13th Australia New Zealand Conference on Geomechanics – Acosta-Martínez & Lehane (Eds.)
© 2019 Australian Geomechanics Society, Sydney, Australia, ISBN 978-0-9946261-0-3

Collapse of the Old Pacific Highway, Piles Creek, Somersby

S.J. Clarke, G. Mostyn & B. Shen
Pells Sullivan Meynink

ABSTRACT: On 8 June 2007 a section of the embankment collapsed along the Old Pacific Highway at Somersby, New South Wales (NSW), Australia. The collapse occurred during a heavy rain storm, at a location where three corrugated steel pipes conveyed Piles Creek through the base of the road embankment. Not long after the collapse, a car drove into the resulting void and all the occupants drowned in the flooded creek.

A detailed forensic investigation was undertaken of the Piles Creek site, and also a similar culvert located further along the highway at Leask Creek.

The inverts of the three pipes at Piles Creek had rusted away over their full length.

The Leask Creek culverts were displaying some features which were known to have been present prior to the failure at Piles Creek, including loss of the pipe inverts and a depression in the road surface above the culverts. The Leask Creek embankment contained substantial erosion gullies, an active landslip, and several voids. These conditions were inferred to be indicative of those likely to have been present at Piles Creek prior to its collapse.

The investigation revealed that the Piles Creek culverts were likely to have been perforated at least six years prior to the final catastrophic failure.

The causes of the failure were the loss of the pipe inverts, and subsequent ground loss into the pipes leading to increased erodibility of the loosened and voided fill, with the failure triggered by the flood event.

1 INTRODUCTION

At approximately 3:30 pm on Friday, 8 June 2007 a section of the road embankment on the Old Pacific Highway at Somersby collapsed and was washed away by flood waters caused by a heavy rain storm. The collapse occurred at a location where three corrugated steel pipes conveyed Piles Creek beneath the road embankment.

Not long after the collapse, a car carrying five occupants drove into the gap in the road and all five occupants drowned in the flooded creek.

Once the flood waters had receded, a gap in the road of 11.5 m length was left, with a void up to 7 m depth below the adjacent intact road surface. The top of the pipes were visible in the base of the void.

The failure led to a forensic investigation to assess the cause. The investigation was undertaken between 2007 and 2008.

Factual evidence was collected by technical experts from Connell Wagner Pty Ltd, Jeffery & Katauskas Pty Ltd and Pells Sullivan Meynink (PSM) representing the NSW State Coroner, Gosford City Council (Council) and the NSW Roads and Traffic Authority (RTA) respectively. The RTA is now incorporated within the NSW Roads and Maritime Services (RMS). PSM was assisted by CTI Consultants Pty Ltd in regards to pipe corrosion.

This paper provides a summary of the key findings of the investigation, with the aim of informing road authorities and other experts of the mechanisms which led to the failure, warning signs of the causative mechanisms, and preventing a recurrence of this tragedy. The paper does not revisit the responsibility for the failure – this has already been determined by the Coroner.

The paper includes PSM's interpretation of the collapse mechanism and causes. Key aspects of PSM's interpretation of the mechanisms and timeline to failure were adopted by the Coroner in its findings.

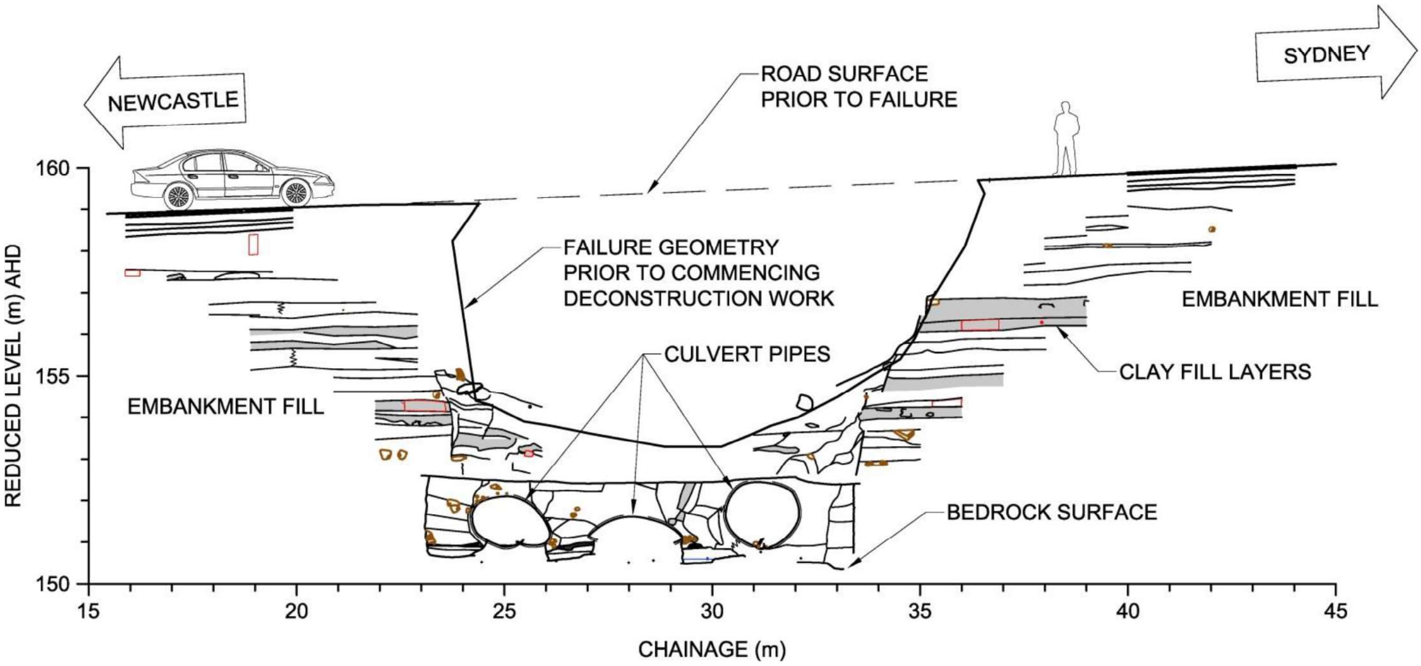


Figure 1. Section along the Old Pacific Highway showing the failure geometry and damaged culvert at Piles Creek.

2 DESIGN AND CONSTRUCTION

The section of the Old Pacific Highway where the failure occurred was completed in 1983 as a deviation to the original Pacific Highway. The work was undertaken for the RTA.

The deviation was required to accommodate the alignment of the nearby F3 freeway (now named the M1 freeway) which was constructed soon after the completion of the deviation.

The Piles Creek culverts comprised three 1800 mm diameter corrugated steel multi-plate bitumen-coated pipes.

Each pipe comprised four quadrants of 3 mm thick galvanised steel which were bolted together. The corrugations were sinusoidal in shape with an amplitude of 50 mm and a wavelength of 150 mm.

At the culvert location, the fill embankment reached a height of 8.5 m above the upstream pipe invert level, with batter slopes of 1.5H:1V. The highway comprised one lane in each direction, with a posted speed limit of 70 km/hr. The road at this location is on a vertical curve, with the low point located approximately 70 m east of the culvert, and 1.5 m lower than the road level above the culvert.

The culverts were sized to pass the 50 year average recurrence interval (ARI) flood event without overtopping the road.

The design required relative compaction of the embankment fill to no less than 95% Standard Maximum Dry Density ratio. The investigation encountered primarily ripped sandstone fill, with post-construction testing indicating densities that achieved the design criteria. Clay fill layers were also encountered during the investigation (Fig. 1).

A nominal 150 mm lime stabilised sub-base layer was present on top of the embankment, comprising sand and sandstone gravel. This was overlain by a 100 mm thick sub-base layer comprising fine to coarse grained igneous gravel. The base course layer was typically 200 mm thick and comprised igneous gravel with a maximum particle size of 25 mm.

