

1 Introduction

A breach occurred in the Rangitiaki River stopbank adjacent to Hydro Road, just upstream of Edgecumbe, at approximately 10am on Sunday 18th July 2004. The river was flowing very high at the time and water poured through the breach causing serious flooding of farmland, the railway, roads, and buildings downstream. The breach in the stopbank widened to about 100m and water flowed through for several days. Photograph 1 shows the breach from the air and Figure 1, prepared by Environment Bay of Plenty (EBOP), shows the extent of the flooding which resulted. As the breach occurred on Sullivan's farm it is referred to as Sullivan's Breach.

Ice Geo and Civil has been commissioned by EBOP to investigate the causes of Sullivan's Breach. This report discusses the following:

- the river and rainfall conditions at the time of failure,
- observations of the stopbank failure,
- the geology of the Rangitaiki Plains,
- the method of stopbank construction,
- the effects of the 1987 Edgecumbe Earthquake on the stopbank,
- the effects of the July 2004 earthquake swarm on the stopbank,
- subsurface and laboratory investigations carried out following the failure,
- computer models of failure mechanisms and
- the influence of the river flow hydrograph on the failure.

The author acknowledges the cooperation of EBOP in providing any data requested as quickly as possible and the Institute of Geological and Nuclear Sciences (IGNS), the National Institute of Water and Atmospheric Research (NIWA) and Trustpower for their provision of information.

Many local residents also gave freely of their time to provide eye witness accounts and local history.

2 Rainfall and River Conditions

Heavy rain began falling in the Bay of Plenty on Thursday 15 July 2004 and continued for the next three days. The rainfall recorded by NIWA at station B76972 in Edgecumbe was as follows (Reference 1):

14 July	5.2mm
15 July	66.1mm
16 July	100.0mm
17 July	91.0mm
18 July	<u>21.8mm</u>
Total	284.1mm

Some parts of the Bay of Plenty experienced over 300mm of rain. Surface flooding occurred in many areas as drainage systems and pumps could not cope with the volume of water. The surface flooding in some areas made it difficult to detect leakage from stopbanks.

The high rainfall caused a rapid rise in the Rangitaiki River. Figure 2 shows the hydrograph measured in the river at Te Teko, about 11km upstream from the breach site. Fortunately the upper catchment of the Rangitaiki River received only about 120mm of rain and not nearer the 300mm experienced in the rest of the catchment.



Figure 2: Flood hydrograph measured at Te Teko (EBOP)

The Matahina Dam can reduce the peak flow of smaller flood events by storing water in the lake and controlled release. However as the available storage volume in the lake is small compared to the volume of water flowing down the river in the July 2004 event, it was not possible to make a very significant reduction in the peak flow. Stopbank failures can be a result of the duration of flood flow in combination with the peak flow. Therefore in large events such as the July 2004 storm, prolonging the peak flows by lake storage and release may not reduce the risk of stopbank failure. This is discussed in following sections.

The stopbanks at the breach site are designed to contain a flow of 780 cumecs (780 cubic metres of water per second) which is the estimated flow resulting from a storm with a 100 year return period. In July 2004 the peak

flow estimated from dam spillage and downstream records is approximately 771 cumecs. The stopbank breached at about 650 cumecs, before the peak flow was reached. Fortunately the river peaked only 10 hours after the breach and dropped steadily over the next five days. This reduced the flow through the breach before it was closed.

The flood flow in the river was the greatest recorded since 1944. In contrast the flood which occurred in July 1998 and caused stopbank problems at the Edgumbe substation and elsewhere was only 464 cumecs (estimated 15 year return period, Reference 2).

3 The Stopbank Failure

3.1 Witness Descriptions

At about 7.25am on Sunday 18th July 2004 the sharemilker managing Sullivan's Farm observed that a fence post at the landward toe of the stopbank was leaning and that there was silty water coming from the ground near the fencepost and along the toe of the stopbank. The volume of water was described as what you would expect from about "10 fire hydrants". "Skid marks" were also observed about half way up the stopbank face. It is considered that these marks were cracks in the face of the stopbank. The sharemilker telephoned EBOP and two staff members were sent out to inspect the stopbank. At this stage there were serious concerns about other parts of the stopbanks further downstream.

The EBOP staff members began by inspecting the stopbank around the Edgumbe Substation about 500m downstream from the breach site as there had been problems there during the July 1998 flood. They had just begun to head upstream when a member of the public told them the stopbank had breached.

At the time of the breach an Edgumbe local was driving south along Hydro Road and saw dirty water spurting up from the paddock "40 to 50 yards" from the landward toe of the stopbank. As he was familiar with stopbank breaches and floods on the Rangitaiki Plains he quickly turned around to head back to Edgumbe. By the time he had turned his vehicle around the stopbank had collapsed and there was a "6 foot wall" of water heading towards Hydro Road.

Another local was driving along McCracken Road towards the stopbank and saw what looked like a huge waterfall coming over the stopbank. This is consistent with the observations of the EBOP staff when they got closer to the site. The "waterfall" was in a confined area and it did not appear that the river had overtopped the stopbank. The EBOP staff observed that the breach was about 20m wide and that there were cracks about a third of the way up the stopbank face adjacent to the breach. In a short time the remains of the stopbank had eroded away and the water levels on each side of the line of the stopbank began to equalise. The exposed ends of the stopbank stood vertically until scour of the underlying soils caused slabs of stopbank to

collapse into the flow. Photograph 2 shows the opening three days after the breach when the river level had dropped significantly.



Photograph 2: Looking downstream three days after the initial breach

3.2 Observations Around the Breach Site

Inland Side of Stopbank

Observations of the breach and the scour hole were carried out on 24 July when the water level had dropped to below the general ground level.

The large volume of water which flowed through the breach scoured out a hole nearly 5m deep where the stopbank had been (Figure 3). The scour hole extended inland about 150m from the stopbank centreline, typically reducing in depth with increasing distance from the stopbank (Figure 4, Photograph 3). About 5m beyond the end of the scour hole was a shallow depression which was later found to be an old abandoned stream channel.

There were large tree stumps and slabs of peat within the scour hole when the water level dropped. These slabs of fibrous peat were up to 2m across and 300mm thick. There were no signs of peat around the edges of the hole and none was found in the later subsurface investigations. It is assumed that the peat and stumps came from further upstream, possibly where the earthquake faultline crosses the river and stumps are exposed in the river bed.

Around the edges of the hole there were fingers of ground where approximately the upper 200mm of soil had been removed. The paddock had been ploughed, rotary hoed and re-sown in February 2004 following its use for growing water melons. The grass roots were not well established and the sown rows of grass were obvious. It is considered that the depth of soil lost in these fingers coincides with the depth of ploughing (Photograph 4).

