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DESIGN OF MINIMUM ENERGY CULVERTS

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THE DESIGN TECHNIQUES ASSOCIATED WITH THE CONCEPT OF CONSTANT TOTAL ENERGY AND COMPATIBLE SPECIFIC ENERGY WITH PARTICULAR REFERENCE TO DRAINAGE STRUCTURES ON HIGHWAYS, WITH A REPORT OF THE PERFORMANCE OF MANY OF THESE STRUCTURES DURING THE QUEENSLAND WET SEASON 1970-1971

by

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INTRODUCTION

For over a hundred years, the basis of channel design has been some form of Chezy's formula, $v = C\sqrt{R\ S}$, where v is the velocity of flow, R is the hydraulic radius, S is the slope of the energy line and C is a variant depending on the channel boundary conditions and the state of the flow. Considerable effort and time have since been spent attempting to define 'C'. Much of this effort has been directed to introducing a constant 'n' which is dependent only on the channel boundary.

The overriding condition to the application of such formulae is that they apply only to long uniform channels - without defining 'long' or 'uniform' and steady flow. Computations relating to natural channels have been carried out on a similar basis, the length being restricted to some small length over which the energy slope is considered as constant and parallel to the bed and water surface. There is, obviously, immediate conflict as small lengths and non-uniformity do not attempt to satisfy the basic analytical requirement. In order to obtain sensible answers, coefficients, somewhat arbitrarily chosen, are applied.

In 1932, Boris Bakmateff introduced the concept of specific energy - the internal energy associated with open channel flow. He showed that for a particular discharge,flow could occur at two depths except for one particular condition - the critical depth. In mild slopes, a small constriction led to a reduction in depth with a corresponding increase in velocity - such that the total internal energy of the flow remained constant. Bakmateff measured this energy from the original channel floor. This concept explained the "hydraulic jump", but apart from using the jump as a mixing facility or to reduce head at a particular place, very little practical use has been made of the concept.

It has always been presumed that the increase in velocity led to an increased 'friction' loss and consequently restrictions which caused such an increase in velocity must lead to an increase in the rate of energy loss and consequently, to be maintained, must require an increase in upstream water level. This would probably be true if no other changes resulted.

Natural streams are invariably non-uniform. It is extremely difficult to measure or to define 'slope'. The cross section changes, often quickly, from point to point. It is often difficult to distinguish a change in section-form and boundary roughness. It has been shown that 'n', originally introduced as a constant, varies radically with stage and from section to section in the same stream. Chow sets out possible variations of n with stage. Figure 1 is a plot of n against stage for the major coastal rivers of Queensland. The basic figures were obtained from the Irrigation and Water Supply Commission. Presumably, they were measured with reasonable accuracy. As the 'n' is a straight multiplier, any computations using a particular n value must be grossly in error.

Overall, the specific energy at a section is given by

$$\frac{A}{B} + \frac{Q^2}{gA^2}$$

where A is the cross sectional area of flow (and measured as defined in fluid mechanics - perpendicular to the velocity).

B is the top water width

Q is the discharge.

Unless the specific energy is compatible section to section, energy dissipation - turbulence - must occur. Thus energy loss can arise purely from a change of section form - not boundary roughness. At the same time, the specific energy of a vertical element is always

$$y + \frac{v^2}{2g}$$

where y is the depth of the element and v the depth-average velocity, so that transverse flow is likely from points of deeper depth (high energy) to the shallower depths (low energy). This transverse flow - again generating turbulence - will dissipate energy. Thus the cross sectional shape can significantly alter the rate of energy dissipation irrespective of the boundary conditions.

The persistence of turbulence is *inter alia* a function of eddy size. Small eddies quickly disappear, but large eddies persist for a considerable time and hence distance. Thus small eddies generated at the boundary will dissipate quickly and will have a local effect only. Large eddies generated by form-change or cross section shape will persist downstream and will make the downstream reach apparently 'rough'. The size of eddies alone does not measure the rate of energy dissipation. The number of eddies is equally significant.

However, provided the specific energy is compatible from section to section, considerable changes of section can occur without turbulence developing - the reach can be comparatively 'smooth'.

