Discussion of "Overtopping Breaching of Noncohesive Homogeneous Embankments" by Stephen E. Coleman, Darryl P. Andrews, and M. Grant Webby

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The discusser congratulates the authors for their authoritative work. Their observations of breach crest curve and streamline pattern in the breach show a challenging similarity with the inlet designs of minimum energy loss (MEL) culvert and weirs on which the discusser will expand. Fig. 1 illustrates two proven designs that have operated successfully for over 30 years. Earlier, McKay discussed the similarity with natural scour: "It became obvious that the forms required [for the MEL design] were common natural shapes," "one natural occurrence is the inevitable bar at the mouth of an alluvial river" while "the same shape occurs in the scour holes below restrictive bridges, culverts and even pipes" (McKay 1970, p. 10). Visser et al. (1990) conducted a prototype experiment with a 2.2-m-high dyke breached during rising tide. Photographs of the breach profile illustrated an hourglass profile similar to the authors' observations and to minimum energy-loss structures. Gordon (1981) filmed lagoon breakouts at Dee Why, illustrating well the hourglass shape. His field measurements showed that the breach width reached up to 67 m for a 150 m^3/s final breach flow.

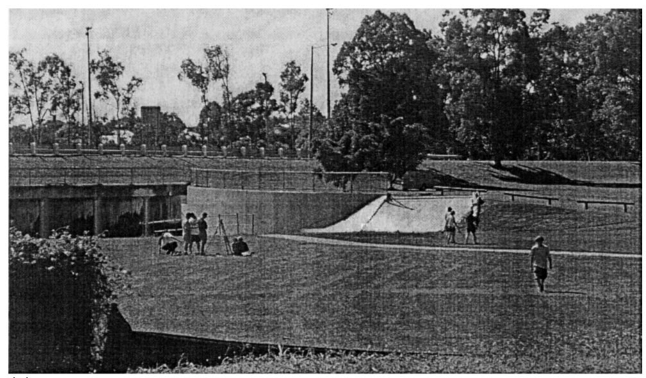
Minimum Energy Loss Culverts

A minimum energy loss culvert is a structure designed with the concept of minimum head loss along the waterway. The flow in the approach channel is contracted through a streamlined inlet into the barrel where the channel width is minimum and then is expanded into a streamlined outlet before being finally released into the downstream natural channel. Both inlet and outlet must be streamlined to avoid significant form losses, and the flow is critical from the inlet lip to the outlet lip. The barrel invert is often lowered to increase discharge capacity (Fig. 1). Professor C. J. Apelt presented an authoritative review (Apelt 1983) and a well-documented audio-visual documentary (Apelt 1994). The discusser has highlighted a wide range of design options (Chanson 2000) and illustrated prototypes (Chanson 1999, 2001) (Table 1).

The concept of a minimum energy loss culvert was developed by Norman Cottman, shire engineer in Victoria (Australia) and the late Professor Gordon McKay of the University of Queensland (McKay 1971, 1978). While a number of small-sized structures were built in Victoria, major structures were designed, tested, and built in south-east Queensland where the natural slope is often very small ($S_O \sim 0.001$), and little head loss is permissible. The largest MEL waterway is located underneath Nudgee Road near the Brisbane airport, with a design discharge capacity of 800 m^3/s. Built between 1968 and 1970, the waterway design was tested in the laboratory with a 1:48 scale model (McKay 1971). Since completion, the structure successfully passed very large floods. The channel bed is grass-lined, and the structure is still in use (Chanson 1999, pp. 384; 421–423). Several MEL culverts were built in southern Brisbane during the construction of the South-East Freeway in 1971–1975. The design discharge capacity ranged from 200 to 250 m^3/s. All the structures are still in use today [Fig. 1(a)] and typically operate several days per year. McKay (1971) indicated further that MEL culverts were built in the Northern Territory near Alice Springs in 1970.

