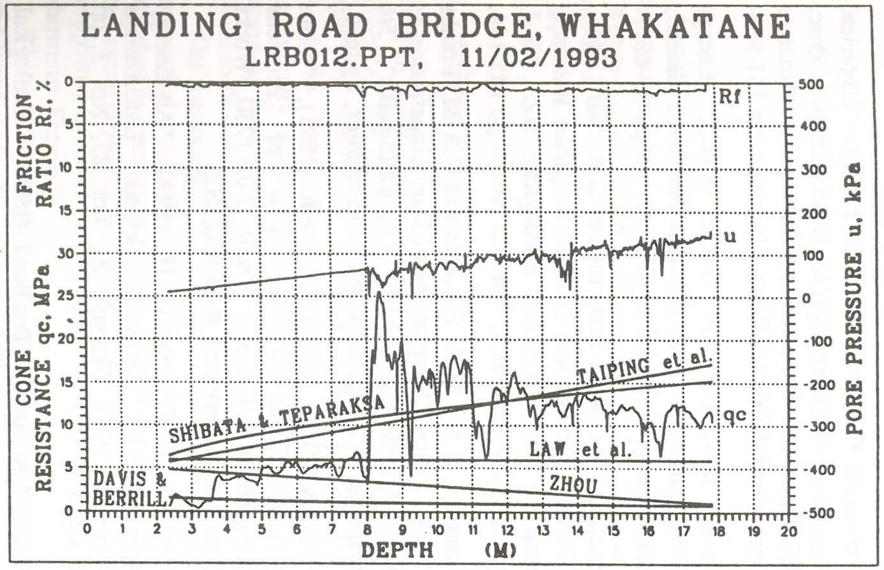

Figure 4: Typical data from CPT with prediction methods superimposed.

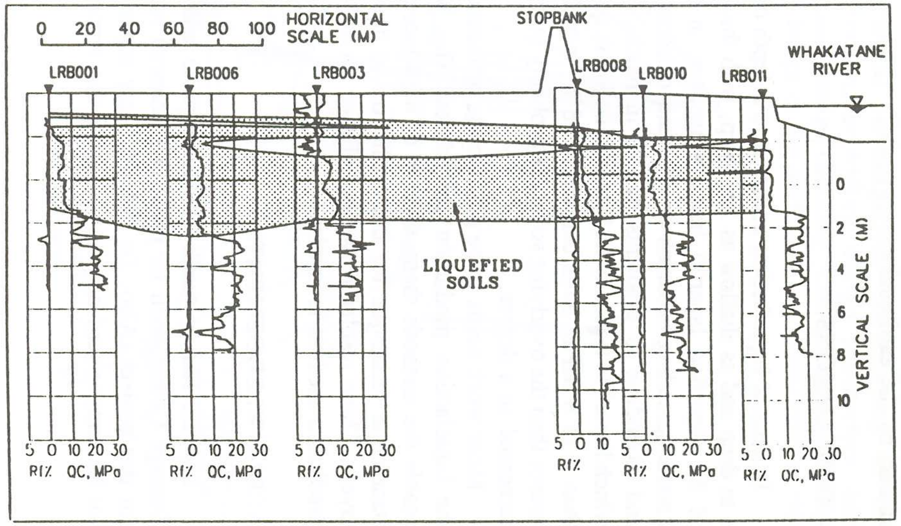

Figure 3: Section AA at Landing Road Bridge site showing CPT data and estimated liquefied soils.

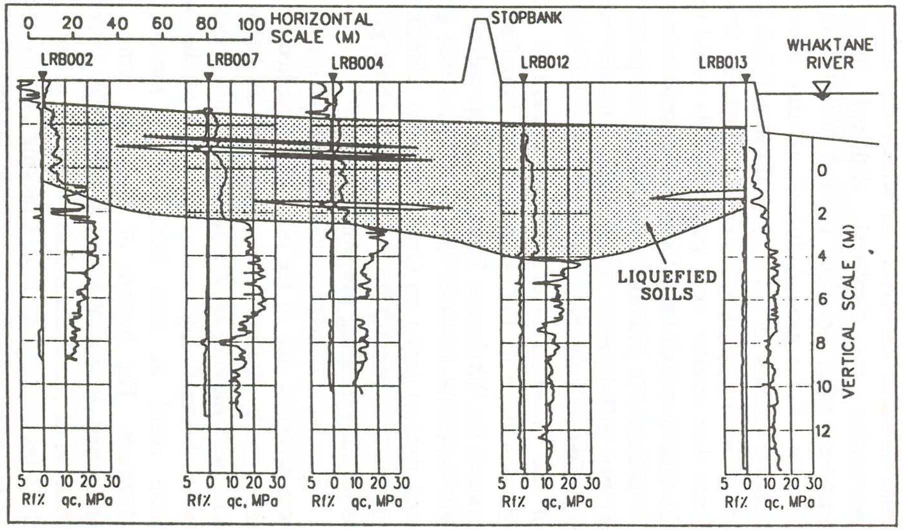

Figure 5: Section BB at Landing Road Bridge site showing CPT data and estimated liquefied soils.

slope towards the river on which the unliquefied overburden was displaced. Although there is a relatively thick liquefied layer(s) (about 5 m), the severity of the ground distortions may have been reduced because of the large grain size (0.3-0.5 mm) which would give rise to faster settling and drainage times than smaller sized particles, this would reduce the time in which lateral spreading could take place.

Section AA provides a more complex picture due to the influence of the bridge structure on the liquefied soils and because of a complex bottom slope of the liquefied layer. Although there is not an obvious slope of the bottom of the liquefied layer towards the Whakatane River, lateral spreading would still be driven towards the river due to the greater total stresses acting on vertical sections beneath solid ground than beneath the river, which would give rise to lateral spreading towards the river.

Structural damage to the northern abutment piles was probably due to the piles being pushed towards the river by the unliquefied soil and fill behind the abutment moving on the liquefied soil. The bridge beams being firmly connected to one another would have acted as strut braced from the southern abutment. The bottom of the piles being firmly embedded in soil that did not liquefy, forced the rotation of the abutment and cracking in the tops of the piles on the front face. Cracking on only one side of the piles indicates monotonic loading caused by lateral spreading of the piles and not cyclic loading caused by the earthquake.

4 CONCLUSIONS

In general liquefaction occurred to a depth of 6 to 8 m deep and as shallow as 1 m deep, with thickness of the liquefied layer(s) being 4 to 5 m in total. Lateral spreading occurred on gradients of about 1%, and extending to gradients of 2% in places. Soils which liquefied typically had cone resistances, qc, less than 7 to 8 MPa. Soils below 6 to 8 m, which are denser than the overlying soils, are probably older and cemented to a degree.

More work needs to be put into the refinement of the liquefaction prediction procedures. This would enable the methods themselves to be used more as a standalone technique for the prediction of liquefied layers. These results show that reliance on only one prediction method is unwise.

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