EDGECOMBE EARTHQUAKE: RECONNAISSANCE REPORT

edited by:

M. J. Pender and T. W. Robertson*

SUMMARY

On March 2 1987, at 01h 42m 34s UT an earthquake of magnitude (ML) 6.3 occurred near 37.91°S, 176.79°E close to the town of Edgecumbe in the North Island, New Zealand. The depth is provisionally estimated to be 12 ± 1 km. Seismic activity in the general area during the previous week culminated in a foreshock on March 2 of ML 5.2 at 01h 35m 37s. Four aftershocks with magnitudes in excess of 5.0 occurred on March 2 at 01h 51m 08s (ML 5.6), 02h 07m 23s (ML 5.1), 06h 56m 32s (ML 5.2) and 07h 55m 09s (ML 5.2).

The earthquakes occurred at the end of summer after a long period of dry weather.

Modified Mercalli Intensities of MM IX have been reported in and around Edgecumbe, with possible instances of MM X. Strong motion accelerographs recorded peak ground acceleration of up to 0.33 g within 15 km of the epicentre. The main shock produced a complex series of surface scarps, the longest being about 7 km long striking SW from Edgecumbe. About 1.3 m maximum extension occurred across the scarp with the area to the north-west being downthrown by about a maximum of 1.5 m which continued to subside slowly. Other smaller normal fault traces have also been detected as well as compressional rolls.

There was extensive evidence of level ground liquefaction and lateral spreading near rivers. Both these phenomena produced eruption of sands at the ground surface. Some wells were observed to have increased flows or increased pressures whilst others were had decreased flows.

General regional subsidence of the alluvial plains in the area up to 2m has been confirmed by levelling completed within three weeks of the earthquake.

Structural damage was confined to the alluvial plains in which the town of Edgecumbe is centred. The depth of sediments on the plains is not less than 350 m.

There was extensive minor damage to roads. Severe damage to many houses and other single storey structures. A dairy factory complex in Edgecumbe, two paper mills in Kawerau and a paperboard mill in Whakatane all sustained damage, in some cases considerable. At present information on the damage in the paper mills is not available.

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FORWARD

Eyewitness statements extracted from newspaper reports:

Whakatane Beacon, Tuesday March 3rd.

"It was the most horrifying frightening experience I have ever had in my life. I thought it was never going to stop. It just went on and on - I felt trapped and I didn't know which way to run. The ground just rolled and rolled."
Mrs. P. Bugden

"I reached a flight of concrete stairs and at that point I felt as if someone just picked me up and chucked me down those stairs. I had no control. I found myself wrapped around a pole and people were trying to get me out."
Mr. M. Morgan.

In Whakatane, eyewitnesses said "The ground was rolling like the sea."

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The Daily Post, Rotorua, Tuesday March 3rd.

"I was driving out of my driveway when one big quake struck and the car was bouncing. I had to stop it. I couldn't go any further."
Mrs. P. Menary.

....

New Zealand Herald, Wednesday March 4th.

Mrs. C. Bakker was in her home, situated on the high ground behind Awakeri and overlooking the Rangitaiki plains, when the earthquake struck.

"I was in there when it happened", she said. "There was a rumbling that was coming from underneath that was terrible. Everything was falling down. Gaps were opening up in the walls and even the toilet started disappearing. It was just like somebody was sucking it down through the floor. The windows were pinging and you should have heard the noise when the bricks from the outside wall started to fall", Mrs. Bakker said.

"All the doors started jamming behind me. I tried to get out the laundry door but I couldn't. In the end I just clawed like an animal at the door surround to get out the back door. I could not walk. I was like a drunk platypus. I really thought that 'that's it for me'."

When she got herself outside, things were no better.

"I couldn't stand up so I ended up sitting and wedging myself up against the door. All I could hear was this incredible rumble."

Chasms 30 - 40cm. wide were opening all around the back lawn. Mrs. Bakkers car, a Ford Sierra, was carried in fits and starts down the slope away from her.

"The gaps were slamming shut like someone clapping their hands", she said. "I was beside myself. I thought I should get into the car but the car was moving away."

1 BACKGROUND

1.1 GEOGRAPHY (M. J. Pender)

A map of the Eastern Bay of Plenty region affected by the earthquake is given in Fig. 1.1. Three rivers flow across the Rangitaiki plains: the Tarawera in the west, the Rangitaiki which flows across the middle of the plains and through the town of Edgecumbe, and the Whakatane in the east which marks the western boundary of the town of Whakatane.

The major centres of population in the area are: Whakatane(12800)[1], Kawerau(8311), Edgecumbe(1825) and Te Teko(572). These towns service the surrounding dairy farming and fruit growing industries. There are about 1000 dwellings on the plains not included in the above figures for the towns. Industrial complexes are located at Kawerau, two paper mills, Whakatane, a paperboard mill, and Edgecumbe, a large dairy factory and associated works.

1.2 GEOLOGY (B. W. Riddolls)

1.2.1 General

The affected area is on the eastern side of the Taupo Volcanic Zone. Hill country in the south and west mainly comprises a range of late Quaternary rhyolitic volcanics, and Mesozoic greywacke in the east. Quaternary alluvial deposits underline the Rangitaiki Plains. Late Quaternary tephras are widespread, Healy et al (1964).

The overall structure is graben-like, i.e. normal faulting. Late Quaternary fault traces have been recorded previously in rhyolitic terrain.

1.2.2 Rangitaiki Plains

There is little published information on the nature and thickness of the deposits underlying the Rangitaiki Plains in the vicinity of Edgecumbe. Logs of groundwater bores, retained by the Bay of Plenty Catchment Commission, provide the most objective indication of geological conditions. They suggest considerable lithological variability both vertically and laterally. The deepest bores range from 73 m to 319 m, and indicate alternating sequences of mainly pumice-derived alluvial sand and gravel with interbedded greywacke gravel, tephra, and marine silt and sand. Siltstone and welded ignimbrite have been recorded in some bores. Greywacke basement was not intercepted.

Most of the deposits underlying the Rangitaiki Plains are saturated, with the water table ranging from near surface in the coastal margin, to about 3 m below ground level in the Te Teko area.

Recently the Geophysics Division of the NZ Department of Scientific and Industrial Research conducted a seismic reflection survey in the Rangitaiki plains on behalf of MWD, Woodward (1985).

This was undertaken to provide information about the depth of the Matahina Ignimbrite beneath the plains, as this was known to be a productive source of groundwater for irrigation.

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[1] Population figures from the March 1986 Census

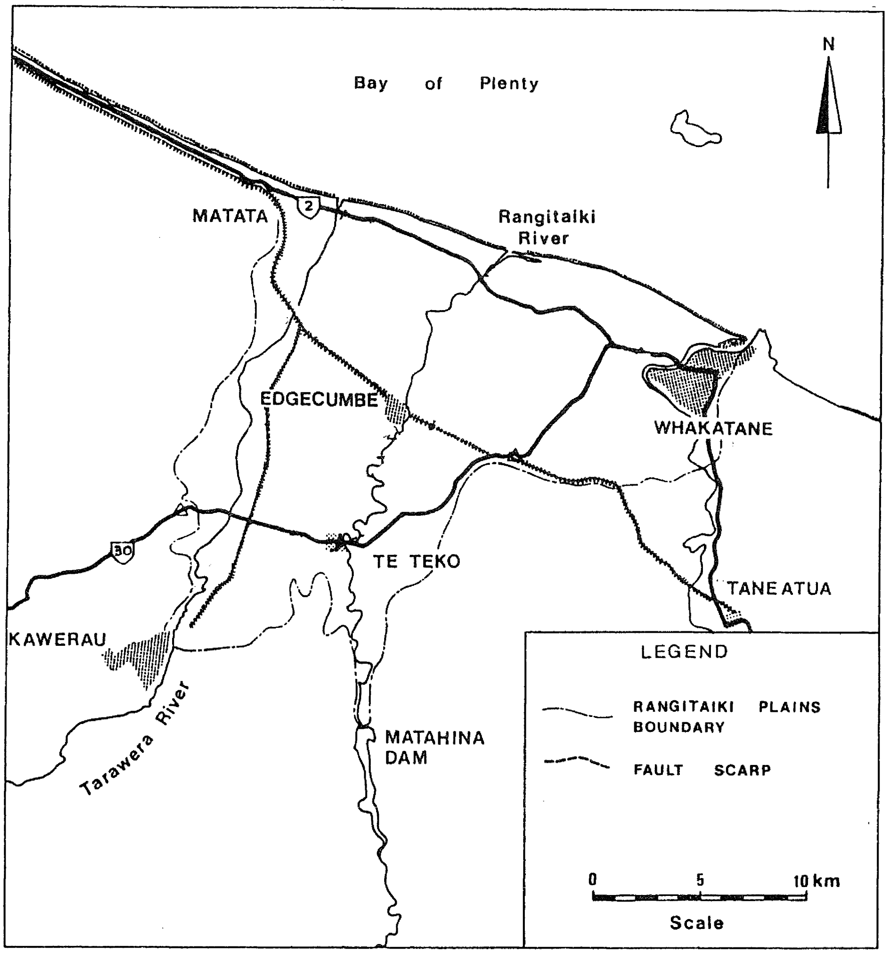


Fig. 1.1 : Map of the Rangitaiki Plans

Seven lines were surveyed across the block of land between Edgecumbe and Te Teko in the north south direction, and between Awakeri and the Tarawera River in the east west direction.

Along the Tarawera River the ignimbrite is about 300m beneath the surface. Along McCracken Rd. between the Rangitaiki River and Awakeri Springs depths of 300 to 360m were obtained. Along a line running parallel to, and a little south of Otkiri Rd., in an easterly direction from the Tarawera River there is a gradual increase in the depth to ignimbrite from 300m at the river to 500m where the line crosses the extension of Omeheu Rd. A north south line which crossed Otakiri Rd. a short distance west of Edgecumbe and ran along Poplar Lane indicated that the ignimbrite was locally rather deeper, but the interpretation of the results from this line is a source of some uncertainty.

In general these geophysical indications of the depth to the Ignimbrite were confirmed by the logs of deep boreholes.

1.3 SEISMICITY (E. G. C. Smith)

The Bay of Plenty is part of the Central Volcanic Region (CVR), which encompasses Lake Taupo and the volcanoes to the south of Taupo, the corridor through Rotorua and out to and beyond the coast across the Rangitaiki Plains. Prominent areas of seismic activity include the western side of Lake Taupo and the Bay of Plenty coast, while the area around Mts Ruapehu and Ngauruhoe has been relatively inactive. Most earthquakes in the CVR for which a reliable depth has been calculated have been shallower than 12 km. In the last 20 years, swarms containing at least four earthquakes of magnitude four or greater have occurred at an average rate of about one every two years, and have accounted for about half the earthquake occurrence in the CVR in this time. The largest known events in the CVR occurred during the swarm of earthquakes near Taupo in 1922 and probably reached magnitude six. There have been substantial changes in the rate of occurrence between periods of about a decade during which it was constant. Fluctuations in the rate of occurrence do not correlate with episodes of eruptive volcanism.

Although there is clear geological and geodetic evidence that the Taupo Volcanic Zone is widening, a dominent strike slip component in the focal mechanisms shows that the zone is not deforming by simple rifting. Surface deformations that accompanied swarms in 1922 and 1983 were very largely normal faulting and subsidence, but composite strike-slip mechanisms for events of the 1983 swarms suggest that weak materials at the surface mask transcurrent deformation at depth.

The mixture of dextral and sinistral faulting inferred from the focal mechanisms shows that there are either local reversals of the regional stress field, or that the zone widens via a complex system of predominantly dextral faults in the west, sinistral faults in the east, and a mixture of these and normal faults in the middle. Further information is given by Smith and Webb (1986).

Fig. 1.2 gives the epicentres in the CVR for shallow earthquakes, magnitude 4.5 and greater in the period 1940-1983.

1.4 SOIL PROFILES (D. N. Jennings & M.J. Pender)

Soil profile information from a number of sites across the plains has been examined in preparing this section of the reconnaissance report. Data from investigations for bridge sites on the state highways and from industrial complexes have been synthesised to provide the following summary soil profile given below. The profile is not specific to any one site, rather it is intended to give an impression of the general soil conditions. At any particular site the depth of the various boundaries may differ from those shown.

1.4.1 Water table

Direct information about water table depths at the time of the earthquake is not available. The earthquake occurred at the end of the summer and after a long period of dry weather. Near the rivers the water table depth would have been controlled by river level. In Edgecumbe the level of the Rangitaiki river was several metres below the surrounding ground at the time of the earthquake. Similarly the Whakatane river was some metres below adjacent ground level.

1.4.2 Soil Profile

A schematic log of the upper part of the soil profile is presented in Fig. 1.3. It appears that there are three distinct layers overlying marine sediments which appear at a depth of 14 - 15m. The marine sediments have very high Standard Penetration Test results in places.

The top of the soil profile is a layer of about 3m thickness which is very loose, having SPT N-values in the range 2 - 10. At the time of the earthquake the water table was probably towards the bottom of this layer over much of the plains.

This is underlain by a layer of medium dense sand with SPT values in the range 10 - 42. This layer is about 4 to 4.5 m in thickness.

The third layer in the sequence is about 7 to 7.5 m in thickness and consists of a complex sequence of loose sand layers interbedded with silt and peat layers.

The logs that are available from the deep well drilling do not provide as much detail as the enginering logs from which the above synthesis was prepared, but they do confirm that the soil profile continues to be complex right down to the underlying ignimbrite.

Examples of the soil profiles from which the idealised picture in Fig. 1.3 was derived are given in Figs. 1.4 and 1.5. In Fig. 1.4 logs and SPT profiles are given for three bores at the Edgecumbe substation (provided by MWD with permission of Electricorp). Figure 1.5 has similar information for one bore and one CPT sounding at the site of the Te Teko school (provided by MWD with permission of Hamilton Education Board.

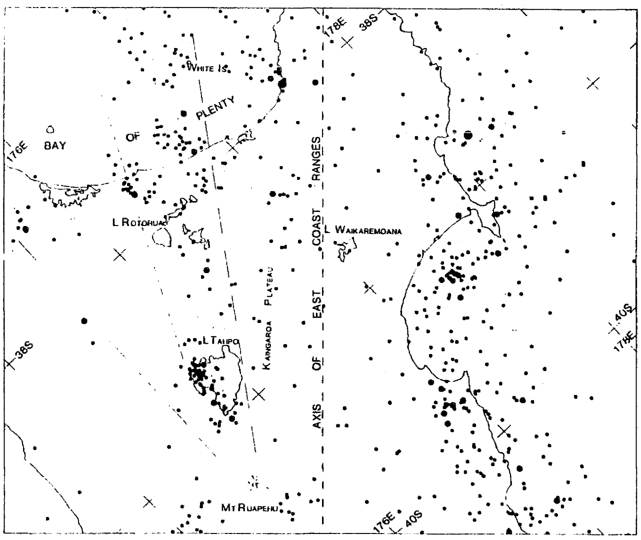


Fig. 1.2 : Epicentres of shallow earthquakes in the CVR , magnitude 4.0 and greater, 1964-1983.

0 ____________________

Loose sand

N 2 to 10

3m ____________________

Medium dense to dense sand

N 10 to 42

7m ____________________

Interbedded sand, silt and peat

N 2 to > 50

14m ____________________

Marine sediments

N up 100

Fig. 1.3 : Simplified soil profile for the Rangitaiki Plains

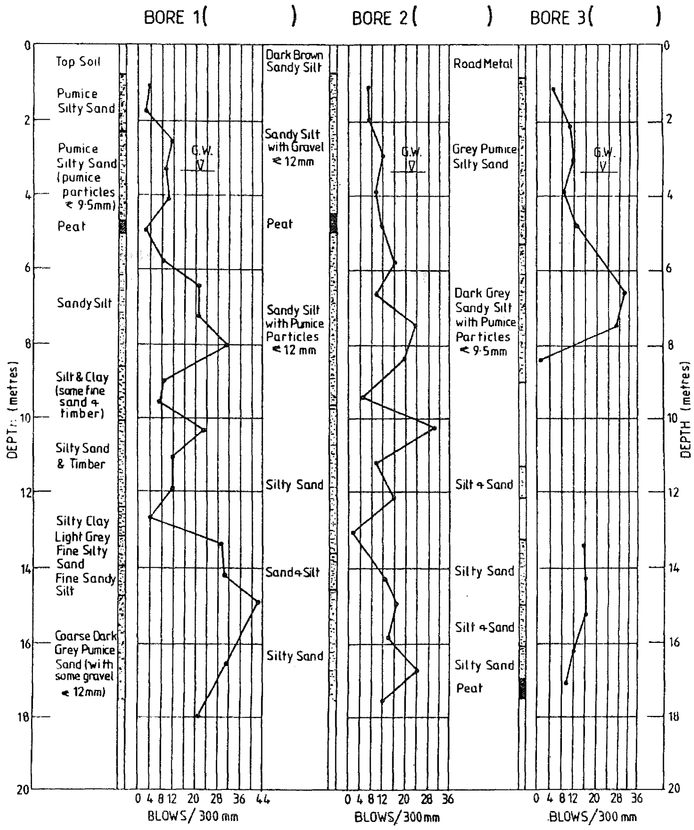


Fig. 1.4 : The soil profile at the Edgecumbe Substation

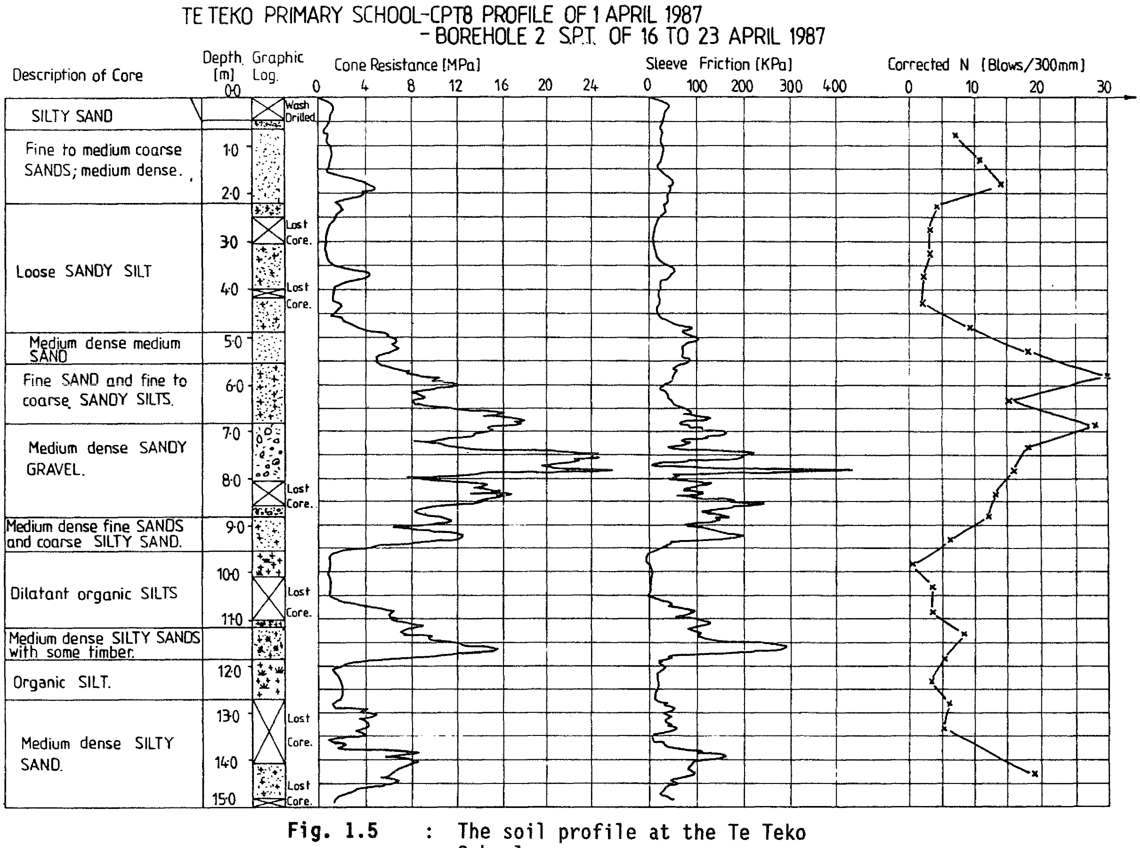


Fig. 1.5 : The soil profile at the Te Teko School

2 THE EARTHQUAKES

2.1 SEISMOLOGY (E. G. C. Smith)

2.1.1 Main Shock and Principal Foreshocks and Aftershocks

The sequence of the earthquakes is given in Appendix A. The main shock occurred at 01h 42m 34s on 2 March 1987 UT. The preliminary epicentre is 37.91^{o}S, 176.79^{o}E. Formal epicentral uncertainty is $\pm$ 5 km. The positions of epicentres of the main shock and several of the aftershocks are given in Fig. 2.1.

Rupture initiated within a few kilometres of the surface and broke through to the surface. Various magnitude estimates are in good agreement: ML 6.3, MS 6.6, Mb (Australia) 6.4. The main shock was accompanied by five others with magnitudes over 5.0.

The chronology of these principal shocks is as follows:

1987 March 02 UT	Magnitude (M_L ± 0.3	Epicentre (± 5 km)
0135 37s	5.2	37.93°S 176.85°E
0142 34s	6.3	37.91°S 176.79°E
0151 07s	5.6	37.89°S 176.76°E
0207 21s	5.1	not determined
0656 31s	5.2	38.11°S 176.63°E
0755 08s	5.2	37.90°S 176.89°E

Cursory inspection of a limited number of local and overseas records suggests that the focal mechanism of the main shock was predominantly that of normal faulting, and that of the largest aftershock (0151) had a similar mechanism. This is consistent with gross features of earth deformation which indicated a NE striking fault downthrown 1 to 1.5 m to the NW.

From ML and MS one can infer fault parameters as follows:

$$\log M_0 = 1.5\ Ms + 9.1$$

thus $M_0 \sim 10^{19}$ Nm

$$M_L = \log u + 6.4$$

thus u ~ 0.4 to 1.6 m (average fault slip)

thus the fault area is about 400 km²,
i.e. about 20 km square

The acceleration spectrum from the base of the Matahina dam indicates a corner frequency of 0.12 Hz or less, implying a source radius of about 10 km, in good agreement with the above.

These figures are included only to give the order of magnitude of these parameters and are to be regarded as very preliminary until confirmed by direct measurement of the seismic moment.

Note that the moment of the principal aftershock would have been about a tenth that of the main shock.

2.1.2 Microseismicity

Preliminary determinations of epicentres of a few early smaller aftershocks suggests activity distributed about the rupture surface with a secondary cluster of epicentres near Kawerau. This has persisted with time while the rupture surface activity has decayed normally. These are almost certainly aftershocks of the M_L 5.2 event at 0656 listed in Section 2.1.1.

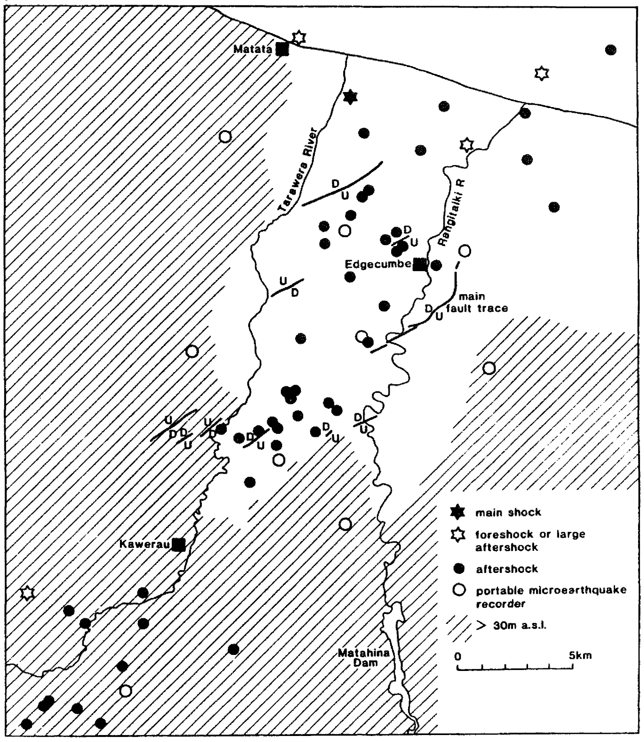


Fig. 2.1 : Epicentres of the main shock and aftershocks

2.1.3 Early Foreshocks

In the week preceding the main shock, earthquakes were felt in the western Bay of Plenty at Te Puke, Tauranga and other places. Separate activity commenced in the Rangitaiki plains area on Friday (27 February), with three magnitude 4.0 events, and continued through Sunday. Very small foreshocks were felt on the Sunday evening at Thornton, at the Rangitaiki River mouth. These were not felt by people a few kilometres away. A full description of the events preceding the main shock must await full analysis of the data.

2.1.4 Felt Intensities

In the last 20 years, four events in the northwestern North Island of magnitude 5.0 and greater have produced epicentral intensity of MM VII or VIII. The main Edgecumbe shock almost certainly produced an epicentral intensity of IX and possibly X, doubt being due to the uncertainty at this stage of which damage is attributable to earth deformation rather than shaking.

It would appear that the Edgecumbe event is consistent with the previously observed isoseismals of earthquakes in this region which feature:

(i) very high epicentral intensities because of the shallow focal depth and soft lithology;

(ii) rapid diminution of intensity with distance from the epicentre, attributable to high attenuation in the mantle or near surface or both.

Note that the principal fore- and aftershocks could have been expected to produce MM VII, and in the case of the largest, MM VIII.

A re-evaluation of the seismic hazard in the NW North Island is planned, based in part on existing macroseismic data and in part on data collected as a result of this occurrence.

2.2 STRONG MOTION DATA (G. H. McVerry, W. J. Cousins, R. T. Hefford)

2.2.1 Strong-motion records

For the first time, strong-motion accelerographs have produced records from a New Zealand earthquake with response spectra comparable to design level 150 year return period spectra for the more seismic parts of the country. The accelerograms obtained at the base of Matahina dam in the M_L 6.3 Edgecumbe earthquake produced 5% damped acceleration response spectra of similar strength to the El Centro 1940 NS spectrum, apart from a deficiency over the 0.5 to 1.2 second period band. The El Centro 1940 NS spectrum has been widely used world-wide as a standard design spectrum.

Five strong-motion accelerographs are permanently installed on Matahina dam as part of Physics and Engineering Laboratories New Zealand wide strong-motion network. The maintenance of these instruments is financed by Electricorp through the Ministry of Works and Development Power Division as part of their dam monitoring programme. Three of the accelerographs are sited across the crest of the dam, one around mid-height in the centre of the dam, and one on the ground in the centre only a few metres from the toe of the dam face.

On the day after the earthquake, two further instruments were installed at the electrical sub-stations at Kawerau (across the road from the Tasman mill) and Edgecumbe. Two days after the earthquake, an accelerograph was installed at Whakatane.

Other permanent instruments in the general region, although not in the zone of damaging motion, are at Maraenui school near the Motu river mouth, Opotiki (peak acceleration recording SP scratch plate), and Rotorua Police Station. An SP instrument at Tauranga had been temporarily removed prior to the earthquake during consg construction work and was reinstalled along with an accelerograph on Monday, 9 March.