Minimum Energy Loss Weirs

The concept of a MEL weir was developed by Prof. McKay to pass large floods with minimum upstream flooding. MEL weirs were designed specifically for situations where the river catchment is characterized by torrential rainfalls and by very small


(a)

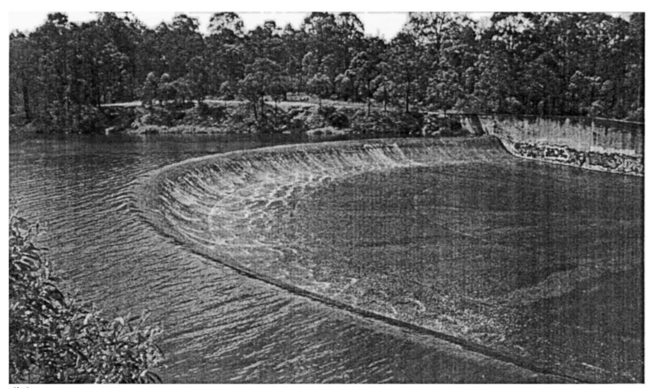

(b)

Fig. 1. Examples of minimum energy loss inlets. (a) Minimum energy loss culvert in Brisbane during some undergraduate student field work on May 13, 2002. Culvert completion, 1975; throat width $=7\times2$ m; barrel height $=3.5$ m; design flow, 220 m^3/s (Chanson 1999, pp. 383 and 425) and (b) minimum energy loss spillway inlet at Lake Kurwongbah (Sideling Creek Dam), Brisbane, Australia on Sept. 12, 1999. Completed in 1969; $H=25$ m; reservoir capacity, 15.5 Mm^3; design flow, 710 m^3/s.

Table 1. Internet Resources on Minimum Energy Loss Structures

Description	URL
The MEL weir design. An overflow earthfill embankment dam	⟨http://www.uq.edu.au/~e2hchans/mel_weir.html⟩
Hydraulics of MEL culverts and bridge waterways	⟨http://www.uq.edu.au/~e2hchans/mel_culv.html⟩

bed slope. The first MEL weir was the Clermont weir built in Queensland, Australia in 1962. The largest, the Chinchilla weir built in Queensland in 1973, is listed as a ''large dam'' by the International Commission on Large Dams. Fig. 1(b) shows the MEL inlet of the Lake Kurwongbah spillway, designed in a fashion somewhat similar to a MEL culvert inlet (McKay 1971). The crest inlet fan converges into a 30.48-m-wide channel ending with a small flip bucket. The MEL crest design allowed an extra 0.4572 m of water storage.

A MEL weir is typically curved in plan with converging chute sidewalls, and the overflow spillway chute is relatively flat [Fig. 1(b)]. The downstream energy dissipator is concentrated near the channel centerline away from the banks. The inflow Froude number remains low, and the rate of energy dissipation is small compared to a traditional weir. For example, the Chinchilla weir was designed to give no afflux at its designed flow (850 m^3/s). In 1974, it passed 1,130 m^3/s with a measured afflux of less than 100 mm (Turnbull and McKay 1974).

MEL weirs are typically earthfill structures with the spillway section protected by concrete slabs; construction costs are minimum. A major inconvenience is an overtopping risk during construction, e.g., the Clermont weir in 1963 and the Chinchilla weir in 1972. In addition, an efficient drainage system must be installed underneath the chute slabs.