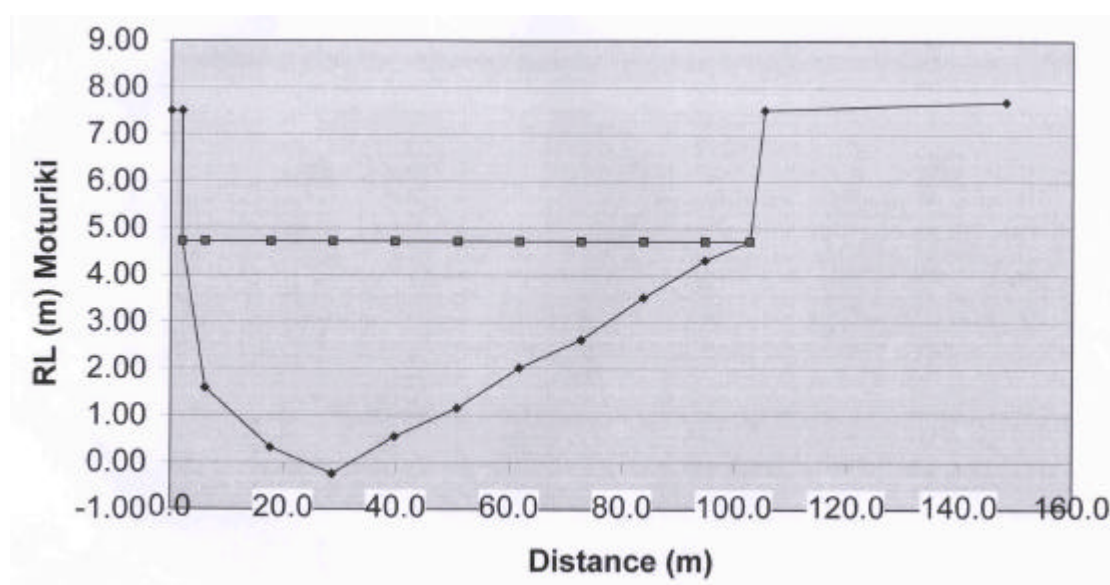


Figure 3: Cross section of scour hole along line of stopbank (EBOP)

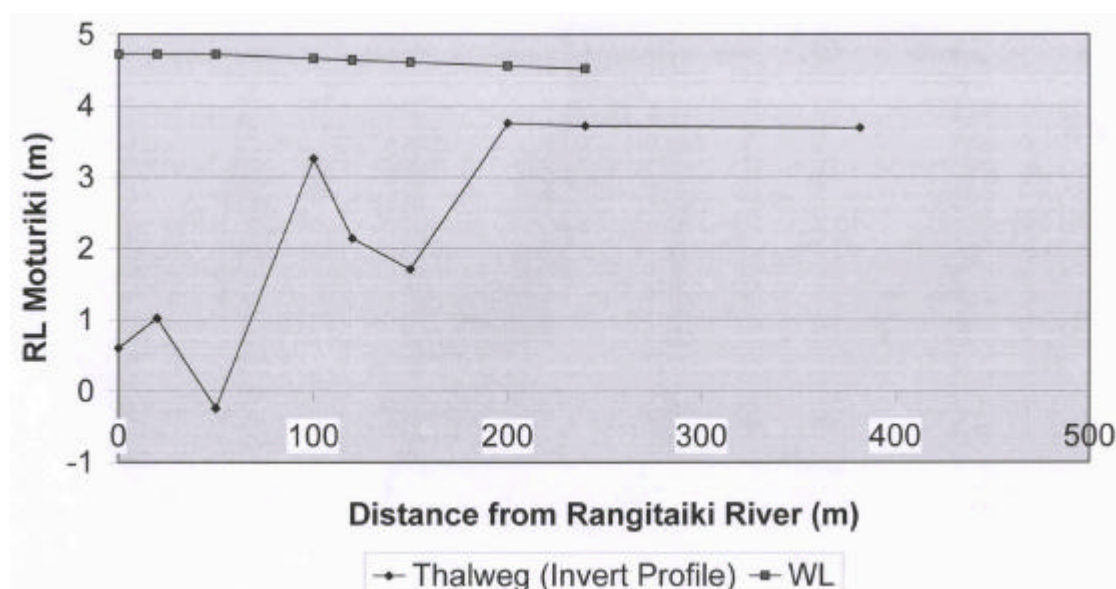


Figure 4: Long section along scour hole from river to Hydro Road (EBOP)



Photograph 3 : Scoured hole



Photograph 4: Loss of upper soil layer

There was often another step in the depth of soil lost from the ploughed depth to about 300 to 400mm below the original ground level. This coincided with a layer of Tarawera Ash across the site. This ash was airfall deposited in 1886 and consists of a coarse abrasive black basaltic sand. It could be seen all around the edges of the scour hole and under the exposed end of the stopbank (Photographs 5 and 6). The thickness of the ash layer varies from about 70mm to 200mm. In some small areas there was a thin layer of fine sand within the predominantly silty soil above the Tarawera Ash.



Photograph 5: Ploughed depth and Tarawera Ash layer

At the toe of the stopbank on the downstream side of the breach it was observed that the surface soils consisted of about 300mm of silty fine sand overlying 75mm of Tarawera Ash and more silty fine sand (Photograph 6).

About 6m from the toe of the breach the combined thickness of these three layers was only about 600mm and they were underlain by a loose fine to medium grained sand. Around the scour hole it could be seen that this sand layer had been removed by water flow causing overhangs and cracks in the slightly cohesive layers above (Photograph 7). Eventually the sand removal would cause collapse of the upper layers. Further out from the stopbank toe the fine to medium sand layer was only about 400mm below the ground surface.



Photograph 6: Exposure at landward toe of stopbank, downstream end of breach showing black Tarawera Ash layer



Photograph 7: Cracking and collapse of slightly cohesive upper soil layers at the far end of the scour hole

At the far end of the scour hole (Photograph 7) where about 200mm of the upper soil had been removed, pieces of broken glass, a large diameter steel rod and a piece of wire cable were found. It was thought these may have come from an old rubbish hole on the farm. Neither the present farmer or the previous owner can recall a rubbish hole in this vicinity. The previous owner did recall a spongy area in the paddock "50 to 80 yards" from the stopbank toe when the river was in flood. An alkathene pipe had been exposed at the end of the breach.

River Side of Stopbank

The debris line on the river side of the stopbank indicated that the river level never rose higher than about 700mm from the stopbank crest. This confirms that the stopbank failure was not due to overtopping. There was no evidence of erosion of the face of the stopbank.

After the water had receded it could be seen that the small pump shed on the berm just to the downstream side of the breach was on a mound 1 to 2m higher than the berm further downstream or upstream. It was later found that this is because there was an old bend in the river downstream from the pump shed and the berm upstream had been used as a borrow area (source of fill) during stopbank construction. The berm between the river and the stopbank widens downstream from the breach site (Photograph 8).



Photograph 8: Pump shed and river berm

Near the pump shed were pieces of steel intake pipe and alkathene pipe. Improperly installed pipes passing through stopbanks can cause stopbank failures. It was later found that the pipe from this pump shed crosses towards the top of the stopbank downstream from the breach.