The original pavement wearing surface was constructed in June 1983 and comprised a 20 mm primerseal. A 50 mm layer of 10 mm asphalt was placed in April 1984 just before the highway was first opened to traffic.

3 FORENSIC INVESTIGATION

3.1 *Scope*

The forensic investigation that followed the embankment failure included systematic deconstruction of the embankments and culverts at Piles Creek and also at Leask Creek.

The purpose of the investigation was to gather information to aid in the assessment of the causes of failure.

The investigation included the following field work:

- Drilling of geotechnical boreholes.
- Mapping of embankment slopes, road surfaces, and excavation faces during deconstruction.
- In situ density testing, permeability testing, and dynamic cone penetrometer testing (DCP) of embankment fill.
- Extensive laboratory testing of embankment fill.
- Geometric survey of embankment, headwalls, and culverts.
- Measurement of pipe corrosion.
- Inspection of corrugated steel culverts in surrounding districts.

3.2 *Maintenance history*

In 1984 the RTA noted that corrugated steel pipes in the deviated section of the Old Pacific Highway were rusting within six months of being brought into use, with the Piles Creek culverts among those identified. RTA photographs of the upstream headwall in 1990 show rusting of the pipes just above the invert.

With the opening of the F3 freeway in mid 1995, the road was reclassified as a local road with responsibility handed over to Council.

In early 2002 Council noted road subsidence above the culverts. Photos show voids above the pipes behind both headwalls prior to any pavement patching work. These voids are estimated to have been up to 1.5 m wide with a depth of at least 0.8 m below the top of the headwalls.

An aerial photograph taken between December 2002 and January 2003 indicates that a repair to the pavement had taken place by that time. Another aerial photo from February 2004 shows an additional patch in the downstream shoulder above Piles Creek. In 2004 a 100 m^2 area of pavement was repaired with a 50 mm asphalt overlay. At about the same time Council noted that the bottom of the culverts had rusted out and advice was sought on how to repair them.

Investigation of the failure included inspection of asphalt fragments which had been washed up to 80 m down the creek. Smaller fragments were recovered while larger pieces (some up to 2 m across) were mapped in situ. These indicated a total asphalt thickness ranging from 40 mm to 230 mm. The thicker fragments displayed several individual layers, indicating distinct episodes of patching. Comparison of the thicker fragments with the adjacent unpatched pavement suggested that subsidence of at least 150 mm had occurred.

3.3 *Police video*

The NSW Police Force provided a video recording to the Coroner taken on 3 June (i.e. 5 days prior to the collapse). The video was from a camera mounted in a highway patrol car, and included two traverses of the section of highway above the Piles Creek culvert. The video showed a rectangular patched section of pavement, laterally deformed guard rails, and a dip in the patched section (estimated to be in the order of 50 mm deep).

3.4 *Hydrology*

Piles Creek has a catchment of 2.3 km^2 upstream of the Old Pacific Highway. The upstream culvert invert is at RL 151 m while the highest point in the catchment is at approximately RL 250 m. The stream bed lies in predominantly horizontally bedded sandstone bedrock, resulting in an irregularly

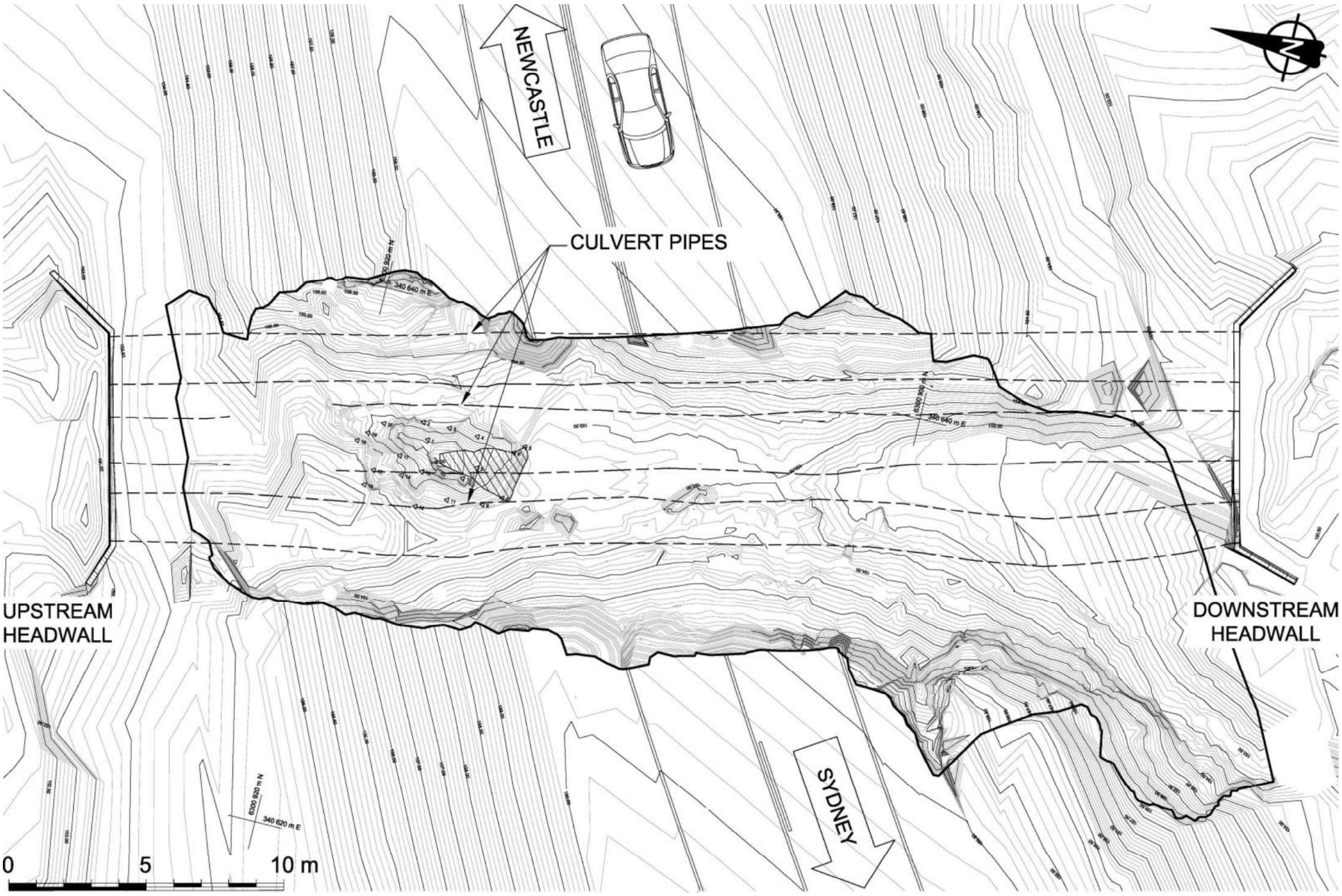


Figure 2. Plan view of the remaining road embankment and failure geometry at Piles Creek.

Table 1. Summary of Piles Creek sandstone fill properties measured during deconstruction.