Minimum Energy Loss Inlet Design of Embankment Breach

In a simple MEL inlet design, the flow is assumed critical from the upstream lip to the throat. At critical flow conditions, there is a unique relationship between the channel breadth B and the bed elevation for a given flow rate and total head (e.g., Chanson 1999, pp. 368; 386–390). For an embankment breach, the breach has a MEL profile if two conditions are simultaneously satisfied. First, under critical flow conditions

$$\mathbf{F}=\frac{Q}{\sqrt{g*\frac{A^3}{B}}}=1 \tag{1}$$

is satisfied at each cross section A selected perpendicular to the streamlines. Second, under the Bernoulli principle, the total head is constant at each cross section

$$H=Z_{w1}+\frac{1}{2}\frac{Q^2}{gA^2}=\text{constant} \tag{2}$$

where Z_{w1} is the free-surface elevation. In a MEL design, the contour lines (i.e., lines of constant free-surface elevation) are equipotential lines, and they must be perpendicular to the flow direction (i.e., streamlines) everywhere. Basically, the inlet design is based on a flow net analysis using irrotational flow theory (e.g., Vallentine 1969). While the design theory is well understood for a rectangular channel, the design of a natural channel is complicated by the irregular cross-sectional shape, but the inlet must be streamlined using a potential flow theory.

The discusser reanalyzed the authors' data of embankment breach for fine gravel ($d_{50}=1.6$ mm) and two breach widths (Andrews 1998, pp. 146–164). Based on photographs, three-dimensional free-surface levels, and breach elevations, the complete flow net of the breach inlet was drawn. An example is presented in Fig. 2, showing some equipotentials and two streamlines. (Note that the breach contour lines are shown only below

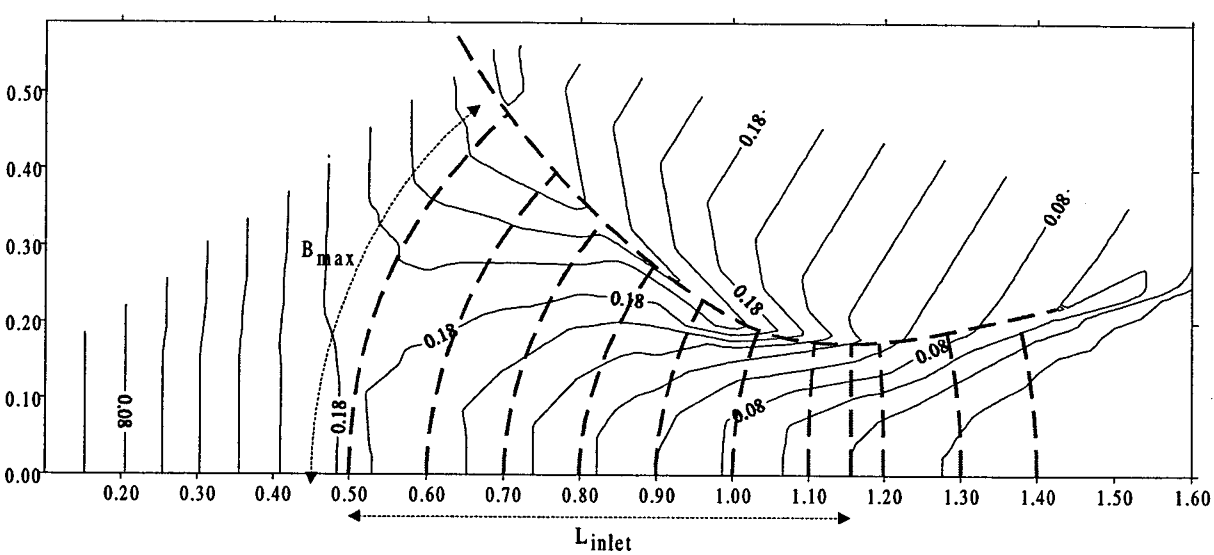


Fig. 2. Flow net analysis of noncohesive embankment breach inlet shape for $Q_{breach}=0.024\ m^3/s$; $t=87$ s; 0.30-m-high embankment, 1.6 mm sand (300 mm breach). Contour lines of the breach are shown only beneath the waterline.

