

The Effect of Lateral Spreading on the Landing Road Bridge in the 1987 Edgecumbe, New Zealand Earthquake

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ABSTRACT

The effects of lateral spreading on a 13-span prestressed and reinforced concrete highway bridge are described. Riverwards movement of a 1 to 2 m thick crust of clayey silt, overlying a 4 m thick liquefied stratum, is prevented by the buried piers and pilecaps of the four northern foundations. Passive failure occurred in this unliquefied crust. Estimates of passive pressure and pile group capacity enable a rough upper bound to be placed on the drag force between the liquefied soil and the 16 inch square precast driven piles.

INTRODUCTION

Lateral spreading of riverbanks probably poses as great a threat to bridges during earthquakes as inertia loads from the superstructure, yet the process of lateral spreading and the loads it can impose on foundations are not fully understood. The aim of this paper is to describe the behaviour of the 13-span Landing Road Bridge, which suffered minor damage due to about 1.5 m of lateral spreading in the 1987 Edgecumbe earthquake, and to attempt to place bounds on the loads generated on its foundations by the liquefied soil.

The M_L 6.3 rupture of 2 March 1987 occurred on a normal fault beneath the Rangitaiki Plains near the north eastern end of the Taupo Volcanic Zone, associated with the boundary between the Pacific and Indian plates, in the North Island of New Zealand. Liquefaction occurred throughout the Plains in 1987 (Figure 1) generally near present or recent former river and stream channels (Franks *et al.*, 1989). At least six bridges ranging in size from a single-span farm bridge to the 240 m long Landing Road Bridge, suffered, mostly minor, damage from convergence of river banks due to lateral spreading.

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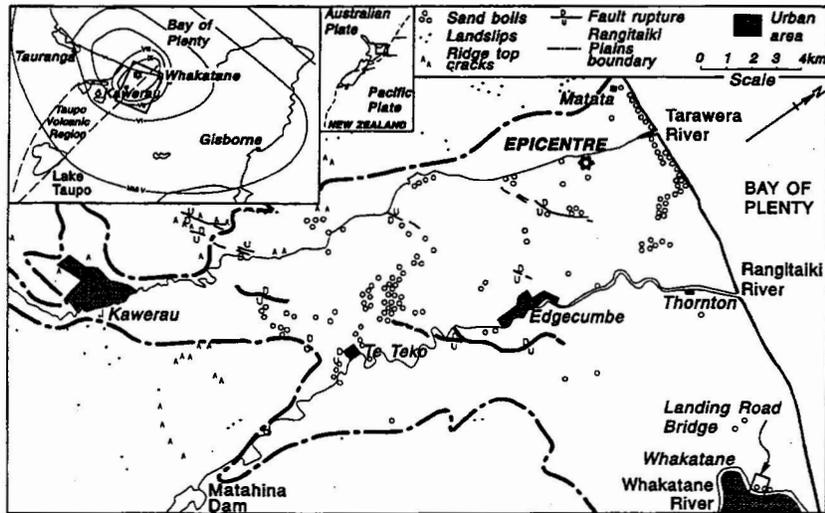


Figure 1. Map of the Rangitaiki Plains showing the ground damage caused by the 1987 Edgcumbe earthquake. Large inset shows Bay of Plenty region and isoseismals from the earthquake.

Figure 1 shows the epicentre near the mouth of the Tarawera River, 17 km from the Landing Road site. Seismological studies suggest that rupture propagated up and to the south from a hypocentre at a depth of 8 km, to break the ground surface on a pre-existing south-westerly striking fault running from about 1 km east of the town of Edgcumbe towards the town of Kawerau. The closest point on the surface rupture is 8 km from the site. There was no strong motion accelerograph in Whakatane; the nearest was at the (damaged) Matahina Dam, due south of the epicentre and also about 8 km from the nearest surface breakage. The instrument at the base of the 80 m high rockfill dam recorded a 0.33 g peak acceleration. Due to rupture directivity effects, the motion at the bridge site was probably weaker than this. Lowry assigned an intensity of MM VIII to Whakatane (Christensen, 1994).

