

## CHAPTER 128

### CONCEPT OF MINIMUM SPECIFIC ENERGY AND ITS RELATION TO NATURAL FORMS

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It is proposed to show in this paper that there is a solution to the problem of non-uniform flow and this solution not only explains in detail many land forms which occur naturally, but thereby, yields a definition of 'form' loss.

If a channel, in which the transverse distribution of specific energy is uniform, converges and/or diverges, and the bed changes so that the flow will be critical at all cross-sections at the same time, the channel appears to be close to being hydraulically smooth.

Many natural forms, particularly estuaries, are readily explicable in this way. The most obvious one is the bar at the mouth of a river. It follows, if the river enters the sea with reasonable uniform grade, which most rivers do, the bed must rise as the flow loses the restricting influence of the banks (i.e. the width increases) if constant specific energy is to be maintained.

It is possible to calculate with considerable accuracy the dimensions of useful structures based on this concept. A large number of full size but nevertheless experimental structures have been built making use of the resultant benefits which develop:- low turbulence, accurate differential water levels and a clearly defined flow pattern, allowing very considerable savings to be made by eliminating expensive protective works.

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INTRODUCTION

For more than two centuries, the calculation of open channel flow, steady or unsteady, uniform or non-uniform, has been based on the concept of a long uniform channel in which the boundary shear in some way determines the total head loss. Euler-Bernoulli principle (1700-1782), Chezy's (1718-1798), Poiseuille's (1799-1869), Darcy-Weisbach's (1803-1871), Bazin's (1829-1917) and Manning's (1816-1897) formulas, all of which are in regular use today, are based on the concept of such a long uniform channel. Adaptation of the relatively modern Karman-Prandtl's (1930-32) theory of boundary layer and velocity distribution to open channels is also based on the concept of the long uniform channel.

This assumption was reasonable in the past for artificial channels since most of the channel works (sewers and storm drains) were constructed so that the cross-section of the channel was often semi-circular and therefore the distribution of the shear stress around the boundary could be considered more or less uniform.

There is strong evidence in the literature<sup>1\*</sup> that the studies of energy losses in open channels have been closely related to the phenomena of the boundary layer. For fully developed flow in a uniform channel of any size, the boundary layer will occupy all of the channel, but in partly developed turbulent flow, the energy loss due to frictional resistance is related to the stage of the development of the boundary layer. Again, past research on boundary layer has been concentrated on flow through circular pipes and past flat plates parallel to the stream.

The practising engineer may rightly question the above philosophies and assumptions, but what alternative has been possible? From Report of Task Force on Friction Factors in Open Channels<sup>1</sup> - "*At least, it could be hoped there would be made available something similar to the resistance diagrams now used for steady flow in uniform pipes and for frictional resistance of ships. It should be stated at the beginning that these hopes cannot be realised at this time. Principle obstacles are the wide range of surface roughness sizes and types encountered in practical channels (from smooth concrete linings to boulder-stream canyons), the effect of bed movement in unlined channels, and the numerous bends and structures that prevent the attainment of steady, uniform, fully developed flow*".

In order to allow a solution to the apparently simple problem of determining the flow capacity of a bridge opening or culvert, the U.S. Bureau of Public Roads<sup>2</sup> publishes a manual of 90 pages of tabulated data. If the bridge type and flow pattern can be reasonably compared with one of those in the manual, a solution can be assessed. Such techniques do little to validate the basic principle.

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\* Numbers refer to the references in the Appendix.

In deriving and applying the basic formulae and principles of uniform flow to open channels, a quantity 'hydraulic radius',  $R$ , representing the channel geometry, is used. Diameter of the pipe,  $D$ , is replaced by  $4R$ . This substitution carries with it the assumption that the distribution of shear stress around the boundary of the channel is uniform. It is well known now that in open channels, especially with non-circular cross-sections, such uniformity of shear stress is a false assumption. This fact clearly points out the failure of hydraulic radius to be the sole geometric quantity representative of the channel cross-section. The effect of cross-sectional shape must be taken into account in the analysis and prediction of the energy losses in open channels. Such studies have been made by some investigators in recent years, among whom are F. Engelund (1964)<sup>3</sup>, H. Rouse (1965)<sup>4</sup>, E.O. Macagno (1965)<sup>5</sup>, C.C. Shih and N.S. Grigg (1967)<sup>6</sup>, E. Marchi (1967)<sup>7</sup>, N. Narayana Pillai (1970)<sup>8</sup> and C.L. Yen and D.E. Overton (1973)<sup>9</sup>. The results of these investigations are yet inconclusive. According to H. Rouse<sup>4</sup>, *"the effect of change in shape upon the resistance function is actually twofold. On the one hand, it produces a change in the wetted perimeter,  $P$ , per unit cross-sectional area,  $A$ , the reciprocal of which is designated by the hydraulic radius,  $R$ . On the other hand, it produces a change in the distribution of velocity and shear; as a result, the shear will generally vary from point to point of the perimeter. Both effects are thus involved in the equilibrium relationship between the gravitational motive force and the surface resistance which the flow entails"*. He supports the validity of the hydraulic radius concept, but concludes that the effect of cross-sectional shape is related to the variation in the hydraulic radius and two coefficients of a semi-logarithmic resistance function. He also refers to the importance of the aspect ratio in the analysis in relation to the effect of cross-sectional shape.