All accelerographs are of the New Zealand MO type. Four of the dam instruments were MO2 accelerographs with one at the true right crest site as MO2A, while MO2A accelerographs were installed at all the new sites. MO2 instruments trigger for an approximately 40 second run, stop, and then retrigger if the vertical motion is strong enough. The MO2A has the advantage of remaining in operation for 20 seconds after the trigger level is last exceeded, and at the end of the run producing a time code with 0.1 minute resolution to allow identification of the event.

The time-codes on the MO2A on the dam crest produced the following end-of-record times, taking the end of the main event as time zero, with identification in terms of the main foreshocks and aftershocks given:

End-of-record time (min. w.r.t. end of the main event)	Event identification
- 7.0	M_L 5.2 01 h 35 m 37 s
main event	M_L 6.3 01 h 42 m 34 s
+ 6.8	
+ 8.4	M_L 5.6 01 h 51 m 08 s
+ 9.0	
+10.0	
+11.1	
+20.0	
+20.8	
+21.6	
+24.7	M_L 5.1 02 h 07 m 23 s
+25.3	
+26.5	
+33.6	
+59.9	
OUT-OF-FILM	

Many further triggerings were recorded after the film ran out. The largest trigger count was 75 up to 1 p.m. on Tuesday, 3 March when the records were recovered. After that, the instruments were visited daily until Tuesday, 10 March, with records produced every day by at least some of the instruments.

Many aftershocks were recorded on the instruments installed at Kawerau and Edgecumbe.

Several aftershocks were recorded in their entirety where the instruments had been triggered by closely preceding events or for test runs. One such record at Kawerau gave a P-S time of 0.7 seconds. None of the Matahina Dam instruments were running when the P-wave from the main shock arrived. Given the size of the main shock however all 5 instruments will have triggered very close to the P-wave arrival, and the trigger-to-S wave arrival interval indicates a source-instrument distance of 23 km. The shortest distance to the main fault trace was much less, about 11 km.

The records from the main shock have been digitised and passed through PEL's standard processing routines to produce time histories of accelerations, velocities and displacements, Fourier spectra of the accelerations, and acceleration response spectra (0,2,5,10,20% critical damping).

The processed record from the dam base accelerograph for the main shock gave the following peak values:

COMPONENT N83E LONGITUDINAL
ACCELEROMETER AXIS (PARALLEL TO DAM FACE)

Acceleration: peak 2775mm/sec² peak FS 1496mm/sec at 2.53Hz.
Velocity: peak 262mm/sec.
Displacement: peak 60mm.

Peak 5% damped spectral acceleration 8570mm/sec² at 5.99Hz (0.17 sec).

COMPONENT N07W TRANSVERSE ACCELEROMETER AXIS (UPSTREAM-DOWNSTREAM)

Acceleration: peak 2364mm/sec² peak FS 1288mm/sec at 2.01Hz.
Velocity : peak 207mm/sec.
Displacement: peak 100mm.

Peak 5% damped spectral acceleration 9830mm/sec² at 2.7Hz (0.37 sec).

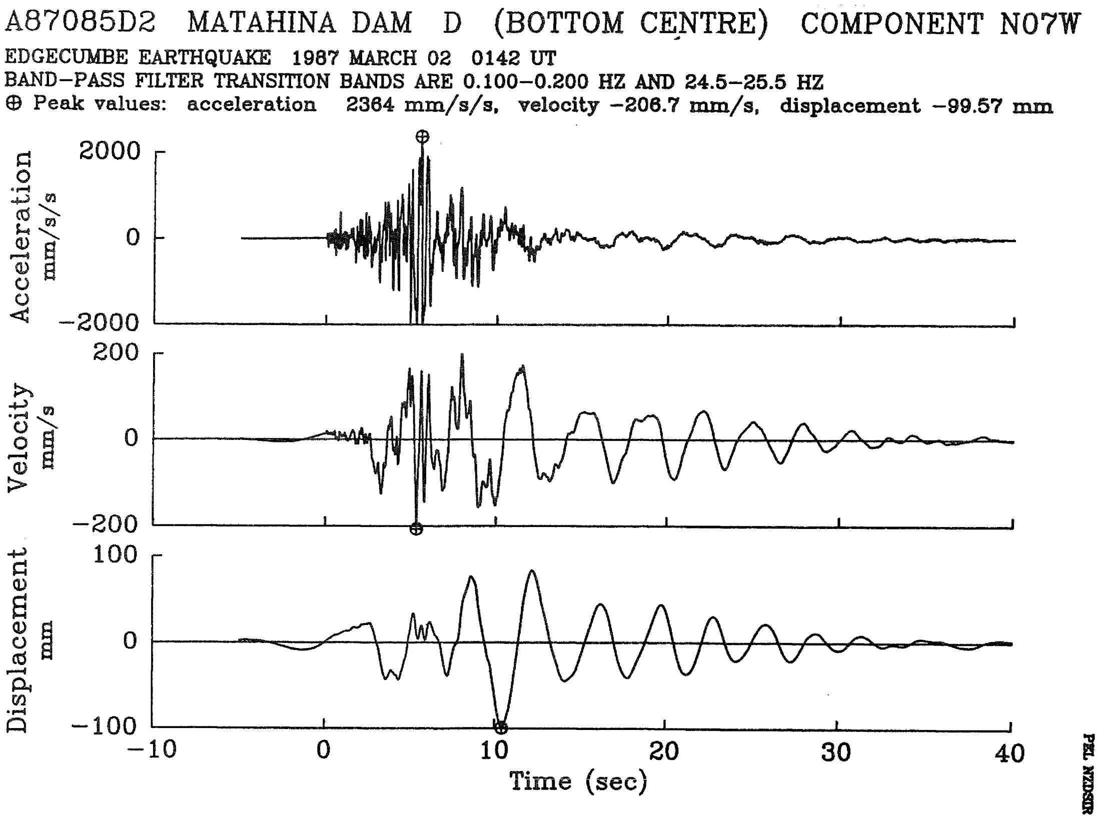


Fig. 2.2 : Time history for ground motion at the base of the dam

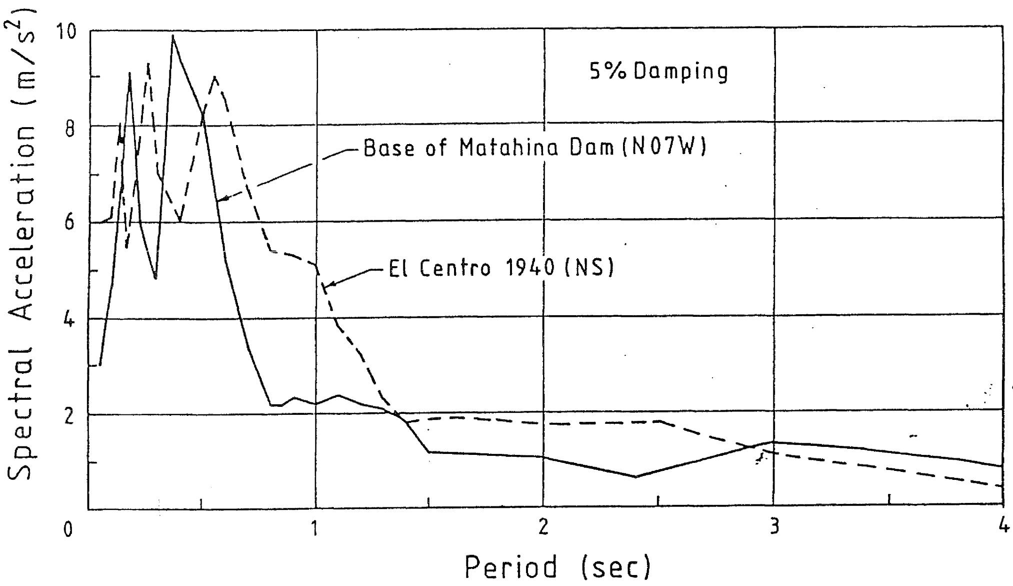


Fig. 2.3 : Response spectra at the base of the dam

COMPONENT VERTICAL

Acceleration: peak 1378mm/sec^2 peak FS 616mm/sec at 2.23Hz.
Velocity : peak 85mm/sec.
Displacement: peak 36mm.

Peak horizontal acceleration 3247mm/sec^2 in direction N37E (37°).
Peak horizontal velocity 291mm/sec in direction N71W (289°).
Peak horizontal displacement 105mm in direction S25E (155°).

Fig. 2.2 gives a time-history plot of the measured acceleration at the base of the dam, and Fig. 2.3 gives a response spectrum for the motion at the base of the dam as well as a comparison with the 1940 El Centro NS spectrum.

The peak horizontal ground acceleration of 0.33 g is the strongest recorded by an accelerograph giving an acceleration time-history in a New Zealand earthquake since the inception of the strong-motion network in 1949. Stronger horizontal ground accelerations have been recorded in New Zealand on peak acceleration scratch plates as follows:

Inangahua	1968	M_L 7.1	Westport	0.42 g
			Murchison	0.40 g
			Greymouth	0.40 g
Westport	1962	M_L 5.9	Westport	0.34 g
Gisborne	1966	M_L 6.2	Gisborne	0.33 g

The 5% damped acceleration response spectra are very similar in strength to the El Centro 1940 NS spectrum.

Comparisons are:

Matahina dam base N83E SAmax = 0.87 g at 0.17 second natural period
N07W = 1.00 g at 0.37 second natural period
El Centro 1940 NS SAmax = 0.92 g at 0.55 second natural period

The Matahina spectra are deficient with respect to El Centro 1940 NS in the 0.5 to 1.2 second period range. They are generally comparable or stronger than El Centro in both the shorter (<0.5 second) and longer (<1.2 second) period ranges.

A striking feature of the time-history plots is the appearance of the calculated ground displacement for the N07W (upstream-downstream) component, Fig. 2.2. It appears like a moderately damped (about 10% critical) sinusoid with a period of about $3^1/_2$ seconds. This unusually long period displacement component appears to be genuine in that the Fourier amplitude spectrum at this period is about two orders of magnitude greater than the noise spectrum amplitude for the digitising system, Fig. 2.4. Explanation of this feature will be required before the accelerogram is used for the design of long-period structures. The extent to which the ground record has been affected by the proximity of the dam to the ground site should also be investigated.

The peak responses for the accelerogram recorded at the centre of the dam crest were:

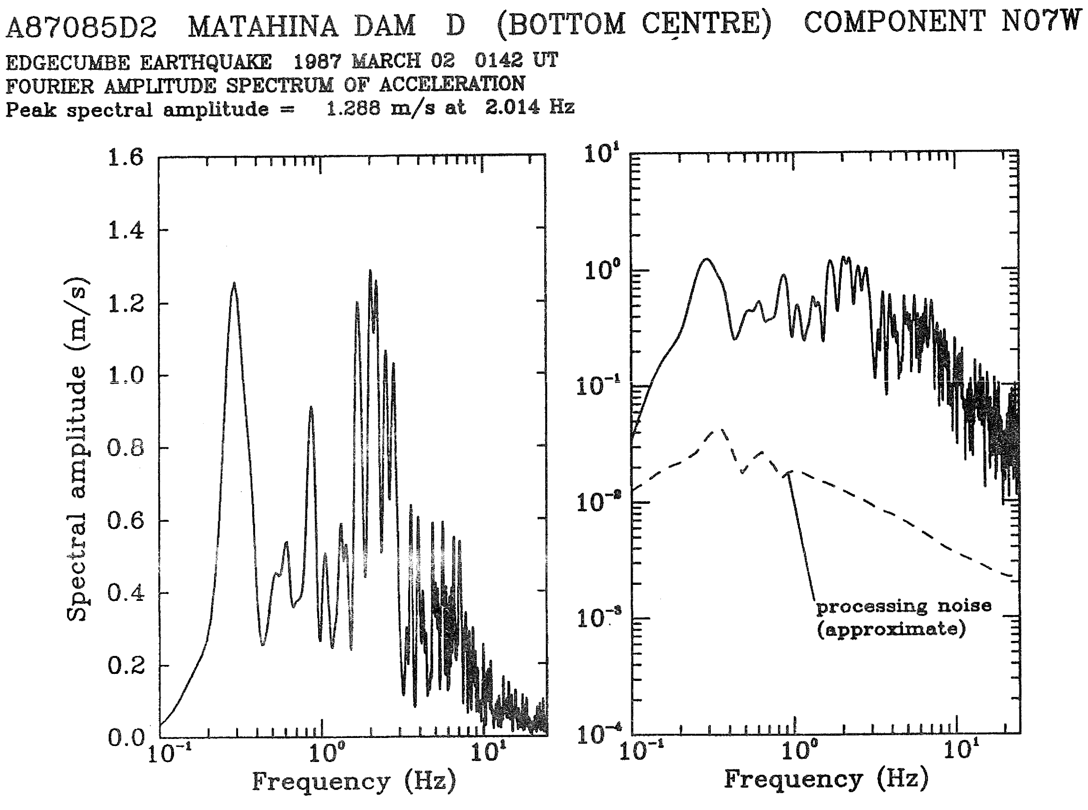


Fig. 2.4 : Fourier Spectrum for ground motion at the base of the dam

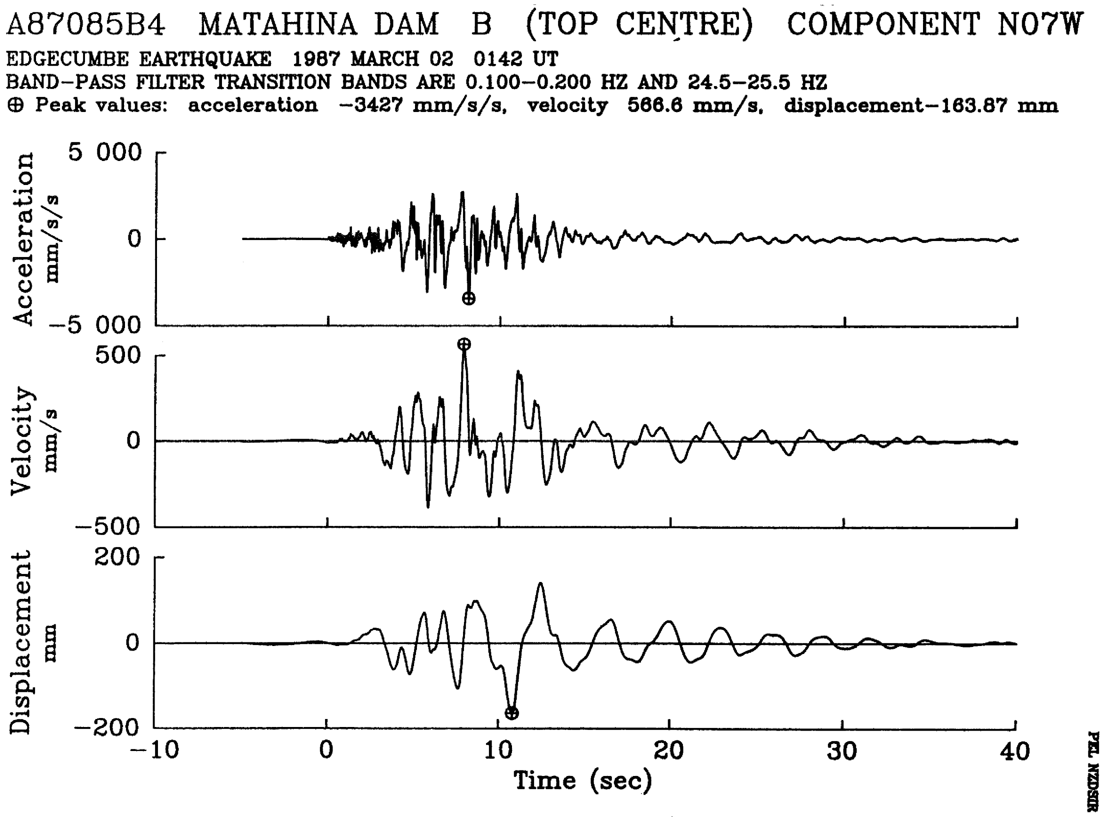


Fig. 2.5 : Time history for ground motion at the crest of the dam

	N07W	N83E	Vert.	Max.	Hor.
a_{max} (mm/sec²)	3427	2766	2824	4155	(S27W)
v_{max} (mm/sec)	566	370	164	664	(N25E)
d_{max} (mm)	164	98	33	166	(S15E)

The peak of the Fourier amplitude spectrum of the upstream-downstream acceleration response of the centre of the dam crest occurred at 0.9 Hz.

Figure 2.5 gives the recorded time-history of the motion at the crest of the dam and Fig. 2.6 gives the response spectrum for the crest of the dam. The peak acceleration response of the crest was amplified by 45 percent in the upstream-downstream direction and more than doubled in the vertical direction with respect to the base motion, while the peak acceleration was virtually unaltered in the across-river direction.

Several of the aftershock records are of sufficient strength to be of engineering interest and will also be digitised. Many of the other records should be of interest to seismologists, especially those recorded near the fault traces at Edgecumbe and Kawerau.

2.3 OTHER INDICATIONS OF THE INTENSITY OF THE GROUND MOTION (M. J. Pender)

(i) Figure 2.7 shows a NZR locomotive that was overturned in the railway yard at Edgecumbe. In addition there were photographs in the newspapers of a loaded timber trailer and trucks overturned during the earthquake.

(ii) Figure 2.8 shows scratch marks on the floor of the Bay Milk factory in Edgecumbe left by the movement of a tank support stand.

(iii) A block of concrete, about 1 m square and 0.6m tall,was reported to have slid 0.2 m across a dry concrete surface in Edgecumbe.

(iv) There were numerous reports from people in the area of waves travelling across ground surface. Mr. G. G. Excell, a civil engineer from Tauranga, was at a site in Edgecumbe when the earthquake struck. He observed the wave corresponding to the drop in ground level (cf. section 3.2) travelling towards him. This was accompanied by a loud roaring sound.

(v) Evidence of vertical acceleration has been suggested in the apparent vertical motion of liquids in storage vessels and the way in which tops were pushed off these vessels. In addition the lift motor in the Power Board Building in Whakatane seemed to have jumped clear of 12 mm retaining pins without damage to the pins (cf. section 4.2.2.4).

(vi) Railway lines suffered numerous instances of compression buckling and sometimes fracture, Fig. 2.9. A similar effect was apparent in the streets in Edgecumbe with the compression of the pavement and buckling of the footpath, Fig. 2.10. Both of these features suggest ground compression in the Edgecumbe area.

(vii) In Edgecumbe and the surrounding countryside in excess of 400 breaks had to be repaired in watersupply and sewage lines. This is covered further in section 6.

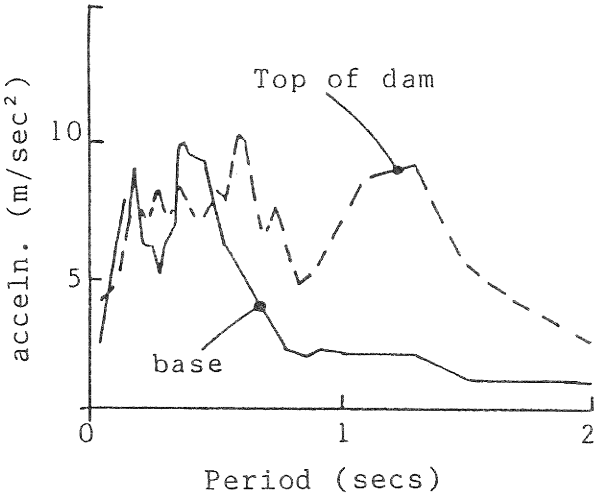


Fig. 2.6 : Response spectra at the crest of the dam

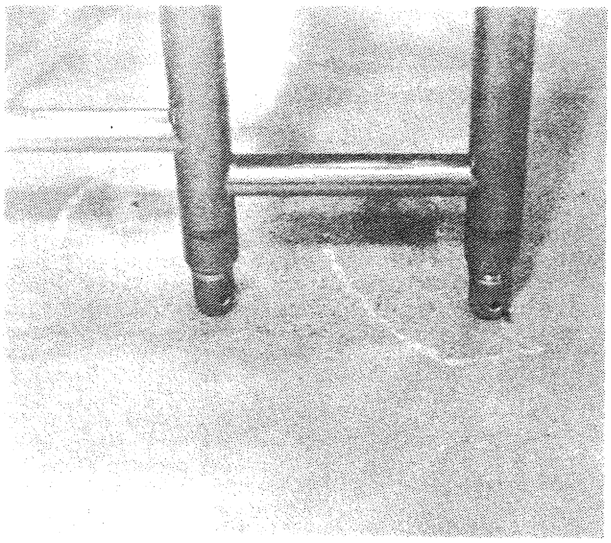


Fig. 2.8 : Scratch marks made on the Dairy factory floor by the legs of a tank stand

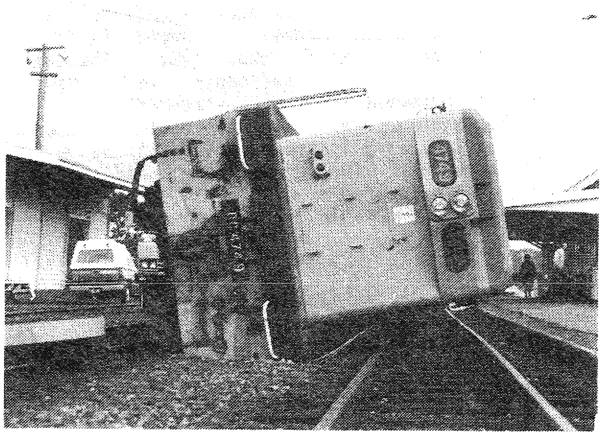


Fig. 2.7 : Overturned locomotive in the Edgecumbe railyard

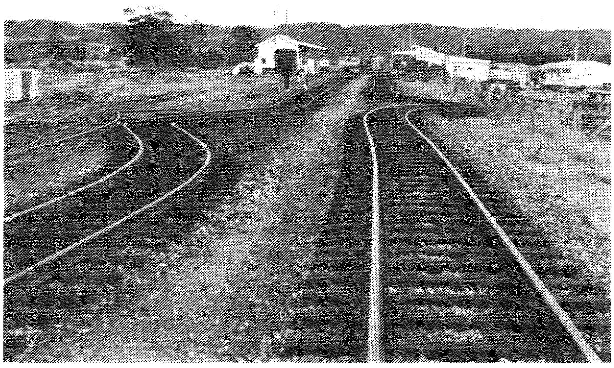


Fig. 2.9 : Compression of railway lines in the Edgecumbe railyard

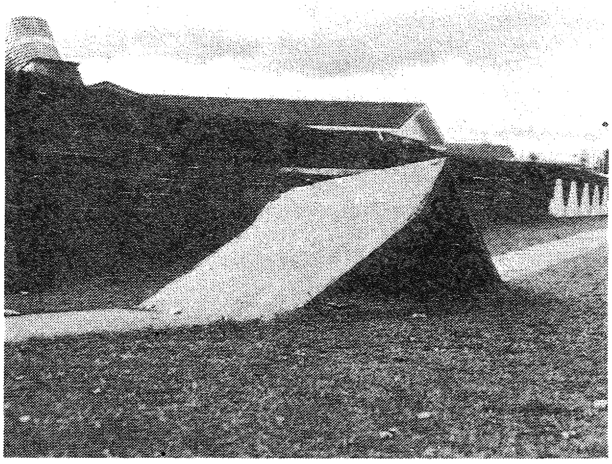


Fig. 2.10 : Compression of the footpath in Edgecumbe

3 GROUND DAMAGE

3.1 SURFACE FAULTING (B. W. Riddolls)

At least seven prominent linear surface ruptures have been recognized across the Rangitaiki Plains trending mainly in a north-easterly direction. The locations of these are shown in Fig. 3.1 and Fig. 3.2 gives an aerial view of the main fault trace between McCracken Rd and the Rangitaiki River. While some of these ruptures can be shown to be tectonic in origin, others are likely to have formed as a result of settlement caused by earthquake shaking. Initial studies by NZ Geological Survey of the longest rupture (Beanland et al , 1987), the 7 km "Earthquake fault", include assessment of aerial photographs taken prior to the earthquake, which show that a pre-existing scarp was present along the line of the new rupture. Trenches excavated across the fault provided stratigraphic evidence of at least one other, and possibly two faulting events in the past 1800 years or so, prior to the 2 March event.

General details of the principal ruptures are given in Table I.

3.2 REGIONAL SUBSIDENCE (D. G. Pemberton)

At the request of the Bay of Plenty Catchment Commission Lands and Survey Department, Rotorua undertook a second-order levelling programme on their Rangitaiki Plains benchmarks soon after the earthquake. This included an additonal line of benchmarks established at the Commission's request along the Matata - Edgecumbe - Awakeri Road.

Changes of benchmark levels, in metres, are shown inFig. 3.1. Negative values signify settlement; positive values denote upthurst. It is apparent that the greatest settlements occurred at Edgecumbe, -2.05 m, and at Te Teko, -.46 m, see Fig. 3.1.

The results given above are of "second order" quality, generally accurate to with $\pm$ 1 cm. The benchmarks are generally about one kilometre apart so these results only indicate trends.

From Matata eastward to around Seccombs Canal, upthrusts to 0.13 m have occurred. From Bennetts Bridge east the coastal area has lowered to a maximum of 0.4 m, the worst affected areas being Bennetts Bridge eastwards to Rangitaiki River. From Thornton upstream the Rangitaiki River to Edgecumbe, settlement increases to the large values of between one and two metres around Edgecumbe township. Upstream, settlement reduces to zero by the Te Teko Fundamental benchmark with slight upthrusts further upstream. Westwards from Te Teko settlement decreases from about 0.4 m to zero near the Mangaoni Stream.

Commission survey work since the earthquake reveals the following average changes at the following locations:

A	Orini Canal, Keepa Road Bridge	- 0.09 m
B	Reid's Central Canal, Halls Pumpstn.	- 0.33 m
C	Estrn. end of Old Rangitaiki Rvr. Chn.	- 0.30 m
D	Lower Awaiti Canal	+ 0.07 m
E	Omehau Canal	- 0.12 m
F	Omeheu Canal	- 0.36 m

In addition there are significant level changes elsewhere which cannot be measured due to lack of pre-earthquake levels; Smith's Drain near Otakiri Rd. is an example. However, in 1985 the Commission completed a comprehensive survey of cross sections and bank levels of the Tarawera River and associated major canals (Awakaponga, Awaiti and Omeheu). Further, at the time of the earthquake a similar survey of the Rangitaiki River and its canals was nearly completed. Both these surveys will be invaluable for comparison with present levels. Resurveying of both river systems is underway.

3.3 MATAHINA DAM (M. D. Gillon)

The Matahina Power Station is located on the Rangitaiki River about 20 kilometres from its mouth. The dam is 79 metres high above foundation level. It has a clayey gravel core, weathered ignimbrite transitions and rockfill shoulders. The dam is founded on alluvial sediments with massive ignimbrite abutments (soft rock strength). The dam is well instrumented with piezometers, seepage monitoring, survey pillars and five strong motion accelerometers. The powerhouse and spillway structures are located in a rock ridge on the left abutment.