PROPERTY	TEST METHOD	NUMBER OF TESTS	UNIT	MEASURED VALUES		
				Min	Median	Max
PARTICLE SIZE DISTRIBUTION	AS 1289 3.6.1					
Passing 0.075 mm			%	6	15	20
Passing 0.600 mm			%	54	71	82
Passing 6.7 mm		28	%	63	84	100
Passing 26.5 mm			%	72	90	98
Passing 75 mm			%	85	98	100
PLASTICITY (fraction < 425 μm)	AS 1289, 3.1.1,					
Liquid limit	3.2.1, 3.3.1, 3.4.1		%	15	19	25
Plastic limit		25	%	11	16	18
Plasticity index			%	1	3	11
Linear shrinkage			%	0.5	1.5	4.5
DENSITY (Nuclear densometer)	AS 1289 5.1.1,					
Field wet density	5.4.1, 5.8.1		t/m^3	1.92	2.11	2.24
Field dry density		41	t/m^3	1.78	1.92	1.99
Density ratio (Standard)			%	92.0	99.0	102.5
DENSITY (Sand replacement)	AS 1289 5.1.1,					
Field wet density	5.3.1, 5.4.1		t/m^3	1.93	2.11	2.23
Field dry density		40	t/m^3	1.79	1.92	2.00
Density ratio (Standard)			%	93.0	99.5	103.5
MOISTURE CONTENT	AS 1289 2.1.1,					
Field moisture content	5.4.1		%	4.6	10.0	14.7
Optimum moisture content		83	%	9.5	11.0	14.5
Moisture variation			%	6.0 dry	1.0 dry	3.5 wet
STRENGTH						
Shear box, peak (3 points per test)	AS 1289 6.2.2	12	kPa	Linear:	$\tau = 7.7 + \sigma_n.\tan 38.9^\circ$	
			kPa	Power:	$\tau = 1.56\ \sigma_n^{0.88}$	
Shear box, residual (3 points per test)		12	kPa	Linear:	$\tau = \sigma_n.\tan 32.8^\circ$	
			kPa	Power:	$\tau = 0.44\ .\ \sigma_n^{1.08}$	
California Bearing Ratio (99% to 100% Standard, 2.5 mm penetration)	AS 1289 6.1.1	8	%	14	37.5	45
PERMEABILITY						
Falling head (99% Standard, OMC)	AS 1289 6.7.2	6	m/s	2.6×10^{-8}	5.2×10^{-8}	1.9×10^{-7}
Constant head (99% Standard, OMC)	AS 1289 6.7.1					
Guelph permeameter		13	m/s	3.4×10^{-8}	9.0×10^{-7}	5.2×10^{-6}
Double ring infiltrometer		2	m/s	4.0×10^{-7}	6.6×10^{-7}	9.2×10^{-7}
ERODABILITY						
Emerson crumb	AS 1289 3.8.1	16	-	5	6	6
Pinhole dispersion	AS 1289 3.8.3	4	-	ND1	-	ND2
UNSW hole erosion procedure (6.5 mm hole)		4	I_{HET}	2.70	2.90, 3.75	3.81

stepped longitudinal profile. Creek banks comprised sands and silts which were generally stabilised by vegetation.

Considerable industrial development had occurred in the catchment but the drainage lines were largely undeveloped. Upstream of the culverts Piles Creek had patches of gravelly sand in the creek bed and banks.

Two weather stations located 4 km north of the Piles Creek catchment record rainfall at 15 minute and 10 minute intervals. These indicated a total rainfall depth of 170 mm and 200 mm for 8 June 2007. The critical duration for the catchment was estimated to be about two hours. Rainfall during the two hour period preceding the failure totalled 65 mm and 85 mm for the two stations, with a maximum rainfall intensity of approximately 80 mm/hr.

Two flood peaks of similar magnitude occurred on 8 June 2007, with the first in the early hours of the morning, and the second from 2 pm to 4 pm. The maximum flow was estimated to be 35 m^3/s to 45 m^3/s, with an ARI of 10 to 25 years. Upstream debris marks suggested a maximum pool level at around RL 155 m, which is 3 m below the low point in the road.

There was no evidence that the highway had been overtopped by the flooded creek.

3.5 *Field mapping and measurement*

Forensic field work included methodical deconstruction of the remaining fill embankment and culverts. A 3 m wide trench was progressively excavated from the road surface down to the base of the culverts in depth increments of 0.5 m.

Geotechnical conditions were mapped at a scale of 1:10, with particular attention paid to the disturbed fill present around the culverts. Some of the disturbance was due to the flooding at the time of the failure, though disturbance to other areas was associated with pre-existing ground loss caused by loss of the pipe inverts.

Detailed measurements were undertaken of the gap width in the pipe inverts (Fig. 8).

3.6 *In situ and laboratory testing*

The embankment fill predominantly comprised compacted ripped sandstone. Table 1 provides a summary of the in situ and laboratory geotechnical testing undertaken of this material and the results obtained. These results are typical in comparison to other engineered fills in the region.

In addition to the sandstone fill, there were also several fill layers comprising low plasticity pale grey clay in the embankment. The thickness of these layers ranged from 0.15 m to 0.8 m. Two triaxial shear tests were performed on samples of the clay fill which indicated effective shear strength parameters of $c' = 8$ kPa and $\phi' = 29^\circ$. In situ pocket penetrometer testing indicated very stiff to hard consistency for clay which was assessed not to have been disturbed, whereas for clay fill which was displaced or disturbed, the consistency was about a third to half that of the undisturbed samples.

Storm events which occurred during the deconstruction work resulted in inundation of the culverts and exposed fill. The pale grey clay fill was highly resistant to erosion during these events.

Chemical testing of the water from Piles Creek indicated normal conditions. Laboratory testing was also performed on samples recovered from the concrete headwalls and steel culverts (Shackel & Retsos, 2012).

4 LEASK CREEK CULVERT

4.1 *Comparison with Piles Creek culvert*

Not long after the failure at Piles Creek, the Old Pacific Highway was closed where it passes over Leask Creek, located approximately 1.4 km west of the Piles Creek culvert. The culvert at Leask Creek was similar to that at Piles Creek, and was displaying features which were considered relevant to the failure at Piles Creek, including a depression in the road surface, distorted guard rails, and loss of the pipe inverts.

Council decided to replace the culvert with a new structure. The embankment and culvert were investigated and deconstructed in a similar, but less detailed, manner to that performed at Piles Creek. Because the embankment was essentially intact, features that were washed away during the failure at Piles Creek were still present. Examination of these offered an invaluable insight into the mechanisms that led to the failure at Piles Creek. The Leask Creek culvert was constructed as part of the same contract as Piles Creek, by the same contractor, and using the same suppliers, fill, and construction methods. Table 2 presents a comparison between the Leask Creek and Piles Creek culverts.

The Leask Creek catchment encompasses the former Old Sydney Town tourist development including a storage dam located about 200 m upstream of the culvert. The dam captures runoff from the majority of the catchment, and has the effect of attenuating flood flows and trapping sediment. The Piles Creek catchment has no

Table 2. Comparison between Leask Creek and Piles Creek culverts.

FACTOR	UNIT	PILES CREEK	LEASK CREEK
EMBANKMENT			
Fill height above pipe obvert	m	6.6	8.0
Batter slopes	-	1.5H:1V	1.5H:1V
PIPES			
Age (at time of Piles Creek failure)	years	25	25
Number	-	3	2
Diameter	mm	1800	1500
Overall width of pipes	m	7.5	4.2
Length	m	41	40
As-built longitudinal gradient	%	1.4	2.7
CATCHMENT			
Land use	-	Industrial and undeveloped	Undeveloped, includes 4 ha dam
Area	km^2	2.3	1.5
Mainstream length	km	1.9	1.4
Overland slopes	%	5 to 15	3 to 12

sediment trapping storage and has been subject to considerable industrial development over the life of the culverts. It was assessed that the sediment load, particularly coarse sediment, at Piles Creek may be an order of magnitude greater than at Leask Creek.

For similar rainfall events, it was calculated that the flow velocity through the Leask Creek culverts was about 130% of that for the Piles Creek culverts. The Leask Creek culvert was therefore similar to the one that had failed at Piles Creek in terms of construction, age, and flow conditions. The main difference between the culverts was the lower sediment load at Leask Creek.

4.1.1 *Pavement, guard rails, and kerbs*

At the time the highway was closed at Leask Creek (June 2007), there was a noticeable depression in the road above the culvert, though the pavement had not been patched. Survey along the road performed in December 2007 showed that the depression was located directly above the culverts, and was about 65 mm deep in the centre of the road, and between 75 mm and 95 mm deep along the continuous paint lines denoting the edges of the lanes. The depression extended about 8 m to 12 m along the road.

The guard rails were observed to have deflected laterally, similar to that observed in the police video of Piles Creek taken prior to the failure. Survey of the guard rails indicated that they bulged away from the road between 40 mm and 120 mm along the section where the embankment had settled.