The junior author in another work\* has proposed other parameters as being more representative of the cross-sectional shape on flow resistance in smooth channels. This method offers a far more rational solution than methods previously proposed by others.

If the flow takes place in a natural stream with erodible bed and banks, the calculations which are at all possible do little to determine the proportions of any change which is likely to occur and in fact hardly assist in allowing predictions of even the general form of the change. Because of the ever varying shape and size of the channel cross-section and unavoidable irregularities in channel alignment, the flow is rarely uniform in natural streams.

It should be appreciated that open channel formulae based on observations of pipe experiments or from small scale physical models of channels and canals or rivers, cannot and do not represent natural situations because :

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\* Under preparation for publication.

- (i) The dimensions and proportions of natural rivers are completely different from those of the pipes for which energy losses have been determined experimentally. In practice the pipes are almost invariably round, ranging in diameter from 1cm to about 10m. The cross-sectional shape of a natural watercourse is quite indeterminate; the width can vary from a few centimetres to many kilometres, while the depth varies from a few millimetres to a very limited amount compared with the width.
- (ii) It is quite impossible to represent all the detailed features and irregularities of a river on the model. We cannot truly conform to the laws of similarity.

Any researcher using large area landscape type models soon appreciates that such models are far too smooth although boundary shear relationships say they should be far too rough.

In small scale models, losses due to boundary shear will always be relatively large because of the effect of scale on both the size and velocity in Reynolds number. However, to assume these losses represent the total energy loss must lead to serious error.

All energy losses will be represented as turbulence. Eddy size is largely a function of the size of the solid boundary generating the eddy. The velocity will determine the number of eddies. Eddies from boundary shear are small and because of their size, dissipate quickly. In contrast eddies generated by channel irregularities and changes in cross-section will be large and will persist downstream adding considerably to the apparent roughness. The reaction of movable boundaries in natural streams to locally generated turbulence cannot be joined with the overall average conditions.

Often the energy losses due to boundary roughness, cross-sectional shape and the boundary irregularities have been confused with one another. For example, H.A. Einstein and N.L. Barbarossa (1951)<sup>10</sup> separated the total energy loss of the natural flow into frictional losses and form losses. Frictional losses were defined as those due to grain roughness and form losses those due to size, shape and spacing of the individual irregularities and presence of sand ripples and dunes. L. Bajorunas<sup>11</sup>, in the discussion of the same paper states that: the roughness factor that reflects the channel irregularities decreases and approaches zero with increasing flow. On the other hand in V.A. Vanoni and Li San Hwang's (1967)<sup>12</sup> results, the bed roughness due to form of the ripples and dunes is the major part of the total roughness and does not approach zero with increasing flow. In the discussion of Einstein and Barbarossa's paper, Sir. C. Inglis<sup>13</sup> categorises the total losses into (i) those due to textural roughness; (ii) those due to ripple roughness; and (iii) those caused by form drag resulting from the major irregularities of the banks, bed, islands and sandbanks. Inglis does not believe in combining groups (ii) and (iii) because of a different time scale by which they change.

The question is, what constitutes the total energy loss in natural rivers and how much of the total loss is due to boundary shear and how much is due to form?

CONDITION FOR NO FORM LOSS

It is proposed to show in this paper that there is a solution to the problem of non-uniform flow and this solution not only explains precisely many land forms which occur naturally, but thereby yields a definition of 'form' loss.

The only channel which gives truly no 'form' loss is of rectangular section. In any other channel the specific energy must vary across the channel. This would generate turbulence not associated with boundary shear. The elements of flow across the section can be shown to respond in different ways to an overall change in section shape.

B.A. Bakhmeteff (1932)<sup>14</sup> showed that if the flow in a channel is tranquil and the channel converges slowly, the surface will fall, the velocity will increase and, if the channel then diverges slowly to its original width, the flow will return virtually without extra loss, to its original uniform flow pattern; i.e. the 'form' loss is zero. If, however, the convergence continues, the flow will ultimately become critical (i.e., the depth  $y_c$  will be  $2/3(y_o + v_o^2/2g)$  when  $y_o$  and  $v_o$  are the depth and velocity in the initial channel,  $g$  is the acceleration due to gravity and  $y_c$  is the critical depth). Any further convergence will lead to a rise in level upstream of the constriction and a corresponding head loss through it.

Similarly, it was shown that if in a uniform rectangular channel a smooth hump is introduced, the water surface level falls, the velocity increases and the depth decreases. The height of the hump for no 'form' loss is limited to that which creates critical conditions at the hump; i.e.

$$y_c = 2/3 \left[ (y_o + v_o^2/2g) - \Delta z \right]$$

$\Delta z$  being the height of the hump. If the height of the hump is increased beyond this limit, the upstream level will rise and a 'form' loss occurs at the restriction.

There is, however, in addition to these particular cases, a condition which offers a wide range of cross-section proportions which will offer no 'form' loss.

If  $v_c^2/2g + y_c = H_o + \Delta z$  everywhere, and the transition is as before reasonably slow, there will be no 'form' loss.

$H_0$  is the initial specific energy =  $v_0^2 / 2g + y_0$

$\Delta z$  is now the change in level of the bed

$v_c$  is the critical velocity

both  $H_0$  and  $\Delta z$  have the same sign and for convenience are measured positively down from the total energy line.