Table I Ground ruptures formed during the Edgecumbe Earthquake (Beanland et al, 1987)

	Length	Av. Vert. Disp. (Provisional)	Pre-existing
Edgecumbe fault trace plus bifurcation	7.0 km 0.7 km	1.5 m	yes
Te Teko fault trace (2 segments)	1.4 km 0.5 km	0.5 m	no
Onepu fault trace	2.2 km	0.5 m	yes
Omeheu Canal fault trace	0.7 km	0.3 m	no
Awaiti fault trace	3.5 km	1.0 m	no
Otakiri fault trace	1.4 km	0.5 m	no
Tasman Forest fault traces (2)	0.5 km 0.5 km	0.1 m	yes

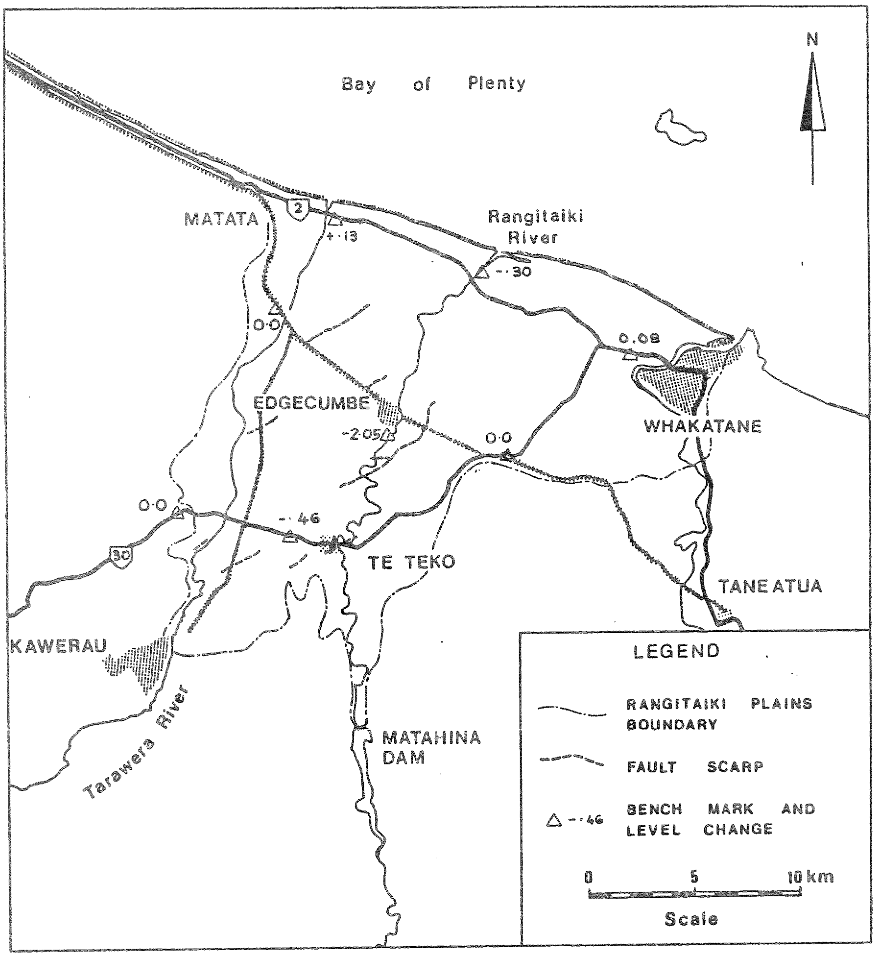


Fig. 3.1 : Map of surface rupture and change of level on the Rangitaiki Plains

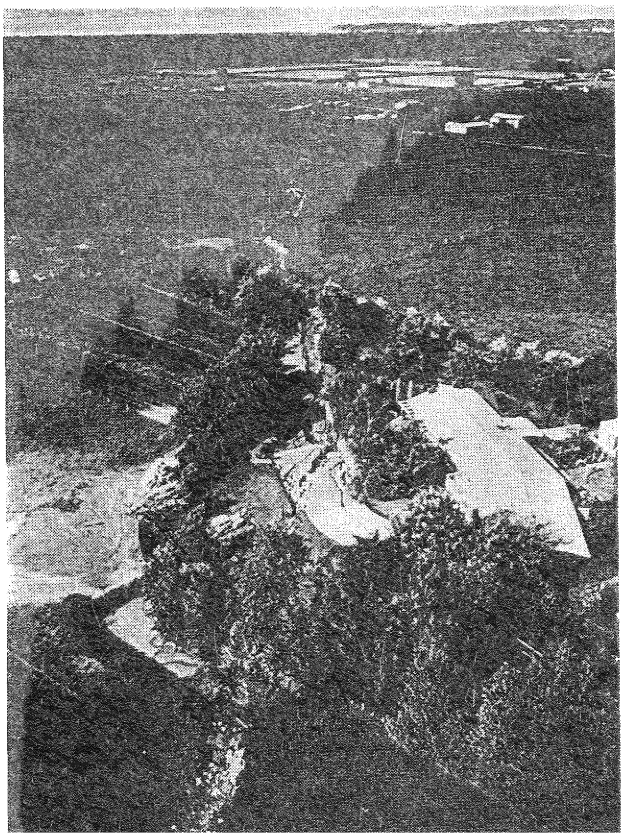


Fig. 3.2 : Aerial view of the main fault trace

The dam is the site of the strong motion instrumentation in the region. The data obtained are discussed in section 2.2.1. As a result of the earthquake the downstream rockfill shoulder moved a maximum of 250 mm downstream with a maximum settlement of 100 mm. The upstream rockfill shoulder also settled. Surface cracking occurred in the abutment areas of the dam but no significant changes in seepage flows or piezometric levels were observed. In a drainage drive in the ignimbrite rock mass adjacent to the spillway seepage flows increased fivefold and have remained at this level. There was no damage to the powerhouse or switchyard.

The base accelerometer is situated on at least 40 metres of alluvial materials, mostly sand and gravel.

3.4 LIQUEFACTION

A number of liquefaction related phenomena were observed across the Rangitaiki Plains. Figure 3.3 shows the locations of sand eruptions observed on aerial photographs of the plains taken a few days after the event; Grapes, Sissons and Wellman (1987). Figure 3.4 shows ejected sand adjacent to the west bank of the Whakatane River near the Landing Rd. bridge.

3.4.1 Level Ground Liquefaction (J. B. Berrill)

Sand boils occurred on level ground in isolated locations throughout the plains, with quite dense concentrations of boils in three areas:

(i) Over an area of several hectares to the east of the mouth of the Tarawera River, between the frontal dunes and the main Tauranga-Whakatane road (SH2). These are shown in part in Fig. 3.5.

(ii) In a band a few kilometres wide and about 10 km long, starting just west of Edgecumbe and running south west to a point south west of the township of Te Teko. In this area, probably the most dense concentration of sand ejection was around Powell Rd, especially on Mr Hohepa's property.

(iii) Along the banks of the Whakatane River in and around the town of Whakatane itself. Here there were both sand boils on level ground, and mass movement of riverbanks on slopes of only a few degrees. These mass movements were accompanied by ejection of sand along fissures parallel to the river, as well as by isolated circular sand boils. Notable areas included:

The Pony Club grounds between Cutler Street and the river.

The flood plain on the inside of the meander loop to the west of Riverside Drive.

Around the north west abutment of the Landing Road Bridge.

The motor camp at the end of McGarvey Road.

Each of these sites is on the inside of a loop in the present river channel, presumably in old point-bar deposits.

Grain sizes of ejected material (determined by hand lens and grain-size comparator) varied from very fine sands and coarse silts (40 to 100 μm) in the paddocks surrounding the Braemar Booster Station at

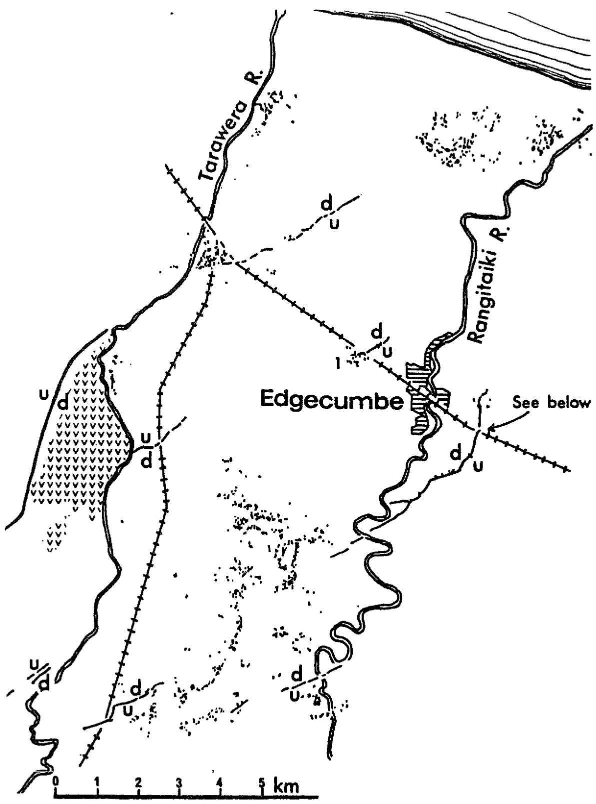


Fig. 3.3 : Occurences of ejected sand on the Rangitaiki Plains after the earthquake. Information obtained from aerial photographs taken a few days after the earthquake. (Diagram provided by H. G. Wellman)

Edgecumbe, to fine and medium sands at other sites. At some sites the ejecta was quite stratified, with interbedded layers of fine and medium sand, often also containing pumice lapilli of up to 20 mm in diameter. In any local area it was common for some boils to eject fine sand while their neighbours ejected medium sand. In stratified ejecta, in some cases the finer material was at the bottom; in others the coarser.

Evaluating the Kuribayashi and Tatsuoka (1975) expression:

$$\log_{10} R_{max} = 0.77M - 3.6, \quad (R_{max} \text{ in km.}),$$

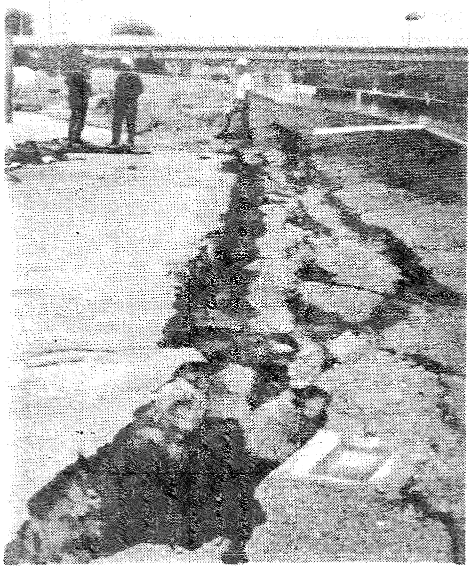
gives the maximum distance at which liquefaction may occur of 18 km for the local magnitude estimate of 6.3; if the surface wave magnitude of M_s = 6.4 is used, the expression gives a distance of 21 km. A 20km radius from the epicentre encompasses all of the plains except Kawerau (where no liquefaction was reported nor ejected sand observable from the air). Thus, given the pervasiveness of recent dune sands and river deposits, it is not surprising that liquefaction was widespread on the Plains.

In calibrating empirical models for liquefaction, the most interesting sites are those just on the limit of the zone of liqueaction. Thus a careful search was made from the air of areas of sediment just around and beyond the 20 km circle. The search focused on the Ohope spit and the flats of the Whakatane River upstream of the town as far as Waikirikiri, 25 km from Edgecumbe, where the river

Fig. 3.4 : Close up of the ejected sand adjacent to the Whakatane Rvr.

Fig. 3.6 : Cracks in the approaches to the Landing Rd. bridge over the Whakatane River.

Fig. 3.7 : The condition of the true right stopbank of the Rangitaiki River in Edgecumbe after the earthquake.



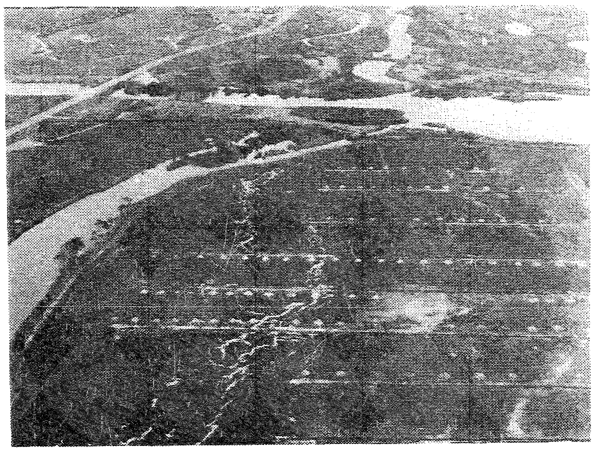


Fig. 3.5 : Ejected sand near the mouth of the Tarawera River. (The elongated white traces are the result of the earthquake. There is also a regular pre-earthquake pattern due to drainage works of the farmer.)

Fig. 3.8 : Cracking in Thompson Rd. which, at this location, is on the crest of the stopbank along the Rangitaiki River

becomes confined. No evidence of widespread sand ejection was found from the air (at an altitude of 200 ft in a light aircraft). A staff member of the East Coast Catchment Board, based in Opotiki and covering the region to the east of Whakatane, reports that he had walked the stopbanks of rivers in the Opotiki area and saw no signs of liquefaction or any cracking of stopbanks.

3.4.2 Lateral Spreading (M. J. Pender)

This is a type of liquefaction phenomenon which occurred adjacent to the river channels. The effect on stopbanks and roads is pervasive although the lateral displacements were generally small. Figure 3.6 is a photograph of the stopbank near the Landing Rd. bridge in Whakatane, Fig. 3.7 is of the stopbank adjacent to the dairy factory in Edgecumbe, and Fig. 3.8 shows the longitudinal splitting in Thompson Rd., which runs parallel to the Rangitaiki river north of Edgecumbe. Such splitting of the roadway was apparent at all locations where the road was close to the river channel.

3.4.3 Particle size distribution of ejected material (D. N. Jennings)

Samples of ejected material were collected from a number of sites throughout the plains. The locations from which the specimens were collected are shown in Fig. 3.9 and descriptions of the specimens are given in Table II. Sieve analyses for the various samples are presented in Figs. 3.10, 3.11, and 3.12.

Comparison of these grading curves with recognised particle size data for liquefaction, Fig. 3.13, shows that the Rangitaiki Plains sands have similar particle size distribution curves to liquefiable deposits elsewhere. The plains sands are a little unusual in that they contain a high proportion of pumiceous material. Thus it appears that the pumiceous nature of the sands does not affect the susceptibility of the material to liquefaction.

3.4.4 Whakatane District Council Sewage Pumping Station (D. N. Jennings)

The Whakatane District Council sewage pumping station is located on the corner of Beach St. and McAlister St. It is some 100 m from the Whakatane River. The pump house is a circular chamber 10 m in diameter and 7 m deep which is split into a wet well and a pumphouse. Alongside the pumphouse is a square chamber which receives the sewer mains and houses valves. The square chamber rotated excessively and the main circular chamber floated up 2-300 mm.

Damage to the pumping station is associated with liquefaction of the uniform fine sand at each site. Gradings of the sand samples collected at and near the pump station (samples 11, 12 and 13) are presented in Fig. 3.10. Evidence of liquefaction in the area was apparent with patches of ejected sand on the sealed street surfaces. Also there was freshly deposited sand on the ground surrounding the pump station.

There was no obvious damage to the houses in the vicinity of the pump station (e.g. no fallen chimneys). Subsurface damage to the sewer mains was apparent. A large quantity of sand was pumped into the station and the pump was severely damaged. Council staff suggested that the power to the pumps failed during the main 6.3 earthquake at 1.42 p.m. but liquefaction and sewer damage may have occurred during the 5.2 foreshock at 1.35 p.m. If this interpretation is true the pumps continued to pump contaminated sand for some 7 minutes before power failure.

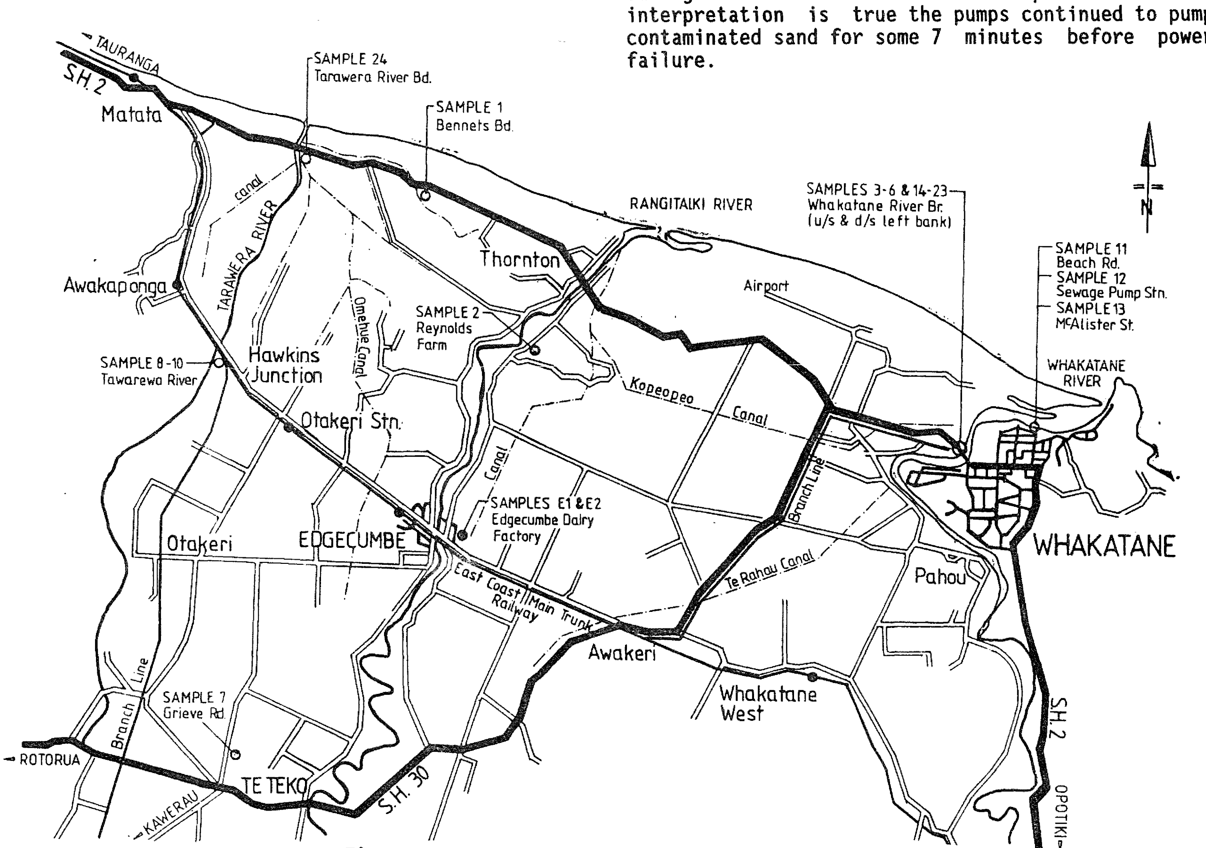
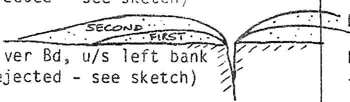


Fig. 3.9 : Location of specimens for particle size distribution

TABLE DESCRIPTION OF SAND LIQUEFACTION SAMPLES EJECTED BY THE 2 MARCH 1987 : EDGECUMBE EARTHQUAKE

Tested by MWD Hamilton

Sample No.	NZMS 1 Reference	Source	Description (Ref : CDP 813/B:1982,) (Colour from Munsell charts)
1	N68 272319	SH 2 Bennet's Bd, right bank approximately 30 m u/s of highway.	Olive grey (5 YR 5/2) fine SAND - sand 50% pumiceous, uniform
2	N68 301281	Reynolds Farm, Rangitaiki River right bank east of East Bank North Road approximately 2 km u/s Thornton Bd.	Olive grey (5 YR 5/2) fine SAND - sand 50% pumiceous, uniform
3	N69 415254	SH 2 Whakatane River Bd, u/s left bank (site 1, first ejected - see sketch)	Light olive brown (2.5Y5/4) medium to fine SAND - sand uniform, (rounded pumice particles to 10 mm)
4	N69 415254	SH 2 Whakatane River Bd, u/s left bank (site 1, second ejected - see sketch)	Light olive brown (2.5Y5/4) medium SAND - sand uniform (no pumice)
5	N69 415254	SH 2 Whakatane River Bd, u/s left bank (site 2, first ejected - see sketch)	Dark grey (2.5Y3/0 fine SAND - sand uniform (no pumice)
6	N69 415254	SH 2 Whakatane River Bd, u/s left bank (site 2, second ejected - see sketch)	Dark grey (2.5Y3/0) fine SAND - sand uniform (no pumice)
7	N77 225171	East of Grieve Rd, opposite Te Teko Racecourse	Light brownish grey (2.5Y6/2) fine SAND - sand uniform (occasional pumice peices to 40 mm)
8	N68 218276	Tarawera River right bank u/s of Edgecumbe-Awakaponga Road (river side of stopbank)	Light brownish grey (2.5Y6/2) fine SAND - sand pumiceous, up to 2 mm, uniform
9	N 68 218276	Tarawera River right bank u/s of Edgecumbe-Awakaponga Road (river side of stopbank)	Light grey to grey (2.5Y6/0) fine SAND - sand pumiceous, up to 2 mm, uniform
10	N68 218276	Tarawera River right bank u/s of Edgecumbe-Awakaponga Rd (river side of stopbank)	Light grey (2.5Y7/2) very fine SAND - sand pumiceous, uniform
11	N69 432261	Beach Rd, Whakatane, off street surface opposite No. 20 Beach Rd	Light grey (2.5Y7/2) very fine SAND - sand pumiceous, uniform
12	N69 432262	Sewage Pump Stn, cnr Beach Rd and McAlister St, Whakatane (WT ≈ 1.5 m)	Grey to dark grey (2.5Y5/0) very fine SAND - sand pumiceous, uniform
13	N69 433260	McAlister St, Whakatane, off street surface opposite No. 6 McAlister St	Light brownish grey (2.5Y6/2) very fine SAND - sand pumiceous, uniform
14	N69 416254	SH 2 Whakatane River Bd, d/s left bank	Light/dark grey (2.5Y6/0) medium to fine SAND - sand, pumiceous, uniform
15	N69 416254	SH 2 Whakatane River Bd, d/s left bank	Greyish-brown (2.5Y5/2) medium SAND - sand uniform (occasional pumice to 6mm)
16	N69 416254	SH 2 Whakatane River Bd, d/s left bank	Grey (2.5Y5/0) medium SAND -sand uniform
17	N69 416254	SH 2 Whakatane River Bd, d/s left bank	Grey)2.5Y5/0) medium to fine SAND - sand uniform
18	N69 416254	SH 2 Whakatane River Bd, d/s left bank	Grey (2.5Y5/0) fine SAND - sand uniform
19	N69 416255	SH 2 Whakatane River Bd, d/s left bank (adjacent approach fill)	Grey (2.5Y5/0) fine SAND - sand uniform
20	N69 416256	SH 2 Whakatane River Bd, d/s left bank (approximately 100 m d/s of approach fill, flat ground)	Grey (2.5Y5/0) very fine SAND - sand uniform
21	N69 415254	SH 2 Whakatane River Bd, (u/s side)	Greyish brown (2.5Y5/2) fine SAND - sand uniform
22	N69 415254	SH 2 Whakatane River Bd, (u/s side)	Grey (2.5Y5/0) medium SAND - sand uniform (some pumice to 2 mm)
23	N69 415254	SH 2 Whakatane River Bd, (u/s side)	Light olive brown (2.5Y5/4) medium SAND - sand uniform
24	N68 238329	SH 2 Tarawera River Bd (u/s right bank, 20 m land side of stopbank)*	Fine SAND
E1	N68 283229	Adj Edgecumbe Dairy Factory workshop, 50 m east of East Bank North Rd	Light grey (2.5Y7/0) very fine SAND - sand uniform, saturated
E2	N68 283229	Adj Edgecumbe Dairy Factory workshop, 50 m east of East Bank North Rd	Grey (2.5Y5/0) fine SAND - sand uniform, saturated



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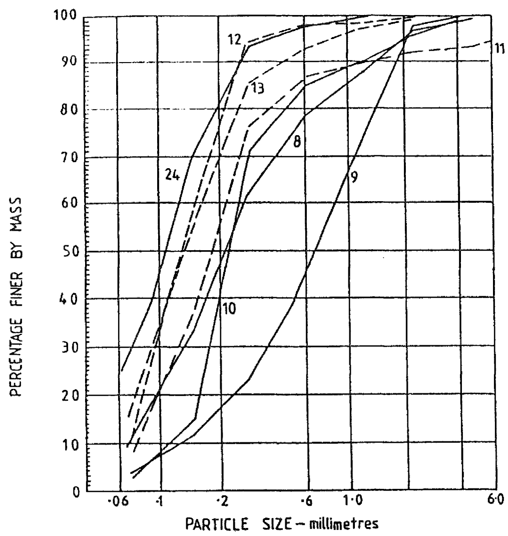


(ALL SAMPLES ARE OF EJECTED SAND)

SAMPLES 8, 9, 10 – TARAWERA RIVER RIGHT BANK U/S OF EDGECUMBE – AWAKAPONGA ROAD.

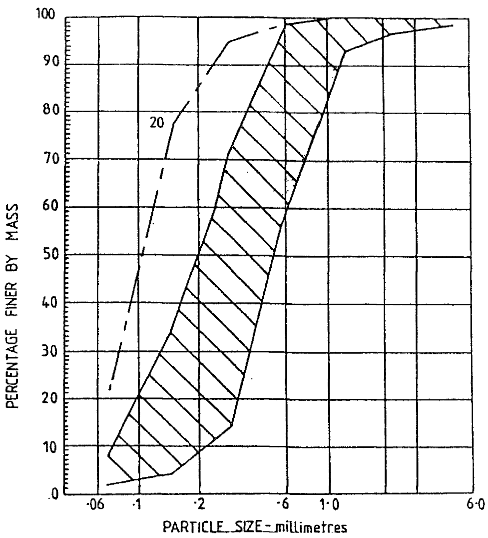
SAMPLE 11 – FROM STREET SURFACE OPP. Nº 20 BEACH ST., WHAKATANE.

SAMPLE 12 – WDC SEWAGE PUMP STN., CNR BEACH ST./McALISTER ST. WHAKATANE.

SAMPLE 13 – FROM STREET SURFACE OPP. Nº 6 McALISTER ST. WHAKATANE.

SAMPLE 24 – S.H.2 TARAWERA RIVER BR.

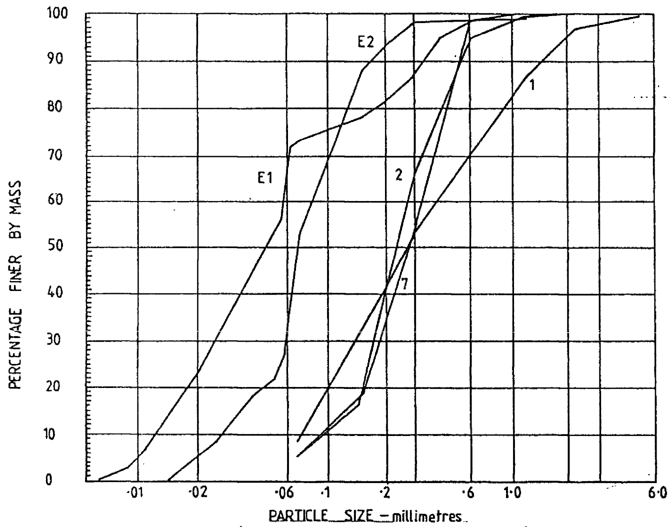
Fig. 3.10 : Particle size distributions of ejected sand



GRADING ENVELOPE OF 13 EJECTED SAND SAMPLES (3, 4, 5, 6, 14, 15, 16, 17, 18, 19, 21, 22, 23)

GRADING OF EJECTED SAND SAMPLE (FROM FLAT GROUND APPROX. 100M D/S OF HIGHWAY, 100M LAND SIDE OF STOP BANK) (20)

Fig. 3.11 : Particle size distributions of ejected sand



(ALL SAMPLES ARE OF EJECTED SAND)

Fig. 3.12 : Particle size distributions of ejected sand

SAMPLE 1 – S.H. 2 BENNETTS BR., RIGHT BANK U/S OF HIGHWAY.