Concrete kerbs were located on each side of the road beneath the guard rails. In December 2007 these exhibited significant tilting, creating a dip of 200 mm to 300 mm, and steps of up to 65 mm between adjacent segments. These movements allowed stormwater to bypass the kerbs and flow down the unprotected slopes and disturbed fill, rather than flowing to the stormwater drain located further along the road.

4.1.2 *Upstream headwall and embankment*

The upstream concrete headwall was undamaged, though the invert of each of the steel pipes was absent, with a gap width of about 0.5 m, and rust extending about 0.1 m from the jagged edges. The black coating inside the pipe was absent on the upstream side of the corrugations over the bottom portion of the pipe.

Initially each of the embankment faces were covered by dense vegetation which significantly impeded observation of surface features. The vegetation was cleared, and the surface mapped (Fig. 3 and Fig. 4). Erosion gullies of up to 2.8 m depth were evident, as well as slip scarps of up to 0.8 m height. The main gully continued up the slope and beneath the concrete kerb, forming a hole up to 0.7 m depth between the edge of the road and the kerb. A void on one side of this hole was measured to extend about 2.4 m horizontally beneath the edge of the road surface (upper void).

Comparison of fill layer levels on either side of the gully suggested that the fill above the pipes had settled at least 0.1 m at this location.

A section of plain concrete was present over the surface just behind the headwall, with a void present immediately beneath it. The void had plan dimensions of 2 m by 4 m, with a height of 0.6 m. One of the pipes was visible along one side of the void.

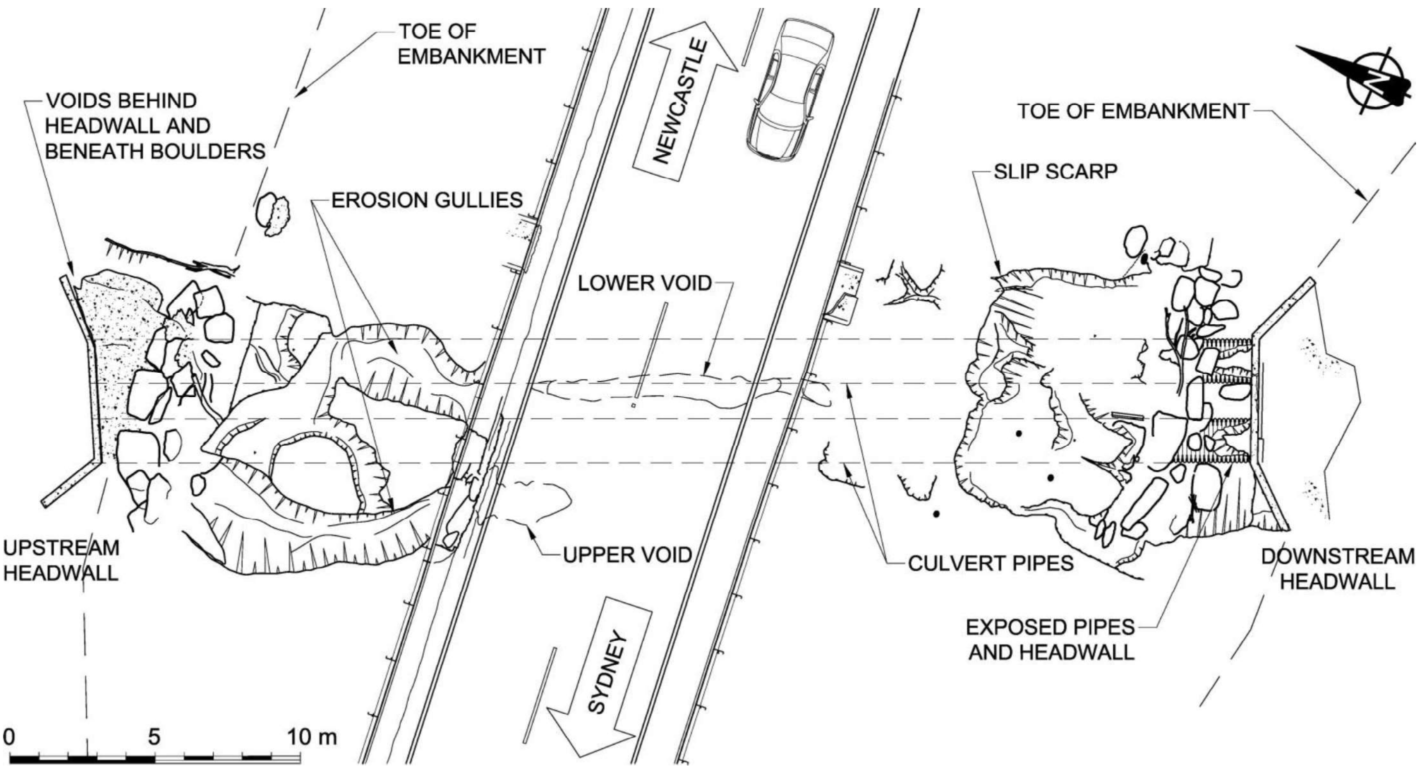


Figure 3. Plan of Leask Creek culvert, showing main distress features found during the investigation.

A tree trunk with a diameter of about 0.25 m was lying horizontally on the embankment face, though was undisturbed where it met the ground surface, suggesting large rotational movement of the ground at this location.

Numerous sandstone boulders of up to 1.5 m size were present on the embankment face above the headwall. Voids were found beneath some of these of up to 1.5 m length, and were located above the pipes.

DCP testing was undertaken to assess fill conditions. This was done both within the area above the pipes, and also on undisturbed regions of the embankment face. Test depths ranged from 1.2 m to 3.7 m. For depths below about 0.5 m the tests away from the culverts gave blow counts of 10 to 15 blows/100 mm, while the tests above the culverts typically recorded blow counts about half this, indicating that the fill was disturbed.

4.1.3 *Downstream headwall and embankment*

The downstream embankment included many of the features of the upstream embankment. The main differences were that the headwall was badly damaged, the pipes were exposed, and a large slip had developed in the embankment slope above the pipes.

The headwall was cracked above and below each pipe, with crack widths of up to 40 mm. Some cracks were stepped by 40 mm to 90 mm. The pipes had pulled away from the concrete, creating gaps of up to 80 mm. The concrete apron had cracked into large fragments and had dropped up to 800 mm relative to the headwall.

A large void was located behind the downstream headwall, between and either side of the pipes, and below the pipes and headwall. The void extended up to 3 m from the rear face of the headwall, and 1 m below the pipe invert level.

A slip scarp was located about halfway down the slope, and had a width of 8 m to 10 m (Fig. 3). The back scarp was up to 1.2 m high. The fill within the slip was saturated.

DCP testing undertaken on the downstream embankment face indicated similar results to that described for the upstream embankment, indicating a zone of loosened fill above the pipes.

4.1.4 *Rates of movement*

Several indicators of rates of movement were available at Leask Creek. These included surface extensometers, road surface survey, survey of targets on the embankment slopes, and photographs.

Six surface extensometers were installed during the investigation to monitor deformation of the embankment faces and downstream headwall. The extensometers on the upstream face of the embankment indicated virtually no movement, though those on the downstream face moved between 20 mm and 50 mm over a period of about two months, with those on the headwall moving 20 mm to 40 mm over the same period. The rates of movement during this period corresponded closely to rainfall, with between 0.1 mm and 0.2 mm movement occurring per millimetre of rainfall.

Survey monitoring of the road surface over a period of four months indicated 13 mm settlement during this period.

Survey results also revealed that the depth of the erosion gully on the upstream face increased by 2 m over a period of three months. The same data indicated that the slip on the downstream face moved about 0.7 m vertically.

An aerial photo of the Leask Creek site taken on December 2006 showed no indication of the hole on the upstream side of the road between the kerb and the edge of the road. Therefore this feature developed within a period of less than a year.