In many natural channels the 'form' loss is the major portion of the total head loss. It is then possible, by building a non-uniformity to this concept to create useful structures which cause no afflux. Any increase in boundary shear loss due to increased velocities can be offset by a reduction in 'form' loss.

The proposition was not determined initially as a result of a theoretical or laboratory research. It arose from the successful solution of a number of ad hoc problems.

#### DESIGNED SMOOTH TRANSITION STRUCTURES

The City of Redcliffe is a satellite town of Brisbane, the capital of Queensland. It has extensive if quite beaches on Moreton Bay and is the nearest seaside holiday resort to Brisbane. The road pattern is just about  $45^\circ$  to the natural drainage lines. One of these drainage lines - Humpy Bong Creek - literally splits the centre of the town in two. In 1958 the sole crossing of this creek was a narrow wooden bridge joining the shopping centre to the south, with the municipal buildings to the north. The shopping centre was along the beach promenade. There were just about enough parking places for the shop assistants' cars. Shopping on any Saturday in the holiday season was quite an adventure.



FIG.1. GENERAL VIEW OF REDCLIFFE AND PROPOSED IMPROVEMENT ON THE CREEK

The flow in the creek was largely sullage water. The banks were steep, ragged and overgrown. Altogether it was a smelly, unpleasant area. It was therefore proposed that a multipurpose improvement be made. (Fig. 1).

- (i) Build a weir to raise the fresh water level in the creek to cover the ragged steep banks and allow the area to be dressed, grassed and easily maintained as a park. At the same time the weir would exclude the tide and eliminate the smell of rotting vegetation.
- (ii) Culvert the creek to remove access problems and create an extensive car park on and adjacent to the culvert. The car park would serve the shops and public offices by day and in addition the proposed Civic Centre and R.S.L. Hall by night.
- (iii) To alleviate flooding of the shops and adjacent area.

An admirable proposal, but how to do it? - particularly at a price the town could afford. The catchment of the creek was rapidly urbanizing and the estimated maximum flow was 910 cusecs ( $25.8 \text{ m}^3/\text{sec}$ ). The general level of the land adjoining, and hence flood level, at the creek mouth was R.L. 8.0 ft (2.440m) L.W.O.S.T. This would preferably also be the level of the car park. High tide was R.L. 4.5 ft (1.370m) L.W.O.S.T. and experience had shown that any outlet with an invert below high tide became stuffed with sand in the dry winter season. It was also necessary that the standing water level in the lake should not be less than R.L. 5.25 ft (1.600m) L.W.O.S.T. to cover all the trash growth on the vertical banks and allow easy maintenance. At that time the whole area flooded at least once every two years and by traditional hydraulics the problem appeared to be unsolvable as the length of the culvert was some 600 ft (180m).

The Department of Civil Engineering, University of Queensland, suggested the solution illustrated in Fig. 2. The logic of this solution was :- The discharge is 910 cusecs ( $25.8 \text{ m}^3/\text{sec}$ ) flowing upstream of the weir at a level of R.L. 8.0 ft (2.440m). If we neglect the velocity head we can take R.L. 8.0 ft (2.440m) as the level of the total energy line at the weir. The weir level was set at R.L. 5.25 ft (1.600m) to give satisfactory pondage conditions. The maximum available specific energy at the weir is 2.75 ft (838mm). Thus the critical depth is  $2/3 \times 2.75 = 1.83 \text{ ft}$  (558mm). The maximum flow per foot width of weir is thus 14.2 cusecs ( $1.327 \text{ m}^3/\text{sec}/\text{m}$  width), so the minimum width of the weir is 64 ft (19.5 m). The culvert barrel was 18.0 ft (5.480m) wide, a Queensland Main Roads' Department standard width. The flow per ft width must be 50.51 cusecs ( $4.7 \text{ m}^3/\text{sec}/\text{m}$ ) so the minimum (critical) depth is 4.30 ft (1.304m). The corresponding velocity head is 2.15 ft (652mm), so total specific energy is 6.45 ft (1.956m). Therefore the level of the culvert invert at entrance is R.L. 8.0 ft, less boundary shear fall, less 6.45 = approximately R.L. 1.43 ft (440mm).

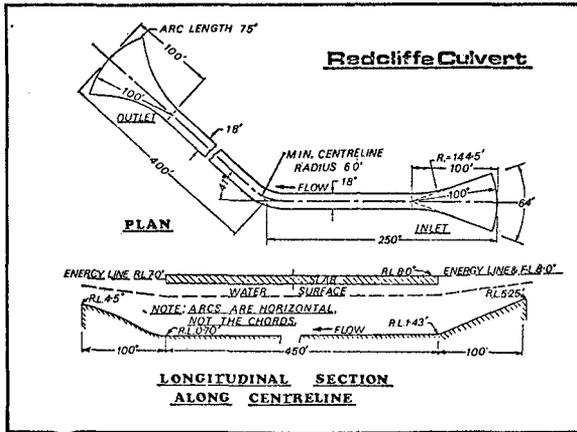


FIG.2. DETAILS OF REDCLIFFE CULVERT

An arbitrary assumption was made that there would be 1.0 ft (305mm) of head loss due to boundary shear so the level of the energy line at the outlet is R.L. 7.0 ft (2.130m). The high tide level is R.L. 4.5 ft (1.370m) L.W.O.S.T. so the available specific energy is  $7.0 - 4.5 = 2.5$  ft (762mm). The minimum depth is  $\frac{2}{3} \times 2.5 = 1.67$  ft (509mm) and the maximum flow per ft width at outlet is 12.25 cusecs ( $1.139 \text{ m}^3/\text{sec/m}$ ) so minimum outlet width is 74.3 ft (22.620m).