SAMPLE 2 – REYNOLDS FARM, RANGATAIKI RIVER RIGHT BANK, APPROX. 2KM U/S THORNTON BR. (S.H.2)

SAMPLE 7 – EAST OF GRIEVE RD. OPP. TETEKO RACECOURSE.

SAMPLE E1/E2 – EAST OF EAST BANK NORTH RD. OPP. EDGECUMBE DAIRY FACTORY. (NOTE SPRINGS FLOWING 4 DAYS AFTER EARTHQUAKE.)

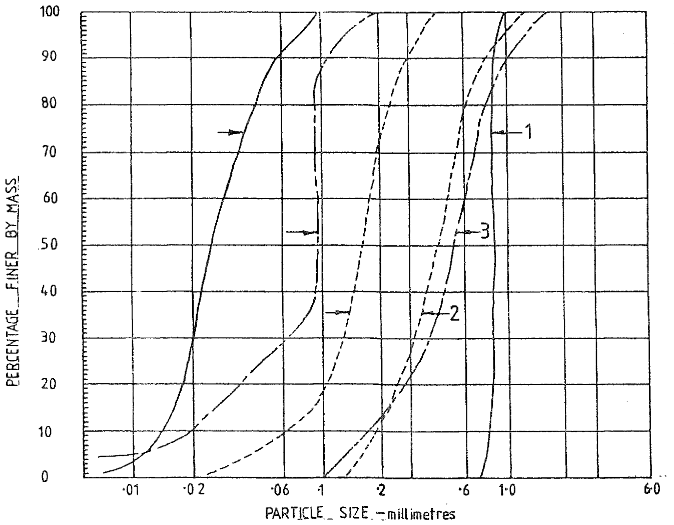


Fig. 3.13 : Grain sizes susceptible to liquefaction

1 - MOST LIQUEFIABLE SANDS IN LABORATORY TESTS, LEE AND FITTON.(1968)

2 - NIIGATA SANDS IN 5 TO 10m DEPTH RANGE, SEED AND IDRISS.(1967)

3 - ENVELOPE OF 19 JAPANESE SANDS WHICH HAVE LIQUEFIED DURING EARTHQUAKES, KASHIDA (1969)

3.5 BEHAVIOUR OF WELLS (M. J. Pender and C. O'Brien)

Figure 3.14 is a photograph of the ground surrounding one of the wells affected by the earthquake. There were three types of response:

(i) Wells with water level below ground level (some had once been artesian) had flows above ground level. Just after the earthquake water gushed 90cm. into the air above the well tube at the Omeheu Demonstration farm. Three days later, by middle of the day the on Thursday March 5th, the waterr was issuing just above ground level. Other wells were still flowing freely on Thursday but the general comment seemed to be that the intensity of the flows was decreasing.

(ii) In the Otakiri Orchards area it was reported that several wells were capped with a pressure gauge attached. The farmers looked at the pressure after the EQ, typically these had risen a few tens of kPa. This excess pressure subsided after a few days.

(iii) Some wells had water gushing from the ground surface near the well. It is not clear if this due to the well pipe cracking at some depth or because of gapping along the well tube and water flowing to the surface from a shallow aquifer. Examples at Otakiri Orchards and kiwifruit farm at corner of Otakiri Rd. and Poplar Lane.

Increased flow was also noted, by the proprietor of the Awakeri camping ground from the thermal springs at the camping ground.

3.6 SLOPES (M. J. Pender, B. W. Riddolls, G. Salt and D. J. Jennings)

No significant problems were appararent with cut slopes although there were many instances of material that had fallen and obstructed the main roads. Between Lake Rotoma and the Kawerau turnoff the road cuttings are through volcanic ash. Although a nuisance these obstructions were not examples of serious slope failure, the material that fell was little more than superficial soil and vegetation. Along SH 2 west of Matata there were several examples of material that had fallen from the high ignimbrite cliffs in that area. The debris seemed to have come from the very top of the cliffs and often brought pohutukawa trees with it. (A typical site is adjacent to the Mimiha bridge.) The ignimbrite is of low quality and reduced to fine silt and sand by the time it reached road level.

A short distance upstream from the Matahina dam is the quarry that was supplied for the dam rockfill. This was left in a series of benches and overall the quarry is a considerable oversteepening of the ignimbrite slopes in the area. (Unlike the ignimbrite along SH 2 near Matata the material at the dam site is at the other end of the spectrum of ignimbrite quality.) Despite the accelerations on alluvial ground at the Matahina dam no slope failure was visible in the ignimbrite at the quarry.

It was reported that damaged slopes behind parts of the residential areas of Kawerau were a slope stability hazard. On examination it was clear that there were conspicious cracks along the ground surface at and near the tops of ridges. These appeared to be a consequence of the soil profile on the slopes. This consisted of a layer of topsoil bound with roots from the grass surface, underlain with a layer of lapilli (from the Tarawera eruption?) which in turn was underlain by volcanic ash. It appeared that the lapilli layer, being cohesionless, had slumped slightly under the EQ loading and consequently caused the surface layer of topsoil to crack. It did not apppear that there was a great volume of soil involved even if a more serious failure had occurred.

Similar slope damage was observed along the Matakina-Kawerau transmission line route. Many of the transmission pylon towers are located on the crests of sharp ridges with concrete cylinder and buried footing foundations. Surface ground cracks were observed in the pumice ash material on the ridge crests, in some cases cracks were up to 150 mm wide. It was noted that ground cracking was more prevalent on narrow sharp steep sided ridges and ground damaged increased in severity towards Kawerau. Only two towers suffered minor structural deformation as a result of ground movements and it is deduced that the ground failures were shallow, generally less than 2-3 metres.

3.7 STOPBANKS

Damage to stopbanks from lateral spreading has been mentioned above in section 3.4.2 and illustrated in Figs. 3.6, 7 and 8. This type of damage occurred on the stopbanks for all three of the rivers that flow across the plains. In addition there were places where the stopbanks were damaged by tranverse fracturing. This was particularly evident where the main fault trace crossed the Rangitaiki River.

3.8 ROADS AND BRIDGE APPROACHES

Spectacular damage to roads was observed where the main fault trace crossed the Edgecumbe-Awakeri Rd., McCracken Rd. and the Edgecumbe-Te Teko Rd. Figure 3.15 shows the abrupt change in level across McCracken Rd.

Elsewhere damage to the roads was repaired (temporarily) with great speed. Most roads were open within a day of the earthquake.

Further comments on roads occur in sections 2.3, 3.4.2 and 3.6.

The subsidence of bridge approaches has been mentioned in section 3.4.2, it is also referred to in section 5.1.

3.9 FOUNDATIONS

Specific comments on foundation behaviour are given in sections 4.1.3.1, 4.2.8.2, 4.2.9.2 and 5.1.

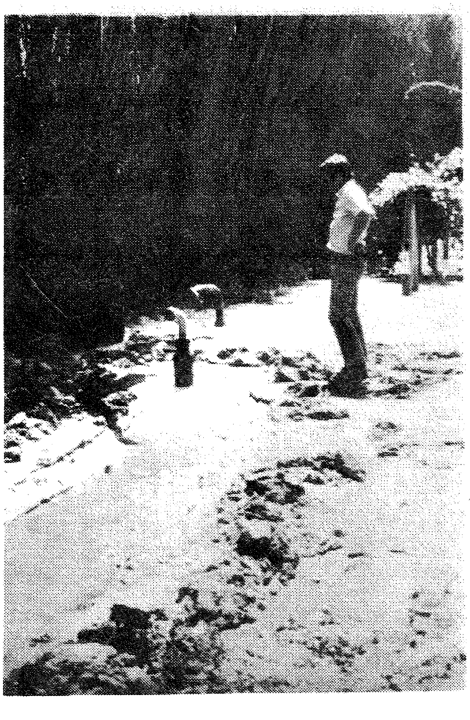


Fig. 3.14 : A well head surrounded by ejected sand

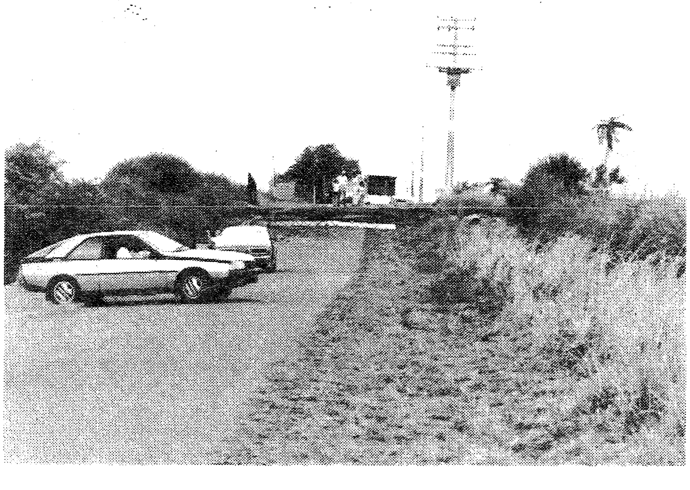


Fig. 3.15 : Damage to McCracken Rd. where it intersects the main fault trace

4 STRUCTURAL PERFORMANCE

4.1 HOUSES (R. C. Cooney)

4.1.1 Introduction

The damage to housing can be broadly put into two categories, being that which occurred to the basic load resisting elements of houses (foundations, sub-floor framing, floors, walls and roofs) and referred to as 'structural damage', and that which occurred to other parts (chimneys, claddings, services etc.) and to contents in the houses, referred to as 'non-structural'. In discussing the damage in this report, that which has been clearly identified as having been caused by permanent ground displacement has mostly not been included. Often though, it was not possible to make such clear distinction.

The majority of houses were not structurally damaged although many of them did receive some non-structural damage. However, many hundreds of houses did receive structural damage of differing kinds with less than 50 suffering substantial structural damage.

Damage was confined to an area approximately bounded by the townships of Whakatane (4183)[2], Kawerau (2324) and Edgecumbe (544). Within this area there are approximately 1000 farm houses and a number of small rural settlements on the Rangitaiki plains, the largest settlement being Te Teko (140). Structural damage to housing was mostly limited to Edgecumbe and Te Teko and the adjacent rural areas. There were also a few instances of structural damage to houses on hill tops at Awakeri and at Whakatane.

Based on reports of damage to NZ houses in previous earthquakes, Cooney (1978), damage to housing in earthquakes had been predicted, Cooney and Fowkes (1981), and these predictions were confirmed in this instance. There were few surprises, to earthquake engineers at least.

The age distribution of the houses did have some bearing on the nature and extent of damage. Approximately two thirds of the houses in the area (Kawerau Borough excluded) were built in the period 1950 to 1979 when the building industry awareness of earthquake resistant construction was not high and appropriate codes of practice were not completely adequate.

In 1978 the Standards Association of NZ published a new Standard (NZS 3604 - Code of Practice for the Construction of Light Timber Frame Buildings not Requiring Specific Design) which is used for most NZ housing. This Standard contains much more specific provisions relating to resistance to earthquakes than did previous codes. These earthquakes, being the first to occur in New Zealand to test seriously the earthquake resistance of houses constructed to NZS 3604, were thus an important event. From observations and reports from the local authorities, it appears that almost all houses reported to have been built in accordance with NZ 3604 were not structurally damaged by the earthquakes. Some exceptions were concrete slab-on-ground floors which are discussed later. In other instances some damage occurred because not all of the provisions of NZS 3604 had been complied with. It must be kept in mind though that most houses did not experience the intensity of ground shaking taken as the design earthquake for the preparation of NZS 3604.

[2] Occupied houses in the 1986 Census

4.1.2 Damage in General

The overall damage to housing, apart from reflecting the nature of construction, also reflected the special characteristics of the earthquakes and of the soil conditions. Four particular aspects, namely ground shaking, ground waves, three earthquakes occurring in short succession and ground rupture have collectively resulted in greater damage than would have occurred with ground shaking alone from a single earthquake. As was the case with other buildings and structures, permanent ground displacement through rupture or subsidence contributed to much of the damage to houses, especially to concrete and masonry foundations, walls and masonry veneers. One significant feature of the ground shaking component was that it appeared from the damage to be extremely variable, even between adjoining properties let alone adjacent streets, given that there apparently were not signficiant differences in the construction of the houses involved. This can presumably only be attributed to highly variable soil conditions. Similar or near identical houses on adjacent properties were not similarly damaged, with sometimes one badly damaged and the other not.

4.1.3 Structural Damage

Substantial structural damage occurred to less than 50 houses, and most of these involved the foundations.

4.1.3.1 Piled Foundations

Houses built on unbraced piles, with or without braced and unbraced jackstuds atop the piles, proved particularly vulnerable. These houses had very little specific provision for lateral load resistance incorporated into the construction of their foundations and sub-floor framing. Any such resistance was usually only achieved by there sometimes being a few poorly connected diagonal timber braces, by cast-in-situ concrete pile footings and items such as cast concrete chimney bases, porches and steps. Less than 30 of these collapsed, damaging their superstructures as well. Approximately a further 100 houses of this type were laterally displaced at ground floor level, Figs. 4.1.1 and 4.1.2.

One house on piles which was badly damaged also had a brick veneer cladding sitting on a continuous perimeter foundation wall which had been constructed independent of the foundation piles. The lack of adequate lateral support to the piles meant that the house moved sideways dislodging the brick veneer from the foundation wall and causing other damage, Fig. 4.1.3.

But not all houses on piles with or without jackstuds collapsed. Many remained standing, but showed signs of having moved considerably. Presumably many of these were saved from collapse by the moderate intensity of the ground shaking which they experienced, and possibly the frequency of the ground shaking and the variable subsoil conditions.

4.1.3.2 Foundation Walls

These were either continuous around the house perimeter or were what is known as corner foundation

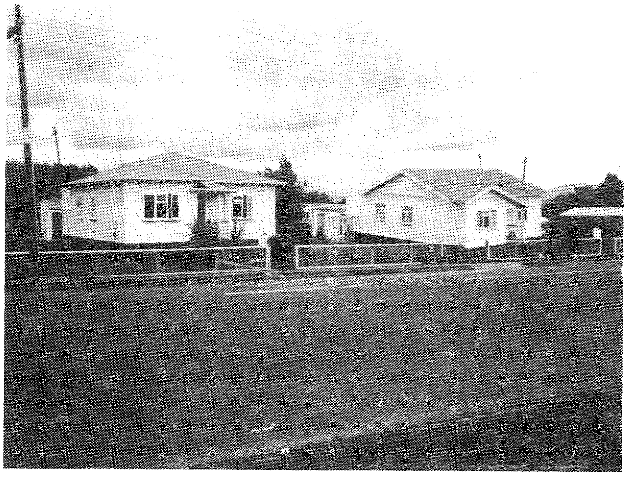


Fig. 4.1.1 : Two near identical houses on lightly braced jackstuds and concrete piles. The house on the right has toppled off, the one on the left is laterally displaced at floor level.

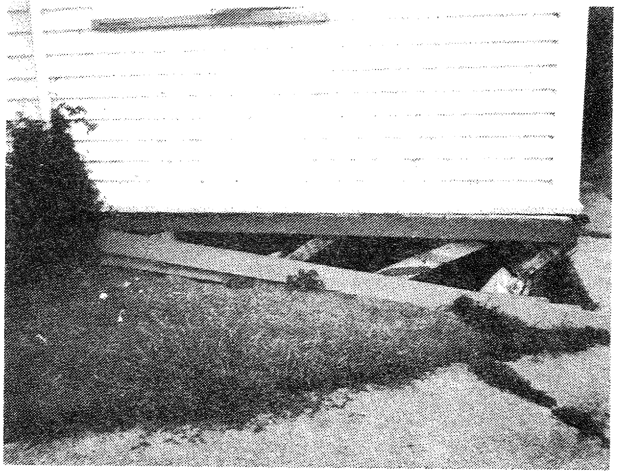


Figure 4.1.2 : Details from the right hand house in Figure 1.

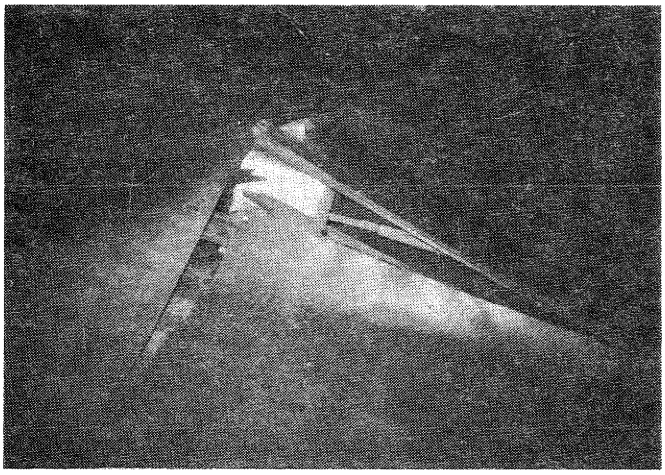


Figure 4.1.3 : Houses which fell off their piles badly damaged the floor, apart from other damage

Figure 4.1.4 : Typical damage to joints between sheets of wall linings

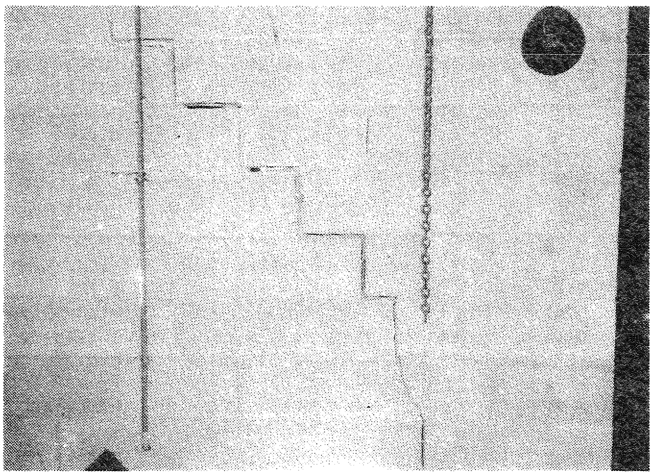


Figure 4.1.5 : Damaged unreinforced concrete block walls

walls. With regard to the latter, one feature of the houses not usually seen in other parts of New Zealand was that additional lengths of foundation walls 1 metre to 2 metres long often occurred in between the corner walls. Consequently, up to 50 per cent of the length of the house perimeter was on a foundation wall of some kind when chimney bases and entrance porches were also taken into account. The floor framing was usually supported on and connected by steel dowels to these walls.

None of the houses with these forms of foundation walls showed any signs of structural damage at that level.

4.1.3.3 Concrete Slab-on-ground Floors

These generally performed well. However, one type of construction permitted by NZS 3604 did not always perform well and based on the evidence from these earthquakes it needs investigating. This construction is that for a concrete slab-on-ground floor for a single storey building where the slab is constructed independent of a perimeter foundation wall and sits on top of that wall. NZS 3604 does not require a connection between the slab and the wall nor reinforcement of the slab. A few buildings of this type of construction were badly damaged when the slabs moved sideways with permanent lateral displacements of up to 300 mm being reported. In one similar case the owner/builder claimed that there had been 6 mm rods at 900 mm centres connecting the slab to the foundation wall and that these had all broken. Observations of strong vertical accelerations during these earthquakes are likely to have contributed to these problems. In another one of these cases a floor slab opened up at all the construction joints by 20 to 30 mm.

4.1.3.4 Walls

Most of the houses had wall framing of light timber frame construction. These walls usually had either diagonal timber let-in or cut-between or steel Ell or Tee shaped braces, and linings of paper faced gypsum board sheets. These nailed sheets are known to provide most of the in-plane racking resistance of the walls.

No instances of collapse or even significant permanent lateral displacement of such walls was reported. However, a considerable amount of damage occurred to the linings of these walls, Fig. 4.1.4. (Also refer to comments on masonry veneers, Section 4.1.6.3.) In many houses the wall linings had moved at their nail fixings due to in-plane racking and had cracked the plaster at their joints and the covering wallpapers. In some instances sheets had come free from the wall framing. This was particularly bad in walls where the linings had not been properly nailed and also where external walls had large areas of openings.

A few walls, usually the lower storey of 2 storey houses, were of concrete block masonry. In only one instance was significant structural damage to such walls observed, and in this case the walls were either unreinforced or poorly reinforced, Fig. 4.1.5.

A few domestic garages with walls of unreinforced concrete blocks or clay bricks collapsed due to the total lack of any reinforcement. In one instance, a concrete block boundary wall which was not cantilevered from a designed footing, collapsed in one piece because it had not been adequately tied to the adjoining house structure and only had a simple strip footing not intended to provide cantilever restraint.

4.1.3.5 Roofs

Some concrete tiled roofs suffered from damaged roof framing due to inadequate fixings and bracing of the framing, Fig. 4.1.6. Both hip and gable roofs suffered equally in this regard.

4.1.4 Building of Irregular Configuration

A trend in the above-average price range of housing in the last two decades has been for a number to have split-level floors and irregular floor plans. This has naturally resulted in buildings of asymmetrical construction.

This trend can be further compounded on hillside sites when pole platforms or pole structures are used to support the house framework and this was evident in the newer suburbs atop the hills adjoining Whakatane township. Whilst this area did receive enhanced shaking compared with other parts of the Whakatane Borough, it appeared not to have experienced quite the maximum intensities of ground shaking as occurred on the Plains. Nonetheless, a few cases of significant structural damage did occur there. In one case a pole platform house fractured the top off one pole and split the tops of some other poles, Figs. 4.1.7 and 4.1.8. In another case a house atop a half lower storey and double carport experienced torsional distortion and fractured the connections of the beams over the carport with the concrete block walls of the half lower storey, Fig. 4.1.9.

In a few other cases it was reported that intermediate floors of split-level houses damaged adjoining walls.

Almost all of the damage to the irregularly shaped houses was as a result of torsional racking of the foundation and wall elements because of the variable stiffnesses and the irregular distribution of such elements in the building plan.

Other examples of damage to irregular buildings were to a 'boomerang-shaped' house which was damaged at the junction between the two wings, and another house part of which was on piles and the remainder being on a concrete slab-on-ground. This house was also damaged at the junction between the two parts.

All the damage indicated that in a code design intensity earthquake, irregularly shaped houses, typical of many in New Zealand and often the subject of specific structural design, would be severely damaged, much more so than the houses of symmetrical plan.

This subject of adequately allowing for stiffness incompatibility in housing is comparatively difficult. Unlike the more traditional engineered structures, the load/stiffness characteristics of many elements used in housing are not known with sufficient certainty. Also, most walls, no matter how they are built, are able to act as quite significant bracing elements.

4.1.5 Houses Under Construction

A few houses were under construction at the time of the earthquake. Most were fitted with metal angle or Tee shaped diagonal braces to the wall framing, but were unlined. Some had only roof cladding on,

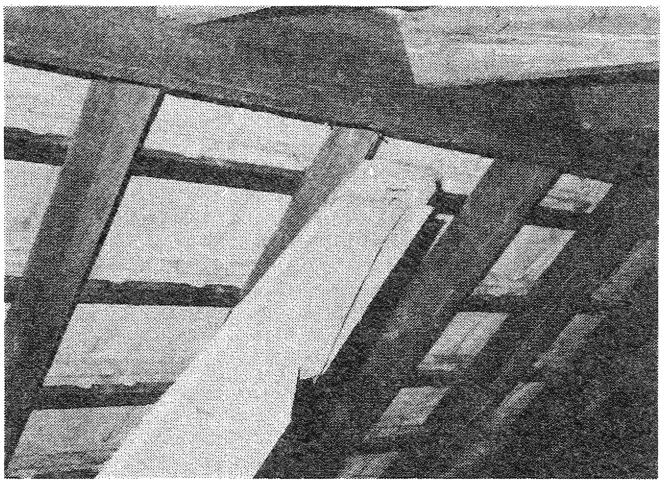


Figure 4.1.6 : A failed connection of a diagonal timber roof brace to the ridge board of a concrete tiled roof

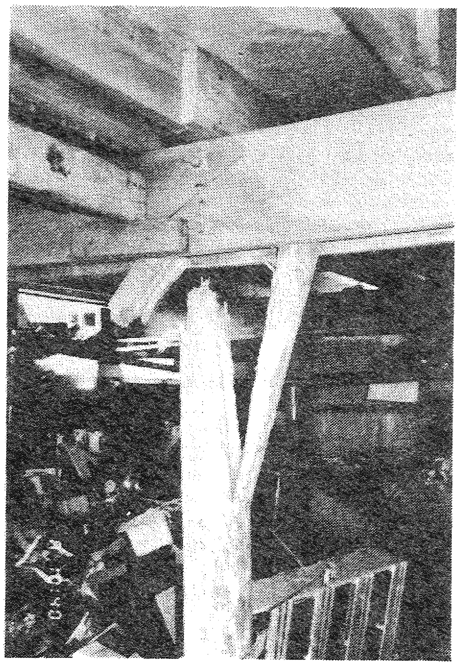


Figure 4.1.8 : A broken pole of the house in Figure 7

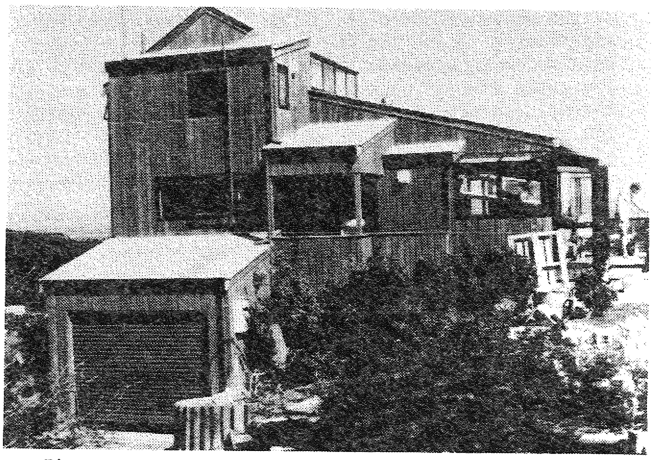


Figure 4.1.7 : A pole platform house of irregular shape

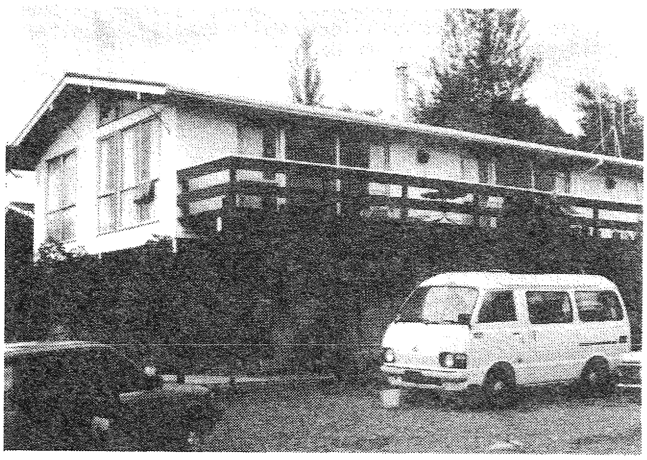


Figure 4.1.9 : An irregular lower storey, half on concrete block and half on cantilevered posts and beams. Failure occurred at the connections of the beams to the block walls.