It is probable the conditions prevailing in natural streams are such that the form loss dominates the flow pattern and that the boundary 'friction' is a minor part of the total energy loss. The range of 'n' values is comparable to the variation of the drag coefficients of solid bodies in turbulent flow, e.g., a flat disc to a streamlined body of the same diameter (1.9 to 0.1).

With this background, the concept of compatible specific energy offers a useful, effective design basis. The only variation proposed to those introduced by Bakmateff is that the specific energy be measured from the total energy line.

The energy line at any point is at an elevation of $\frac{Q^2}{2gA^2}$ or $\frac{V^2}{2g}$ above the water surface. This can be an actual water surface or a chosen design level at which the design discharge Q is to flow under existing conditions. The level of the water surface,actual or design is usually easy to establish. The discharge at that level is, however, more difficult to determine, but a change in average velocity from say 4 ft/sec. to 5 ft/sec. at the same level, i.e. 25% increase in discharge,leads only to $\frac{25}{64}$ ft - $\frac{16}{64}$ ft = $\frac{9}{64}$ ft (lbs/lb) difference in the level of the energy line. No river works can, at present, be designed to this accuracy. This type of error can be carried through the works and the design chosen always to reduce its significance without undue cost increases.

Establishment of the existing total energy level above and below the proposed works determines the 'energy line'. What in fact happens to this line in detail is not significant - it cannot rise above the upper level nor fall below the lower. The concept of compatible energy simply is that significant changes in section can be made without disturbing unduly the total energy levels. The changes made are those best suited, cost wise, to achieve the object of the proposed works.

In general, the specific energy at a cross section is given by

$$\frac{A}{B} + \frac{Q^2}{2gA^2}$$

where Q is the total (design) discharge and is constant, but A (the area) and B (water surface width) can vary. The minimum size is obtained when

$$\frac{A}{B} = \frac{Q^2}{gA^2} \quad \text{or} \quad \frac{A^3}{B} = \frac{Q^2}{g} \text{ which is known.}$$

This can be attained by decreasing A with B constant, increasing B with A constant, or varying both in a particular proportion.

As the elevation of the total energy line is to remain unchanged and the specific energy of a vertical element is always $y + \frac{v^2}{2g}$, the level of the lowest point of the section is

$$\text{Elevation of energy line} - (y_{max} + \frac{v^2}{2g})$$

where y_{max} is the maximum depth

v = velocity.

This condition determines the relative vertical position of the whole section.

For a geometrical section-form, A and B can be written in terms of y, the depth which leads to a simple arithmetical solution. The designer then is required to determine the form of the channel and the depth or the width (but not both), whichever may govern the design in the particular circumstance.

After model testing, a number of structures have been built to this procedure and have been subjected to flows of some magnitude during the reasonably wet summer of 1970-1971. It is proposed to give the detail design of some of these structures and a report of their observed behaviour. As the design procedure is not restricted to road structures, the behaviour of other structures is also reported as their performance is essentially linked to the validity of the basic concept.

1. NUDGEE ROAD BRIDGE, BRISBANE.

The area to the north east of Brisbane is low lying. Recent improvements associated with the flood alleviation in the Kedron Brook led to rapid industralisation of this area and Nudgee Road has become the main arterial road from this area. It crosses the Kedron Brook where the flood plain is 1400 feet wide (Fig. 2). Ground level is generally R.L.10.0. Flood level is R.L.15.6, when the discharge (1:30 year flood) is 30,000 cusecs. Previously the road was at a low level, R.L.13, with an open causeway and bridge over a 35 feet wide drainage canal, with approach embankments on either side. This approach was untrafficable at least once per year. The state of this structure and the rapid increase in heavy traffic required the immediate construction of a new two lane bridge (subsequently to be four lane). An extensive new shopping town had already been constructed less than a mile upstream. The 1967 flood rose to within one foot of the agreed floor levels in this shopping centre. It was essential that no further rise was induced by the new bridge. However, the cost of bridge in excess of embankment amounted to approximately $1,000 per foot at present, with a further $1,000 later.

Design data: Flood level R.L. 15.6
Design discharge 30,000 cusecs
Ground level R.L.10.0.

Slope is surface fall only and approximately 8 feet in 7 miles.