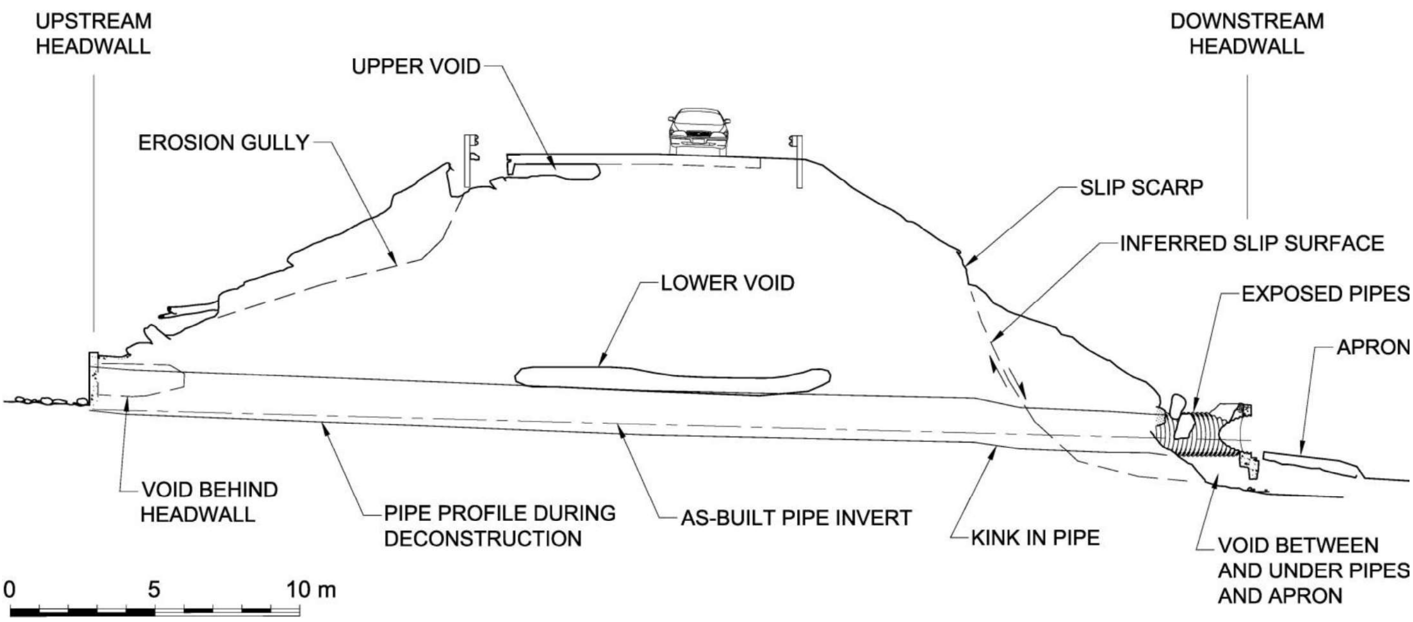


Figure 4. Long section through Leask Creek culvert, showing features found during the investigation.

4.2 *Subsurface features*

The embankment and culverts were deconstructed in a systematic manner similar to that employed at Piles Creek, except that excavation occurred in 1.5 m depth increments. Features observed included voids, cracks, disturbed fill, and displaced and damaged pipes.

Voids behind the headwalls and beneath the boulders on the upstream face were apparent prior to the deconstruction works, as described above. Two additional voids were found during deconstruction of the embankment (Fig. 3 and Fig. 4).

4.2.1 *Upper void*

The roof of the upper void comprised a stabilised pavement layer, about 0.4 m below the road surface. The void had a height of 0.6 m, and extended 1.5 m along the road axis, with a width of 3 m, most of which was beneath the traffic lane. This feature is essentially a continuation of the main erosion gully observed on the upstream embankment face. The surfaces on the inside of the void included horizontal scum lines indicating periods when it had been partially filled with water. There were also rotted grass and leaf residue on the roof of the void which suggested that it had been filled with water at some time.

A series of subvertical cracks up to 50 mm width and striking parallel to the pipes were observed close to the upper void. The floor of the void and underlying material comprised a zone of loose sand, which was inferred to be disturbed and transported fill material. Additional excavation revealed that the cracking and pockets of disturbed fill (infill) extended down to about 4.3 m below the road surface and up to 8 m either side of the road centreline, coinciding with the base of the erosion gully on the upstream face. Zones of orange-brown and red-brown staining extended up to a few metres from the cracks.

Cracking was observed on the Sydney side of the culverts, though not in the 3.5 m zone above the pipe obverts.

4.2.2 *Lower void*

The lower void was encountered just above the obvert level of the pipes (Fig. 4). It ran parallel to the pipes over a distance of about 10 m, located in the centre of the embankment, with a span of 0.7 m and a height of 0.8 m. The roof of the void was arched and the floor was flat and underlain by a zone of disturbed fill that extended down and around the pipe to the invert.

4.2.3 *Disturbed fill*

Zones of disturbed fill were found either side of the pipe on the Newcastle side, as well as beneath the lower void.

Some of the cracked zones encountered during deconstruction included enlarged zones (up to 0.5 m width) filled with disturbed material which was inferred to have been transported by flowing water along cracks in the embankment.

4.2.4 *Pipe deformation*

The pipes had not moved at the upstream headwall. The as-constructed position of the pipe invert was less than 0.4 m above bedrock level over most of their length, but for the downstream portion this increased to about 1 m.

Survey of the pipes during deconstruction indicated that the downstream headwall had settled by about 0.6 m, and that the pipe on the Newcastle side had settled by around 0.35 m over much of its length. The pipe on the Sydney side had settled less than the Newcastle side, and may explain why the cracking within the embankment was only observed on one side of the culverts.

4.2.5 *Downstream embankment slope failure*

The pipe survey also indicated the presence of a change in level of about 0.2 m over a length of 1.5 m about 9 m from the downstream headwall. Based on this kink, DCP test results, survey monitoring, and the scarp observed on the surface, the location of a slip surface was inferred (Fig. 4).

Stability analysis of the slope indicated that failure would not occur for the original embankment slopes, strength parameters corresponding to compacted fill, and low groundwater levels. Further analysis indicated unstable slope conditions associated with over-steepening from loss of material around the headwall, reduced strength of the disturbed fill, and higher groundwater conditions, consistent with the observed slip surface.

5 PIPE CORROSION MECHANISM

5.1 *Service life*

The actual service life of the culverts at Piles Creek and Leask Creek was approximately 20 years (i.e. to complete loss of invert).

There are a number of methods available to assess the design life of corrugated steel culverts. Birtles (1968) tabulates observations of 116 corrugated steel culverts in NSW. These observations include the condition of the culvert, structural and material ratings, and estimated remaining life. Most of the culverts inspected were less than 10 years old at the time of his inspection. Review of Birtles' data suggests that 8% of culverts had an estimated life of less than 20 years, with a median life of approximately 50 years.

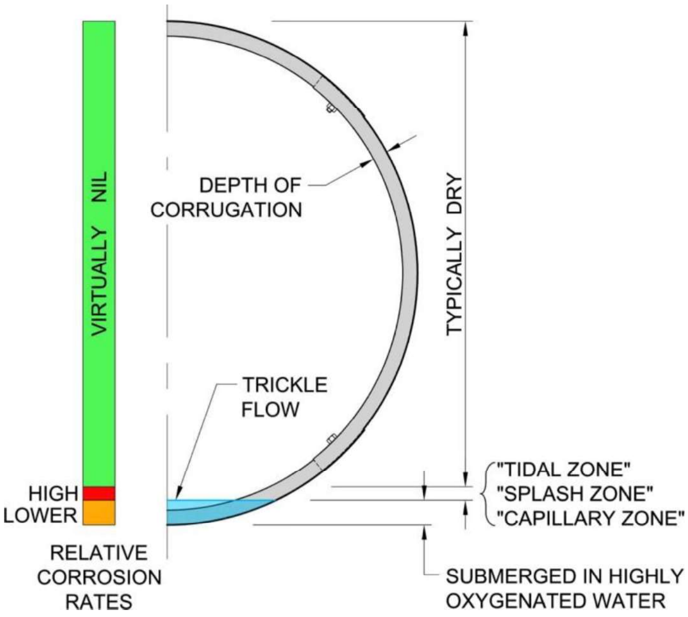


Figure 5. Relative corrosion rates in corrugated steel pipes.