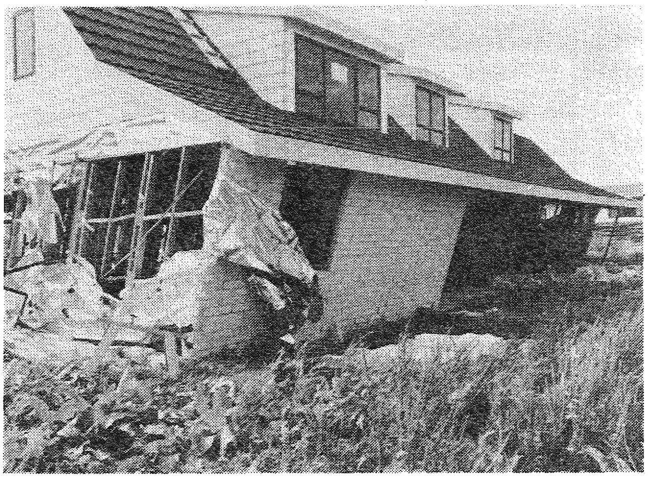


Figure 4.1.10 : House under construction

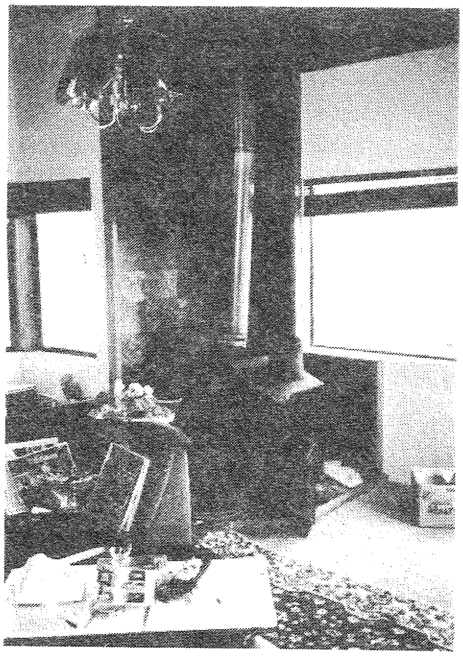


Fig. 4.1.11 : A dislodged free-standing solid fuel stove

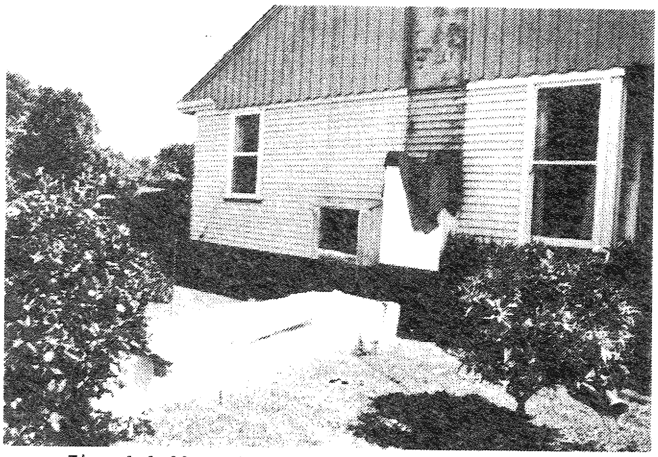


Fig. 4.1.12 : An unreinforced pumice concrete chimney

Fig. 4.1.14 : A collapsed panel of brick veneer

Fig. 4.1.13 : A reinforced pumice concrete chimney where the poorly grouted reinforced rods had partially withdrawn

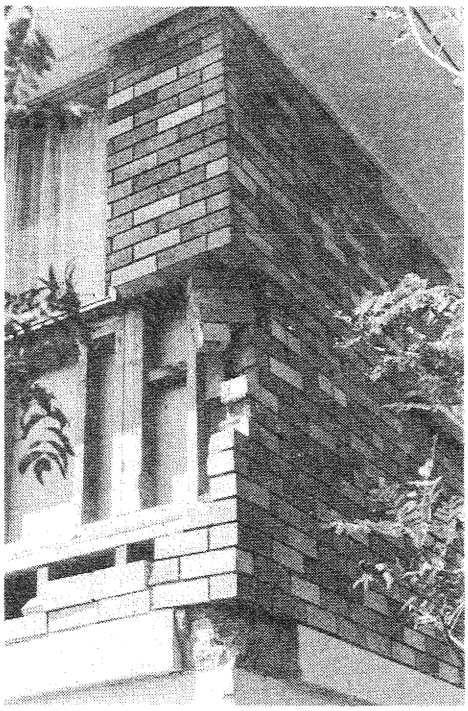


Fig. 4.1.15 : Corner damage of brick veneer

others wall cladding as well.

In all cases seen, some of the metal braces had either buckled or had torn loose at an end. In only one case was a house near collapse and this was a "Cape Cod" house with a clad roof and an upper floor placing a significant large mass on a comparatively small amount of unlined lower storey walls in the direction in which it collapsed, Fig. 4.1.15.

These cases help to illustrate the importance of additional temporary bracing during construction of a house, often done to help resist wind loads more commonly experienced, and that properly nailed wall linings of timber framed houses play a significant structural role in resisting lateral loads imposed by earthquakes and winds.

4.1.6 Non-Structural Damage

4.1.6.1 General

Perhaps the largest monetary loss, and certainly that which placed considerable emotional strain on the people involved, was the damage to the contents of house.

In the worst hit areas cupboards and shelves emptied their contents on to the floors, some cupboards were wrenched from the walls, whiteware was overturned and electrical appliances such as microwave ovens, videos, televisions and sound systems all fell to the floor and were badly damaged.

Of interest was the damage which occurred to waterbeds. A number of these burst their retaining boards and split the membrane, spilling the water into the houses which added to the damage.

A number of free-standing solid fuel burning stoves moved on their hearths and some became disconnected from their flues, Fig. 4.1.11. Similarly some solid fuel stoves installed into open fireplaces were dislodged. This too could have added to the problems had the earthquakes occurred when the appliances were alight.

4.1.6.2 Chimneys

Most houses had at least one chimney, usually of precast pumice concrete. Some were of clay brick construction. Most chimneys in and near Edgecumbe and approximately 700 in Kawerau were extensively damaged, being either totally or partially collapsed or cracked as a result of the ground shaking.

Chimneys usually fractured at the roof line or at the gathering level, Figs. 4.1.12 & 4.1.13. Many located on external walls fell to the ground, but some, and a number of their being internal chimneys, fell onto the roof. If the roof was concrete tiled, then there was considerable damage to the roof.

The reason for these collapses was largely because the bylaw requirement for vertical reinforcement of the chimneys and that they be either self-supporting or be securely tied to the house at roof and ceiling levels had not been complied with. This particular aspect of non-compliance has been noted in previous major earthquakes (Cooney) and wind storms (Cooney) and unfortunately will continue to be a feature of earthquake damage to existing houses in many years to come, even though some remedial work could be carried out to lessen the life risk of falling chimneys. Happily there was no loss of life or injury from falling chimneys in these earthquakes.

Had these earthquakes occurred during a cold winter evening with fires going, it is more than likely that some house fires would have resulted to further add to the disaster.

The damage to chimneys will mean that all chimneys, even if apparently undamaged, will need to be thoroughly inspected before they can be used again. Many hundreds of them, perhaps even 1500 or 2000 will probably need to be completely rebuilt.

4.1.6.3 Masonry Veneers

Masonry veneer claddings attached to timber wall framing were a feature of a signficant number of the houses with non-structural damage. In Kawerau for instance, it has been reported that of the 600 odd houses with masonry veneer, approximately 100 had damage to the veneer, other than slight cracking.

In most instances where there was considerable damage to masonry veneer, this could be attributed to the type of wall tie used, the manner in which the tie was fixed to the wall framing and the spacings of the ties. Many of the veneer ties in these cases were of a twisted wire variety shaped and fixed to the studs in such a way as to offer little strength or stiffness resistance to face loading of the veneer. In the case of one new house, the veneer ties, whilst of a type which comply with the current code, were not fixed as specified being at larger centres and having fewer, smaller diameter nails attaching them.

In some other instances the veneer had slid on the supporting foundation wall due to the lateral movement of the house superstructure.

Whilst the majority of houses with masonry veneer appeared at first glance to be undamaged, closer inspection revealed that many of them had newly cracked mortar joints and bricks, indicating that the supporting house structures had moved in the earthquakes. This type of damage at least is expected with a rigid brittle veneer supported by and connected to a comparatively flexible timber framed structure. In extreme cases the movement between the building frame and the veneer was so great that the distortion of the window frame caused the glass to vacate the opening.

The veneer panels between windows, in many buildings were shed through a combination of the inplane racking of the building with the window frames bearing against the veneer together with insufficient wall ties to resist the large out of plane forces induced in the ties adjacent to the stiffer parts of the wall framing.

In some instances small panels of veneer between openings fell away, even though many of them were reasonably well tied to the wall, Figs. 4.1.13 and 4.1.14.

Where wall veneers weren't shed, diagonal cracks in the veneer originating from windows and veneer damage at corners and re-entries were observed in the old and new buildings.

4.1.6.4 Concrete Roofing Tiles

Many concrete tiled roofs lost some of their tiles because the tiles had not been fixed to the battens. The loss of roof integrity also allowed rain water to enter the houses resulting in further damage.

4.1.6.5 Water Supply

In the rural areas, many houses relied upon water contained in tanks on stands. Large numbers of these tanks either burst, toppled from their stands or the stands collapsed. A number of header tanks in roof spaces or on top of hot water cylinders were dislodged, as were some hot water cylinders, and their contents lost. In many instances the water supply piping to these containers was found to be leaking, this being particularly noticeable once the mains water was reconnected, a consequence of the containers having moved in the earthquake.

For these houses this emergency supply of drinking water was not available following the earthquake.

4.1.6.6 Sewerage

Many houses on piled foundations moved sideways sufficiently to fracture their sewerage connections. In some houses the porcelain toilet bowls were fractured at their base fixings.

4.1.7 Summary

The performance of the housing stock was largely as expected. Inadequate bylaws for earthquake resistance in past decades, non-compliance with bylaws and a number of poor construction practices have been observed in other parts of the country and a similar situation was observed in this instance.

Fortunately the intensity of shaking for most areas was not at code design level, and this meant that the vast majority of houses did not suffer much or any signficant structural damage.

With the exception perhaps of some aspects of chimney and masonry veneer damage, most of the damage due to ground shaking is readily preventable, as was witnessed by the good performance of well-built older houses, and most of the newer houses which had been constructed to NZS 3604. The situation regarding some concrete slab-on-ground floors needs investigating.

There have been a number of changes to codes in recent years which aim to reduce the damage to housing in earthquakes. These include improved foundations, connecting the superstructure to the foundations, bracing of framing and the securing of concrete roofing tiles, header tanks, hot water cylinders and free-standing stoves. For existing houses there are published recommendations, Cooney (1982), which, if followed, would greatly reduce the extent of the type of damage which occurred in these earthquakes.

In the light of these earthquakes all these additional code requirements and recommendations appear to be justified. If complied with they should ensure that houses are much less damaged and will be more likely to be habitable, even after a large earthquake.

4.2 NON DOMESTIC BUILDINGS AND OTHER STRUCTURES
(T.W. Robertson)

4.2.1 General

This section of the reconnaissance report covers buildings (other than housing) and miscellaneous structures of interest. Generally the report is restricted to the Edgecumbe and Whakatane areas as the reconaissance team did not have access to the Caxton and Tasman mills at Kawerau. Structures of special interest such as houses, bridges and tanks are covered in other sections of this report.

Generally, with some notable exceptions, it can be said that buildings and industrial structures performed well, particularly those of newer construction. Damage usually was restricted to low rise buildings of rigid construction and to non-structural elements. However, for every generality, there were exceptions, which is one of the particularly noticeable things about the earthquake - it was selective in its damage with often adjacent similar structures being affected quite differently.

This section of the report describes in detail items considered to be of interest not covered elsewhere. Sometimes the interesting aspect is the lack of damage rather than the extent of it.

4.2.2 Architectural Observations

4.2.2.1 Windows (E. B. Lapish)

The most widespread and noticeable damage sustained by buildings was broken windows. This problem was more prevalent in low-rise and single storey buildings than taller ones, and the few structures in the area of five to six storeys suffered only a couple of cracked windows between them. Since none of the buildings had seismic separation in their windows, this relationship may seem surprising but is attributed to the quite considerable ground distortion which racked the relatively rigid small buildings, whereas for taller buildings the sway was insufficient to cause damage to the glazing. Silicone jointed mitred glass corners in shop fronts failed spectacularly whereas often the adjoining windows survived. Obviously the locking up of the glass panels at corners with silicone jointing could not provide for the movement mechanism necessary for the glass to survive induced simultaneous "in and out of plane" deflections. Much of the broken glazing shattered, in many instances onto the street and the fact that there were no reported injuries from falling glass was good fortune and a consequence of the relatively light pedestrian traffic in small towns together with the advance warning of the foreshocks. The need for adequate seismic separation detailing for glazing in buildings of all sizes is clearly demonstrated.

4.2.2.2 Ceilings

Except for the Riverslea Shopping Centre, suspended ceilings fared fairly well. The Whakatane Hospital lost a number of tiles in random locations and the Bay of Plenty Power Board building lost only one small group of tiles. Both these buildings were five storeys and had heavy plaster tiles in unbraced suspension systems. However, both buildings were well compartmented by internal partitions which effectively limited accumulated distortion and prevented wholesale ceiling collapse. The Whakatane War Memorial Complex Sports stadium, a single storied portal frame structure lost 10 percent of its ceiling panels. Fortunately the stadium was not in use at the time. The Riverslea Shopping Centre sustained considerable and widespread collapse of the suspension system, Fig. 4.2.1, which, of course, also brought down the lights and wiring. These ceilings were lightweight but covered large unpartitioned areas and were unbraced. The Riverslea building itself was strained to such extent that probably no ceiling design would have survived unscathed but provision of adequate bracing and positive fixing between rail components could have limited damage to relatively small perimeter areas. As the earthquake occurred in the middle of the business day, only the fact that the tiles were lightweight prevented a signficant injury statistic.

4.2.2.3 Partitions

Partition systems were not a particular problem and, except across seismic breaks between building blocks, little damage was sustained. Whakatane Hospital suffered several split partitions but these were superficial and easily repairable. BoP Power Board building reported no damage to partitions.

4.2.2.4 Services

There were a number of examples of inadequate fixing down of building services and plant resulting in considerable disruption to the immediate reuse of the building affected. Since these included both the hospital and the Power Board building, the functioning of which are important after seismic disaster, the current requirement to attend to such matters in new projects is well justified.

The Power Board building lift motor/generator was located by vertical pins through holes in each floor but was not bolted down, Fig. 4.2.2. The motor lifted clear of the pins, a jump of at least 30 mm, and landed clear of them. As the pins were relatively light but were not bent, it was concluded that the motor experienced a major vertical accleration prior to any significant horizontal component. This may have been a local effect but there was other notable evidence of a strong vertical component as will be mentioned later. Unfastened water tanks in the hospital shifted, rupturing the piping connections. The consequential flood caused the most significant damage within the hospital as well as losing a water resource that would have been important following a major earthquake.

Cast iron stops on antivibration mounts of hospital plants components proved to be inadequate and were prone to fracture.

4.2.2.5 Shelf Contents and Filing Systems

Shops, industrial and commercial premises and the hospital all experienced widespread loss of shelved contents and filing systems though generally the shelves themselves remained standing. To prevent this loss would require fiddle rails along all shelves plus additional restraint to the shelving to carry the additional load of restrained contents. Clearly the inconvenience to normal usage created by fiddle rails would be unacceptable in some (particularly retail) cases. However the potential loss of vital medical supplies and equipment could bring about a serious situation following a major earthquake and consideration of means of restraining such supplies is clearly warranted. In some instances building contents fell across doorways inhibiting egress and re-entry by emergency rescue services.

Figure 4.2.1 : Riverslea Shopping Centre suspended ceilingcollapse

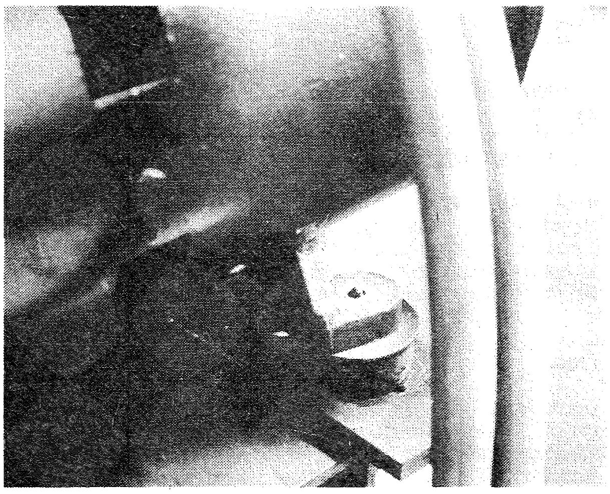


Figure 4.2.2 : BoP Power Board Building lift motor foot showing lack of holding down

4.2.2.6 Sprinkler Systems

Sprinkler systems performed very poorly in many installations, with failure of the piping itself, usually at screw-threaded connections and failure of the supporting brakets. In one major building "the sprinkler system ended up in plan on the floor", to quote the Engineer who showed the team around the site. This would have relieved the load on the supporting structure (which would in all probability have been designed for and actually capable of supporting the load) at the expense of considerable damage to building contents and a costly re-installation of the sprinkler system itself.

4.2.2.7 Light and Light Fittings

There were many instances of the fluorescent tubes falling out off fittings, even flush ceiling - mounted fittings. Arc and fluorescent lights suspended from open hooks in the roofs of industrial buildings fell down and caused widespread damage. Some of these units weighed 75 kg each and thus represented a major threat to life and equipment.

4.2.2.8 Structural Separation

There were numerous cases of considerable separation gaps between structures of parts of a single structure caused by the earthquake.

In smaller structures the prime cause appeared to be differential ground movement, especially where structures were supported on pads. In larger structures, especially those supported on piles, differential superstructure movement played a significant role in causing separation cracking.

4.2.3 Timber Structures

There were three significant industrial timber structures of interest in Edgecumbe. The two new structures survived unscathed, whereas the older structure collapsed.

The Harvest Packaging Co. building, approximately 18 months old, consisted of hipped plywood box beams supported on cantilever timber poles. There was no damage to structure nor cladding.

The BoP Fertilizer Co. bulk store was built of laminated timber two bay portals with ply gusset plate knee joints. This building similarly suffered no damage to structure or cladding.

The Timber Company dry timber store was a very old knee braced portal with trussed roof. Its collapse was total, largely due to its age and rickety state of repair.

The Tunnicliffe Timber Company store comprised pole columns supporting timber trusses with no longitudinal or transverse bracing. Stability was provided by cantilever action of the poles together with some frame action with the trusses. No major damage occurred. However the floor slab was considerably spalled and cracked surrounding many of the poles providing significant ductility, Fig. 4.2.3.

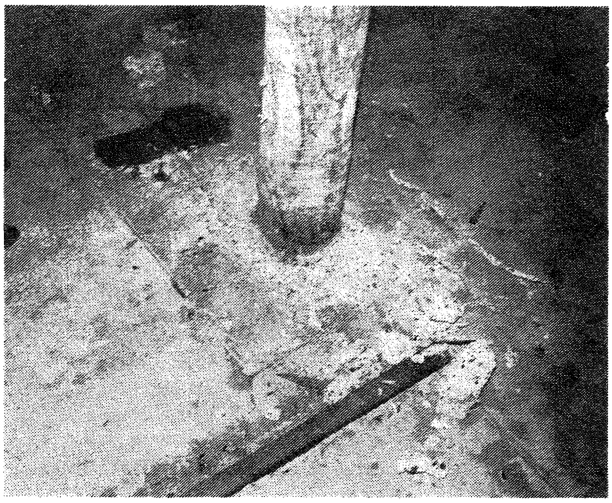


Figure 4.2.3 : Tunnicliffe Timber Company pile foundation

4.2.4 Steel Framed Structures (G. C. Clifton)

The majority of major industrial buildings were steel framed, usually with simple unnecked angle cross bracing - usually, though not always, on piles. There was almost no sign of yielding or buckling within these structures except where some other object (tank, produce) had collapsed onto them, and even in these cases the steel showed considerable toughness preventing further collapse.

Apparently without exception steel structures designed to current and less recent seismic design procedures performed extremely well. Only minor damage was suffered all of it within the primary seismic-resisting members and often such damage was repaired, in some cases within 24 hours of the earthquake, by simply replacing the primary seismic-resisting elements.

Damage to steel structures, where moderate or extensive was caused by the following factors:

(a) Secondary damage due to impact of contents and/or adjacent structures. In some instances this was quite severe, although in all cases observed, the structure behaved in a very ductile manner under, at times, very high impact loading. Among the causes of secondary damage were impact of water onto a structure from an adjacent elevated tank, impact of plant, equipment or contents of storage systems contained within the structure and collapse of silos onto the structure.

(b) Failure of brace members due to fracture of braces (threaded CHS members), tearout of end bolts or failure of welds. Even where total brace failure occurred damage to structure and cladding was only moderate at worst.

(c) Corrosion-induced failure of structural members of gusset plates. In some instances this was very severe and was due to inappropriate corrosion protection and/or inadequate maintenance.

4.2.5 Unreinforced Masonry Buildings
(T. W. Roberston)

Time did not permit a detailed survey by the reconnaissance team of buildings in Whakatane that could come within the category of earthquake risk buildings as defined in Section 624 of the Local Government Act. However the Whakatane Assistant County Engineer advised that no significant damage had been reported to him. A street inspection confirmed this.

Some unreinforced concrete masonry buildings failed spectacularly while a number of others were badly damaged.

Two such buildings are illustrated, Figs. 4.2.4 and 4.2.5, and need very little comment, except that corner reinforcement prevented overall collapse of a concrete block garage even though starter bars from the foundation into the reinforced cavities were not provided. Basketing reinforcement could have

Fig. 4.2.4 : Unreinforced masonry garage

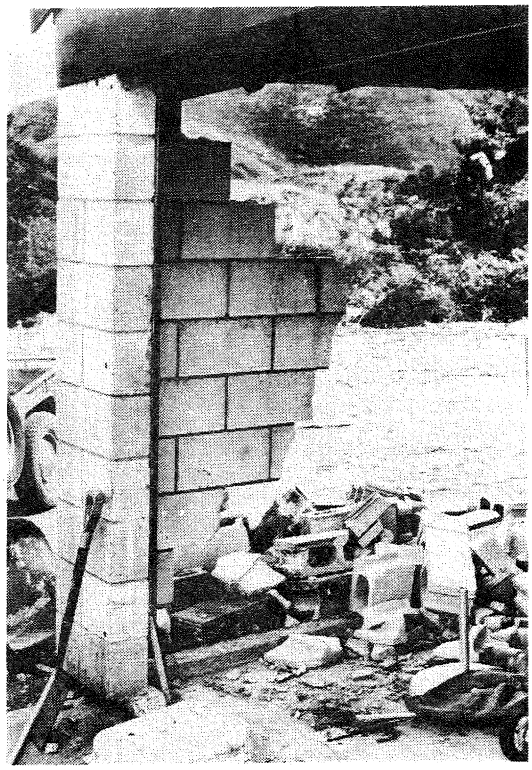


Fig. 4.2.5 : Unreinforced concrete masonry shed

prevented the large collapsed areas of blockwork.

A full storey height unreinforced concrete masonry construction supporting a light timber frame upper storey clad with an iron roof and wall veneer also suffered substantial diagonal cracking of the unreinforced masonry load bearing walls.

4.2.6 Other Structures (G. C. Clifton)

(a) Reinforced concrete structures generally performed well, except for some minor cracking. Some localised damage at the connections of reinforced concrete corbel beams to main steel frame columns occurred. An elevated 18 m diameter x 8 m high reinforced concrete water tank built 15 years ago was undamaged, however the knee joints in a rigid cradle supporting an elevated water intake pipe at the water surface level were severely damaged. They were reinforced with plain round longitudinal bars and inadequately confined with transverse stirrups and at the time of visiting the site this cradle was slowly collapsing into the tank.

(b) There was one major precast concrete structure with diagonal steel braces inspected, along with precast concrete water tanks, several steel frames with precast cladding panels and one sunken precast concrete water tank currently under construction. All performed extremely well, except for minor cracking around the connections of the precast cladding panels to the steel frame and damage to some of the panels of the sunken water tank under construction. The steel bracing in the precast concrete structure suffered some yielding as expected allowed for in design and was quickly replaced.

(c) Reinforced blockwork walls that appeared to be well constructed performed well, although many were left with fine cracking which may cause corrosion of the rebar in the future if left untreated. There were numerous instances of major reinforced blockwork walls, mainly used as fire walls, separating from supporting steel columns and requiring pulling back into alignment after the earthquake. The separation gap was over 100 mm in some instances, near the top of the walls and showed inadequate connection between the blockwork wall and steel frame.

4.2.7 Timber Company Water Tower

This was an old structure comprising a reinforced concrete octagonal water tank supported on a three storey concretre frame with eight external columns and four internal columns, all interconnected by beams, Fig. 4.2.6. The water content at the time of the shake is unknown but reported to be "a reasonable quantity of water".