THE LANDING ROAD BRIDGE AND SITE

The Landing Road Bridge, shown in Figure 2, carries State Highway 2 over the Whakatane River. The bridge was constructed in 1962 and is of a standard design used widely in New Zealand. It comprises 13 simply supported spans 18.3 m long, carrying a two-lane concrete deck plus two footpaths. The superstructure consists of 5 precast post-tensioned concrete I-beams, interlinked with linkage bolts through diaphragms over the piers and through the abutment backwalls, and bearing on 16 mm rubber pads. The beams are tied down using holding bolts at all piers and abutments forming a quite stiff, monolithic structure. The substructure comprises concrete slab piers supported by eight 406 mm square raked prestressed concrete piles. The abutments are also supported by 406 mm square raked piles, 5 piles on the river side and 3 on the approach side. 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 1.1 m diameter concrete cylinders each, around 1985 after one pier had been undermined by flooding. The underpinned piers (piers H to M) can be seen in Figure 3.



Figure 2. The Landing Road Bridge, looking north east to the mouth of the Whakatane River.

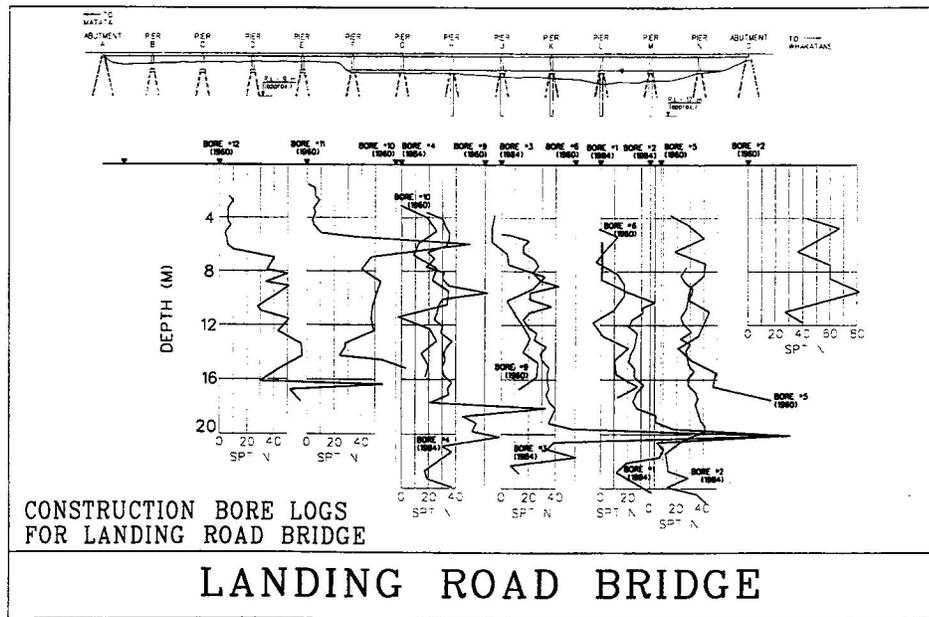


Figure 3. Section along the centre-line of the Landing Road Bridge showing the SPT logs which were used for the bridge design and redesign. Abutment A is at the north end of the bridge.

During the past 100 years the river has prograded about 100 m towards the south, leaving behind loose sands and silts, in a classical point-bar structure, to a depth of about 6 m; it is these soils which liquefied. The bridge piles were driven into denser sands below that depth. These older and denser sands are also found extending to the ground surface at the south abutment. Gross features of the soil profile are seen in the SPT logs reproduced in Figure 3.