In plan the inlet weir was joined to the culvert with an arbitrary shape and the culvert to the outlet weir likewise. The inlet and outlet floors were designed so that everywhere we had critical flow. The culvert slab fitted very conveniently between R.L. 8.0 ft (2.440m) and the water surface to give ample freeboard. Despite much critical comment the Department was commissioned to build a model. A very big model  $1" = 1 \text{ ft}$  (1:12) was built. Very quickly we learned that the inlet and outlet had to be part of flow nets. With this single modification the model performed perfectly. There was an amazing correlation between the calculated and model water levels, everywhere within 0.01 ft (3mm on the model). There was apparently no difficulty in imposing this flow system.

The culvert was built. No flow measurements have been taken but no flooding whatsoever has occurred since, although there have been at least three occasions when the design flow has probably been exceeded. This includes 1974 when Brisbane suffered devastating flooding, both from the local creeks and the Brisbane River.

Some years later we were asked to investigate the augmentation of the water supply for a small town, Clermont, in Central Queensland. With this request was attached a very odd condition - that any weir in the river must not cause flooding at a lesser flow than at present nor at more frequent intervals. There was good reason for this condition. Clermont lies in the junction of two streams, the Belyando River and Rocky Creek. Typical of western Queensland the highest land for some miles around is the river bank. Once the flood breaks out it spreads through the town and over the adjacent

country. In 1916 a major flood peaked on a Saturday night and nearly one hundred people were drowned. Naturally Clermont is now a little sensitive to flooding.

There is always a minimum storage below which storage has little purpose. This minimum storage determines the minimum weir height and, in this case, the required weir was relatively high compared to the banks of the river. The whole region is alluvium and consequently maintaining the weir in the river was also an equally difficult problem. Having tried a whole array of strange shapes and arrangements, we failed to satisfy these conditions and reported accordingly.

However, reflecting on the Redcliffe outfall, there were second thoughts. If this culvert were cut in two and placed end to end, it gave a system of weir without afflux. The bank full flow could be taken as the design flow,  $Q$ . The slope of the water surface at this stage is known. The velocity,  $V$ , is  $Q/A$  when  $A$  is the cross-sectional area at 'bank full'. The energy line will then be  $V^2/2g$  above and parallel to the bank full-flow surface. The height of the weir crest; i.e. Storage Level, is already determined. The difference between storage level and the energy line level is the specific head. Forthwith the critical depth, the maximum flow per unit width and the minimum length (width) of the crest can be calculated. This crest width was very much wider than the river itself. By choosing an arbitrary plan shape, the height at any other transverse section of the weir can be readily calculated and the longitudinal profile determined; alternatively a profile of the weir can be chosen and the widths calculated.

The Clermont Weir (Fig. 3) was 20 ft (6m) high, 350 ft (106.8 m) wide along the crest. The bank slopes were, of necessity, flat to obtain a smooth transition. The problem now was not the adequacy of the design but how to build it. The only possible solution for such a big volume structure was somehow to build the bulk of it in earth.



FIG.3. PHOTOGRAPH OF CLERMONT WEIR

This was done (not without incident) and the earth protected by a filtered concrete slab. No other protection to bed and banks was found

necessary. Again no measurements have been made but there have been many flows over the weir and at least there have been no complaints in the twelve years of its existence. A much larger but similar weir has since been built. This second weir (Fig. 4) is at Chinchilla, South-West Queensland. It is 40 ft (12.2 m) high and has a crest of 750 ft (228.300m). It stores 7,500 acre feet ( $3 \times 10^6 \text{ m}^3$ ) and the crest is actually level with the adjacent bank contour. A bank six feet high ties the end of the weir to higher ground some distance away. When the flow starts to overtop this bank; i.e. when flooding commences upstream, the flow is some 35,000 cusecs ( $1000 \text{ m}^3/\text{sec}$ ) and the measured afflux was 4.0 inches (100 mm). Below the design flow the excess energy is dissipated in a single roller on the face of the weir. No other protective works are provided. Above the design flow, all trace of the weir disappears.

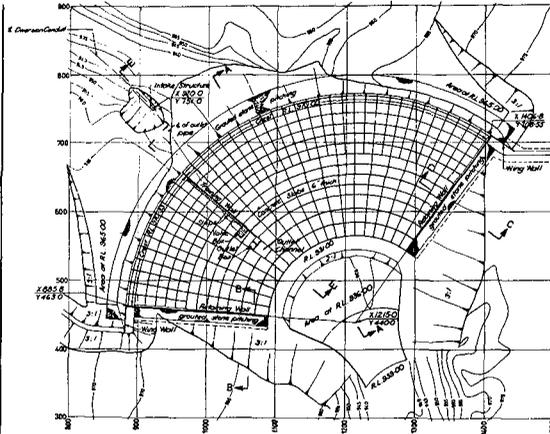


FIG.4. PLAN OF CHINCHILLA WEIR

The strangely smooth turbulent free flow over these weirs posed the question, was the concrete protection necessary? Eventually the opportunity arose to build an earth weir protected only by grass. It was designed to have no effect at a level at which the flow would cause flooding or nuisance if exceeded - not the maximum flow. It has been completely successful. The earth formation was completed on the 5th November, 1967. The bank was sown with Greenleaf Sudan - a fast growing sorghum and sprigged with Kikuyu. The first flow over it occurred exactly five weeks later. In January, 1968, two months after completion, the weir was submerged completely early on a Saturday evening and remained completely submerged until Tuesday mid-day. The photo (Fig. 5) was taken on the Wednesday. The growth was adequate protection and the weir has survived ever since. In order to secure these weirs in the early vulnerable period before the grass is established they have been reinforced with a cheap plastic mesh.