The approach, being so wide, was considered rectangular, i.e. $\frac{A}{B} = y$, the depth.

The flood level 15.6 was taken as a limit approximately 1000 upstream of the proposed bridge where the width was 1320 feet.

The approach velocity, $v = 30,000/1320 \times 5.6$

$$= 4 \text{ ft/sec.}$$

$$= 15.6 + \frac{v^2}{2g} \quad .$$

R.L. energy line upstream = 15.85.
It was taken as 15.6 at the bridge and 15.4 downstream.

While the ground remains at R.L.10.0, the flow can be converged until the critical depth, y_c, is reached.

Than y_c = 2/3 Specific Energy

$$= 2/3\ (15.85 - 10)$$

$$= 3.9 \text{ feet}$$

whence q flow/ft width $= y_c\sqrt{g\ y_c} = 43$ cusecs

minimum width = 30,000/43 = 688 feet.

If the flow is to be further constricted, then deepening below ground level is essential. The Department of Primary Industries advised that the lowest level at which a reasonable grass cover (kikuyu) could be established was R.L.7.5. These levels are to a low water datum and high high-tide is R.L.8.0. The choice of this level of R.L.7.5 determines the total span length.

Specific Energy R.L.15.6 - R.L.7.5 = 8.1 feet.

Critical depth $= \frac{2}{3} \times 8.1 = 5.4$ feet.

Maximum flow per foot width $= 5.4\sqrt{g \times 5.4} = 71.0$ cusecs/foot.

Minimum width = 423 feet.

The design was nine spans each of 50 feet with round pile piers and 1:1 battered abutments which were stone pitched.

The stream approaches to the Nudgee Road were the subject of a secondary experiment - the use of grass as a protective material. There was no restriction of space. The lip of the inlet fan was 400 feet above the bridge. The slope of the approach was constant. Ground level at 10 was linked to the bridge at R.L.7.5. Irrespective of the length of the approach, the levels at the quarter points and consequently the width of the approach can be readily calculated.

TABLE I

Point	Lip	1/4	1/2	3/4	Bridge
Ground (Bed) level R.L.	10	9.38	8.75	8.12	7.5
Energy level	15.85	15.78	15.72	15.66	15.6
Specific Energy ft	5.85	6.4	6.98	7.54	8.1
Critical depth ft	3.9	4.3	4.65	5.03	5.4
Flow/ft width cusecs	43	50	57	64	71
Width ft	688	595	529	470	423

As the natural ground fell about the same amount as the total energy line, the outlet becomes an image of the inlet.

Figure 2 shows a sketch of the arrangement.

These measurements are in the direction of a flow net.

Model tests carried out at a scale of 1:48 natural gave results which agreed with the computed results with 0.2 feet. Such models are extensive as they must be truly three dimensional. As the energy of whole flow is involved everywhere, it is not possible to study a 'typical' longitudinal section.

The model also showed that, if the stream approaches were not depressed with a 450 feet span, the upstream flood level rose above R.L.18.5 - the minimum road approach level - and overtopping occurred. The computations showed that without the depression, a bridge 688 feet long would be required to pass the flow at an upstream level of R.L.15.6.

The model also showed the remarkably smooth, turbulent free flow pattern which is a feature of these designs - and hence the decision that grass protection alone would be adequate except under the bridge, where lack of sunlight would inhibit growth. This area was stone pitched.

It was also found that the effect of the training banks at the edge of the fam was not large. A head loss of about 3 inches at the design flood was the total effect. The value of the land behind these banks was, however, far in excess of the formation cost, as ample fill was available and the banks were formed from the material dozed from the depression. The banks were stone pitched at 1:1 batter at the bridge, but were quickly turned to a flatter batter suitable for grassing. The cost of all the earthworks was $10,000. The cost of the bridge was approximately $500,000.

The bridge and approach roads were designed by Blain, Bremner & Williams, Consulting Engineers for the Brisbane City Council.

Owing to past experience of siltation of pipes and culverts, it could be expected that this tendency would be accentuated by the depression. However, the situation is completely different. The present design simply introduces a reduction in section. At low flows the velocities in the culvert can well be less than the approach flow. With the constant energy design, the velocity in the narrow section will, at all flows, exceed that of the approach flow.