Application of common design methods indicates that the Piles Creek and Leask Creek culverts would have an expected service life of around 50 years.

5.2 *Site observations*

Three zones of corrosion are shown in Figure 5:

1. The upper portion is normally dry and exposed only to atmospheric corrosion. The corrosion rate is virtually nil. The bituminous coating was still present over most of this area at both Piles Creek and Leask Creek.
2. The lowest portion is permanently submerged by trickle flows. It is exposed to highly oxygenated water and thus presents a moderately corrosive environment. It is subject to periodic erosion by hydraulically transported sand, which is sufficient to remove the products of corrosion (e.g. rust), particularly on the upstream sides of the corrugations.
3. A small zone is located above the trickle flow level and subject to continual wetting and drying. Various terms are used to describe the zone just above the water level, e.g. tidal zone, splash zone, and capillary zone. It experiences high corrosion rates as well as periodic erosion, particularly on the upstream sides of the corrugations.

There was minimal corrosion observed on the outside surface of the exhumed pipes.

Figure 6 compares the rate of metal loss for an erosive environment against non-erosive conditions (e.g. stagnant). In stagnant conditions the rust layer inhibits diffusion of oxygen to the metal, such that the rate of material loss reduces with time. For an erosive environment the corrosion products are periodically removed, such that the process recommences at the initial higher rate of corrosion. This erosion mechanism is associated with removal of the weaker products of corrosion rather than of the base metal itself. It follows that an erosive environment results in an increased rate of material loss compared to a non-erosive environment.

Figure 7 shows the application of the above principles to a corrugated steel pipe subject to a continuous low flow with occasional high sediment flows. There are seven stages shown on the figure, though one stage dominates in the culvert at any time. The bituminous coating is lost after a few storms on the upstream side of the corrugations, exposing the zinc coating.

Corrosion of the zinc starts along the waterline, followed by corrosion of the underlying steel. The extent of the corrosion increases and follows the upstream side of the corrugations in the invert. The first perforations occur on the ridges along the waterline, and then coalesce to form a broad “U” shape, before finally joining to liberate a “canoe” shaped section of corrugated steel. Thus the progression from initial perforation to complete loss of the invert is quite rapid.

The mechanism illustrated in Figure 7 was supported by observations of the recovered Piles Creek and Leask Creek pipe culverts, as well as several other corrugated steel culverts inspected in nearby districts which were at varying stages of this process. The liberation of the canoe-shaped section occurs over a short period of time, such that the chance of observing this stage is low, but “canoes” were recovered during the investigation. Gadsby (1974) describes corrosion mechanisms observed in culverts in Victoria that are consistent with that shown in Figure 7.

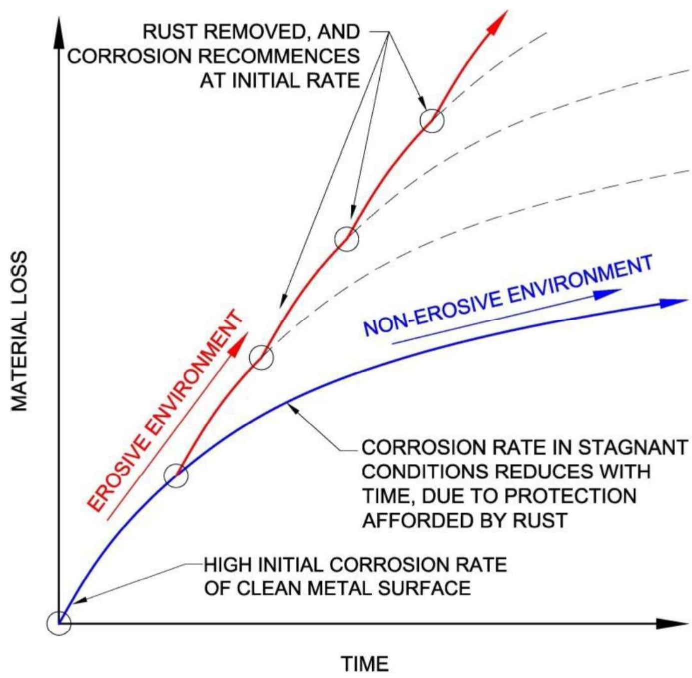


Figure 6. Rates of material loss from corrosion in erosive and non-erosive environments.

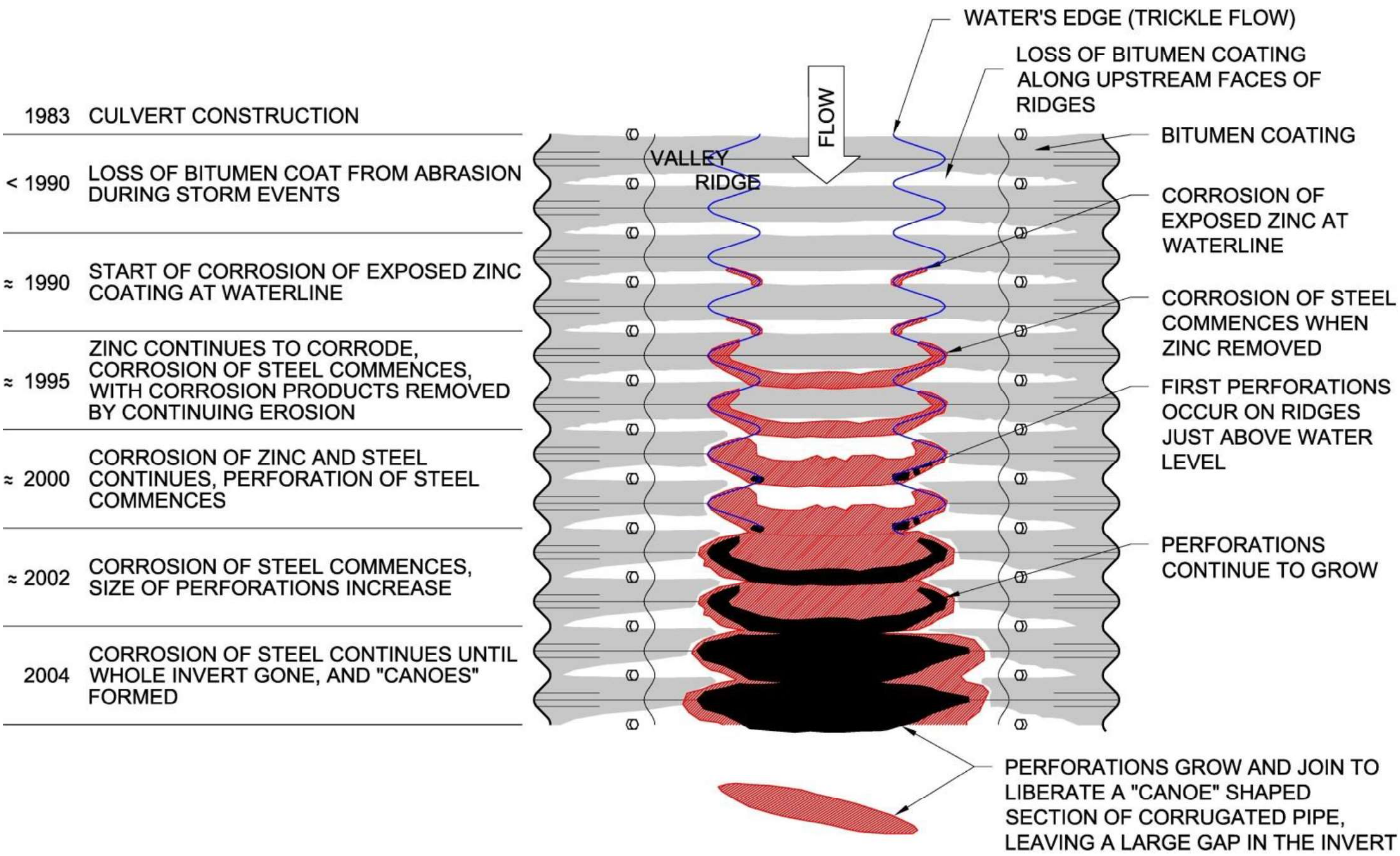


Figure 7. Plan of pipe invert showing progressive stages of corrosion. These stages are shown on one section of pipe for the purposes of brevity only – in reality only one stage occurs at a time.