As grass had proved completely adequate as protection in these smooth flow structures, they became far more attractive cost-wise, despite the strange shape, than more traditional designs. A number of smaller but identical weirs have been built and protected by grass only. The cost is \$50-\$100 per million gallons stored ( $500\text{-}1000 \text{ m}^3$  per dollar). This type



FIG.5. PHOTOGRAPH OF YULEBA WEIR

of weir makes possible extensive storage at sites which have previously been discarded. The optimum site is the *widest, shallowest*, and hence highest part of the river bed.

The farm dams were not high, averaging 5 ft (1.5 m); the highest has been nine feet (2.74 m). At small overflows the bank is subjected to velocities close to  $\sqrt{gh}$  when  $h$  is the height of the dam. Even at a height of five feet, this amounts to over 12 ft/sec (4m/sec). There was never suspicion of scour.

The calculations associated with these designs are indeed simple. The accuracy of the calculations is very good and the performance completely predictable. The confidence gained from these successes enabled much more exotic structures to be designed and built to achieve solutions not previously possible, in particular culverts and bridges without afflux but discharging at high velocities so that the span is minimized.

Typical is the Nudgee Road Bridge over the Kedron Brook in Brisbane. At the point of the crossing the natural stream channel had completely degenerated having left its well defined steep watercourse and entered the coastal swamp. Long since the area had been drained by a small canal-like waterway to a well defined tidal inlet some miles away. This channel could not even carry the annual flow. The design flood of 30,000 cusecs (850 m<sup>3</sup>/sec) spread over a width of 1320 ft (404m) increasing downstream. The road crossed the swamp on a low embankment rising to a short timber bridge over the canal, and the floods rose over the embankment and cut the traffic.

A major shopping complex had been built upstream of the crossing but still within the swamp zone. The council had required a considerable portion of their area to be retained as a flood relief channel and the remainder had had to be filled to a considerable depth. The area had continued to develop and Nudgee Road became a major traffic route and at this point the timber bridge failed. The situation required the immediate construction of a new two-lane all-weather crossing (subsequently to be four-lane). There could be no raising of the flood level for fear of damage in the shopping complex; the 1967 flood having risen to within 1 ft (300 mm) of the floor levels. The difference in cost of embankment

to bridge structure was \$1000 per ft length.

The constant energy design gave certainty of calculation and the absolute minimum span. The ground (bed) level was determined by the lowest level at which it was possible to grow grass - the area being tidal. This was taken at R.L. 7.5 ft (2.280m) L.W.O.S.T. The design flood (30,000 cusecs) level was R.L. 15.6 ft (4.750m); the existing ground level was R.L. 10.0 ft (3.047m). The slope is surface fall only and in flood, amounts to less than 1 foot per mile. The approach, being so wide, was considered rectangular.

Approach velocity =  $30,000 / (1320 \times 5.6) = 4.0$  ft/sec (1.22m/sec)

R.L. Energy line is 15.85 ft (4.830m)

For ground level at R.L. 10.0 ft (3m),  $y_c = 3.9$  ft (1.19m) and critical unit flow  $q_c = 43$  cusecs ( $4m^3/sec/m$ ), so flow can be restricted to 688 ft (210m) width only. Under bridge the available specific head  $H_s = 8.1$  ft (2.470m),  $y_c = 5.4$  ft (1.645m) and  $q_c = 71$  cusecs/ft ( $6.6 m^3/sec/m$ ), therefore the minimum width is 423 ft (129m). The bridge was built with 9 x 50 ft (9 x 15.26m) spans with round piers.

The lip of inlet fan was 400 ft (122m) above the bridge. The shape of the fan is shown in Fig. 6. A model scale 1:48 gave results which agreed with the computed results to within 0.2 ft (61mm). These models must be truly three-dimensional as it is no longer possible to study a 'representative' longitudinal section. The model showed that if the approaches were not depressed with the 450 ft (137m) wide opening, the