At the sides of the scoured hole the soils were found to consist of about 300mm of fine sand with some silt/ silty fine sand overlying fine to medium sand with fine pumice lapilli. About 25 to 50mm of fine hard gravel was found on the surface of the river berm downstream from the breach.

Stopbank

Well compacted silty fine sand was exposed in the ends of the stopbank which were standing vertically. There was no evidence of any cracking, seepage paths or poorly compacted layers.

3.3 Possible Failure Mechanisms

Failure Mechanisms at the Stopbank Toe

The early observations of water coming from the landward toe of the stopbank and the cracks in the face of the stopbank suggest a piping type of failure. Piping occurs when there is an outlet for flowing water and there is enough hydraulic gradient to remove soil particles. Continuous soil particle removal can result in the formation of a "pipe" or "tomo" under a stopbank. The arching effect of soil can allow the pipe to reach a significant size before the soils above collapse into the hole and a stopbank breach can result.

The critical hydraulic gradient for soil loss depends on the weight of individual soil particles and whether they have any cohesion. The light, predominantly pumiceous, sandy soils of the Rangitaiki Plains are susceptible to piping. The hydraulic gradient at the landward toe of the stopbank is dependent on the head difference across the stopbank and the permeability of the soils within and below the stopbank.

It is considered that at the breach site the Tarawera Ash and the deeper sand layers may have been sufficiently permeable to allow a high hydraulic gradient and high water pressure to develop at the toe of the stopbank. A surface layer of cohesive soil can prevent piping developing if it remains intact and the water pressures below it do not exceed the weight of the soil layer. Where water pressures exceed the weight of the layer, heaving can occur. This results in the ground surface being lifted and a "spongy" surface. Sufficient water pressure can result in the surface layer fracturing, allowing water and soil particles to flow out.

At the breach site it is possible that the fence posts (which had only been driven a week earlier than the breach) penetrated the upper slightly cohesive soils to a higher permeability layer below, effectively creating a short circuit.

A good root structure and the build up in humus under pasture helps bind soil aggregates and some tensile strength can result in the upper soils (Reference 3). At this site there was no mature mat of pasture roots due to the use of the paddock for growing watermelons and subsequent ploughing and sowing of pasture in February. Vegetable crops and the associated rotary hoeing result in rapid breakdown in soil organic matter and a sparse root system.

Ploughing can result in a "smear pan" at the base of the ploughed depth. At the breach the observations around the scour hole suggest that the ploughing effectively split the slightly cohesive upper soil layer into two. It may be possible that whereas relatively uniform seepage could occur through an undisturbed natural soil layer, more concentrated flows could develop where pressure builds up under a thin smear pan causing water to break through in localised areas. The ploughing was carried out right up to the stopbank toe.

It is possible that heave and fracturing of the thin upper soil layer at the stopbank toe allowed the escape of water from the Tarawera Ash layer and/or deeper high permeability layers. The failures observed in the face of the stopbank could have been due to:

- the collapse of the stopbank foundations due to piping,
- and/or high pore water pressures developing in the face of the stopbank due to internal seepage making it unstable.

Failure Mechanisms in the Paddock

One witness observed the final stopbank failure originating in the paddock some distance inland from the stopbank. This failure is considered to be due to heave of the upper soil layer and the rapid escape of water from layers of high permeability below. This type of behaviour has been observed at other sites on the Rangitaiki Plains in previous floods. One local who used to be a farming contractor recalls how the paddock where the breach occurred was spongy whenever the river rose to the level of the river berm.

The paddock falls about 1.2m from the toe of the stopbank to Hydro Road. Therefore the head differential from the river to the ground surface increases with distance from the stopbank. The head difference between the river at the time of failure and the landward toe of the stopbank was only about 1.8m, but at Hydro Road this would be nearer 3.0m. Also it was observed that the thickness of the upper slightly cohesive soil layer varied significantly and there could have been an area where the layer was particularly thin.

Once the upper soil layer is lost the non cohesive layers of light soil below could rapidly erode back to the stopbank. This would cause the stopbank, made of more competent soil, to drop down into the hole formed below and be overtopped. From the witness observations it appears that it was this process that caused the final failure and not the piping occurring at the stopbank toe. As the stopbank is constructed of silty fine sand, overtopping would cause rapid scour down to foundation level.

4 The Geology of the Rangitaiki Plains

The Rangitaiki Plains are the most northern extent of the Taupo Volcanic Zone on land (Reference 4). They lie in the Whakatane Graben (depressed block of land) with hills to the east, west and south. Geological studies indicate that the graben has been widening at an average rate of 7mm per year with associated small subsidence. The dune system and alluvial lowlands within 10km of the coast have formed in only the last 8,000 years. The plains are therefore geologically very young and unconsolidated which means they are highly susceptible to liquefaction and settlement following significant earthquakes.

The plains have been built up by over-bank alluvial deposits from meandering rivers in flood, wind blown sand, airfall ash deposits and peat formation in depressions. The active volcanism of the upper Rangitaiki River catchment and the high erodibility of the volcanic tephra result in a large source of sediment which can be deposited on the plains. The outcome of the plain formation processes is a wide and unpredictable variety of soil types and grain sizes within small horizontal and vertical distances. Figure 5 (Reference 5) illustrates the floodplain environment of a meandering river with bank erosion and cut-off channels.

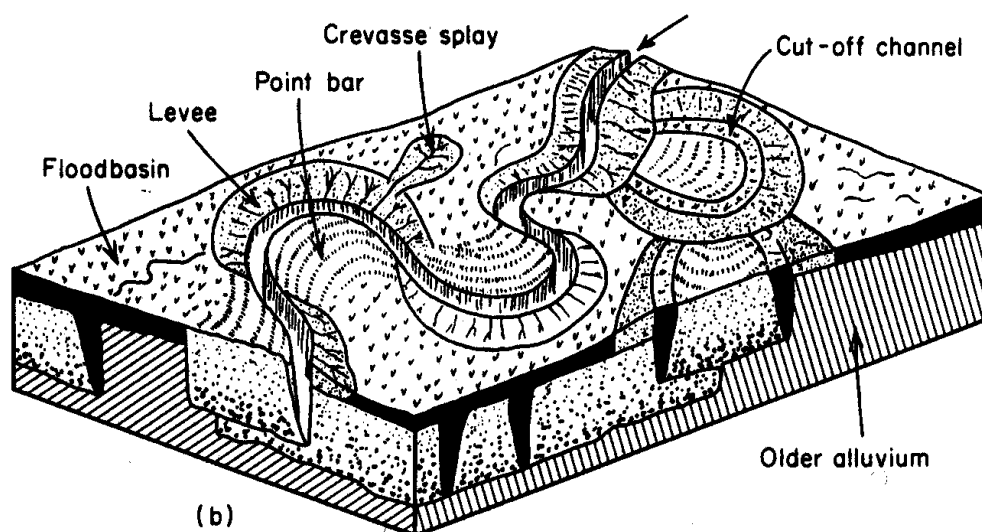


Figure 5: The floodplain environment of meandering rivers
(Figure 4.2 Freeze & Cherry (1979) Reference 5)

A critical soil property for stopbank design is the permeability of the underlying soils. Most of the soils within the plains are silts, sands and gravels with moderate to high permeability. Much of the soil is of pumiceous origin and there are many layers of pumice gravel ranging from lapilli size (up to say 10mm) to cobbles about 100mm in diameter. These layers have very high permeability. A surface layer of more cohesive low permeability silt is found in many areas and this is often the only reason there are not excessive flows

under stopbanks. The rapid variation in soil types and layering makes it very difficult to design a stopbank system which can take into account all possible foundation conditions.