On Sunday, 25th October, 1970, a small flood occurred in Kedron Brook in the early hours of the morning. Slides 1, 2 and 3 contrast the resulting situation at three successive bridges on the Kedron Brook, viz., Melton Road, Hedley Avenue and Nudgee Road (refer to Fig. 3). The debris collection was common to all bridges on this creek, large and small. At some crossings structural damage resulted and a car was swept from an approach.

On Sunday, 8th November, 1970, another similar flood occurred. Slides 4, 5 and 6 show the situation at the same three crossings just after the peak of the flood. Again the collection of debris at Melton Road and Hedley Avenue.

The summer overall was very wet, but the distribution of rain was such that, while an excessive number of small floods (1 in 2 year frequency) occurred, the biggest was about 12,000 - 15,000 cusecs (1-5 year to 1-10 year). A film was taken at the Nudgee Road Bridge at about the peak of this flood. The smooth flow is very apparent - a short shot of the flow downstream of the bridge is worthy of note. No flood damage occurred to the grass protection. No debris was deposited in the area. A barbed wire fence below the bridge was swept away and the cattle confined by the fence moved into grassed area - possibly before the flood had completely subsided - and chopped up the area with their hooves. This healed without difficulty and no permanent damage appears to have been suffered.

At the end of the wet season, the area was completely free of any debris.

It would appear at this stage that this design is behaving according to the model predictions and there is no reason to believe that it will not do so at higher flows. Of matters not resolved by model studies, nothing has arisen to impair the practical feasibility of the use of this concept. Slide 7.

2. CULVERTS ON THE STUART HIGHWAY NORTH OF ALICE SPRINGS, NORTHERN TERRITORY, DESIGNED BY CAMERON, McNAMARA & PARTNERS, CONSULTING ENGINEERS FOR THE COMMONWEALTH DEPARTMENT OF WORKS.

The Stuart Highway was to be upgraded to an all weather road. It crosses McGrath and Harry Creeks, both of which have flood plains about two miles wide. The creeks each have catchments of about 100 square miles. The area is arid so slopes are small and even small flows carry large amounts of very fine clay. The main channels are only capable of carrying about a once in two year flood. Floods greater than this spill out into the flood plain and inundate the road for long periods. The main channels have built up levees about four feet higher than the remainder of the flood plain which prevents the return of water from the flood plain into the main channel. The bed level of each creek is in fact the same level as the flood plain.

Owing to the level difference upstream and downstream, severe scours occurred in the embankment. Raising the road embankment through the flood plain merely leads to a rise in the upstream water level until overtopping occurs. As all stone has to be imported long distances, adequate protection of the downstream face is prohibitively expensive.

In 1970, minimum energy twin cell culverts were built where the road crossed the main creeks and single cells in the flood-plain embankments. The road surface was strengthened, but not raised,and resealed. In the first week of January, 1971, a large runoff occurred after severe storms in the area and flows occurred through all culverts for some days. The water level rose close to the bitumen, but did not overflow so far as is known. Quote from Engineer's report - "Despite the fact that there was a lot of clearing up at the inlets and outlets to be done at the time of the storm, the culverts all worked well, carried large flows with no measurable loss of head and after the flow had ceased there was only about 2 inches of silt remaining on the floors of the culverts".

Photographs of road during this flood flow are believed to have been taken on Polaroid, but copies are not yet available. Photographs taken after cessation of flow could be of interest. (Slides 8, 9, 10, 11, 12). The captions (page 21) are those of the Consulting Engineer, Mr. Ian Cameron.

↑ Ask Prof. APELT

3. CULVERT AT JERRY'S DOWNFALL, BRISBANE - BEAUDESERT ROAD. DESIGNED BY UNIVERSITY OF QUEENSLAND TO MAIN ROADS DEPARTMENT STANDARD 7' X 5' CULVERT. BUILD BY MAIN ROADS DEPARTMENT.