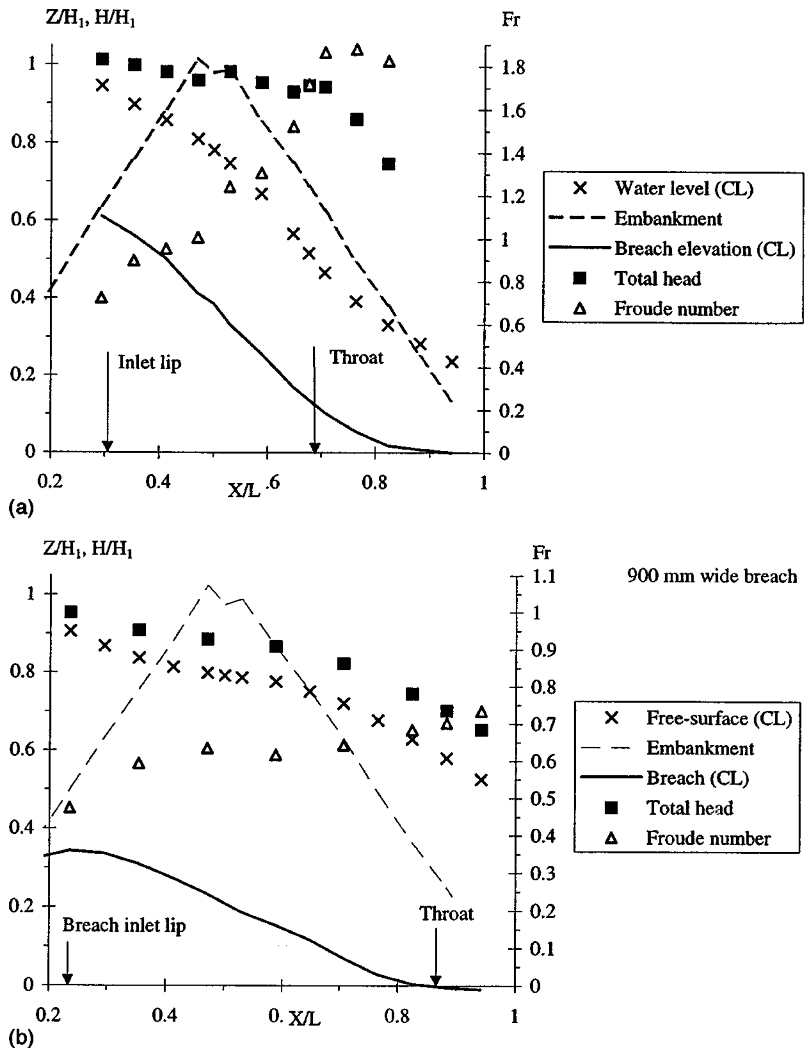


Fig. 3. Analysis of noncohesive embankment breach inlet shape. Cross-section averaged Froude number and dimensionless total head H/H_1 as functions of the dimensionless cross-section coordinate X/L on the centerline $(Y=0)$. (a) 300-mm-wide breach; $Q_{\text{breach}}=0.024\,\text{m}^3/\text{s}$; $t=87\,\text{s}$; 0.30-m-high embankment; 1.6 mm sand and (b) 900-mm-wide breach; $Q_{\text{breach}}=0.071\,\text{m}^3/\text{s}$; $t=147\,\text{s}$; 0.30-m-high embankment; 1.6 mm sand.

the water line.) For the 300-mm-wide breach (Fig. 2), photographs and measurements suggest that the upper lip of the breach intersects the centerline at about $X\sim 0.5\,\text{m}$ from the upstreamembankment toe while the throat is located at $X=1.15\,\text{m}$, where the breach throat is defined at the narrowest flow cross section and the breach inlet lip is defined as the first well-defined equipotential. Flow cross-sectional areas A and free-surface widths B (Fig. 2) were measured along each equipotential, and the cross-section averaged Froude number and total head were calculated based on Eqs. (1) and (2) (Table 2). Results are shown in Fig. 3 where the Froude number and dimensionless total head H/H_1 are plotted as functions of the dimensionless centerline location of the cross section, where H_1 is the upstream total head and L is the embankment base length. The results are compared with the measured breach invert elevation on the centerline, embankment profile, and free-surface profile.