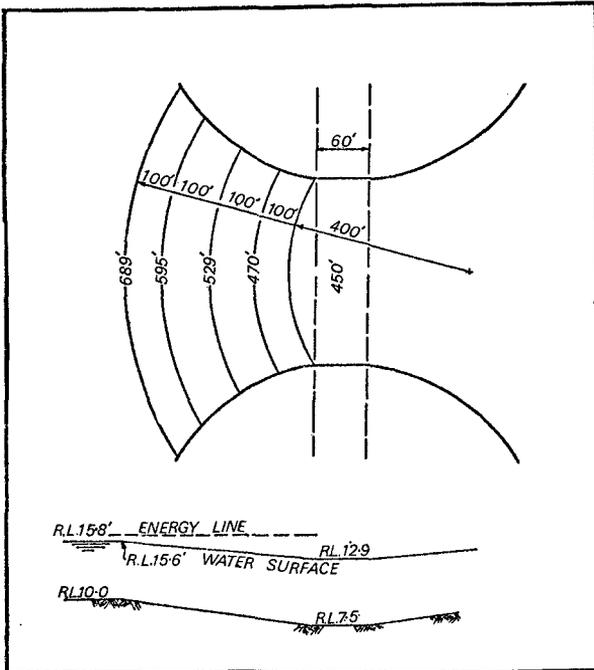


FIG. 6. NUDGEE ROAD BRIDGE INLET

flood level rose above R.L. 18.5 ft (5.650m), the minimum road embankment level, and overtopping occurred. The minimum span to pass the flood at ground level R.L. 10.0ft (3m) was 688 ft (210m). The model again showed the remarkably smooth turbulent free flow and as a result grass only was used as protection on the inlet and outlet fans even though the velocity exceeds 13 ft/sec (3.96 m/sec). The outlet fan was an exact image of the inlet fan.

The cost of the earthworks in the fans was \$10,000, the saving on the bridge about \$240,000.

The design procedure was used in different circumstances in Stawell, Victoria, Australia, by N. Cottmann, Shire Engineer. A bridge on the Stawell-Newington road had a capacity of 800 cusecs (22.62 m<sup>3</sup>/sec) before the road overtopped - which happened in every flood. The bridge approaches were redesigned so that the same bridge could carry 5,000 cusecs. Fig. 7 shows the bridge carrying 4,300 cusecs (122 m<sup>3</sup>/sec) in February, 1975. Although the water level under the bridge was well below the approach level it recovered the level and passed through virtually without afflux. Again, despite 15 ft/sec (4.57 m/sec) velocity and minimal protection, no scour occurred.



FIG.7. STAWELL BRIDGE CARRYING 4300 CUSECS (122 m<sup>3</sup>/sec) IN 1975

The concept lends itself to use in dual or multipurpose structures, in particular the combination of flood alleviation with stream crossings. The South East Freeway out of Brisbane was deliberately routed through the valley of the Norman Creek in order to minimize the number of houses it was necessary to resume. But the valley was free of houses because the area was subject to severe short floods from the adjoining urban areas. Retardation basins in the form of playing fields had been established along much of the length of the creek. The freeway not only crossed the creek on numerous occasions but the embankment occupied a significant portion of the available retardation basin area.

Because of the certainty of the calculation, minimum energy culverts were used throughout. The inlet fan to each culvert was so arranged as to act as a minimum energy weir and to discharge a particular flood at a particular level; i.e. the highest possible. (Fig. 8). The flow through one culvert is not affected by the backwater of the culvert/weir



FIG.8. PHOTOGRAPH OF CULVERT ENTRANCE

downstream. The loss of detention area was amply compensated by the increased depth made available.

A different version of this same theme, flood alleviation and a stream crossing, is that at Settlement Shores, Macquarie, N.S.W. This is a very popular holiday area conveniently situated north of Sydney. The Hasting River flows out of the ranges and meanders in a large loop through the coastal swamps. Even in small floods the area presented was flooded and is unsuitable for development.

A large channel will be built to short circuit the loop but at the lower end a minimum energy weir is to be built high enough to prevent egress of the tide. At the same time the weir allows the free discharge of the flood water virtually without head loss at a lower level and thus frees the area from danger of inundation. Immediately below the weir a very rapid convergence allows economic convenient bridging as access to

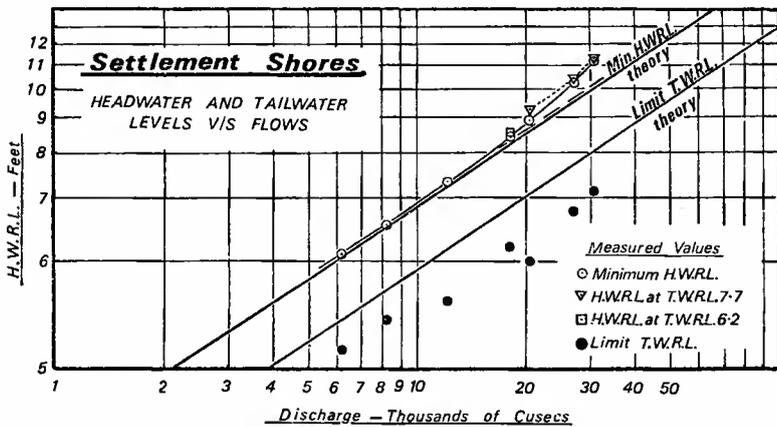


FIG.9. SETTLEMENT SHORES - HEAD-DISCHARGE RELATIONSHIP

the area. Fig. 9 shows the relationship between the calculated and measured head water/tail water on the model. From our experience, the prototype losses will be relatively far less.

#### NATURAL SMOOTH TRANSITIONS

It became increasingly clear that each of these structures had counterparts in nature. The Redcliffe tidal outfall is surely an idealized and miniaturized river bar formation. If a river enters the sea with reasonably uniform energy, then as the river loses the constricting influence of the banks, the bed must rise to maintain constant energy. If the divergence of the banks is uniform the bar is in fact curved. If the banks are curved the bar is straight. In the days of sail, it was always said that 'ships sailed up-hill over the bar', and how true this could be. Equally it is claimed, again with some truth, that it is always choppy over the bar.