Figure 8 shows a graph of the measured circumferential gap in the invert of the Piles Creek and Leask Creek pipe culverts, together with the arc length of the intact pipes submerged by trickle flows. It can be seen that the extent of metal loss coincides with the width just above the trickle flow level.

6 FILL DISTURBANCE AND PAVEMENT SETTLEMENT MECHANISMS

6.1 *Process overview*

Ground loss is the term used to describe soil which is completely removed from its initial position, rather than just being displaced. In an earth fill road embankment, ground loss occurring at culvert level results in fill disturbance, void formation, and pavement settlement. These mechanisms are inter-related and may occur simultaneously.

6.2 *Ground loss*

Ground loss mechanisms are associated with complete loss of the pipe invert and are shown diagrammatically in Figure 9:

- Vertical pipe translation.
- Pipe contraction.
- Silo action.

Vertical pipe translation involves the downwards movement of a rigid pipe section, with the material beneath the pipe removed by flowing water. This is a similar mechanism to how caissons are sunk into the ground. The Newcastle pipe at Leask Creek exhibited this type of behaviour.

Pipe contraction refers to the inwards movement of the pipe walls due to the external soil load applied to the pipe and the reduction in stiffness and strength resulting from total loss of the invert. The material loss is the difference in area between the original pipe size and the deformed pipe geometry.

Silo action refers to fill moving down around the outside of the pipe, beneath the pipe, and finally removed by water flowing along the pipe. During the deconstruction at Piles Creek the silo action mechanism was evident as a layer of sandstone gravel located immediately around the outside of the pipe. This was probably caused by the coarser particles becoming trapped between the pipe corrugations, such that only the sand and finer sized particles were transported all the way around the pipe wall to the invert.

A fourth mechanism involves flushing of finer particles from the fill around the outside of the pipe. Once the invert is lost, water in the pipe is hydraulically connected to the groundwater in the fill around the pipe, such that variations in water level in the pipe result in water flowing in and out of the invert as the water level outside the pipe equilibrates with the level inside the pipe. This

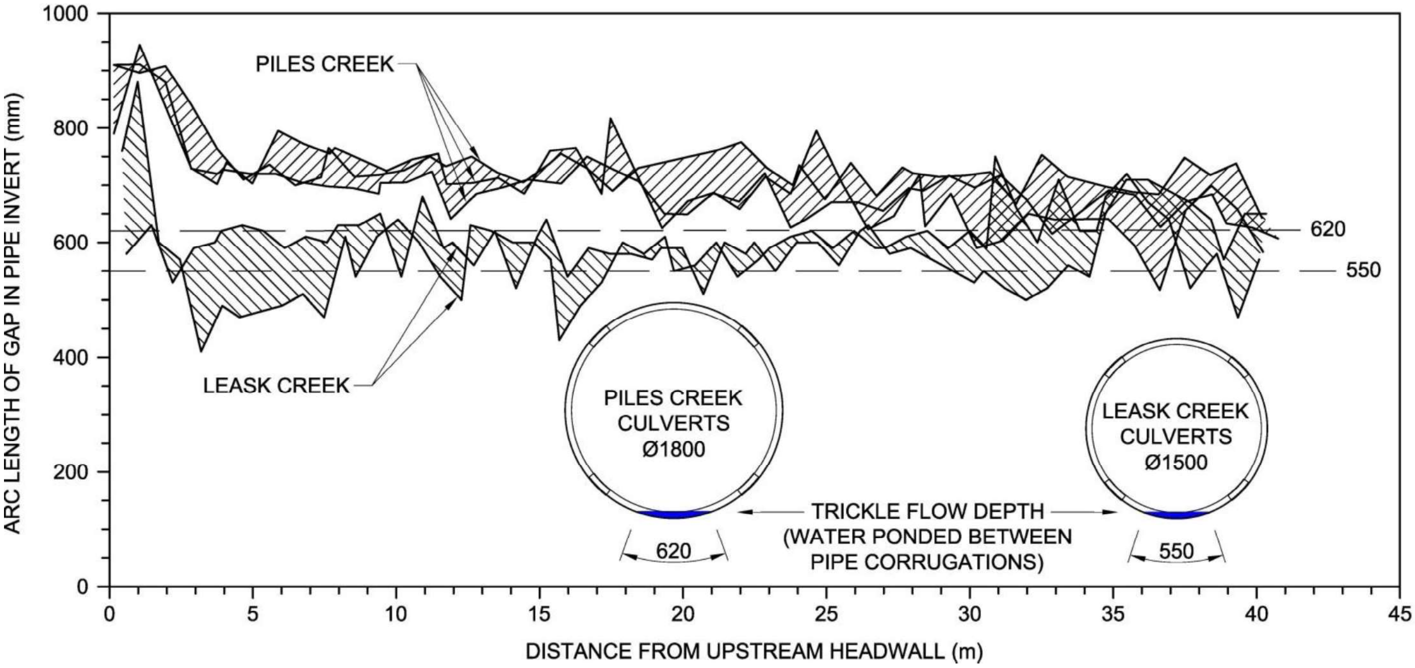


Figure 8. Comparison of gap length in Piles Creek and Leask Creek pipes with length of pipe invert inundated by trickle flows.

flushing action tends to remove the finer sized fill particles, leaving coarse grains around the pipe, as was observed during construction (Fig. 9c). The rate at which the silo action occurs is dependent on the rate of water level changes, and on the frequency and magnitude of such events. Based on the comparison of hydrology presented in Table 2, Leask Creek would be much more benign in these respects than Piles Creek.

6.3 *Fill disturbance and void formation*

Fill becomes disturbed when it undergoes transportation or reworking and loses its original structure, with associated reductions in stiffness, strength, and density (i.e. loosening). It may also experience segregation in which some of the particle sizes are separated from the remaining fill (Fig. 9c).

The ground loss mechanisms shown in Figure 9 result in disturbance to the fill where it fails and displaces towards the "lost" material, or formation of voids where the fill has sufficient strength.

The fill adjacent to the pipes is likely to be periodically saturated by minor flood events, such that it loses strength that it previously had due to negative pore pressures, and is thus less likely to be able to sustain formation of voids.

Fill at higher levels (e.g. above the pipe obverts) is less susceptible to being saturated, and thus more likely to be able to form and sustain voids for significant periods (e.g. as was observed at Leask Creek).

The very stiff to hard clay fill layers found at Piles Creek would have provided stable roof conditions for voids.