5 Original Stopbank Construction

5.1 Rangitaiki – Tarawera Rivers Major Scheme

As discussed above the Rangitaiki Plains have been formed by several processes and in particular by flooding. The earliest flood recorded following European settlement was in 1904. Other significant events occurred in 1924 and 1944. Between 1944 and 1964, 15 floods occurred causing extensive inundation and river bank erosion (Reference 2). Older residents can recall floods in the Mc Crackens Road area.

The Bay of Plenty Catchment Commission initiated formal stopbank construction along the river in the 1960s as part of the Rangitaiki – Tarawera Rivers Major Scheme. The scheme was designed to:

- alleviate flooding and
- to control the river channel and reduce the loss of land due to erosion.

The Reids Central Canal floodway was designed to divert some water from the river. The entrance to the floodway is just upstream from the breach. It does not have the capacity to carry the flow which passed through the breach.

Formal stopbanks were built along the Hydro Road section of the river in 1977. Drawing R450/22/2 shows the layout of the stopbank with the breach site, small pump shed and borrow area on the river berm in the lower left corner of the drawing. Prior to formal stopbank construction there was a small stopbank around Mc Crackens Bend, which is an abandoned river channel. The new stopbank was built across the two arms of the bend and the old stopbank removed and placed as a surcharge on the old river bed to prevent excessive flows under the stopbank.

The stopbank was typically 1.5m above the general ground level, with 3H:1V side batters (Reference 6). Most of the soils used in stopbank construction were sandy silts and silty sands removed from the river side of the stopbank. Figure 6 shows the approximate original surface, the stopbank and the depth of excavation from the berm (approximately 1.4m). A small bench was left in front of the stopbank.

The stopbank appears to have functioned satisfactorily until the 1987 Edgecumbe Earthquake. During this period the largest flood in the river was 372 cumecs (Reference 2)

5.2 Regulations Concerning Stopbanks

The highly permeable foundations of the stopbanks within the Rangitaiki Plains have been well recognised and efforts have been made through regulation to reduce the risk of stopbank failure. Clause 9.1 of the Bay of Plenty Regional Council Floodway and Drainage Bylaw 2002 states:

No person shall, without the prior written authority of the Council undertake any of the following activities-

(a) The digging or maintenance of any drain, or any excavation within 150 metres from the landward toes of the Rangitaiki River Stopbanks:

The clause goes on to give the following explanation:

The lower reaches of the Rangitaiki and Tarawera Rivers have layers of soils that are susceptible to piping failures beneath the stopbanks during flood events. The purpose of this rule is to minimise the risk of such failures occurring.

Clause 4.1.5.2 of the Whakatane District Plan Rural contains more detailed requirements. Normal farming activities such as ploughing and fencing are allowed.

6 The Effects of the 1987 Edgecumbe Earthquake

6.1 The Earthquake

On 2 March 1987 the Rangitaiki plains experienced a Richter magnitude 6.2 earthquake. This was preceded by a series of foreshocks as big as Magnitude 5.2 starting on 23 February and was followed by several significant aftershocks. It was estimated that the main earthquake was centred within the 10km of the earth's surface.

The earthquake caused reactivation of the Edgecumbe, Onepu and Rotoitipakau Faults and several new surface ruptures ranging in length from 0.5 to 7.0km (Reference 7). Typically the north-western side of the rupture dropped relative to the south-eastern. The largest displacement was near the centre of the Edgecumbe Fault rupture which is as close as 700m to the Sullivan's Breach site. This displacement was up to 2.5m vertically and 1.8m horizontally (Figure 7). The rupture crossed the Rangitaiki River about 3km upstream from the breach site.

The earthquake swarm caused extensive liquefaction of the young alluvial deposits within the plains. Liquefaction occurs when loose granular deposits are shaken into a denser state during an earthquake. Where the deposits are below water a denser state can not be achieved immediately as water is effectively incompressible. As a result the water pressure rises until it can

equal the weight of the soil above. The soil loses its strength and essentially behaves like a liquid. The liquefied soil and water can be squeezed up through surface fissures and form small volcanoes (sand boils) on the ground surface. Many sand boils were found around the Rangitaiki Plains following the earthquakes. Other evidence of liquefaction includes cracking of river banks and movement towards the river (lateral spreading) and the activation of springs.

Following an earthquake the ground surface will settle as excess water pressures dissipate and a new denser state is achieved by the alluvial soils. Following the Edgecumbe Earthquake 1 to 2m of ground settlement was recorded extending from where the fault crossed the river to downstream of Edgecumbe. Near McCrackens Road this settlement was measured at up to 7mm a day. Local farmers are still experiencing ground movement, 17 years after the major event and regularly have to re-hang some gates. Some of the springs that were activated are still flowing.

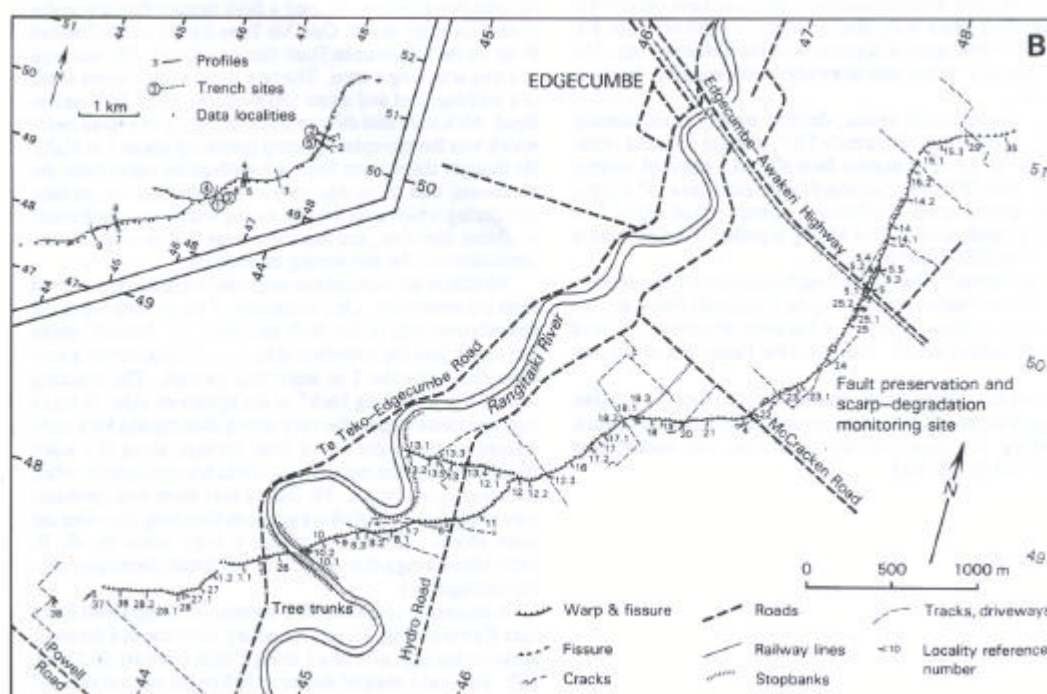


Fig. 1B Locality numbers for the Edgecumbe Fault; profile and trench sites shown in inset.