It makes no difference whether the tide is ebbing or flooding, the same shape is demanded by the concept of constant energy. It is not surprising that C.D. Floyd (1968)<sup>15</sup> reports on river mouth training in New South Wales, Australia to the Public Works Department. *"A summary is given of the results of training sixteen rivers in an endeavour to increase bar depths. The bars are of simple crescent formation fed by littoral drift.*

*Whilst the training works have improved conditions for navigation, they have not resulted in any appreciable increase in bar depths.*

*Despite the complex mechanisms involved in bar formation a consistent simple relationship is found to exist between channel and bar depths. This correlation seems to apply to all rivers and inlets with simple bar systems and extends over a range from a bar depth of two feet to 60 feet!*

*The majority of the work (the training) was carried out in the period 1880 to 1910 with minor changes and additions in the period 1910 to 1930. Commencing in the 1950's, a major training scheme was started on the Clarence River and also a programme of development of small river entrances for fishing craft."*

The inlet and outlet features of the culverts are surely the corresponding scour so often appended by nature to man-made structures. A perfect fan completely formed naturally is given in Fig. 10. This is below a small portal bridge over a gravel bed stream after a short sharp fresh flow. The complete culvert form is the shape which always develops between the tidal lagoon and the ocean. This is so well illustrated by Per Bruun (1966)<sup>16</sup>. He writes of 'the gorge' and the 'shoals' and illustrates it profusely when he is searching for a relationship for the stability of this strange shape which is deeper in its middle than at either end. (Fig. 11). The entrance to San Francisco Bay, both inside and out, is a perfect major example of this form (Fig. 12).

That this complete culvert shape is a not uncommon geological feature, many arch dam builders have found to their cost. The Gordon River in Tasmania, Australia, is typical. The gorge narrows to provide the perfect abutments but the solid floor is a long way below that at the entrance to the gorge. Of world renown because it is so well advertised, is that mysterious gorge on the River Aare in Switzerland. At places



FIG.10 FAN AT BLYTH CREEK - ROMA

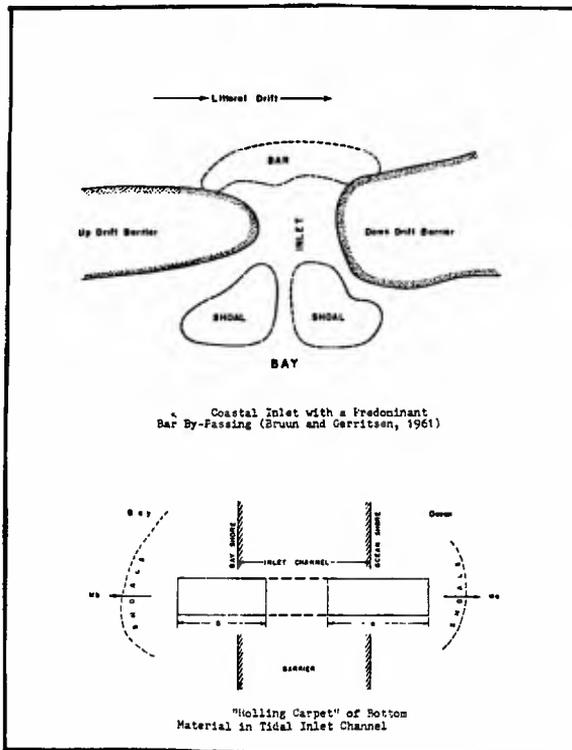
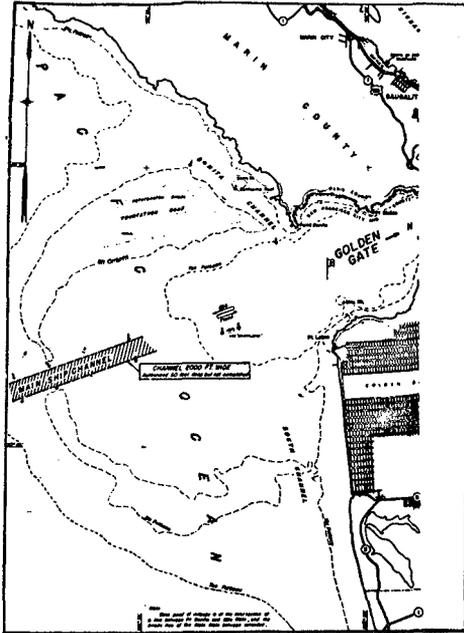


FIG.11. TIDAL LAGOON - (FROM REF. 16)



The Golden Gate and its Ocean Shoals (U.S. Corps of Engineers Annual Report, San Francisco District, 1952).

FIG.12. SAN FRANCISCO BAY - (FROM REF. 16)

this is little more than a meter wide and its depth is a hundred metres below the river bed at each end.

*"The Gorge of the Aare near Meiringen (by Professor Dr. P. Arbenz (Berne)). Half an hour beyond Meiringen the Haslital is blocked in the whole of its breadth by a barrier, on the steep walls of which grey chalk is everywhere to be seen. It is not the result of a landslip, nor is it a moraine, but rather a crossbar of rock, a small mountain range in the valley, abounding in small peaks and valleys and a labyrinth of wooded indentures and defiles. The river Aare pierces through this rocky obstacle in the famous Gorge of the Aare. It has eaten its way through the rock at so great a depth, that the fall on its way through the gorge is but slight. A level path leads to Innertkirchen on the other side of this rocky crossbar."*

How else could a 'blind' channel be formed across half tide sand banks in an estuary?