The support structure had behaved in almost textbook fashion, i.e. the inadequate, out of date details had behaved as theory would have predicted. Most columns showed column base hinging with significant spalling of cover concrete, Fig. 4.2.7. The plain steel reinforcing was not confined with consequential buckling of the column bars after yielding in tension. Several beam/column joints had failed in joint shear with reversal creating diagonal corner to corner cracking, Fig. 4.2.8. The amount of joint steel is unknown but judging from the age of the structure it is likely to be light to non-existent. One external beam had hinged considerably with large loss of concrete due to lack of confining stirrups. An internal beam column

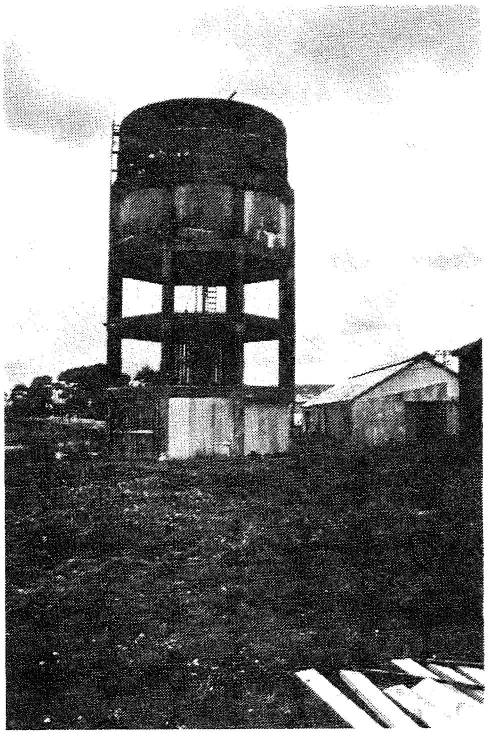


Fig. 4.2.6 : Timber Company water tower

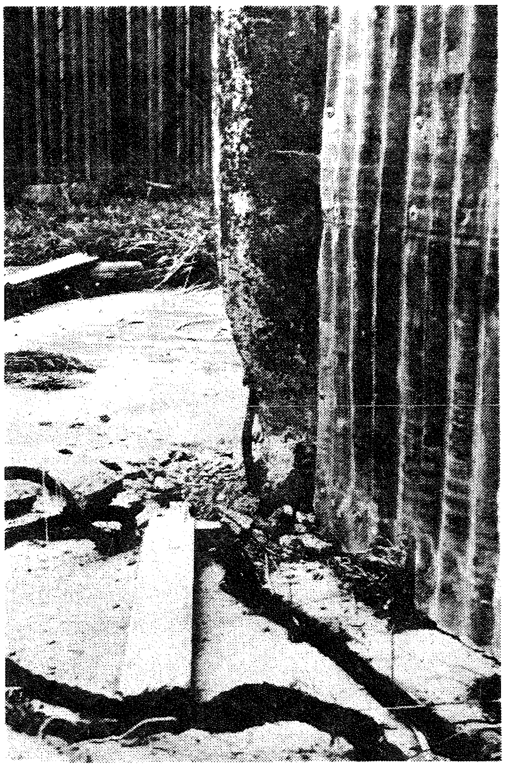


Fig. 4.2.7 : Water tower column base hinge

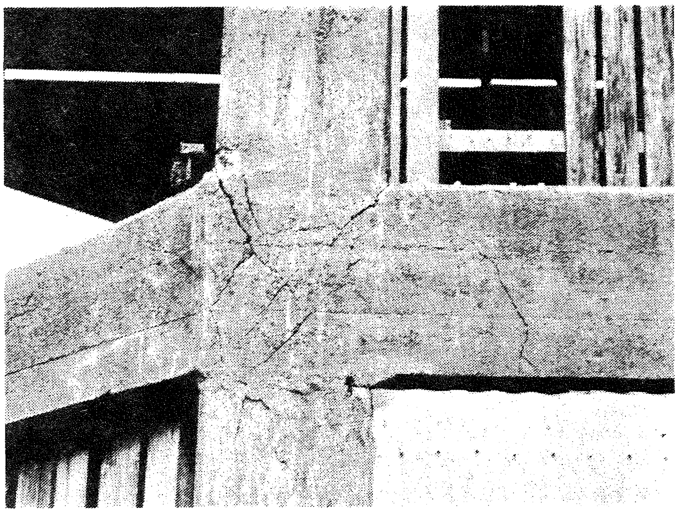


Fig. 4.2.8 : Water tower beam/column joint hinging

joint showed signs of beam hinging in both directions, with minor spalling of beam cover concrete. The structure remained stable but is expected to be demolished.

4.2.8 Bay Milk Products Ltd. - Dairy Factory

4.2.8.1 Overview

The dairy factory is situated on the outskirts of Edgecumbe, immediately alongside to east bank of the Rangitaiki River and about 2 km from the main rift. The company employs 400 people in the manufacture of butter, milk powder and casein, whey products and treatment of milk. One of these products is distilled into alcohol at an adjacent plant. Damage was extensive, though concentrated mainly on thin wall tanks (see Section 6), older buildings and foundations. New buildings, properly designed for seismic forces, generally performed as intended and clearly demonstrated the adequacy of current design theories. However, the earthquake was unforgiving and sought out and exposed any deficiencies in detailing.

The western side of the factory site suffered significantly more structural damage than the eastern side. This relationship is due principally to the simple fact that the western site was more extensively developed but there was also a more extreme ground distortion on that side due to the proximity of the river and the phenomenon of lateral spreading (refer Section 3.5).

4.2.8.2 Foundations

The subsoil generally consists of various sand layers extending to some depth and this has been described in detail earlier in this report (Section 1.5).

Shallow footings were racked and underwent differential settlement, which was reflected in damage to the supported superstructures.

Piled foundations generally continued to support their gravity loads though many underwent significant hinging at the pile cap and possibly deeper. Although the whole landscape was downthrown by about 1.5 - 2.0 metres the ground surface settled about 300 mm more than the founding level of sand, some 15 m below surface, which left pile supported structures significantly higher than surrounding pavements, and in a few buildings resulted in collapse of unsupported floors or shallow plant footings.

4.2.8.3 Storage Warehouse

This building suffered extensive damage due to toppling of stacked produce.

The building consisted of a long series of simple steel portals. Longitudinal bracing was cross braced steel flat with turnbuckles in the plane of the columns and across the roof in various bays.

A 200 thick concrete block fire wall ran down one of the long sides, supported laterally by small angle fixings to the steel portals. The rest of the cladding was corrugated steel sheet.

The building's contents was bagged produce (understood to be milkpowder) on pallets with bags seven high on each pallet, and pallets stacked four high. The stacks had shifted and toppled, and surely would have resulted in loss of life if the building had been occupied at the time. The produce had fallen against the end wall pushing the building sideways. The cross bracing was inadequate for such loading and broke away at the fixings. The corrugated sheeting then held the building up preventing total collapse, Fig. 4.2.9. The displacement of the columns ruptured the fire wall fixings, Fig. 4.2.10, and the fire wall then collapsed. It was noticed that the fire wall was very lightly reinforced and that many of the reinforced cells were not properly grouted, Fig. 4.2.11.

While it is unlikely that any bracing normally put in a simple portal industrial frame would withstand the type of loading that occurred here, a more robust and ductile type detail would have aided the building to suffer less damage. Indeed, it would seem that for warehouses being used in such fashion some consideration should be given to the effects of the contents toppling, either from earthquake or human error.

4.2.8.4 Pipe Bridge

An interesting collapse occurred with a particular pipe bridge within the milk treatment depot. Initial collapse occurred at a section where a tank fell upon it, which certainly would not come within a normal design loading. However, the longitudinal and lateral displacement thus induced brought about brittle failure of the welds at the base of the cantilever RHS stanchions well away from the point of impact. Had these behaved in a ductile manner, then consequential damage to the bridge and pipework would probably have been much less. The RHS posts were simply welded to the heavy base plates by fillet welds without gusset plates, stiffeners or the like. Although the base plates had been yielded, the welds had failed to generate the strength of the RHS and the welds tore progressively from a corner location without so much as distorting the RHS, Figs. 4.2.12 and 4.2.13.

A pipe bridge elsewhere in the plant with CHS columns gussetted to the base plate had performed well.

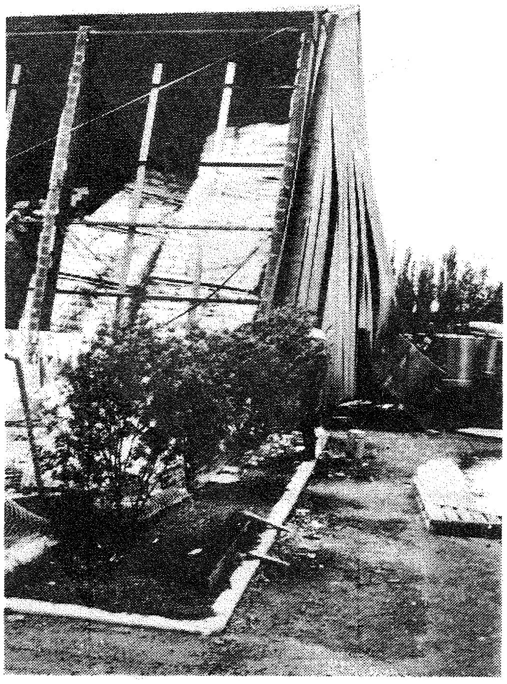


Fig. 4.2.9 : End wall of Milk Company warehouse

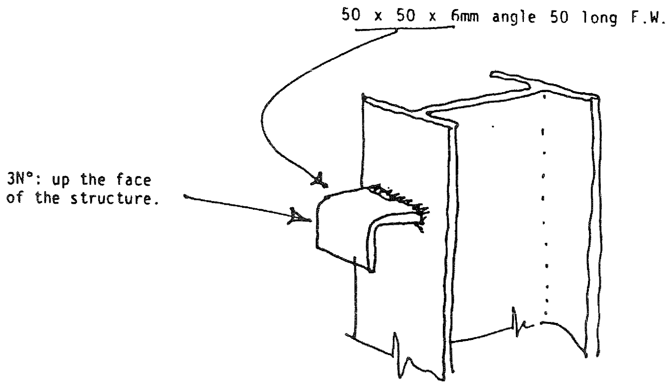


Fig. 4.2.10 : Detail of blockwork securing angle

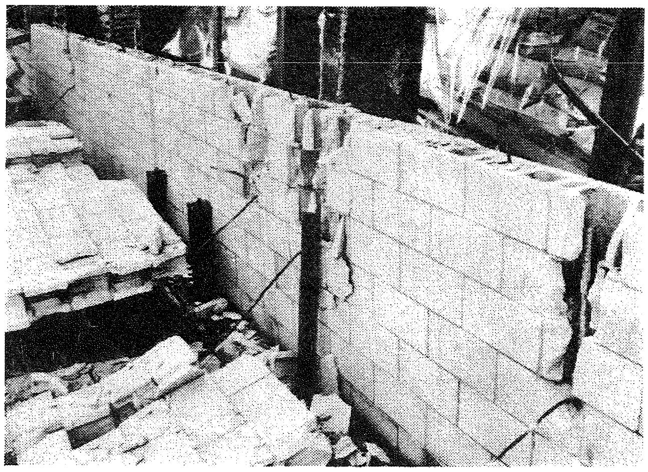


Fig. 4.2.11 : Collapsed fire wall. Note spacing of reinforcing and incomplete grouting of reinforced cells.

Fig. 4.2.12 : Pipe bridge - general view

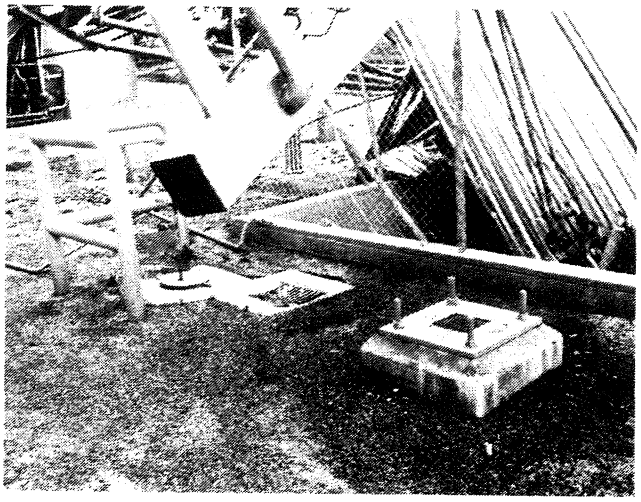


Fig. 4.2.13 : Pipe bridge - column and baseplate

4.2.8.5 Bunds

Most tanks within the Bay Products Ltd depot and the Distillation Company were unbunded. Where concrete or blockwork bunds were provided, they were usually ineffective due to damage caused by the tanks falling on them. Since bunds are generally to provide confinement of spilt contents in an emergency, they need to be able to withstand impact forces from tanks or be positioned beyond the fall line.

An earth bund at the distillation plant was undamaged in spite of impact.

4.2.8.6 Pallet Racking Systems (G. C. Clifton)

There were two fully loaded pallet racking storage systems both of which collapsed. These were one each in the cold store and cool store.

The system was a standard demountable cold-formed drive-in system, situated down each side of the building adjacent to a central access aisle. To this system seismic bracing had been added, consisting of mild steel angled across the top and back face of each rack system. The racks were not connected to the enclosing portal frame.

Damage to the pallet racking was very severe, with partial or complete collapse and almost total spillage of contents. Collapse was due to column buckling at one level throughout the system, always immediately below a main cross beam. The cold-formed columns suffered local buckling over a length equal to the column width with resultant loss of load-carrying capacity.

The seismic bracing at the top of each separate system performed well, holding the top stable in plan, however the bracing at the rear failed, allowing the sidesway collapse observed. The rear bracing was welded to the main cross beams and had failed by pulling these off the columns, thus depriving the system of any longitudinal bracing and locally weakening the rear columns.

One method of potentially overcoming the method of collapse observed would have been to brace the racking system back to the main structure, through the seismic bracing in the horizontal plane at the top of the racks. As it was the racks collapsed against the structure, which contained the impact loading but only with considerable local deformation and damage to the insulation layer. Collapse of stored contents was widespread and would have severely threatened the life of anyone working in the building at the time of the earthquake.

4.2.9 Whakatane Hospital Multi-Storey Buildings
(A.J. Macdonald)

4.2.9.1 Introduction

This sub-section covers the results of an inspection made of the Whakatane Hospital Medical Services Block and the adjoining Tower Block. Described is the soil type which exists under the hospital buildings and how the foundations were originally constructed, then the damage sustained by the buildings.

4.2.9.2 Soil Properties

A report dated December 1965 indicated that the soils underneath the then proposed buildings were medium to fine oxidised sands and the groundwater table was approximately 15 feet below the ground surface. The sands underneath the buildings were comparatively loose and so measures were taken during construction to compact these sands. In the case of the Tower Block, vibro-flotation down to a depth of approximately 16 feet was used, whereas in the case of the Medical Services Block, a full basement was excavated below existing ground surface, surface, then a further excavation 5 feet below that was undertaken which was then backfilled with sand in layers and compacted to maximum density as the filling took place.

Both buildings were built on spread footing foundations on top of the compacted sand. In the case of the Medical Services Block, the bearing pressure used under spread footings was limited to 2 tons/ft^2 (215 kPa) per square foot in order to attempt to minimise the dynamic effects of an earthquake on the compacted sand sub-grade.

4.2.9.3 Structure

The structure of the Tower Block is of the shear wall type and hence the building is extremely rigid. It extends six floors above the ground with one basement and is approximately square in plan shape.

The Medical Services Block adjoining it is of the same height, and was designed to be lightweight so that the bearing pressure could be kept down to 2 tons per square foot. Consequently, it was designed as a beam and column structure with lightweight floors and is comparatively flexible. A seismic joint approximately 100 mm wide was constructed where the Medical Services Block abutted up against the Tower Block.

4.2.9.4 Damage

Generally, the building did not suffer any structural damage that could be detected. The majority of damage which occurred was superficial and took place at the location of the seismic joint between the two buildings. Some minor cracking of the first floor beam column junctions has been reported, which may indicate strains approaching yield values. On one floor the brass floor plate crossing the seismic gap dropped through the gap to the floor below. Other damage was confined to several ceiling tiles (refer earlier), and to flashings between the Tower Block and the Medical Services Block where one building moved relative to the other.

Examination of the concrete frame indicated it had behaved elastically.

Assessment of the relative movement between the Tower Block and the Medical Services Block indicated that a movement of approximately 75 mm took place between the two buildings, 25 mm of which remained as permanent set which may have been brought about by differential foundation movement.

Examination of the fixings for precast concrete panels on the east of the building was made. No discernible damage was apparent.

Some damage occurred to solid plaster which had spalled from the undersides of the escape stairs on the eastern end of the building.

Refer also to 'Architectural Observations', Section 4.2.2.

4.2.10 Riverslea Shopping Mall

This was a relatively modern building (about 15 years old), generally single storey but some two storey sections. We did not have access to the construction details but observed steel roof trusses acting as portals with universal columns, concrete encased. Foundations were spread pads. It is unknown whether the foundation system included tie beams between pads. The building was extensively damaged principally due to relative movement of the foundations. Construction joints in the floor slabs had opened up, some by as much as 100 mm. It is easy to be critical of a modern lightweight single storey building being so extensively damaged. However, there were eye witness reports of surface waves in the ground and there was ample evidence that the ground in this area had experienced extension or compression in locations in close proximity to each other. These phenomena are discussed in more detail elsewhere but must be kept in mind when considering the performance of large area structures.

4.3 TIMBER TANKS (R. J. W. Granwal)

A 250 cubic metre timber store tank near Edgecumbe failed spectacularly, Fig. 4.3.1. It was 3.6 m high by 10.3 m in diameter and had a butynol liner with no sole to the ground. Evidence is now accumulating that failure of a timber tank subjected to back and forth vibration can be due to a strong liner working its way under the staves and bodily lifting the tank. The liner will burst when the hernia becomes sufficiently large, hence ripping at the base. This is very likely to be accompanied by water surge attempting to detatch the roof diaphragm, leaving the barrel to collapse inwards by landing on one edge. This tank has now been reinstated with much reuse of the original materials.

However, there were many other timber tanks in the area (reported to be over 30) only three of which sustained any damage and only the Edgecumbe one with extensive damage. There were several examples of timber tanks having slid back and forth by about 90 mm.

Fig. 4.3.1 : Edgecumbe timber tank

4.4 STEEL AND STAINLESS STEEL TANKS (B.J. Davidson)

Most tanks inspected were those of Bay Milk Products Ltd. which is located at the edge of the town approximately 5 km from the epicentre of the earthquake. The company was the owner of a large number of thin wall stainless steel tanks which ranged in size from the smaller; approximately of height (h) = 1.5 m and a diameter(d) = 1.0 m to the larger h = 14.0 m and d = 4.7 m. The smaller tanks generally were used to contain acid and caustic material and the larger tanks were used for milk products and water.

The support method for the large tanks was essentially the same with details varying from tank to tank. Each tank sat on a concrete pedestal (sloping to allow drainage), Fig. 4.4.1, which was part of a piled foundation slab. The floor edge of the tank was radiused upwards (formed by rolling) and butt welded to the wall; a skirt lapped onto