DAMAGE OBSERVED AFTER THE 1987 EARTHQUAKE

Extensive liquefaction, marked by lateral spreading and ejection of sand, occurred around the bridge and for a few hundred metres downstream on the northern (true left) bank of the river. Cracks associated with the lateral spreading show clearly in Figure 4. The cracks tended to be parallel to the river channel except immediately beside the bridge where they swung around to approximately 45° to the bridge centreline. Near the bridge, there were about five major crack sequences (Figure 5) which were in excess of 200 mm wide in a strip extending back about 300 m wide from 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 upstream and downstream beside the bridge had D_{50} 's ranging from 0.2 to 0.5 mm. Some of the ejecta contained pumice particles, while others did not. The stopbank, seen in Figure 4 running through the bridge abutment, had longitudinal cracks which were in excess of 100 m in length in places. Intensive cracking and subsidence of the north-west road embankment ran from the bridge abutment to past the intersection of SH2 with Keepa Road. An eye witness states that this approach was passable immediately after the main earthquake, but one hour later it was no longer passable in a vehicle, suggesting that lateral spreading, here at least, took place under static conditions in persistently liquefied soil, after the shaking had stopped. Gaps between piers and soil of up to 600 mm appeared on the river side of the piers on the berm of the left bank of the river; on the landward side, soil piled up behind the piers.

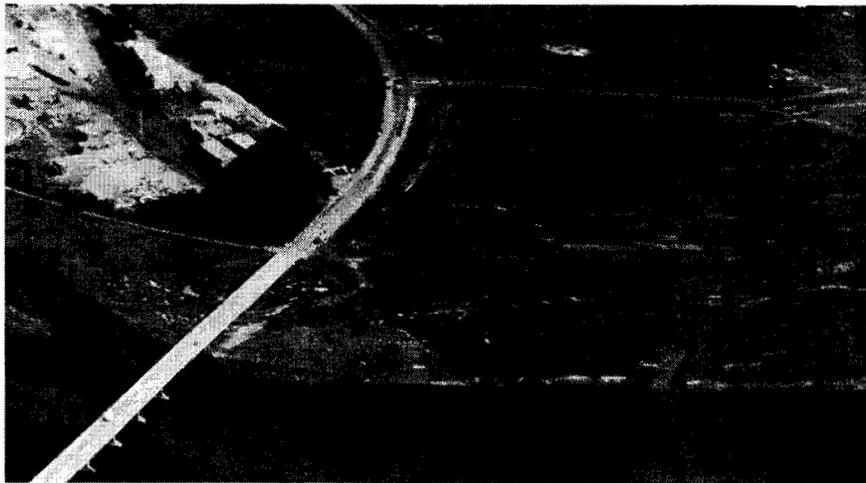


Figure 4. Aerial view looking towards the north-west of the Landing Road Bridge site. The Landing Road Bridge can be seen in the left foreground. Ground cracking can be seen on the berm beside the bridge and in the fields beside State Highway 2.

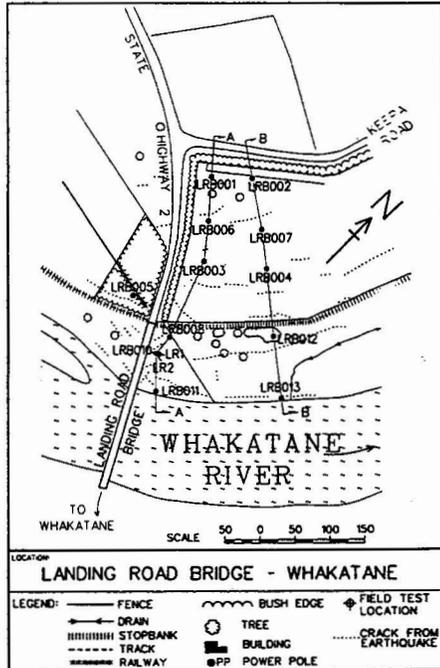


Figure 5. Map of the Landing Road Bridge site showing lateral spreading cracks and the location of *in situ* tests and cross section ties.



Figure 6. Front raked pile of the western abutment of Landing Road Bridge showing settlement of the ground. Small cracks can be seen through the front of the pile face (courtesy of H Chapman).