Table 2. Embankment Breach Inlet Characteristics

Location	Q (m^3/s)	t (s)	X (CL) (m)	Z_{breach} (CL) (m)	Z_{w1} (CL) (m)	1/2 A ([a]) (m^2)	1/2 B ([a]) (m)	H (m)	F (m)
300-m-wide breach	0.024	87							
Breach upper lip (inlet lip)			0.5	0.183	0.284	0.0384	0.506	0.300	0.72
Breach throat			1.15	0.042	0.155	0.0151	0.1725	0.284	1.7
900-wide breach	0.071	147							
Breach upper lip (inlet lip)			0.4	0.103	0.272	0.134	1.06	0.286	0.47
Breach throat			1.45	0.0	0.181	0.0849	0.570	0.216	0.69

[a]Half-breach dimensions; embankment height=0.30 m; embankment base length=1.7 m; and material=1.6 mm sand.

The results (Fig. 3) show that the flow is near-critical in the breach (i.e., $0.5<\mathsf{F}<1.8$). Basically, the total head remains constant throughout the breach inlet up to the throat. Head losses occur downstream of the throat when the flow expands and separation takes place at the lateral boundaries. Separation is associated with form drag and head losses, and the assumption of 1D flow becomes invalid. Such a result is well known in MEL culvert design where the design of the outlet is critical to prevent flow separation and large head losses (Apelt 1983; Chanson 1999).

The breach inlet length, measured along the breach centerline between the inlet lip and throat, satisfies $L_{\text{inlet}}/B_{\max}=0.5$ to 0.6, where $B_{\max}$ is the free-surface width at the upper lip. The result is close to the minimum inlet length recommended for MEL culvert design, "the minimum satisfactory value of length/$B_{\max}$ is 0.5" (Apelt 1983, p. 91). For a shorter inlet length, separation may be observed in the inlet.

In summary, the analysis of breach profiles demonstrates that the breach inlet flow operates in a similar manner as in a MEL structure. That is, the total head is basically constant from the inlet lip to the throat, the flow is streamlined (photographs in Andrews 1998), and the flow conditions are near-critical ($0.5<\mathsf{F}<1.8$). This suggests that during a noncohesive embankment breach the movable boundary flow tends to an equilibrium that is associated with minimum energy conditions and maximum discharge per unit width for the available specific energy.

Acknowledgments

The discusser thanks Stephen Coleman for providing his data and exchanging ideas. He acknowledges helpful discussion with Colin J. Apelt (University of Queensland).

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The writers thank the discusser for the interest expressed in their work. It is always interesting to compare different phenomena, such as the present naturally constructed breach channels and culverts and weirs designed to minimize energy losses, and to observe similarities and differences that give insight as to how nature behaves across phenomena and scales.

The discusser's expansion on the background of minimum energy loss (MEL) culverts and weirs is appreciated, as is his highlighting of additional field studies of dyke breaching and lagoon breakouts that complement the writers' work and findings.

Using the writers' data, the discusser presents calculations illustrating that in terms of minimal head losses and flow nature along the channel (as well as channel form and discharge capacity as highlighted by the writers), breach inlet flow is similar to inlet flow for a MEL structure. This reinforces the concept, highlighted in the writers' work, that a noncohesive breach boundary will deform to approach conditions of minimum energy loss and maximum discharge per unit width for the available specific energy.

In regard to the discusser's work, the writers would like to clarify that the contours shown in the discusser's Fig. 2 were determined by a contour-fitting package (assuming ideal fluid flow), with any contours shown outside the submerged breach channel (e.g., on the dry downstream face of the embankment) being questionable. As highlighted in the writers' paper, any application of discharge-capacity expressions for MEL channels to breach channels requires adoption of the MEL definitions of breach-crest length and head on the breach crest for the associated discharge coefficient to be appropriate.