Figure7: Edgecumbe Fault location (Beanland et al., 1989)

6.2 The Effects on the Stopbank

Following the earthquake all the stopbanks along the rivers within the plains were inspected for damage and a damage rating system was developed (Reference 8). In the inspection it was noted that many sections of the stopbank had highly permeable foundations and surcharges had been placed behind some sections to prevent excessive flows under the stopbank. Type I and VIII damage was recorded at the Sullivan's Breach site.

Type I damage consists of river bank slumping. As shown in Figure 6 there is about a 15m difference in the location of the river bank between the time the stopbank was originally built and when it was repaired after the earthquake. This difference may be due to the river bank slumping and subsequent erosion.

Type VIII damage indicates that the stopbank itself was lightly cracked in areas with transverse and longitudinal cracks less than 75mm wide. The required repair was described as "importation of granular fill, overspreading, watering and compaction with a vibrating roller".

Before the repair work to the stopbank was carried out it was realised that the depression created by ground subsidence in the Edgecumbe area meant that the stopbanks were no longer high enough to retain a 100 year return period flood. The stopbank at the breach site had to be raised 1.2m to re-instate adequate flood protection. The work was carried out in 1989 mainly using local silty sand/ sandy silt soils. The re-instated profile is shown in Figure 6 (Reference 9).

6.3 The Influence of the 1987 Earthquake on the Breach

It is considered that the 1987 earthquake and the damage caused by it had no direct influence on the breach. There was no evidence of residual cracking or poor construction of the raised part of the stopbank in the exposed ends on each side of the breach. Indirectly the earthquake may have influenced the outcome on 18 July 2004 as the stopbank was 1.2m higher relative to the natural ground surface than the pre-1987 stopbank. Therefore the head difference across the stopbank from the river to the ground on the landward side was higher than it would have been pre-1987. Allowing for the approximately 0.7m of freeboard when the breach occurred, the water level was 0.5m above the top of the pre-1987 stopbank. The increase in head differential may have been enough to cause the breach. The pre-1987 stopbank did not experience any floods of a similar size to that in 2004.

7 The July 2004 Earthquake Swarm

At about 4pm on 18 July 2004 a small but significant earthquake rocked the Bay of Plenty. This coincided with the arrival of the flood peak at Sullivan's Breach. The breach had occurred six hours earlier. The Institute of Geological and Nuclear Sciences has confirmed that there were three small foreshocks between magnitude 2.0 and 3.1 from 12.38am to 3.37am. These appear to have been centred at between 7 and 8km depth and were not felt (Reference 10). It is considered that these foreshocks did not contribute to the breach.

8 Insitu and Laboratory Investigations

8.1 Insitu Investigations

A subsurface soil profile is required to confirm the causes of the stopbank failure using a model of the seepage processes through and under the stopbank. The scour caused by the stopbank breach removed the natural soils from the river bank to over 100m from the landward side of the stopbank. The precise soil profile at the breach site is therefore unknown and an assumed soil profile has to be derived from the exposures around the scour hole and investigations on each side of the breach.

A small window sampler was used to determine the soil profile to up to 7m depth from the present ground surface. Investigations were carried out at each side of the breach and at each side of the stopbank. All the tests were carried out right at the stopbank toe except for that on the landward side of the stopbank on the upstream side of the breach. Stopbank repair activities here meant that this investigation was carried out 34m from the toe of the stopbank. Figures 8 and 9 show the assumed soil profiles on each side of the breach based on these investigations and the logs of the investigations are included in Appendix A. Some shallow hand augers were also carried out to confirm soil profiles. The investigation holes were backfilled with bentonite.

A constant head permeability test was carried out in the stopbank to provide a comparison with permeability estimates based on soil particle gradings.

8.2 Laboratory Testing

Laboratory particle grading analyses were carried out by Opus International Consultants Laboratory on six samples of soils taken from various layers and the stopbank. The grading tests provide data for the estimation of soil permeabilities using published relationships.

Two specific gravity tests were carried out on the Tarawera Ash and on the stopbank fill. Particle specific gravity is required to estimate the critical hydraulic gradient at which particles can be washed out of the soil mass.

The results of the testing are included in Appendix B.

9 Investigation Results

9.1 Soil Profile

It can be seen from Figures 8 and 9 that the soil layering under the stopbank is complex and variable from one cross section to another. There is however a general trend of clayey to sandy silt overlying Tarawera Ash, then fine sandy silt/ silty sand, fine to medium sand and layers of coarser sands, gravels and pumice lapilli. Minimal organic material was found in any of the investigation

holes. Some thin clayey silt layers were found at over 5m depth and a thin hard pumiceous silt layer was found at over 6m depth.

The soil profile found at Sullivan's Breach is similar to that found by Opus at the substation just downstream (Reference 11).

It can be seen from the upstream cross section (Figure 8) that the upper silt and sandy silt / silty sand layers were removed when the river berm was used as a borrow area. This has left the highly permeable Tarawera Ash layer exposed at the face of the stopbank and fine sands at the toe of the stopbank. The downstream cross section shows consistency in the upper soil layers under the stopbank and a slight fall towards the river.

The soil profile assumed for seepage analysis of the breach site is given in Table 1.

depth (m)	soil
0.0	clayey SILT
0.3	Tarawera Ash
0.4	fine sandy SILT
1.3	fine SAND
2.7	medium to coarse SAND

Table 1: Assumed soil profile

It should be noted that the depth to the fine sand layer was observed to be as little as 0.4m in the edge of the scour hole. It has been assumed that the deeper silt layers are not continuous and do not form an effective flow boundary.

9.2 Soil Permeabilities

Several methods were used to estimate the soil permeabilities from the laboratory soil grading curves. These methods were that of Hazen, Masch and Denny (Reference 5) and Alyamani and Sen (Reference 12). The accuracy of these methods is dependent on the soil grain size with a tendency of increasing accuracy with increasing grain size.

The estimated permeabilities were also compared to some measured in laboratory tests by Opus when investigating the problems with the stopbank around the substation just downstream (Reference 11) and for the stopbank, the constant head test. Table 2 gives the permeabilities assumed in the seepage analyses.