Full details of the design of this culvert were given as an appendix to a paper "Pavement Drainage", A.R.R.B. Conference, Canberra 1970. The object was to show that a standard Main Roads Department (Queensland) culvert could be used successfully as a minimum energy culvert. In normal use the standard 7' x 5' culvert designed to be structurally adequate for up to 5 feet in height is cut off in height (bottom portion excluded) to suit the road level and ground level. The floor of the culvert is placed at ground level. The cost advantage of using this type of structure is considerable. Standard formwork is available, but largely because a gang can be trained to a procedure and the whole culvert can be fabricated by largely non-craft labour very quickly and without undue difficulties.

The completed roadworks are shown in Slide 13. The roadway is level within 0.10 feet for 1000 feet, at R.L.153.77. The ground across this length is also level at R.L.147.5, but at the northern side there is a slight depression - the remnants of a drainage line and the culverts were set on this side to provide drainage. It was estimated that 2050 cusecs would flow at a level of R.L.150.83, and eight cells of culvert were considered necessary using the full height of 5 feet. The floor was set at R.L.145.40. Longitudinally the grade was very flat and total drainage of the culvert was not reasonably possible. A pipe, however, was set through the downstream fan in an attempt to reduce the water level as the flow approaches were protected only by kikuyu.

The culvert must have been subjected to some flows between October 1970 and February 1971, but each time a visit was made to the culvert after heavy rain, no flow was then taking place. Apart from some washings from the table drains deposited on the approach fans, no deposition or debris was visible. On February 24th, after long period and medium heavy rain, a local storm (3.56 inches at Beaudesert) produced heavy runoff in this area. Unfortunately, the flow commenced about 2.00 a.m. and the peak passed about 4.00 a.m. so no photographs are available. The Area Foreman reported that at its worst the water was 6 inches below the top of the concrete head upstream and 9 inches below on the downstream, which was a little disturbing. No flow whatsoever took place across the road. Some surface scour was also visible on the downstream fan.

Subsequent levelling showed the flood height, measured by the considerable debris marks on a fence 75 feet downstream of the road formation, to be R.L.151.00. This was an average of a number of levels, the maximum variations being 0.03 feet. This flood level is thus higher than design R.L.150. In addition, it was found that a barbed wire fence heavily laden with debris had been swept down by the flood. One end went through culvert one, the other through culvert five, so that a partial block of culverts 2,3 and 4 had occurred.

In view of this, discharge performance could be rated as high. The upstream fan was unscathed. At the downstream fan, a portion of the kikuyu had been torn off, but the maximum depth of scour was about 1 foot. The structure was in no way endangered. There is no doubt that the partial blockage had given rise to considerable turbulence which would cause both the downstream scour and the unduly high levels. It also became apparent that the wing walls could be raised with advantage as at these high levels some spillage must have occurred over the wings interfering with the major flow. The scour of the downstream fan was less severe than the scour round a telephone pole 75 feet downstream and some distance to the side of the line of culverts.

4. CULVERT UNDER THE BURNETT HIGHWAY ON THE ONE MILE CREEK SOME FOUR MILES NORTH OF GOOMERI. THIS CULVERT WAS DESIGNED AND BUILT BY THE KILKIVAN SHIRE ENGINEER, R.W. BOYS, WHO SUPPLIED THE DESIGN FIGURES, PHOTOGRAPHS AND REPORT OF ITS PERFORMANCE.

The creek catchment is farming area 4000 acres in extent, hilly at the source and lightly undulating at the culvert site. The design discharge was 1140 cusecs at R.L.62.00. Bed level in the drainage depression was R.L.59. The road is level R.L.62.5. Any increase in flood level would inundate adjoining fields which are sown to lucerne and some other high priced crops, which deteriorate rapidly when flooded.

The slope of the inlet and outlet fans was chosen as 1:4 in order to accommodate the structure within the limits of the road boundaries and avoid resumptions.

Table II - Design Tabulation
R.L. Energy Line 62.00
(neglect approach velocity)

Point	Lip	1/4	1/2	3/4	Culvert
R.L. Ground (invert)	59	58	57	56	55
Specific Energy H_s ft	3	3	5	6	7
Critical depth $y_c = \frac{2}{3} H_s$ ft	2	2.67	3.33	4	4.67
Flow per ft width $y_c\sqrt{gy_c}$ cusecs	16	24.7	34.4	45.3	57
Width ft	72	46	33	25	20*

*Chosen.

The design procedure is as follows.