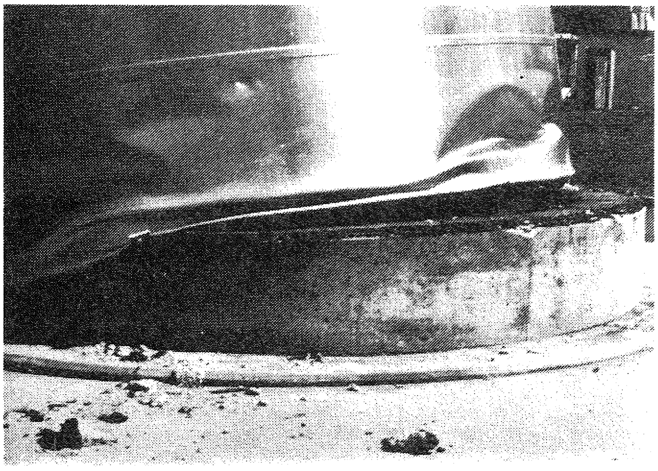


Fig. 4.4.1 : Typical tank pedestal

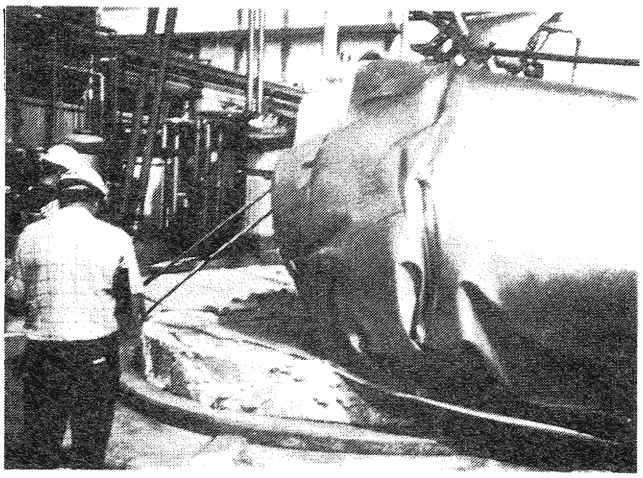


Fig. 4.4.2 : Pedestal with foam lining

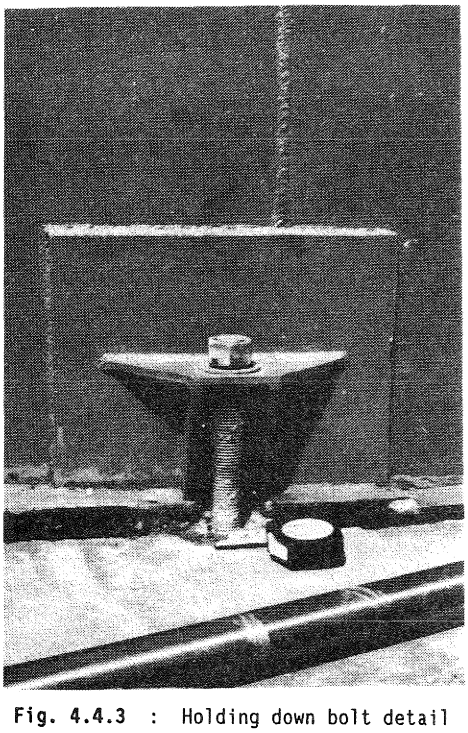


Fig. 4.4.3 : Holding down bolt detail

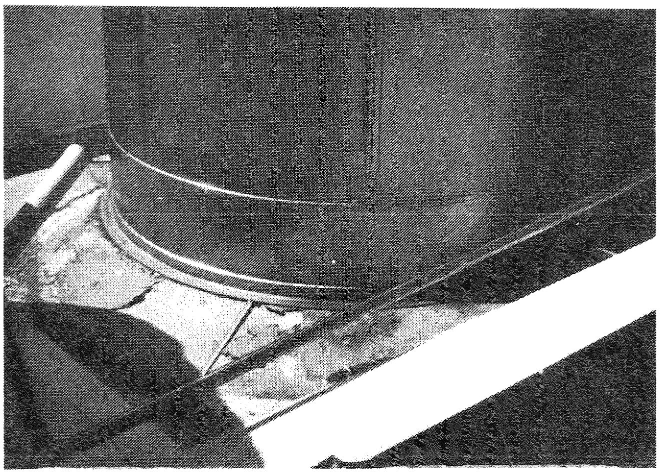


Fig. 4.4.4 : Horizontal tank slippage

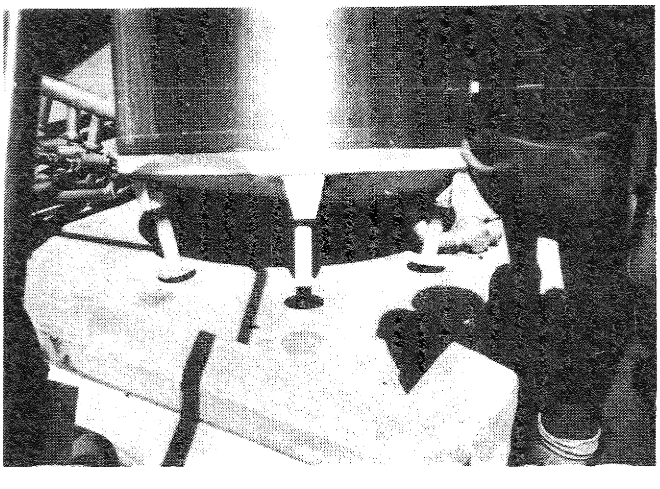


Fig. 4.4.5 : Damage to tank on legs

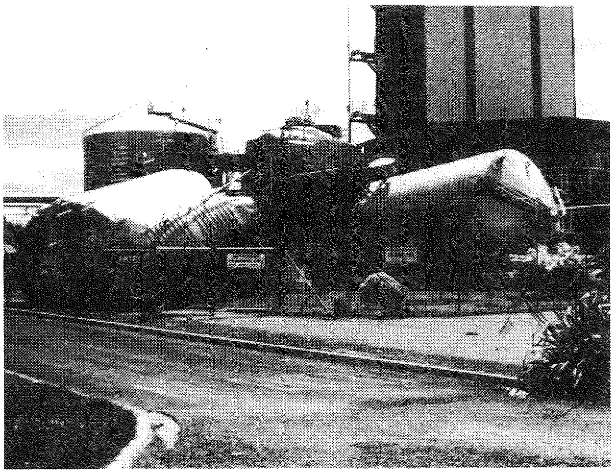


Fig. 4.4.6 : Bay Milk Products - overturned tanks

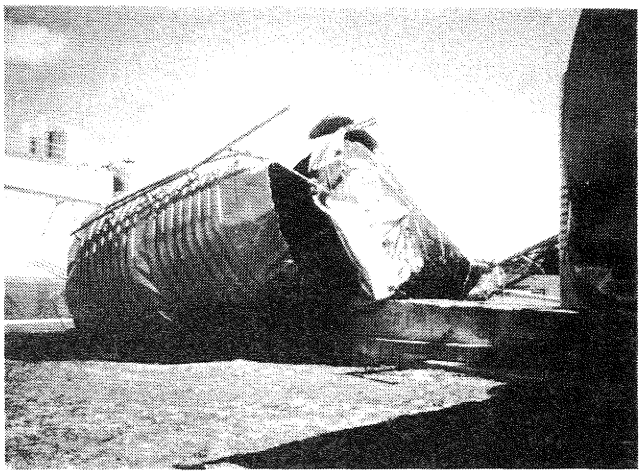


Fig. 4.4.7 : Bay Milk Products - overturned tanks

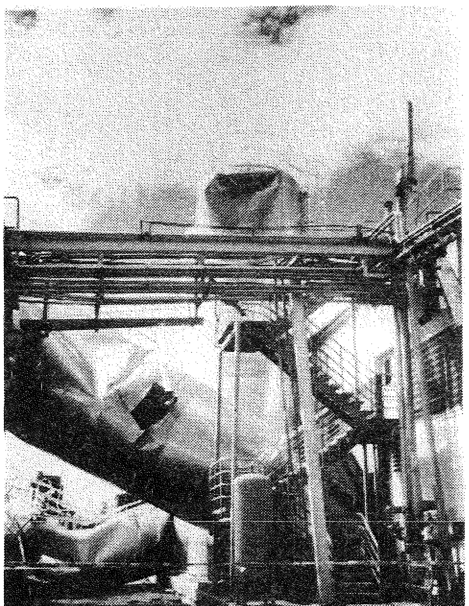


Fig. 4.4.8 : Bay Milk Products - overturned tanks

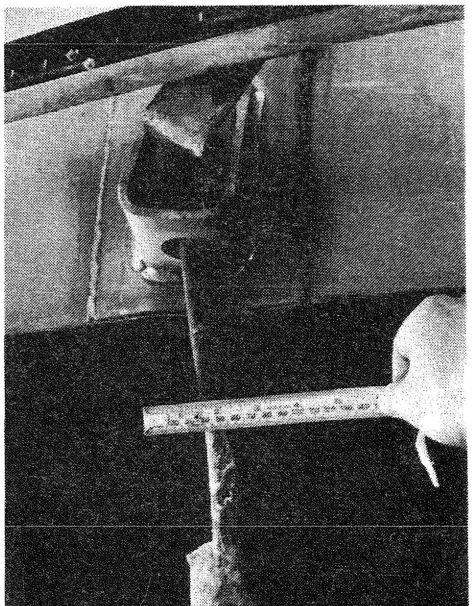


Fig. 4.4.9 : Holding down bolt, pull out failure

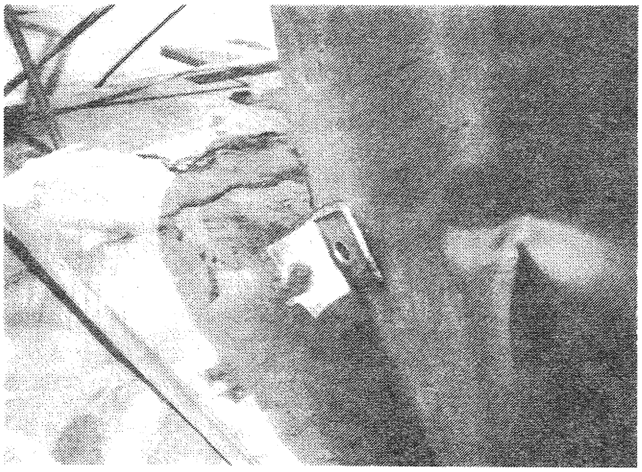


Fig. 4.4.10 : Holding down lug, ripped

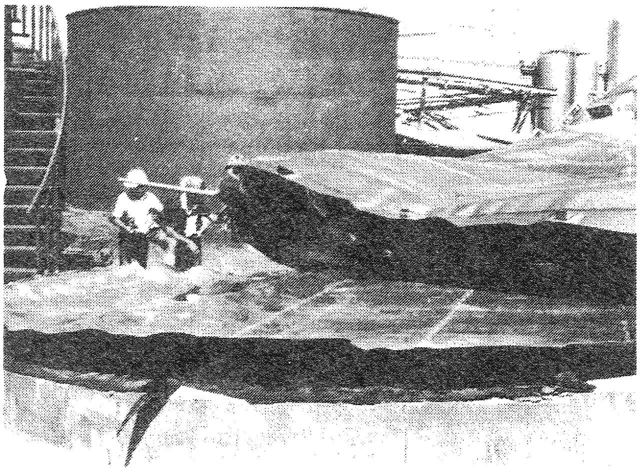


Fig. 4.4.11 : Shell of tank torn from base

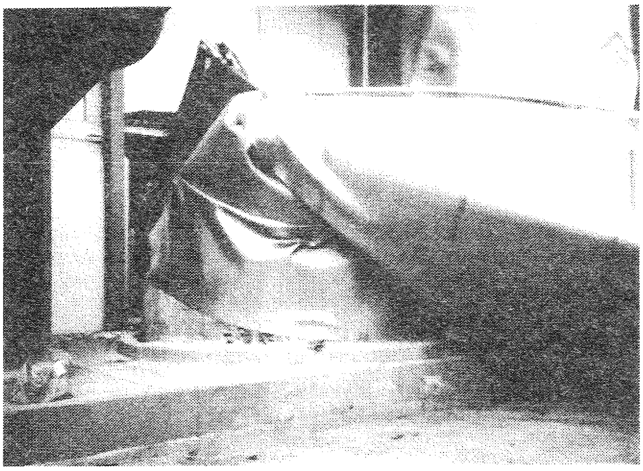


Fig. 4.4.12 : Cantilever collapse

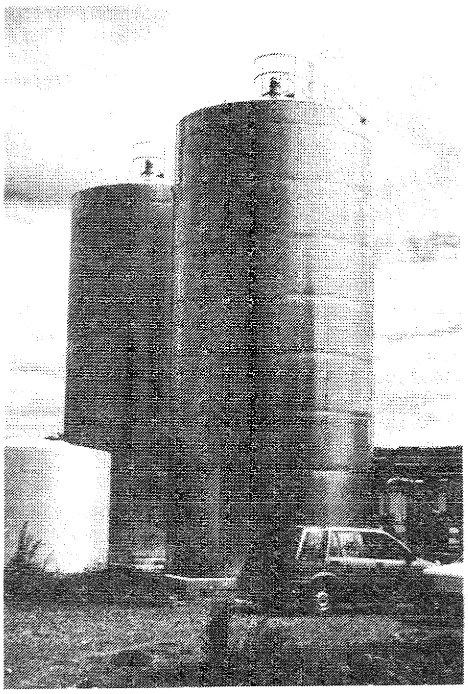


Fig. 4.4.13 : Omeheu Demonstration Farm tanks

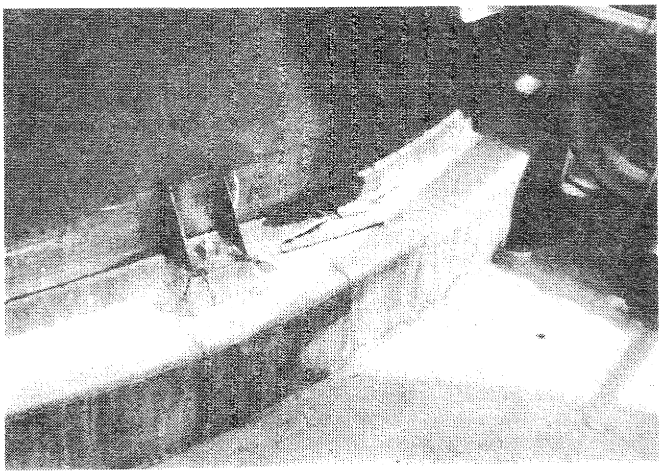


Fig. 4.4.14 : Base of tank at Omeheu Demonstration Farm

the wall and extended down to the foundation pad. Usually there was some insulation material (cork or foam plastic) between the metal floor and the pedestal, Fig. 4.4.2. The skirt in turn was fastened to the foundation slab by holding down bolts, Fig. 4.4.3. There were usually about 9 No. 20 mm bolts for the larger tanks. These bolts acted through lugs that were welded to the skirt (a variety of details), and then fastened into the concrete. The bolts were made mainly of mild steel, but some of stainless steel were used. Some tanks had the bolts cast in-situ or grouted in while others used a "terrier type" fastening.

The smaller tanks were generally free standing or supported on short legs, sometimes anchored and sometimes unanchored. These failed primarily in their method of support. Those free to slide, slid, Fig. 4.4.4. Those on legs, slid, tipped over, or tripped, bent a leg and then toppled, Fig. 4.4.5. The tanks that had tipped over suffered minor damage, often the connection area to a bent leg caused indentation to the tank wall. These tanks were, in the main, of robust construction and damage of a ductile nature was observed. The sliding behaviour as a means of base isolation appeared to have worked well for low aspect ratio tanks but not so well on taller tanks. Connection to piping was maintained, indicating strong connections and a flexible piping configuration.

The larger tanks, if they contained substantial fluid, collapsed; their behaviour during the earthquake being so violent that they often damaged nearby tanks and structure. Most of these tanks were lying on the ground, completely ripped off their bases with their contents lost, Figs. 4.4.6, 4.4.7, 4.4.8. Some were left standing, leaning on some supporting structure. It is difficult to categorize the sequence or primary cause of failure. For all tanks, the holding down detail had failed. It had failed in every possible way: the bolts (whether cast in-situ or of the "terrier" type) had been ripped from the foundation slab, the bolts had failed in tension and shear, there were cases of the nuts being pulled off the the threads; the bolts had pulled through the hole in the lug, Fig. 4.4.9, ripped the lug hole open, Fig. 4.4.10, or pulled the lug completely from the tank (refer to Section 4.4.3 for more detail). Parts of the tank walls were buckled; quite frequently the tank base had unzipped from the wall along the weld, Fig. 4.4.11; in cases the tank walls were torn; or had buckled in a cantilever fashion, Fig 4.4.12.

4.4.2 Omeheu Demonstration Farm

Located approximately 4 km west of Edgecumbe, this farm used two silos which were similar to many observed at Bay Milk Products Ltd., Fig. 4.4.13. These were constructed of stainless steel and had a support configuration using a pedestal, skirt and holding down bolts similar in detail, to those as described previously. There was some damage to these silos but it was only superficial. There was a little cracking of the mortar around the holding down bolts and possibly a small amount of indentation at the base of the tank due to axial compression forces in the walls, Fig. 4.4.14. These tanks were approximately 40% full at the time, for which calculations indicate the base moment would be only 12-15% of that for the full tank.

4.4.3 Hold Down Bolts (G. C. Clifton)

The following points were noted with respect to poor performance of holding down bolts due to failure of the bolts themselves or failure of the bolt anchorage into the foundation slab or pad:

(a) Pullout of bolts due to insufficient anchorage length for the bolt or anchorage into the supporting grout ring only rather than the foundation concrete.

(b) Pullout of bolts due to sufficient anchorage length and insufficient size of head on bolt. Instances of failure of headed bolts up to 16 mm occurred, even when a normal sized washer was placed over the bolt prior to embedment in the concrete. In one example the washer was extruded back over the head of the bolt during the earthquake and the bolt pulled out.

(c) Cracking of the foundation slab due to the holding down bolts being positioned too close to the edge of the slab and/or insufficient reinforcement being placed around the bolts to prevent cracking due to uplift - induced lateral tensile forces in the concrete.

(d) Tensile failure of the bolt initiated at a point of severe localised corrosion.

(e) Tensile failure of the bolt at the beginning of the threaded section (normal cut thread). This only occurred in stainless steel holding down bolts, whereas in mild steel holding down bolts considerable ductility was observed, even with normal cut threads.

(f) The nut pulling off due to thread stripping. This was only observed in stainless steel holding down bolts.

One aspect of poor hold down bolt installation that was observed was insuffient length of bolt as installed to allow at least 3 complete threads to protrude beyond the tightened nut. In one installation, no bolt was observed to protrude beyond the tightened nut at all and several nuts had only half their threads engaged. This example appeared however to have been designed to resist seismic-induced loading and had performed very well.

The following points were noted with respect to poor performance of holding down bolts due to failure of the cleat connecting them to the main structure (in all cases connection to silos).

(g) Failure by cleat tearing due to insufficient edge distance to the bolt.

(h) Excessive cleat distortion due to an apparent lack of design for uplift (seismic-induced tensile) loading.

The following aspects of cleat design were noted as being less than satisfactory even though they did not contribute to poor performance of the particular item during the earthquake.

(i) An angle cleat formed from two fault sections connected with a one-sided fillet weld.

(j) Mild steel or galvanised steel plates welded to stainless steel silo walls, thus causing accelerated corrosion of the cleat due to galvanic action.

(k) Non-passivated welds to stainless steel members leading to accelerated corrosion along the weld line.

4.5 PLANT AND EQUIPMENT (G. C. Clifton)

Damage to plant and equipment designed to resist seismic action by way of direct anchorage to piles and/or through seismic braces into the surrounding structure was on the whole, minimal. Shearing of bolts in some seismic braces occurred, especially in one instance where the strength of the diagonal brace from machine to floor in tension/compression greatly exceeded the strength of the bolts in shear. The bolts at both ends of the brace sheared through; those at floor level were replaced and the upper end of the brace fillet welded to the machinery. This repair had been undertaken on March 3 and had withstood all aftershocks to the date of inspection, however this should not have been considered as a long-term solution.

Damage to plant and equipment either inadequately anchored or not rigidly anchored to the foundation/ground slab appeared to be much more extensive. This was mostly due to considerable horizontal movement of equipment, with resultant fracturing of piping and ducting. However one company commented that the observed damage to piping etc. was in fact less costly and time-consuming to repair than the damage that would have resulted to the working parts of equipment rigidly anchored and thus exposed to greater seismic forces. In this case a conscious decision was made to leave equipment free to move to save the equipment at the expense of the connecting piping and ducting.

There was one piece of equipment observed mounted on commercial seismic isolation pads. This was a computer hard disc unit. General damage to the surrounding equipment meant that it had not been tested at the time of inspection.

One interesting observation was the considerable ductility shown by stainless steel piping, some of which was flattened and/or severely twisted without fracture.

Screw thread couplings in piping showed examples of non-ductile failure at the threaded joints, especially where a coupling was situated on or immediately adjacent to a change of direction of the pipe. One PVC pipe carrying chiller fluid sheared clean through at the base of a chiller unit in a completely brittle failure.

5 BRIDGES

5.1 GENERAL (T.W. Robertson)

The most common form of failure that occurred with almost all bridges in the locality, both road and rail, was settlement of the soil behind the abutments. Virtually all bridge superstructures survived with only minor damage, but in many instances the bridge was put out of use by this settlement. The solution to this problem is not as obvious as it may seem. In the author's opinion, it is unlikely that the earthquake caused extra compaction but more likely that lack of confinement of the soil plus abutment movement allowed the settlement to occur. Perhaps the most striking aspect of the reconnaissance trip was the speed with which the local community responded to pick itself up and particularly this was so with the reinstatement of bridge approaches. To design for confinement of the backfill soil would be expensive. This expense needs to be weighed up against the relative ease with which the bridges were recommissioned.

Kawerau bridge suffered damage mainly due to displacement of the west embankment pressing against concrete piers both cracking and displacing them.

5.2 TE TEKO BRIDGE (R.I. Skinner and H. E. Chapman)

Throughout the region of severe shaking, during the 2 March 1987 Edgecumbe earthquake, the above road bridge was the only one which was provided with isolation of the superstructure. Because of a construction deficiency it lost the action of one of the two laminated-rubber bearings at the western abutment. The resulting unbalance and reduced horizontal constraint caused moderate structural damage. In other respects the bridge performed in accordance with its design criteria.

The earthquake severity, as measured by an accelerometer at the Matahina dam some 10 km south of the bridge site, was comparable to the NS component of the 1940 El Centro earthquake. Flexible ground surrounding the bridge site may have increased the severity of the longer period accelerations, those from 1.0 to 1.5 seconds, which would have dominated the earthquake response of the bridge.

The reinforced-concrete bridge crosses the north-flowing Rangitaiki river just east of Te Teko. The five 20 m spans used precast U-Beams with a cast-in-place deck and end blocks. Span ends are connected by cast in rubber sheathed linkage bars in the deck slab. Each end of each span is supported by a pair of cylindrical laminated-rubber bearings, spaced transversely at 4 m. Each pier has a single cylindrical column with an upper cross-beam which supports the four lead-rubber bearings and a lower cap supported on 13 m raked piles.

The two laminated-rubber bearings which were supported on the pier capping beams at each abutment were raised on mortar pads about 50 mm high. Each end of every bearing was surrounded by a retaining ring to prevent horizontal sliding. The retaining rings were 20 mm deep and had a radial clearance of 20 mm from the bearings.

As designed; all retaining rings had an effective height of 20 mm.

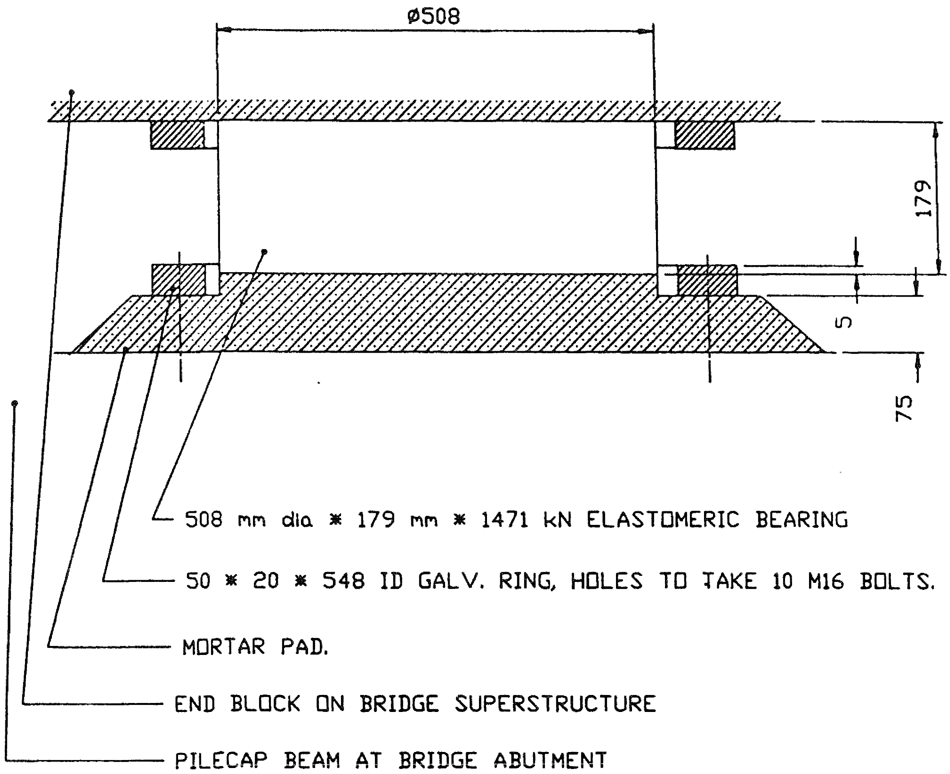


Fig. 5.2.1 : Te Teko bridge lead-rubber bearing - as installed

As constructed; a 10 mm to 15 mm upstand was included on the two mortar pads at the western abutment, as shown in the accompanying drawing, which reduced the effective height of the lower retaining rings to 5 mm to 10 mm, Fig. 5.2.1. The effective depth of retainment was a little greater for the downstream bearing.

Comment; A retainer height of 20 mm is the minimum required for the bearings of this bridge. Some structures require greater constraints on the ends of their isolating bearings.

Along-stream earthquake accelerations would cause non-resonant rolling forces on the superstructure. A peak acceleration of 0.33 g would change the bearing vertical loads by about ±20%. Vertical accelerations would further increase these load changes, but the greatest reduction is most unlikely to have reached -50%.

As the superstructure moved upstream it evidently dislodged the down-stream bearing only from its lower retainer. An upstream lurch of the superstructure would have increased the load on the upstream bearing and reduced the load on the downstream bearing favouring dislodgement of the latter. Had the along-stream ground accelerations been reversed in direction it is thought that the upstream bearing would have been dislodged instead.

During the earthquake the bearing moved from its installed position to one which was 800 mm upstream and about 50 mm lower (having shifted off its supporting pad with upstand).

A preliminary estimate indicates that the loss of one western-abutment bearing would have increased along-stream motions at this end by about 25% and at the eastern abutment by about 10%. There was some loss of cover concrete towards the base of the single column on the western pier, with lesser effects on two other pier columns. The column damage was associated with upstream motion of the dominantly transverse mode of the unbalanced superstructure. If the action of one abutment bearing had not been lost it is unlikely that the columns would have suffered any visible cracking.

The cross-stream motions of the superstructure pushed the abutment knock-out (fuse) beams about 75 to 100 mm from their installed positions. The bridge foot-path at the western end was cracked, apparently by an upward load, presumably due to over riding the abutment wall.

The bridge was rendered serviceable by jacking the western end of the superstructure and replacing the laminated-rubber bearing, by repositioning the knock-out beams and by filling and compacting the approach material displaced by these beams.

6 MUNICIPAL SERVICES

6.1 NATURAL GAS LINE (A. G. Smaill)

The compressed natural gas pipeline, which runs between Awakeri and Edgecumbe, crosses the main fault trace at roughly right angles to the strike. The pipeline suffered very little damage. Deformation of the pipeline was confined to gentle warping over a relatively wide fault zone.

6.2 WATER SUPPLY (J.L. Morrison, J. Hunt and P. D. Leslie)

6.2.1 Intake and Trunk Mains

The main water supply for the town of Edgecumbe (population 2,000) comes from Braemar Springs, approximately 11 kilometres west of Edgecumbe, near the Tarawera River. The scheme has a capacity of 90 litres per second, and supplies a large rural area as well as the town of Edgecumbe. The scheme was completed in 1984.

The town of Te Teko is on a separate water supply system, also supplied by springs, with a capacity of 20 to 30 litres per second. This system was not investigated.

At Braemar Springs, a main pumping station boosts the flow in the pipeline. This pumping station was undamaged. The delivery pipeline consists of asbestos cement pipes, with a maximum diameter of 375 mm.

Damage reported on this pipeline was related to compression failures at pipe joints. According to one Contractor, the 11 km of pipe is now about 6 metres shorter. Damaged sections of pipe were cut out and replaced as appropriate.

The Braemar water supply is not chlorinated. Emergency chlorination equipment was supplied following a request to the Ministry of Works and Development at Trentham, who provided assistance with its installation.

6.2.2 Reticulation Water Mains

The reticulation water mains in Edgecumbe were generally laid in 1968, prior to the commissioning of the Braemar scheme. The pipes initially installed were asbestos cement, 100 mm diameter, laid to a cover of about 750 mm. These particular pipes have a socket glued to the pipe barrel, with a rubber ring for connection to the next pipe. Subsequently other watermains were installed, of PVC or asbestos cement.

Damage to the reticulation water mains was major. Because of the considerable ground compression in the area, pipes have failed at their joints. In many cases pipes have been squeezed together by up to 250 mm at the joint, but generally 100 mm, causing the collar to split. There were no reported instances where pipes had failed by being pulled apart. The failure locations for pipes did not seem to be related directly to ground deformations, which were almost all of a cracking nature. No explanation was obvious as to why the pipes should have suffered this type of compression damage.

Repairs were effected by cutting out the damaged joint, and replacing with a piece of asbestos cement pipe, and coupling this back into the water main, Fig. 6.2.1. Additional line valves were installed at appropriate locations. In some streets, if damage was too extensive, a complete relaying of the water main was undertaken, Fig. 6.2.2.

As repairs were completed, no attempt was made to locally chlorinate the water main in the area of the repair. In some areas of the town, 100 mm PVC water main had been used. These generally performed quite well, but conversely the area in which they were laid had suffered less damage.

6.2.3 Rider Mains and Service Connections

Rider mains in Edgecumbe were either galvanised iron or 50 mm asbestos cement, laid at a cover of 300-400 mm. These mains, particularly the latter, performed very poorly. In many areas, rider mains were being replaced by 50 mm dia. PVC pipes, with glued joints. These replacement rider mains were being installed very rapidly in the grass berms, Fig. 6.2.3.

Service pipes were generally polyethelene. Some damage occurred at the connection of the service pipe to the reticulation water main.

The damage to the shallow rider mains may have been caused by the considerable effect of the surface wave from the earthquake.

6.2.4 Reservoirs and Pumping Stations

There was no report of any damage to main concrete storage reservoirs at Whakatane or Kawerau. Edgecumbe has no concrete reservoirs.

A timber stave water storage tank, of 250 m³ capacity, 10.3 metres diameter by 3.6 metres high at the Putiki Booster Station just east of Edgecumbe failed completely. It appeared that the tank or its roof had flattened the north side compound fence, before discharging its contents in a southerly direction. This flow of water completely displaced a galvanised steel shed on the adjacent gas regulator station site.

A small modern timber framed pumping station building immediately adjacent to this timber tank appeared to have suffered no damage either to the structure or to the installed pump set pipework and control valves. Other timber tanks throughout the district were apparently undamaged.

6.2.5 Pipelines on Bridges

Damage to pipelines on bridges was generally caused by the settlement of the bridge approaches or by relative movement between the deck and the abutment.

At the Rangitaiki River bridge at Edgecumbe, the approach had settled considerably. A 150 mm dia. A.C. pipe in a duct under the bridge footpath lost its support adjacent to the abutment, and had to be reconnected.

At the Landing Road bridge at Whakatane, a 250 mm diameter steel rubber ring jointed pipe which was supported from the bridge deck without upwards restraint fixings passed through an open sleeve in the abutment. Settlement of the approach to the bridge has forced the pipe downwards, causing it to pivot about the abutment and consequently rotating the first rubber ring joint under the bridge. This joint has been rotated by about 15 degrees without leaking. The abutment of this bridge appears to have moved both longitudinally and laterally during the earthquake, Fig. 6.2.4.

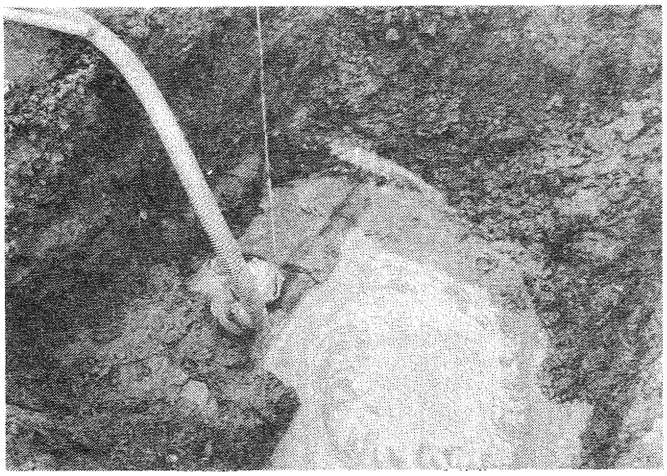


Fig. 6.2.1 : Repair to asbestos cement water main

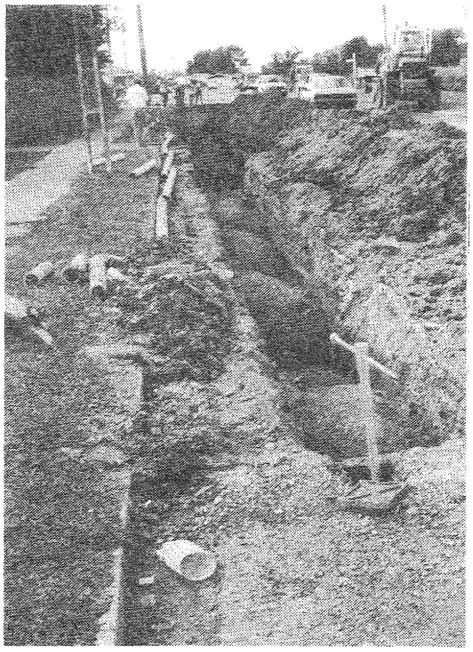


Fig. 6.2.2 : Relaying of extensively damaged water main

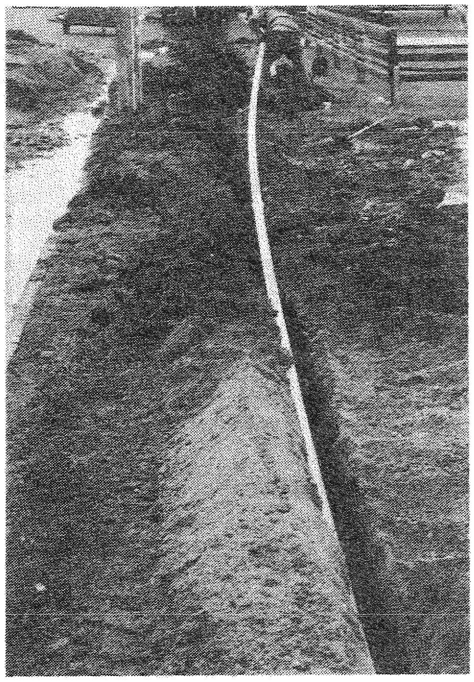


Fig. 6.2.3 : Replacement of water rider mains

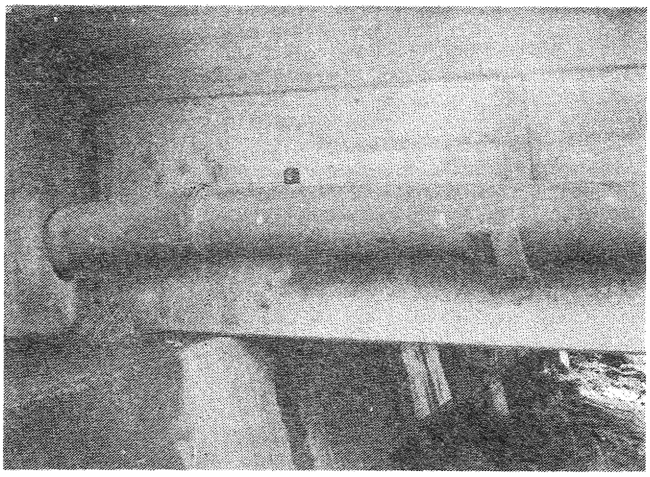
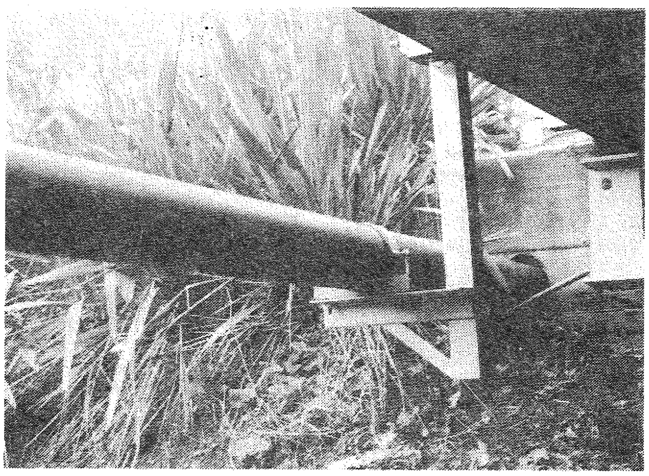


Fig. 6.2.4 : Pipeline under Landing Road Bridge

Fig. 6.2.5 : Sewer pipe on Kawerau Bridge



At Kawerau, the bridge over the Tarawera River adjacent to the Borough Works Depot suffered some damage to the services. The bridge deck appeared to have moved about 100 mm towards the left bank of the river. The 250 mm diameter steel water main underneath the bridge passed into a sleeved hole in the abutment. The movement of the deck taking the pipe with it has caused the pipe to jam on the top of the sleeve. Efforts were being made to enlarge the opening to reduce the loading on the pipe. Also on this bridge a 250 mm dia. steel sewer was suspended by angle brackets from the side of the bridge. The pipe was seated in shaped timber blocks, and was well secured in position. According to the workmen on the job, the end bracket was badly deformed and the pipe jammed against the sleeve. Repair work consisted of straightening the bracket, and reducing the height of the timber support block under the pipe by about 100 mm, while keeping the sewer in service, Fig. 6.2.5.