The writers comment that the expressions presented in the paper can be used to simulate breach evolution for an embankment. They would note, however, that in regard to numerical modeling of breach development by erosion, conventional

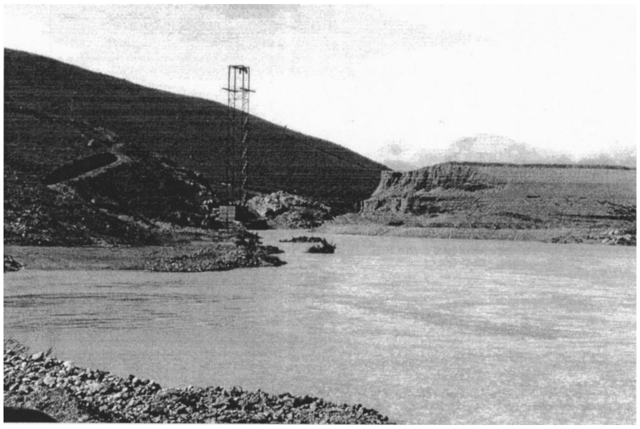

Fig. 1. View of breach in Opuha Dam from upstream

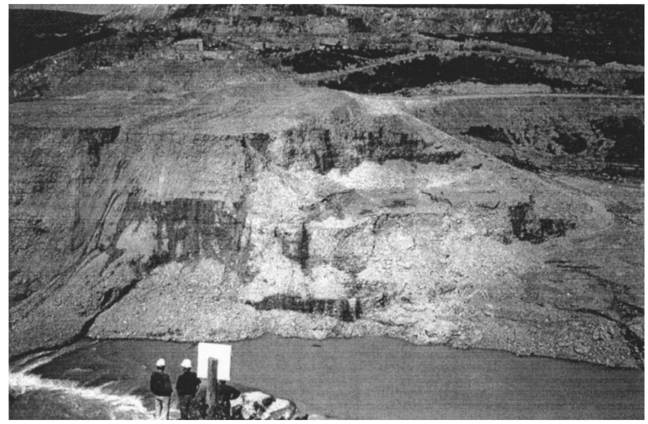

Fig. 2. View from left abutment looking across breach in Opuha Dam toward remaining part of embankment

sediment-transport formulas are limited, in that they are foundedon the mechanics of steady uniform flow for the erosion of rivers and channels of mild slopes. An alternative formula for sediment-transport rates for rapidly accelerating, three-dimensional flows along steep slopes of sediments is then required for dam-breach modeling. A preliminary expression, based on the present measured breaching of noncohesive embankments for constant reservoir levels, is presented in Andrews (1998).

The discusser's observations on the similarity of the breach channel shapes observed from the laboratory experiments of model dams with the shapes of MEL structures prompt a very pertinent question: How does the shape of experimentally observed breach channels compare with the shape of prototype breach channels? In the conclusion of this closure to the discussion of the original paper, the writers examine this issue with respect to a dam failure that occurred recently in New Zealand.

The partially constructed Opuha Dam on the Opuha River, South Canterbury, New Zealand was overtopped in the early hours of the morning on February 6, 1997. The embankment forming the dam was a zoned earthfill structure. The dam materials comprised outwash (silty) gravels, river gravels, and rockfill excavated from the dam foundations (Pickens and Grimston 2001). These materials were all essentially noncohesive.

Prior to failure, the earthfill embankment forming the Opuha Dam was about 30 m high, some 20 m short of its final design height. Heavy rain in the upstream catchment over the preceding 3 days had swelled flows in the Opuha River and its tributaries and, with limited diversion capacity past the dam, caused a pond behind the partially constructed dam to start filling. As the level of this pond increased, the contractor raised the upstream crest of the embankment 2–3 m to increase storage and to try and contain the flood event. This proved unsuccessful, and, when overtopping appeared imminent on the evening of February 5, 1997, the contractor cut an emergency channel through the compacted fill material adjacent to the left abutment of the dam. This emergency channel progressively increased in size until a full-scale dam breach formed. The breach eroded down close to the dam foundation level adjacent to the left abutment. Postfailure analysis indicated the flood that caused overtopping of the partially constructed dam was in excess of a one in 10 annual exceedance probability event (Pickens and Grimston 2001).