The earth weirs arise in the central reaches of many rivers where a well formed channel suffers a change of grade, widens and shoals quite

severely to form pools in low flow periods - the silt banks of Australian streams - below which the stream will converge again to a well-defined channel. These are the sites to be chosen for farm dams and far from scouring, many of the grassed weirs are actually growing.

A very good example of accidental minimum energy culvert must surely be the old London Bridge (Fig. 13). As little as 25% of stream area was available to the tide passing through the bridge. The tidal range is high but up to high tide the plan shape is ideal and the Thames mud was no doubt duly scoured to give the correct profile. This bridge survived from 1209 to 1825 A.D. For five centuries it was the only bridge.

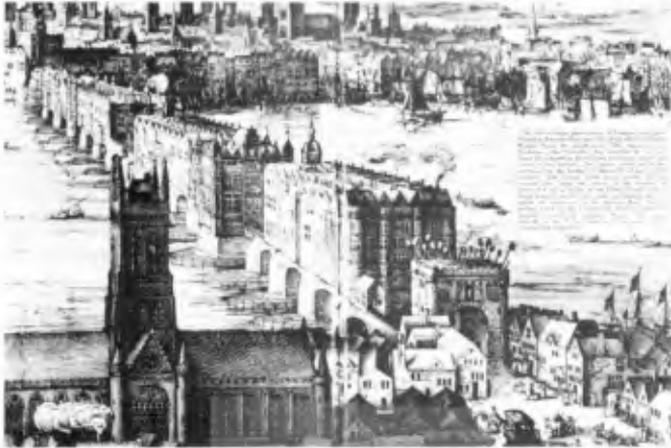


FIG.13. OLD LONDON BRIDGE

Obviously if a form persists there must be some mechanism by which it becomes self sustaining and able to resist any destructive forces. The concept of minimum energy is just such a mechanism. Minimum energy introduces critical velocities; i.e. the maximum velocity which can exist at that energy condition or for that flow per unit width. It is wrong to assume that critical velocities are high, they can be so small that they are unable to move even sand particles. The concept allows the velocity to diminish without reducing the total discharge. Thus the natural fan shape at both inlet and outlet becomes quite stable and self sustaining.

As the velocity of the flow in the connecting deeper channel is always greater than the approaching flow, any bed load is carried through the deeper section. Any floating material will move much easier in the deeper channel so anything which is carried in will be carried out. If the total flow increases for some reason, the inlet fan will erode back to a higher, longer lip. The central section will either become deeper or wider and the eroded material will be carried up the outlet fan to deposit there again to form a higher, longer lip.

The concept does not restrict a river, alluvial or otherwise, to one particular shape so much sought by many researchers - G. Lacey (1930)<sup>17</sup>, T. Blench (1957)<sup>18</sup>, F.M. Henderson (1963)<sup>19</sup>, D.B. Simons and M.L. Albertson (1960)<sup>20</sup>. Any rectangular shape is acceptable. Provided there is everywhere

the right relationship between width and depth the energy grade will be uniform. The river can become narrower if it deepens or can become wider provided it shallows. The actual cross-sectional shape is a function of the material of the bed and banks. If you add the hypothesis of L.B. Leopold and W.B. Langbein (1966)<sup>21</sup>, that meanders are curves of minimum energy in bending (and there are any number of suitable curves for any situation), then a river has almost complete choice of cross-section and position and can still satisfy the overall concept.

#### CONCLUSIONS

- 1 - There is no doubt that the concept of minimum and constant energy unfolded here does allow the quantitative determination of smooth transitions in open channels and these transitions can have useful applications.
- 2 - It was shown that in addition to the previously known applications of specific energy, there is a condition which offers a wide range of cross-section proportions without additional 'form' loss. This condition is the reduction of ground (bed) level to satisfy a further convergence of the flow; i.e. to reduce span or to drop the water surface to increase headroom.
- 3 - A solution was offered to the problem of non-uniform flow and an explanation was given for the occurrence of many land forms in nature, such as formation of a river bar and estuaries.
- 4 - The transition shapes are essentially self sustaining. They are 'smooth' with no 'form' loss. There is no energy change and there is no turbulence, hence no scour. As any other shape would require more energy to convey the flow, there is also no deposition.
- 5 - To impose an unacceptable change on such a system is courting disaster as is evidenced by the many vain attempts to change estuarine conditions which are essential to the estuary's very survival. Equally futile is to attempt to rid rivers of the 'bar'. We can learn much by studying the natural shapes.
- 6 - By simply using constant energy rather than constant discharge, we can accurately predetermine the dimensions of the appropriate shape and this shape will be without 'form' loss. Structures can be built to other forms but they will have to be defended, to retain their form, by solid protection or continual maintenance.
- 7 - Always the shape is the same - a strange fan shape. The shape of a scour hole below a culvert. The shape of the old London Bridge. The shape of a tidal inlet to a lagoon. The shape outside San Francisco Bay; of a channel very much deeper than the lakes it connects or the channel through a sandy estuary. All these shapes are, we are sure, conforming to the concept of  $v_c^2/2g + y_c = H_0 + \Delta z$ .

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