6.2.6 Land Drainage

Much of the Eastern Bay of Plenty has very minimal grades for drainage. The Rangitaiki Drainage Board has about 35 land drainage pumping stations. These pumping stations typically have a 450 mm or 600 mm discharge pipe. In many cases, there was no flexible joint installed between the pumping station building and the pipeline, and failures have occurred at these locations. The pumping equipment in these pumping stations generally performed satisfactorily. At headwalls of culverts, in many cases the culvert has separated from the headwall.

6.3 SEWAGE

6.3.1 General

Damage to these could not always be directly correlated to the degree of surface damage and varied from street to street.

The central/western part of Edgecumbe suffered the worst damage with Te Teko suffering some damage, but to a much lesser extent, but there was little or no damage to underground services elsewhere.

6.3.2 Sewers

There was very serious damage to pipes in Edgecumbe which were mostly asbestos with fixed collars and rubber ring joints of fibrolite type mid-1960's construction and mostly 150 mm dia. (but some 200 mm). Generally, these were found still to be usable, as they had retained their relative concentricity except for some serious compression failures at the collars caused by foreshortening, rather like the effect of driving a mandrel into the ends causing some degree of splitting, on average at least 12 joints per 100 mm. Several hundred metres of 150 mm earthenware pipe is scheduled for complete replacement during the 1987-88 summer because nearly every individual pipe was damaged. This was determined by T.V. camera check of all mains as they were brought back into use. Heavier infiltration will be accepted meantime, but further repair work will be carried out at fracture points in due course (on the basis of video tape record of damage) and probably as blockages occur.

6.3.3 Pumping Stations

There were 10 of these incorporating Flygt subersible pumps with wedge type fittings and operating with a series of short lifts into sequential gravity sections forming the next stage of the sewer reticulation system. These were found to be undamaged and are being brought back into service stage by stage as checked.

6.3.4 Manholes

There was some slight upward movement of these (25 mm), with some spalling of surface blocks, but apparently little or no shearing from the pipes themselves.

6.5 DOMESTIC ELECTRIC POWER

About 10% of house leads had pulled off and were restored by the Power Board who also inspected every house and carried out minor repairs where necessary (e.g. broken hotpoints) before people were allowed to return.

Power was restored last week after a maximum of three days in any area.

7 ELECTRICAL SUPPLY INSTALLATIONS

7.1 DAMAGED EQUIPMENT AT EDGECUMBE SUBSTATION

(A. L. Rutledge, L. T. Pham)

7.1.1 Control Room and Workshop Area

7.1.1.1 Control Room

Three rows of 12 control panels fell forward. These panels are 450 mm depth x 2140 mm height and the width varies from 600 mm to 760 mm. On reposition, the cable connection appeared sound and little damage was done to the perspex cover of the instrument inside. Most instruments inside were found to be in working order.

The panels fell because they were not secured properly to the ground and anchored against the wall. Other control panels at the 45 ton workshop end of the control room supported on Unistrut stayed intact.

7.1.1.2 80 Ton Workshop

Broken windows are the only visible sign of damage to the building. The 80 ton crane can only travel in one direction but appears to have suffered no damage and is reported to have rolled along the rails and smashed into the limit switch.

Inside the workshop, three unsecured spare Rade Koncars (out of six) still in their shipping wooden crates fell over but the CT's appeared to be undamaged. Crates of tools, nuts, bolts and other odds and ends also fell over.

7.1.2 220 kV Structure

7.1.2.1 ASEA Capacitor Voltage Transformer (C.V.T.) - Kawarau No. 3 Line

The red phase ASEA C.V.T., type CT08 1/22 of the Kawerau No. 3 line broke at the base of the capacitor's support casting. The yellow and blue phase C.V.T's remained intact.

7.1.2.2 Oerlikon Circuit Breaker - CB 482 - Tarukenga No. 2 Line

All six porcelain support columns of the Oerlikon 220 kV, type FS 9C1 circuit breakers broke.

This failure is a classic bending failure of a weak porcelain column supporting a heavy interrupter head at the top. The Oerlikon breaker is one of many circuit breakers which was due to be earthquake strengthened.

7.1.2.3 Balteau Current Transformer

The broken interrupter heads on the blue phase of the Oerlikon circuit breaker strained the Balteau type SEX 200 and caused considerable distortion to the base of the CT.

At the other two poles, the interrupter heads fell off completely and pulled the flexible connection to the CT's through the clamp. As a result, the yellow and red phase CT's did not experience the same strain.

7.1.2.4 T4, T5 Transformers Bank (Fig. 7.1.1)

The tall and very heavy (149300 lbs) Westinghouse single phase 50/3 MVA transformers survived the earthquake. There are seven transformers. Six form the T4, T5 bank and one spare. The transformers did not come off the rails but the holding down brackets (designed in 1948) suffered considerable distortion.

These transformers survived because the brackets are made of heavy gauge steel plate (3/4") and the bolts are fairly substantial (7/8 and 1" dia.).

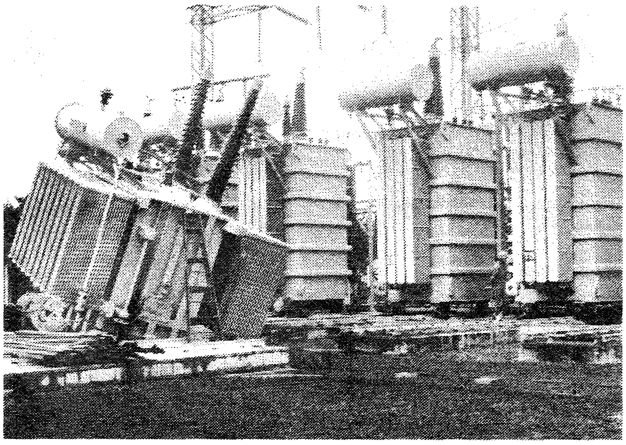


Fig. 7.1.1 : Damaged Savigliano 220 kV 16667 MVA Transformer

7.1.2.5 T6 Transformers Bank

The three British Thomson Houston transformers which form the T6 bank are the heaviest transformers (total weight 87 tons). The failure of the 3/4" (20 mm) bolts which secured the holding down brackets onto the rails allowed the yellow and red phase transformers to move along the rails and ram the radiator support structure. The total movement was about 860 mm. The movement broke the oil pipe connecting the radiator and the transformer. Note that the blue phase transformer stayed intact.

The 3/8" thick brackets were also badly distorted.

It is obvious that the 3/4" bolts do not have adequate strength. The large clearance between the 1" dia. hole in the brackets and 3/4" bolts would tend to increase the shear loading by dynamic impact.

7.1.2.6 T8 Transformer Bank (Fig. 7.1.2 and 7.1.3)

All three Savigliano transformers plus the spare came off the rails when the holding down arrangements failed completely. All of the 3/4" Whitworth studs securing the holding down brackets to the wheels failed in shear as shown in Fig. 7.1.3. The studs would have been subjected to very high shear loads under horizontal earthquake load in the direction of the rails because of the lever action of the relatively long bracket on the narrowly spaced studs, Fig. 7.1.4.

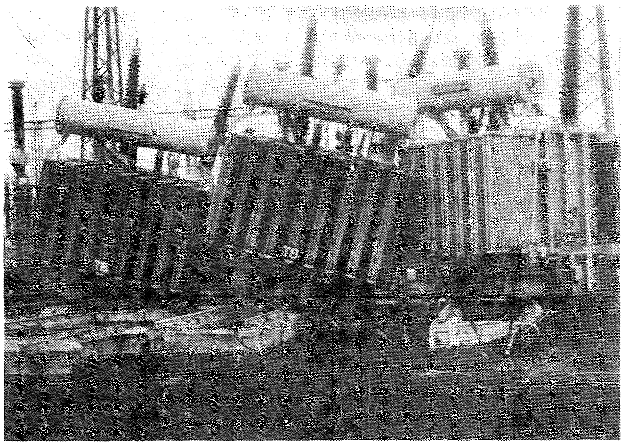


Fig. 7.1.2 : T8 Bank of Savigliano 220 kV transformers

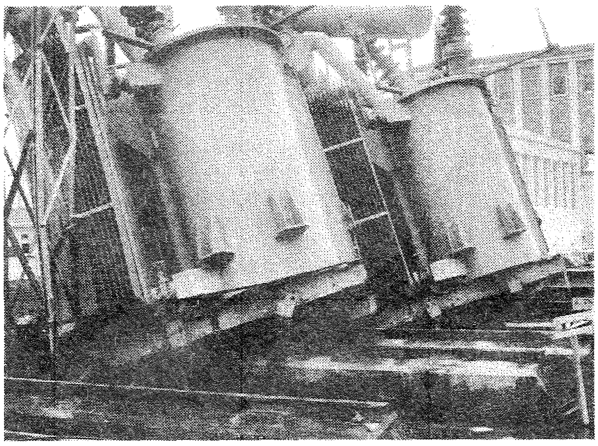


Fig. 7.1.5 : Two of the three Met-Vickers 110 kV 3333 MVA transformers. Note hold down bracket.

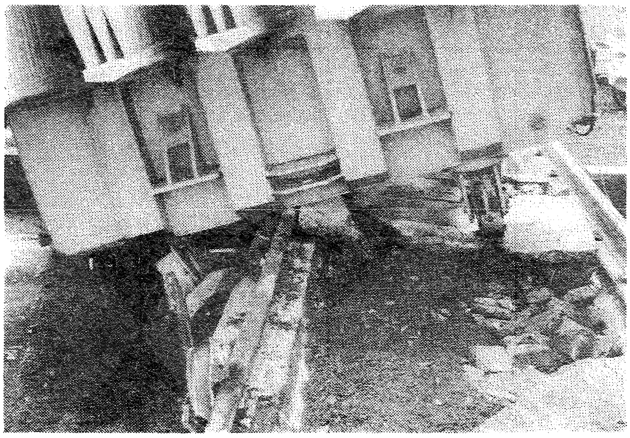


Fig. 7.1.3 : Close up view. Note twisted holding down bracket

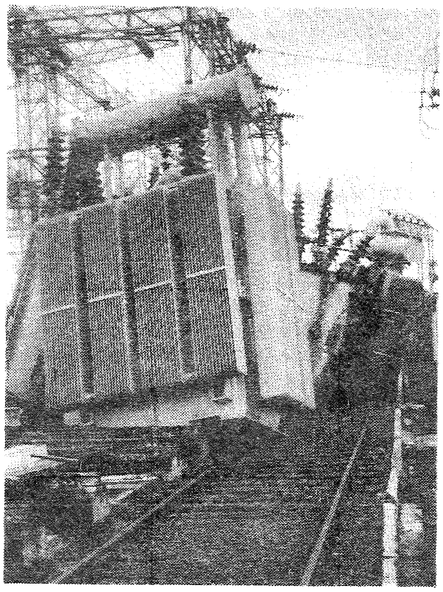


Fig. 7.1.6 : Spare transformers bank in the 110 kV yard. The Feranti in the foreground and the three Metropolitan Vickers in the background.

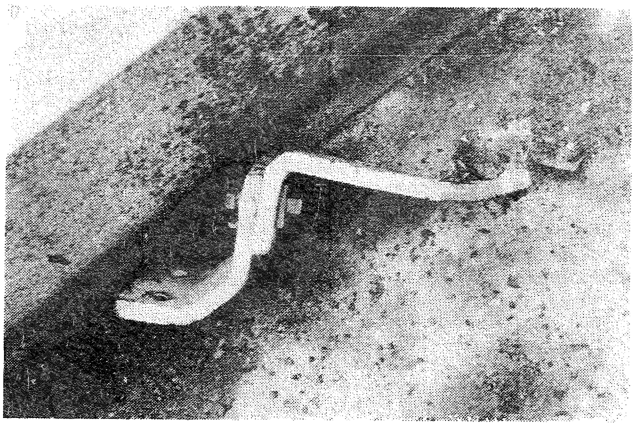


Fig. 7.1.4 : Holding down bracket shows evidence of prising of fixing bolts

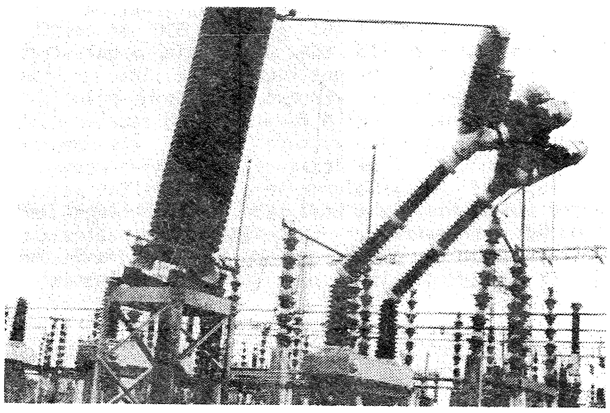


Fig. 7.1.7 : 220 kV Oreliken CB-Blue Phase. Red and yellow phases were on the ground.

7.1.3 110 kV Yard

7.1.3.1 Metropolitan-Vickers 110 kV 3333 kVA Transformers (Fig. 7.1.5)

All three transformers sheared the 5/8" bolts securing the holding down brackets to the undercarriage and the rail. As a result the transformers came completely off the rails.

The original brackets are quite tall compared with the horizontal spacing of the 16 mm dia. bolts and horizontal earthquake load in the direction of the rails would subject these bolts to very high shear forces.

7.1.3.2 Ferranti 110 kV 5000 kVA Transformer (Spare)

The 16 mm dia. bolts holding the transformer to the undercarriage sheared and the transformer slid off the undercarriage and came to rest against the adjacent Metropolitan-Vickers transformer.

The resistance of the 16 mm dia. bolts to horizontal shear was considerably decreased by a 50 mm thick spacer between the transformer base and the undercarriage.

7.1.4 Conclusions

Most of the damage at Edgecumbe was to transformers that do not appear to have been designed to current Electricorp earthquake standards for electrical equipment. For example, the Savigliano earthquake clamps were designed in 1967 and the clamps for the BTH were designed in 1953. The Ferranti clamps date back to 1951 and the Westinghouse clamps are even earlier. On the other hand the Osaka holding down design was installed in 1977 and survived well.

A more detailed analysis of the failures will be carried out but at this stage it would seem that if the holding down brackets had been upgraded to current E.D. standards for earthquake resistant design it is likely that most of the damage to transformers would have been avoided.

7.2 EARTHQUAKE DAMAGE AT KAWERAU SUBSTATION

(A. L. Rutledge, L. T. Pham)

7.2.1 Control Room and Control Panels

Three control panels out of 35 or so panels fell over. The panels are of modern construction - 600 mm depth x 2200 height and 600 to 800 mm width. They are secured to the floor via a uni-strut support system but are not tied at the top. This type of support is current ED design practice. However, the panels which fell over had counter-sunk head screws without washers whereas the proper design employs 10 mm hexagonal bolts with washers. The force of the earthquake tore holes in the panels where the counter-sunk head screws went through and toppled the panels. Fortunately, the cables and instrument in the panels appeared undamaged and the panels were repositioned soon after the earthquake.

7.2.2 220 kV Switchyard

7.2.2.1 CB622 - Merlin Gerin 220 kV FA2 Circuit Breaker

The blue phase of CB622 was totally destroyed. The support insulator broke - not at the base above the stand where the bending moment due to earthquake forces is highest but about half way up. Signs of arcing between the interrupter housing and the stand are clearly visible as the interrupter head fell down.

This breaker was installed in 1979 and its design met ED current earthquake design specifications. Its failure is unexpected and a detailed investigation should be carried out to establish the cause.

7.2.2.2 Rade Koncar Current Transformers (CT)

The six Rade Koncar CT's on the north side of the switchyard all leaked oil. One Rade Koncar CT on the south side - near the T12-T13 banks leaked oil badly but the other five were unaffected. A close inspection of the affected CT's did not show any sign of cracking of the porcelain column. The holding down bolts were found to be stretched and two turns were required to tighten them.

7.2.2.3 220 kV Line Traps

The insulators on top of the two line traps (red and blue phase) at the north end of the switchyard were broken. The failure was due to the excessive stress caused by the relative movement between the line trap and the solid conductor connected to it.

7.2.3 110/11 kV Transformers

None of the in-service 110/11 kV transformers came off their rails but some of the holding down arrangements and fixing of the undercarriages to transformers did fail or partially fail. The resulting displacements were large enough to cause some damage to insulators and tap changers.

The holding down arrangement for the Ferranti transformers was designed in 1981 to current ED earthquake resistant design standards. The undercarriage is solidly fixed to the foundation via channel section cross members which are bolted to universal I beams set into the foundation. There was no sign of damage to the holding down steelwork but the M16 bolts securing the transformer base to the undercarriage sheared.

The spare Breda transformer was not secured and came completely off the rails. The spare Ferranti and AEI transformers suffered the same damage to holding down arrangements as the in-service units.

7.2.4 Conclusions

Many of the 110/11 kV transformer rail clamps which failed at Kawerau were not designed to current Electricorp earthquake resistant design standards. For example the T1 and T3 ASEA clamps were designed in 1955 and the clamps for the Ferranti T2A were designed in 1962. The fact that none of the in-service units came completely off the rails may have been because the predominant direction of ground shaking was parallel to the rails.

A more detailed analysis of the failures will be carried out but it would seem that if the transformer fastenings had been upgraded to current Electricorp standards for earthquake resistant design most of the damage to bushings, insulators and tap change mechanisms would have been avoided.

The failure of blue phase on CB 622 (Merlin Gerin 220 kV FA2) requires further investigation because it was installed in 1979 and the strength complied with current Electricorp specifications for earthquake resistance. There are 34 FA2 circuit breakers of the same rating (13400 MVA) around the country installed between 1976 and 1980. There are a further 48 FA2 circuit breakers with a higher rating (15000 kV) installed between 1980 and 1985.

The failure of the control panels was due to bad working practice. The large number of surviving panels shows that the uni-strut system is a satisfactory method of protecting panels against earthquake forces.

7.3 220 kV BUSWORK SUPPORT POST PERFORMANCE
(J. N. O. Coad)

7.3.1 Summary

During the earthquake on 2 March 1987 numerous concrete buswork support posts in the Kawerau substation 220 kV yard cracked at the base. These posts are designed to yield under Electricorp's standard earthquake. Information available to date, from the DSIR, suggests that the ground acceleration at Kawerau was within 20% of the peak ground acceleration assumed by Electricorp's design earthquake of 0.4 g. The damage to the posts under this relatively severe test was generally minor and has since been repaired.

7.3.2 Design Philosophy

Concrete buswork support posts are designed to yield before damage to the supported equipment can occur. Under design seismic loadings a plastic hinge forms at or near ground level. Closely spaced stirrups in this region ensure ductile behaviour and provide concrete confinement.

7.3.3 Concrete Post Damage

At Kawerau substation 50% of the concrete support posts cracked. The majority of the cracks occurred in posts supporting airbreak isolators though some buswork support posts also cracked. In most cases the cracking was minor, Fig. 7.3.1, though the line termination posts have more serious cracks due in part to the extra base moment generated by the line tension load. In all cases the cracking occurred within 300 mm of the ground in the ductile region of the pillar.

A variation in crack size and occurrence was noticeable through the switchyard. Almost all the ABS isolator posts showed signs of cracking and some had signs of yielding, yet other isolator posts had no cracks at all.

An inspection of the cracked posts showed only small offsets of the post tops. Substation Design section considers a permanent post top deflection of 13 mm as a failure under seismic loadings. Measurements with a rudimentary plumb bob indicated that where a measurable deflection was present it was less than this value and generally not more than 6 mm, though this was hard to assess given the limited accuracy of the measurement procedure. That some post top deflection is present implies yielding of the reinforcing steel has occurred.

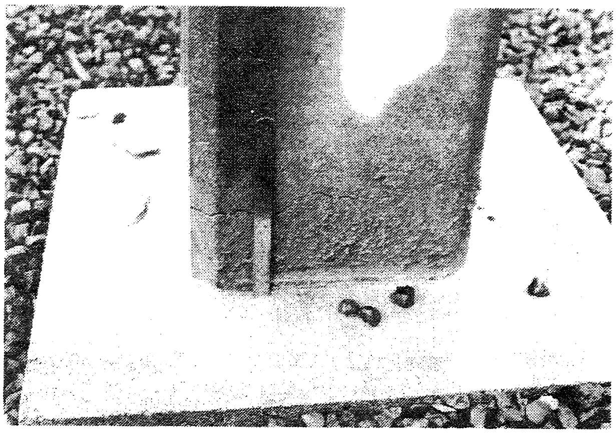


Fig. 7.3.1 : The base of a concrete pillar supporting an airbreak switch. This crack is smaller than a major crack and is typical of an intermediate crack.

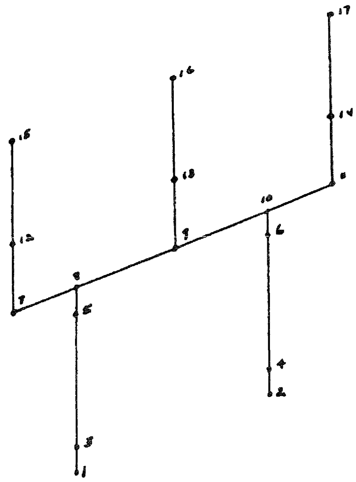


Fig. 7.3.2 : The computer model of the airbreak switch support used for the Strudl-Dynal dynamic analaysis

7.3.4 Analysis of Post Damage

The DSIR has reported that the earthquake at Matahina dam had a peak ground acceleration of 0.33 g and a form very similar to that of El Centro 1940. To determine the possible loadings during this earthquake, the concrete posts were computer modelled using the El Centro earthquake. On this basis the base bending moment on the concrete pillars at Kawerau may have just exceeded 22 kNm. Using the same model, Fig. 7.3.2, and a scaled El Centro earthquake, it was found that the ultimate bending moment of 26 kNm is only exceeded when the ground acceleration exceeds 0.43 g.

The posts are designed to yield at a base bending moment of 22 kNm. Observations at Kawerau indicate that yield has occurred in some of the posts as predicted, and cracking has occurred in most of the posts. It can be shown that initial cracking of the post occurs at a base bending moment of 9 kNm and as the predicted maximum moment is 22 kNm it is surprising that all posts do not show signs of cracking. Inspection shows some correlation between position in the yard and post cracking exists. All the support posts for the air break switches on the north-east side of the yard were slightly cracked while posts on the south-east side remained uncracked. Buswork support posts showed a similar pattern of failures with no cracking on the south-east side. There is, however, a gradation of crack magnitude within posts supporting similar equipment which does not appear to correlate with placement within the yard. This suggests that while most of the variation in cracking may be attrributed to differences in the intensity of ground movement some may be due to variations in post strength.

Despite there being an apparent difference in post strength of undetermined magnitude, no posts appear to have performed worse than the design requirement. The size of the earthquake was sufficient to cause yielding of concrete posts and this was observed.

7.3.5 Repairs

It was envisaged at the design stage that in the event of an earthquake causing posts to yield that repairs could be effected by pulling posts straight and grouting. The post top deflections at Kawerau were so small that switchgear still operated and it was not necessary to straighten the posts. Repairs were completed by injection grouting the cracks using a commercial epoxy at a rate of about 10 posts per day. The MWD believe that the injection grout will prevent water access to the crack and reinforcing steel and that the post strength will be similar to an uncracked post.

Some post foundations were excavated and no damage to the footings were found.

7.3.6 Conclusion

The concrete buswork support posts at Kawerau were subjected to earthquake accelerations which were at least 80% of the design level and may have exceeded that level. Cracking occurred throughout the substation yet there are no signs that posts yielded to the nominal failure deflection of 13 mm. Some post lean did occur but this appears to have been due to footing movement rather than post failure. There was some variation in the magnitude of cracking which in part can be attributed to strength variations and partly to differences in ground movement.

Based on the performance of posts at Kawerau and the results of computer modelling it is clear that posts will yield at peak ground accelerations 0.43 g.

8 ACKNOWLEDGEMENTS

Much valuable information was provided to the Reconnaissance Team members by the people of the Rangitaiki Plains, despite the shattered state of their lives, homes and workplaces.

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APPENDIX A: Chronological Sequence of Events with Magnitudes Greater than 4.0.

Data supplied by the Seismological Observatory of the Geophysics Division of the NZ Department of Scientific and Industrial Research.

Chronological Sequence

Date	Time	Magnitude
February 27	0102 hours	4.4
	1126	4.4
	1140	4.2
February 28	2349	4.9
March 02	0135	5.2
	0141	4.7
MAIN SHOCK	0142	6.3
	0151	5.6
	0203	4.7
	0207	5.3
	0212	4.3
	0214	4.2
	0216	4.2
	0218	4.6
	0223	4.0
	0237	4.0
	0241	4.1
	0242	4.4
	0250	4.1
	0309	4.6
	0311	4.3
	0313	4.1
	0315	4.0
	0327	4.8
	0330	4.6
	0346	4.2
	0511	4.2
	0513	4.5
	0519	4.3
	0538	4.0
	0601	4.0
	0626	4.0
	0641	4.7
	0644	4.9

Chronological Sequence

Date	Time	Magnitude
	0648	4.3
	0653	4.0
	0656	5.3
	0700	4.5
	0728	4.0
	0755	5.2
	0844	4.1
	0931	4.5
	0934	4.6
	1031	4.3
	1033	4.1
	1517	4.3
	1706	4.3
	1715	4.4
March 03	0048	4.0
	0057	4.4
	0115	4.1
	0115	4.4
	0514	4.0
	0517	4.0
	1214	4.5
	2344	4.0
	2344	4.1
March 04	0838	4.5
March 06	1541	4.4
March 17	1107	4.3
March 18	2258	4.1
March 26	1036	4.0