The bridge superstructure did not undergo any significant distress. Compression was indicated in one instance by the buckling of a pair of concrete footpath slabs. Deck joints over the berm on the left bank generally showed tension, while the joints over the river piers varied between tension and compression (pers. comm. H Chapman). 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 (Figure 6). There was no indication of cracking on the bridge 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. At the time of the earthquake this abutment was thought to have rotated (pers. comm. L McCallin). This observation was supported in 1994 with the measurement of $\frac{1}{2}^\circ$ rotation of the bottom of the abutment face towards the river. The south-eastern abutment was not inspected after the earthquake. In 1992 the authors found that the tops of the first two piers from the northern abutment were leaning towards the river by about 1° , while the remaining piers appeared vertical. Horizontal cracks near the base of piers H and J, (which are the first two piers additionally underpinned from the left bank of the river), were not noticed by the highway authority until some years after the earthquake but were considered to have occurred as a result of it.

SITE INVESTIGATIONS

In 1993, five cone penetration tests (CPT) and eight piezocone (CPTU) probes and two rotary borings were carried out at the locations shown in Figure 5 at the Landing Road Bridge site. The log of piezocone probe LRB010 is shown in Figure 7 overlain with liquefaction criteria from several authors. Clearly these prediction procedures, which for the near-field condition of this site at least are highly inconsistent, are of little help in determining which layer liquefied. Another approach is to compare the particle size distribution (PSD) of ejected soil with that of samples recovered from the borings. However, here this method was not reliable since a precise record was not kept of exactly where samples were taken in 1987. In the event, the most useful evidence came from the comparison of grain sizes in a close up photograph of a three-layered sand boil taken in 1987 near pier C with samples retrieved from a rotary boring at the same location. This established that the loose clean sand strata shown shaded in Figure 8, with q_c up to about 8 MPa, liquefied in the earthquake.

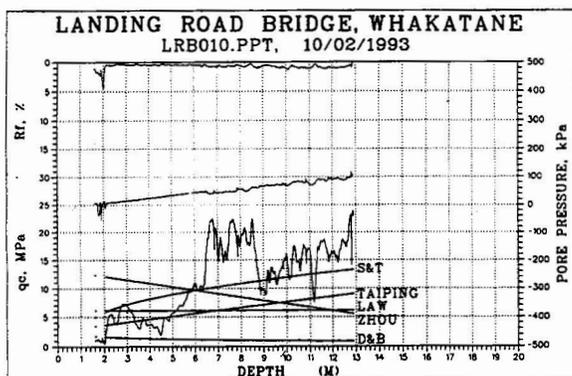


Figure 7. LRB010.PPT graph for 2 March 1987 Edgecumbe Earthquake.

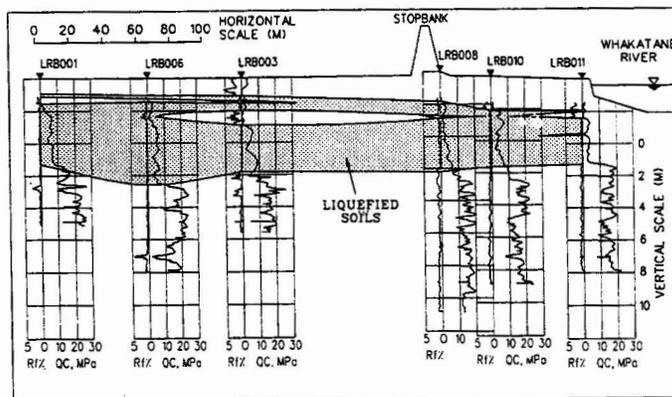


Figure 8. Cross Section AA at Landing Road Bridge showing the estimated liquefied strata. Cross Section BB is generally similar, with the loose sand layer slightly thicker.