The horizontal permeability of the stopbank soil has been assumed to be greater than the vertical due to the layering effects of stopbank construction. The lower sands are distinctly layered due to variations in particle grading between sand deposition events. Therefore it has been assumed that water

can flow more easily along layers than vertically through layers with slightly different grain size and permeability characteristics.

soil	horizontal permeability	vertical permeability
stopbank	2×10^{-6} m/s	1×10^{-6} m/s
silt	3×10^{-7} m/s	3×10^{-7} m/s
Tarawera Ash	1×10^{-4} m/s	1×10^{-4} m/s
silty sand / sandy silt	1×10^{-6} m/s	1×10^{-6} m/s
fine sand	5×10^{-5} m/s	2×10^{-5} m/s
medium to coarse sand	5×10^{-4} m/s	1×10^{-4} m/s

Table 2: Assumed soil permeabilities

The seepage characteristics of an unsaturated soil layer change with time as the voids between soil particles are filled with water. The greatest permeability, in terms of water travelling a given distance, is achieved when the soil becomes saturated. Soils above the normal ground water level are unsaturated therefore relationships are needed between water content, permeability and water pressure. These relationships are called characteristic curves. Opus carried out laboratory testing and derived characteristic curves for a range of soil types as part of their study of sections of the Rangitaiki stopbanks (Reference 11). These curves have been assumed in the analyses of Sullivan's breach. (Some slight changes to Opus' curves were required to avoid instability in the numerical soil model.)

9.3 Specific Gravity

The specific gravity of the soil within the stopbank was found to be 2.44 and that of the Tarawera Ash, 2.7. The significant difference is because the stopbank is made from soils of alluvial origin with a pumiceous content and the Tarawera Ash is a volcanic basaltic scoria.

Investigations on pure pumice sands found in the Waikato have shown their specific gravity to be as low as 1.77 due to the voids within the soil particles (Reference 13). This specific gravity may be relevant to some of the soil layers found in the Rangitaiki Plains.

Table 3 gives approximate critical hydraulic gradients for the three soil types.

soil	critical hydraulic gradient
typical sands and silts	0.75
Tarawera Ash	0.9
pumiceous sands and lapilli	0.4

Table 3: Approximate Critical Hydraulic Gradients

10 Computer Analyses

10.1 Introduction

The soil profile estimated from the insitu investigations and the soil permeability characteristics derived from the laboratory testing were used to develop a computer model of the stopbank and underlying soils. Analyses of the model were carried out to determine if failure mechanisms consistent with witness evidence could be predicted. The computer programme used was Geo-Slope Seep/W Version 5.

There was approximately a 1.2m fall from the landward toe of the stopbank to Hydro Road. This fall has been included in the computer model. It was assumed that the thickness of the upper silt and Tarawera Ash layers remained constant and the thickness of the underlying silty sand layer reduced to produce the fall.

The soil model used was complex due to the need to model the thin Tarawera Ash layer and to extend the model well past the landward side of the stopbank to see if it would predict the types of failure observed. The results of the analyses are included in Appendix C.

The slope stability programme Geo-Slope Slope/W Version 5 was used to confirm failure at the downstream toe of the stopbank.

The flow through the breach removed the natural soils under and on both sides of the stopbank, therefore the soil profile at the breach site has been estimated from profiles which may be 50 metres or more from the original breach. In addition it is very difficult to estimate the permeability of complex soil layers to within one or two orders of magnitude. The computer model of the stopbank failure presented can not therefore be considered precise.

10.2 Flood Flow Hydrograph

The flood flow hydrograph at Te Teko discussed in Section 2 above was used to model the flood up to the time of the breach. The hydrograph time was adjusted for the two hour time difference in river flow from Te Teko to Sullivan's Breach. The hydrograph levels were adjusted to allow for the 0.7m freeboard at the time of the breach. The resulting hydrograph is shown in Figure 10.

10.3 Steady State Analysis

An initial steady state analysis was carried out to estimate the ground water conditions before the flood occurred. It was assumed that the river was at the level at 8am on 16 July (22.6m) and the water level was 2m below ground level 50m from the landward stopbank toe.

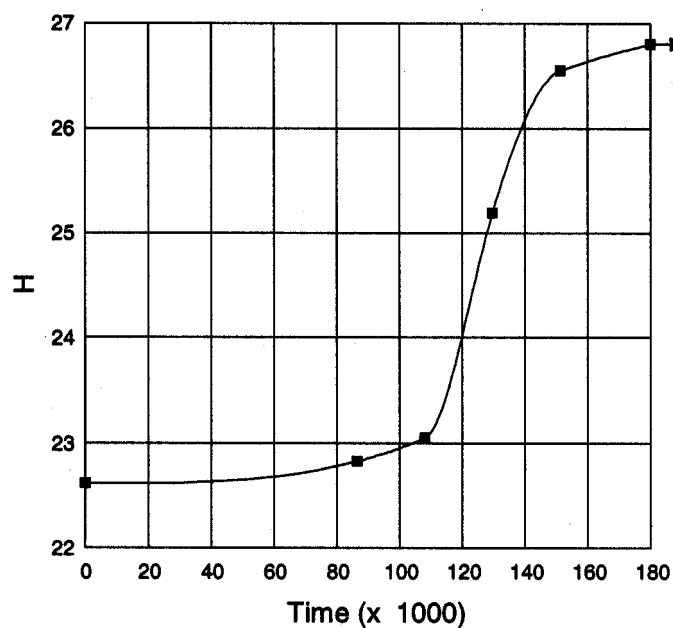


Figure 10: Flood Flow Hydrograph
(time in seconds from 8am 16 July 2004, stopbank crest 27.5m)

10.4 Seepage Conditions at 7.30am 18 July 2004

A transient analysis of water flowing through and under the stopbank was carried out using the flood flow hydrograph. This analysis started with the initial steady state conditions and modelled the response at half hour intervals. Seepage review nodes were specified at the landward toe of the stopbank and to about 10m from the toe. These nodes allow adjustment of the pressure head if the phreatic surface reaches ground level and surface seepage occurs. Further review nodes were specified from about 20m from the stopbank to the edge of the model. Infinite elements with no boundary conditions were specified at the inland edge of the model.

Stopbank Toe

The phreatic surface (ground water level) predicted by the model at 7.30am is shown in Appendix C. The predicted rise in water pressure at the toe of the stopbank is not sufficient to cause heave of the upper two silt layers and as the phreatic surface does not reach the toe of the stopbank no seepage is predicted. The inherent assumption in the model is that there is an intact 1.3m layer of soil of reasonably low permeability above the more permeable sands.

The pressure gradients in the lower silt are high enough to cause piping if there is an outlet for the water and soil particles. Two grading tests were carried out on the samples from the layer below the Tarawera Ash. Comparison of these grading curves with the grading curve for Tarawera Ash shows that the Tarawera Ash would act as a filter for the sandier of the two samples. This means that the ash would prevent piping. However particles

from the siltier of the two samples could be washed into the voids in the Tarawera Ash.