Fig. 1 here shows the embankment breach viewed from upstream with the intake tower for the dam outlet structure in the foreground. The left side of the breach was formed by natural country on the left abutment of the dam while the right side was formed by exposed fill material. Fig. 2 here shows a view across the breach from the left abutment of the dam. This illustrates the zoned structure of the embankment forming the dam and the shape of the upstream end of the breach channel. Fig. 3 here shows a view of the breach looking downstream from a similar location to Fig. 2. The dam outlet pipe (which also acted as the diversion facility during construction) leading to the buried circular powerhouse is visible in the base of the breach channel.

Fig. 4 here shows a postfailure contour plan of the breached dam with contours at 2.5 m spacing. The shape of the breach channel evident in this plan can be compared with the breach channel shapes observed from the laboratory experiments of model dams formed from noncohesive materials, as evidenced in the present paper (Coleman et al. 1997; Andrews et al. 1999). The comparison is strictly only valid for the right side of the breach of the Opuha Dam embankment, as the left side was formed by natural country on the left abutment of the dam.

Fig. 4 indicates some rounding of the contours at the upstream end of the right side of the prototype breach channel. However, this rounding is much less accentuated than that observed in breach channels from model-dam experiments and those featured

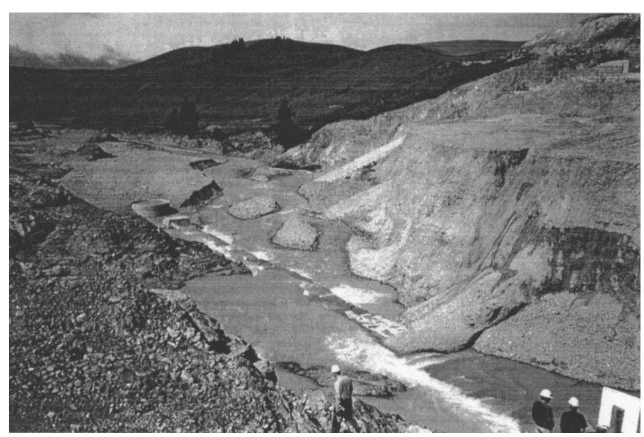

Fig. 3. View from left abutment of downstream end of breach in Opuha Dam

Fig. 4. Topographic contour plan of breached Opuha Dam immediately following dam failure

in MEL structures. The breach channels in the model-dam experiments were formed for a constant-level reservoir. The writers consider that the less rounded nature of the upstream end of the prototype breach channel was primarily attributable to the falling level of the upstream reservoir during the course of breach development. The prototype breach channel appeared to be oriented at a slight angle to the main along-valley axis of the dam, and this would have contributed to the rounding of the channel side on the right at the downstream end of the breach channel. The material forming the right side of the prototype breach channel at the downstream end stood quite steeply.

In summary, the writers' perception from the prototype evidence is that falling reservoir levels during the course of embankment-dam breach development induce a less rounded channel shape at the upstream end compared to the breach channel shape observed from the model-dam experiments described in the original paper. Final breach cross sections of triangular and trapezoidal shapes (as seen in closure Figs. 1–4) can arise for falling reservoir levels during the breach development process, with the breach channel maintaining the curved cross sections described by the writers below the waterline as the breach develops (Andrews 1998).

Acknowledgments

The permission of Opuha Dam Limited (dam owner) and Eliot Sinclair and Partners Limited (survey company) to publish the topographic contour plan shown in Fig. 4 is gratefully acknowledged.

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