Settlements due to the liquefaction and lateral spreading were estimated by photogrammetry to be of the order of 400 mm, which is consistent with the settlement measured at the northern abutment (300-500 mm) shortly after the earthquake in 1987. Estimation of the horizontal displacements was also attempted using photogrammetry but could not be accomplished with any accuracy due partly to the lack of good markers. Using a rough estimate of crack widths, seen in a number of terrestrial photographs from 1987, a horizontal displacement of 1.5 to 2.0 m is estimated. Liquefaction-induced settlements for probe sites LRB010.PPT and LRB012.PPT were estimated using the procedure of Ishihara (1993) at 170 and 310 mm respectively. These are of the same general order as the average value of 400 mm measured photogrammetrically.

BEHAVIOUR OF THE STRUCTURE

The observations described above indicate that the 1 to 2 m thick crust of partially-saturated silty material was carried towards the river as the underlying sands liquefied and spread, and that passive failures occurred in this silt crust where its progress was impeded by the four northernmost buried piers. A rough calculation of the passive force, based on the measured $q_c \approx 500$ kPa in the silt, gives a value of the order of 0.5 to 1 MN per pier. To transfer this force from the underlying liquefied stratum requires a shear stress of about 2 kPa, which surely could be sustained, even in the liquefied soil.

At first, we speculated that the passive load applied to the northern abutment and the four partially buried piers on the northern berm was transmitted to the superstructure through the stiff pier and deck system and thence on to the strengthened piers H to M, with some force possibly reaching the south abutment. However, the capacity of the pier-to-girder connection and of the cracked section of piers H and J is such that only about 100 kN could be transferred from pier to superstructure, for less than the 0.5 to 1 MN estimated above.

Then, given the observation that the piers remained near vertical after the earthquake, we are led to the conclusion that any load on the buried section of pier, and on the piles themselves, must have been carried mostly by the pile group itself, and that a collapse mechanism had not formed in the pile group. Some compressive load, sufficient to yield piers H and J, must have found its way into the superstructure; but the greater part of the soil loads must have been dispersed through the pile system itself.

Simple checks suggest that the system of 8 raked piles could not carry a passive load of the order of 1 MN without some damage. Specifications from the early 1960's call for pretensioning to 750 psi, reduced to 600 psi (4.14 MPa) with creep and shrinkage, for these standard piles. An elastic analysis of the group suggests first tensile cracking of the piles would occur under either a passive load of about 300 kN applied at the pile cap assuming no lateral load on the piles themselves, or with a uniformly distributed load of 50 kPa on each pile assuming no passive load at the pilecap. Since the displacement in the unliquefied crust of the order of 1 M should be sufficient to fully mobilize passive pressures against the buried section of pier, it is likely that cracking occurred at least in the landward row of piles, at their inner surface below the cap.

Further rough calculations of the moment capacity of the 16 inch pretensioned piles suggests that a collapse mechanism would form under a point load at the pilecap of about 600 kN. This in turn implies that the drag force from liquefied soil must be small if collapse is to be avoided. For example, if the crust exerts 0.5 MN at the top of the pilecap, the drag pressure is limited to about 30 kPa. This is a very rough estimate, coming from the difference of two already imprecise large numbers. Clearly,

we should return to the site to obtain a more precise definition of geometry and better estimates of soil properties, as well as excavating at least the tops of the piles.

CONCLUSIONS

1. The Landing Road Bridge, a robust, standard design from the 1950 and 60s, survived lateral spreading of about 1.5 m with only minor damage.
2. Hamada's method predicts this magnitude of spreading and Ishahara's procedure predicts the magnitude of settlement well.
3. The greater stiffness of the strengthened piers H through M attracted greater loads than their neighbours, causing piers H and J to yield.
4. Passive failure behind piers B to E in the 1 to 2 m thick unliquefied crust of the north berm as it was carried towards the river on underlying liquefied soil would have imposed loads of the order of 0.5 to 1 MPa on the buried, solid section of the piers. These loads are sufficient to cause cracking of the inland piles of each group, but are not great enough to form a collapse mechanism. That the piers remained essentially vertical confirms that the pile system did not fail completely.
5. Assuming a passive force of 0.5 MN per pier, a very rough upper bound of 30 kPa can be placed on the drag force exerted on the piles by the liquefied soil.

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