The water pressure predicted at the toe of the stopbank is not high enough to cause the observed failures in the stopbank face.

Paddock

At about 40m out from the toe of the stopbank, where it was assumed there was 0.5m of silt below the Tarawera Ash, the phreatic surface reached the ground level at about 6am. The hydraulic gradients at the ground surface are larger than 1.0 which is enough to cause loss of soil particles. The witness who observed the failure at 10.00am noted the dirty water spouting up. It is possible that water had been seeping upwards for some time before the failure but had been unobserved. Surface water due to the heavy rain would hide smaller seepages. The hydraulic gradient in the Tarawera Ash is less than that required for piping to occur.

Beyond 40m from the stopbank toe the pressure head under the silty layers is less than that required to cause heave and fracturing of the layers. It is considered that fracturing of these layers must have occurred to allow the observed rapid escape of water and subsequent stopbank failure. However the predicted pressure head at the top of the Tarawera Ash is sufficient to lift a 300mm layer of silt. It is possible that a small area of silt was lifted and the pressure below the remainder of the layer was relieved. If a large area of surface silt was lost, and the soil model is correct, the pressure at the base of the lower silt layer would have been sufficient to allow more major heave to occur.

Rainfall

An effort was made to incorporate the heavy rainfall occurring at the same time as the flood into the model. This predicted a rise in the phreatic surface to about 0.9m above the natural ground level at the landward toe of the stopbank. This rise is enough to cause failure of the face of the stopbank but the numerical model was not very stable, even when the time step was halved, and did not allow for overland flow towards Hydro Road. Therefore no further attempts were made to incorporate rainfall.

Model Adjustments

It was observed at the downstream edge of the scour hole, about 6m from the stopbank toe, that the top silt and Tarawera Ash layers were only 600mm thick. The soil model at the toe of the stopbank was therefore changed to reflect this. Analysis of this model produced a rise in the phreatic surface up to the toe of the stopbank.

A slope stability analysis of the stopbank face indicated a failure occurring about half way up the face. This is consistent with the crack and leaning fence post observed by the sharemilker at about 7.30am. The soil parameters used in the slope stability analysis are given in Table 4.

soil	density	cohesion	effective friction
stopbank fill	18kN/m ³	0kPa	33°
silt	17.5kN/m ³	5Pa	27°
Tarawera Ash	18kN/m ³	0kPa	36°
silty sand/sandy silt	17kN/m ³	0kPa	30°
sand	17kN/m ³	0kPa	33°

Table 4: Soil strength parameters

It is possible that water travelled up the cracks caused by the slope failure and the pressure on the base of the silt layers was relieved sufficiently to prevent further heaving. It is possible that most of the water bubbling from the toe of the stopbank had travelled through the Tarawera Ash layer. There would therefore be a limited volume of flow compared to flow through the thicker sand layers below the silt.

The Tarawera Ash layer may have acted as a filter preventing loss of soil particles (in the short term at least) from the layers below. The silt in the water observed by the sharemilker may have been from the surface silt layer.

10.5 Seepage Conditions at 10.00am 18 July 2004

The transient analysis of the flood flow was continued to 10.00am. By this time the water pressure under the silt layers from 30m from the stopbank toe to the edge of the model has risen sufficiently to allow heave of the three surface layers and the exposure of the underlying sands. A possible chain of events is:

- formation of a large crack or hole in the upper silts,
- the washing out under pressure of the highly permeable sands below,
- the formation of a hole below the upper silts,
- the collapse of the silts into the hole,
- repetition of this the hole formation and collapse sequence to the toe of the stopbank,
- formation of a hole under the already weakened face of the stopbank and
- either slumping of the downstream face of the stopbank including the crest, thus causing overtopping or
- the formation of a large hole under the whole stopbank and vertical collapse of the stopbank into it.

10.6 The Effect of River Berm Removal

The use of the river berm as a source of stopbank fill resulted in the exposure of highly permeable sand layers at the river side toe of the stopbank and effectively halved the seepage path from the river under the stopbank. This

may not be critical in floods of short duration where there is not enough time for high water pressures to develop on the landward side of the stopbank, but for larger events this seepage path reduction could be critical. An analysis of the soil model was carried out with the flood hydrograph assuming the river berm was left intact. The post 1987 earthquake river bank position was used as the river bank failure is likely to have occurred whether or not the upper soils had been removed from the berm.

This analysis did not predict heave of the top three soil layers in the middle of the paddock at 10.00am but possibly smaller heave of the top silt layer above the Tarawera Ash, as found at 7.30am in the original soil model. The phreatic surface reached the ground level at 7.30am, one and a half hours after the original model. The water level in the river continued to rise for ten hours after the breach at 10.00am. It is therefore considered that a breach would still have occurred if the river berm had been left intact, possibly before midday.

10.7 The Effect of the Flood Flow Hydrograph

If Trustpower had received earlier warning of the magnitude of the storm it may have been possible to drain some water from the Matahina Dam to enable storage of a greater portion of the incoming flood peak. This would have reduced the size of the downstream flood peak. If Trustpower had foreknowledge of the entire hydrograph of flow into the lake behind the dam, including the peak river inflow and the total volume, the following hypothetical discharge regime could have been possible (Reference 14).

time	discharge
10.00pm 16 July	120 cumecs
9.00am 17 July	320 cumecs
7.00pm 17 July	584 cumecs, held to 3.00am on 19 July when all storage would be used

Table 5: Possible discharge regime from Matahina Dam

This discharge regime has been converted into a level hydrograph at Sullivan's Breach allowing for flow times and inflows downstream of the dam. Figure 11 shows the resulting hydrograph. Although this regime may have reduced the peak flood flow it would not have significantly reduced the river level at the time the breach occurred, prior to the peak.

It can be seen by comparing the Trustpower modified hydrograph (Figure 11) to the recorded hydrograph (Figure 10) that modification would have resulted in higher river flows on 16 and 17 July. The seepage model was analysed using this hydrograph and it was found that the phreatic surface would have reached ground level in the paddock at 11.00pm on 17 July, compared to 6.00am on 18 July with the measured hydrograph. Heave of the top three soil layers in the paddock could have occurred at about 2.00am.

The comparison of the two hydrographs highlights the influence on seepage response of time at higher than normal river levels in contrast to the influence of actual river level.