6.4 *Embankment settlement*

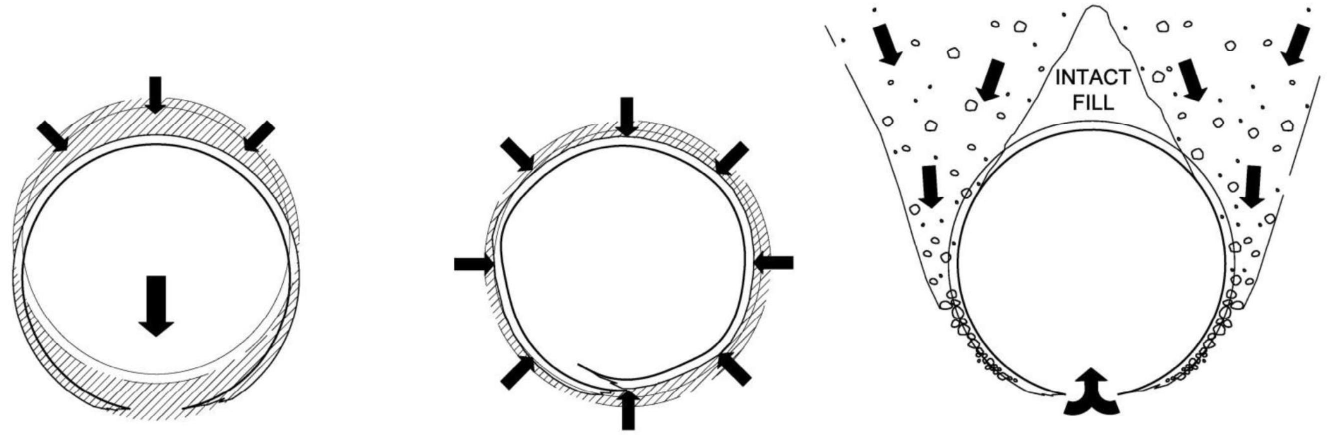
The disturbance to the fill around the pipe from ground loss resulted in settlement of the overlying fill embankment. A total surface settlement of 280 mm was assessed to have occurred at Piles Creek with 65 mm measured at Leask Creek.

Finite element analysis was performed to investigate this mechanism and assess the degree to which road surface deformation is indicative of ground loss occurring at pipe level.

The analysis involved simulation of ground loss by incrementally lowering the base of the finite element model over the width of the culverts, then calculating the magnitude and extent of the resulting surface settlement.

This analysis showed that, for the same degree of ground loss, the surface deformation at Leask Creek was about 70% of that calculated for Piles Creek. This difference is explained by the relative width/cover ratios of the two locations. Piles Creek has a ratio of 7.5/6.6 = 1.1, whereas Leask Creek is 4.2/8 = 0.5, meaning that the Piles Creek geometry is closer to being one dimensional, in which case all the ground loss would be exhibited as surface settlement. The Leask Creek geometry is more amenable to arching of the fill at pipe level. The significance of this is that the magnitude of ground loss at pipe level is related to both the culvert geometry and the observed surface settlement.

The analysis also showed that the ground loss at Piles Creek was more than double that at Leask Creek.



(a) Vertical translation (caisson sinking) (b) Contraction (c) Silo action

Figure 9. Ground loss mechanisms associated with loss of pipe invert.

7 PIPING AND EROSION

Piping is the term which describes an internal erosion mechanism in an earthfill dam, or similar structure. The process involves water flow through an embankment that initially removes fine particles, eventually creating a longitudinal void (pipe), which then permits much more rapid erosion and dam failure. Only the latter part of this process is relevant to the collapse at Piles Creek, because the longitudinal flow path was already formed by ground loss mechanisms associated with loss of the pipe inverts. The disturbed fill was thus highly susceptible to rapid erosion and collapse during flood events.

A series of laboratory tests were undertaken on compacted samples of the embankment fill to assess their resistance to piping failure and erosion.

Emerson crumb tests indicated ten soils with Emerson Class 6 and six of Class 5. Fell et al (2005) states that "soils with Emerson Class 1 to 4 need to be treated with extra caution in dam construction".

Four pinhole dispersion tests were completed, with two giving a classification of ND2 "non-dispersive", and two of ND1 "completely erosion resistant". Fell reports Sherard et al (1976) as finding that soils classified as ND1 or ND2 had not suffered piping failure in earth dams nor severe erosion damage by rainfall in embankments and natural deposits.

These standard tests are relevant for intact compacted earthfill, though less applicable to fill which is disturbed.

Additional erosion tests were undertaken using a methodology developed at the University of New South Wales (UNSW) which employs a 6.5 mm hole drilled through the sample rather that the 1 mm pinhole used in the standard test. The four test results indicated erosion rate indices described as "moderately rapid" to "very rapid".

The failure of the Piles Creek embankment involved the erosion of approximately 3000 tonnes of fill within a few hours. The results of the UNSW tests appear to provide a closer analogue to the rapid erosion which occurred at Piles Creek.

8 PILES CREEK EMBANKMENT FAILURE

The failure sequence of the highway embankment commenced with the corrosion of the three corrugated steel pipes, which started soon after construction. Corrosion continued for about 20 years, until complete loss of the invert occurred over a short period around 2004, instigating various ground loss mechanisms. The ground loss process continued over several years, causing disturbance of the fill around the pipes, formation of voids above the pipes, and propagation of disturbed fill up through the embankment leading to settlement of the highway pavement. It is likely that stable voids were present beneath the clay fill layers. Disturbance of the embankment slopes at the upstream and downstream ends resulted in slope instability, potentially loss of material from around the downstream headwall, and rotation of the guard rails. This represents the likely condition of the embankment in the days prior to the collapse.

The flood of 8 June 2007 would have encountered a void-riddled embankment with disturbed fill daylighting in the embankment slopes above the upstream and downstream headwalls. The floodwaters saturated the loose disturbed fill and inundated open voids. The head differential between the upstream and downstream ends of the culvert resulted in longitudinal flow through the fill. The flowing water rapidly eroded the disturbed fill, creating a large void above the pipes and undermining the highway pavement. The pavement bridged this void for a while before subsiding, cracking, and collapsing in rapid succession to reveal a torrent of floodwater about 2 m below the adjacent road surface.

9 CONCLUSIONS

The Piles Creek and Leask Creek culverts were not unusual:

- Sandstone fill employed in their construction is typical of the region.
- Creek catchments were not abnormal.
- Water and fill chemistry was benign.
- Nature and rate of corrosion of the corrugated steel culverts was similar to that described in the literature.

The complete loss of the inverts from corrugated steel pipes occurs over a relatively short period, and initiates several ground loss mechanisms which develop quickly and lead to disturbance of the fill embankment and surface settlement.

Void formation is initially a two-dimensional process, and is not associated with longitudinal flow (apart from removal of the displaced fill).

Embankment distress indicators associated with the loss of the inverts from the pipe culverts included:

- Presence of voids behind headwalls.
- Embankment slope instability and localised erosion features.
- Outwards deformation of vehicle barriers.
- Settlement of road surface.

The presence of extensive zones of loosened fill and longitudinally extensive voids results in a greatly increased susceptibility of the embankment to rapid erosion during flood events.

The culvert deterioration and embankment failure processes observed at Piles Creek and Leask Creek are unlikely to be unique. Other occurrences of embankment collapse are probably not investigated to the level of detail at Somersby, and may be incorrectly attributed to embankment overtopping or washout.

Repair techniques involving invert replacement or lining of pipes which are undertaken once ground loss mechanisms are well-established (e.g. causing pavement settlement) may be too late as the fill has already loosened and is likely to contain voids. Such repairs need to be undertaken before the embankment fill is disturbed and its resistance to rapid erosion compromised.

ACKNOWLEDGEMENTS

The authors wish to acknowledge the collegial attitude of the other experts involved in the Coronial investigation, and the professionalism of the Gosford City Council workers undertaking the painstaking deconstruction work. This paper includes specialist work undertaken by Fred Salome on culvert corrosion, hydrological assessment by Bill Meynink, and review of historical documents by Dr Philip Pells. RMS have provided permission to publish this paper, though it is noted that the opinions expressed in this paper are not necessarily endorsed by RMS.

REFERENCES

Birtles, J. 1968, A survey of corrugated steel culverts in New South Wales with special regard to durability, July – August 1967, *Local Government engineers Conference, April 1968*.

Fell, R., Stapledon, D., and Bell, G. 2005, Geotechnical engineering of dams, *AA Balkema.*

Gadsby, W.P. 1974, The corrosion of galvanised steel culverts, *7th Australian Road Research Board Conference, Adelaide*.

Shackell, C., and Retsos, P. 2012, Piles Creek embankment collapse – The investigations, the Expert's findings and subsequent improvements in asset management of culvert structures on major roads in NSW, *Australian Geomechanics Society, Sydney Chapter Symposium, October 2012*.