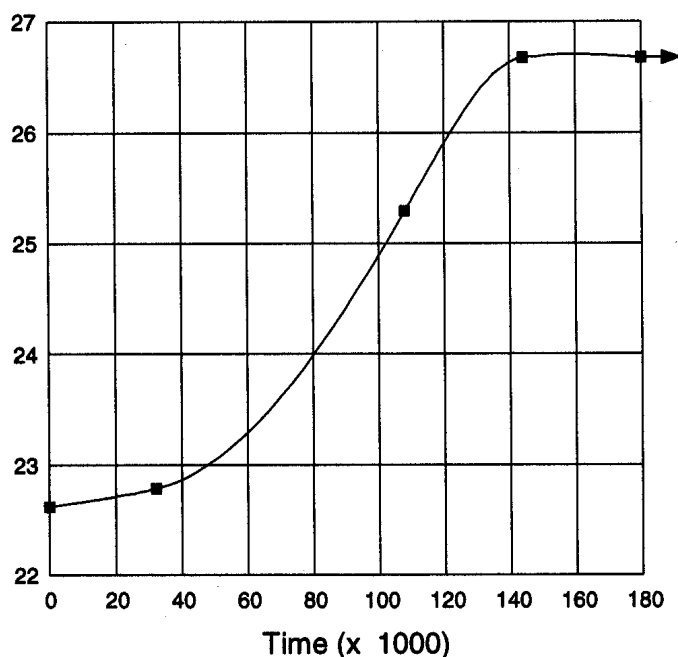


Figure 11: Trustpower modified flood flow hydrograph
(time in seconds from 8am 16 July 2004, stopbank crest 27.5m)

Trustpower has also advised a second scenario which may have been operationally possible if it had been known well in advance that inflows over 500 cumecs were likely. This prediction would apparently have been possible if the quantum of rain in the MetService forecast had been accurate and the Aniwhenua River level gauge had not failed at a critical time. Under this scenario water would also have been drained from the Matahina Dam (but at a lesser rate than discussed above) and the peak discharge from the dam would be reduced to 700 cumecs. In terms of the effects on the stopbank at the breach site this scenario is intermediate between what actually occurred and the hypothetical scenario discussed above.

11 Discussion

The flood during which the stopbank failed was the biggest experienced since the stopbank was built. Previous smaller floods had caused problems at the stopbank around the substation just downstream of the breach site. Following the 1998 flood this section of stopbank was buttressed and filter drains were installed. No problems were observed there in the July 2004 flood. McCrackens Bend lies between the substation and the breach site. The old river bed on both sides of the bend was surcharged for about 50m back from the stopbank toe during initial stopbank construction in 1977. This was carried out because it was recognised that the old river bed formed a potential

seepage path under the stopbank. Sullivan's Breach occurred immediately upstream from McCrackens Bend.

There was local knowledge that the paddock well out from the toe of the breached section of stopbank became spongy in even small floods and this had also been noticed within the section of McCrackens Bend between the two surcharges. It is unlikely that this knowledge was ever passed on to any Regional Council staff.

It is possible that a failure would not have occurred if the stopbank was still at the original height built in 1977. Raising the stopbank to re-instate the desired level of flood protection after the 1987 earthquake increased the head across the stopbank in the design 100 year return period flood by 70%. There would have been some urgency to raise the stopbank in 1989 as flooding could have occurred due to events with relatively short return periods. Fortunately there were no big floods between the earthquake and the time the stopbank was raised. There was a large amount of stopbank reinstatement work required after the earthquake. It is unknown how much consideration was given to site specific investigations and stopbank design.

Due to the highly variable nature of the soils within the Rangitaiki Plains there is always the possibility that soil conditions will be less favourable between sites where investigations are carried out. Therefore even if investigations had been carried out at the breach site, the weakness in the stopbank system (the presence of thin areas in the surface low permeability soil layers) may not have been identified. The holes augered on the landward side of the stopbank at each side of the breach for this investigation showed the upper low permeability soil layers to be a minimum of 1.3m thick. The report writer had the advantage of approximately a 200m length of exposure of the surface soils around the edges of the scour hole which showed areas where the upper layers were as thin as 400mm.

It is difficult to imagine more unfavourable conditions at the time of the stopbank breach than actually occurred. These conditions include the following:

1. The thin layer of low permeability soils above high permeability soils in some areas (less than 1m).
2. The magnitude of the flood in the river.
3. The saturation of the ground by the high rainfall.
4. The fall of the ground away from the landward toe of the stopbank.
5. The use of the river berm as a borrow area.
6. The lack of a good root structure in the surface soils which may have added some minor amount of tensile strength and reduced piping potential.
7. The row of fence posts along the stopbank toe possibly penetrating to the high permeability soils.

It is considered that given the magnitude of the flood the first condition would have been sufficient to cause stopbank failure. The remaining conditions may have just made the failure occur earlier than would have happened otherwise.

12 Conclusions

- It is considered that the stopbank breached in Sullivan's Farm due to the presence of thin areas in the layers of low permeability soils in the paddock behind the stopbank which overlay layers of high permeability soils extending to the river. The designers of the stopbank repairs following the 1987 earthquake appear to have been unaware of the existence and significance of these thin areas of indeterminate extent in the layers of upper soils.
- The stopbank breach initiated in the paddock well out from the toe of the stopbank where it appears the surface soil layers were heaved up by water pressure, allowing water and soil to escape. The high water flow from the underlying highly permeable sands enabled the hole to rapidly work back to the stopbank, causing it to collapse and be overtopped.
- The use of the river berm as a borrow area may have slightly accelerated the failure but did not cause the failure.
- The stopbank would have failed whether or not modifications were made to the river flow rate at the Matahina Dam.

References

1. Tredea, K. (Aug 2004) personal communication, NIWA.
2. Environment Bay of Plenty (2000) Review of the flood carrying capacity of the Rangitaiki River below Edgecumbe, Operations Report 2000/09.
3. Haynes, R.J. (Aug. 1995) Soil structure breakdown and compaction in New Zealand soils, MAF Policy Technical Paper 95/5.
4. Soons, J.M. & Selby, M.J. (1982) Landforms of New Zealand, Longman Paul.
5. Freeze, A.F. & Cherry, J.A. (1979) Groundwater, Prentice Hall International.
6. Bay of Plenty Catchment Commission (1976) Plan R450/22/3 Edgecumbe to Te Teko stopbanks right bank Rangitaiki River.
7. Beanland, S., Berryman, K.R. & Blick G.H. (1989) Geological investigations of the 1987 Edgecumbe earthquake, New Zealand, NZ Jnl of Geology and Geophysics, 1989, Vol. 32.
8. Water and Soil Conservation Authority (1987) Bay of Plenty, Earthquake damage: March 1987, Catchment Works, 75/12/20, Works Committee Submission 381.5.87.
9. Bay of Plenty Catchment Board (1989) Plan R602, Rangitaiki River Scheme – post 1987 earthquake, right bank restoration work cross sections 1800m to 2050m.
10. Ferris, B. (Aug 2004) personal communication, IGNS.
11. Opus International Consultants Ltd (2000), Stopbank assessment Rangitaiki River, Edgecumbe, Geotechnical Report No 2069.
12. Alyamani, M.S. & Sen, Z. (1993) Determination of hydraulic conductivity from complete grain-size distribution curves, Groundwater, Vol. 31, No. 4.
13. Larkin, T.J. et al. (2001) Some geotechnical engineering properties of a pumice sand, Short Course: Geotechnical properties of residual and volcanic soils, University of Auckland, 2001.
14. Lilley, P. (Aug 2004) personal communication, Trustpower.

Appendix A

Soil Logs

Appendix B

Laboratory Test Results

Appendix C

Computer Analysis Results