At the entrance to the inlet fan, ground level is R.L.59. As there is no reason for it to be otherwise, this level is chosen as that of the lip.

Hence, the Specific Energy, H_s, here is 62.00 - 59 = 3 feet. Hence critical depth = 2/3 H_s = 2 feet. Therefore flow per foot width q is $y_c\sqrt{gy_c}$ = 16 cusecs; thus width is $\frac{1140}{16}$ = 72 feet.

At the culvert entrance the design choice is 20 feet width.

Hence flow per foot width q = $\frac{1140}{20}$ = 57 cusecs

$$\text{thus} \quad y_c = 3\sqrt{\frac{q^2}{g}} = 4.67$$

$$H_s = 3/2\ y_c = 7,$$

thus culvert invert must be R.L. (62 - 7) = R.L.55.

Thus the R.L. of the 1/4, 1/2 and 3/4 distances in the entrance fan must be, irrespective of the slope, 58, 57 and 56 respectively. This allows the calculation of the widths at these points as for the lip.

The design culvert selected was a 3 cell standard 7' x 5' used to the full height. The approaches were modified to 22'4" to match the culvert (Fig. 4).

The estimated cost was $3,000 made up as follows:

	$
140 cu yds excavation @ 40¢	56
40 cu yds rockfill in scours d/s	80
51 cu yds concrete in R.C. culvert	2080
164 sq yd grouted stone pitching	492
12½% overheads.	

The culvert was constructed in November 1969 at an actual cost of $3176. A 4" diameter drain was provided to drain the culvert as this was considered to be a mosquito breeding hazard. Normally a culvert opening of 200 sq.ft would be required for 1200 cusecs discharge, i.e. 6 cells 7' x 5' and in fact the Main Roads Department (Queensland) has provided a culvert of this size, on this creek, about a half mile upstream. The cost advantage at $600 per additional cell is particularly obvious in this case.

Following some smaller flows, a discharge close to design occurred early in January 1971.

"Although the photographs and indeed the culvert may not look impressive, the effect in actual operation is really most spectacular and successful. I was not able to visit the site at full flow, the peak appeared to have passed and the flood was several inches below the design head and falling. The velocity, however, seemed to be considerably in excess of the calculated 12.1 ft/sec. It was a very spectacular sight and there was no wave action to indicate the critical velocity was exceeded. There had been several smaller flows prior to this: there was no debris in the culvert. In any case, I think any debris would be only light trash and weeds from the farming area. No heavy debris comes down this far. Before the culvert was built, scours were developing in a short gully through which the creek flows into Nangin Creek. These were protected by rock fill. There is no evidence of further scour.

I might add that the Councillors who saw the culvert in action were obviously impressed and in view of the cost and results were very pleased. This is the second effort in this method - the first was a much smaller one as a trial run and appears to be operating quite successfully".

No details are available of this first culvert.

5. DESIGN OF CULVERTS FOR THE SOUTH EAST FREEWAY FROM BRISBANE

The second stage of the South East Freeway from Brisbane passes into the valley of the Norman Creek and continues generally in the line of the creek. The Norman Creek (catchment) is one of the major drainage systems of the Brisbane Area and is rapidly becoming fully urbanized. As a result of this urbanization, severe flooding occurs in the lower reaches of the valley. While the Freeway does not make a significant change in the catchment, the structure causes large changes in the valley. In the upper reaches complete realignment with many culverts is necessary. Lower, four major crossings of the valley are necessary and the embankment occupies a significant area within the flooded area. Model tests showed that the time of concentration was materially reduced consequently increasing the probability of a particular flood. This undesirable effect could be countered by the provision of increased storage.

It is proposed, at this stage, to use constant energy culverts at four major crossings. Not only does the use of this concept offer considerable cost advantage in the culverts, but also a very close control of the flood height can be achieved automatically. These four culverts have been designed for the Main Roads Department, Queensland by Messrs Rankine & Hill, Consulting Engineers. Two marked "X" in Table III were examined in detail by model. As the results of the model tests were so close to the computed figures, it was agreed that detailed model examinations of the other two, which had no complications, was not necessary.

TABLE III

Freeway Crossings - Norman Creek

Culvert Designation	Design Flow cusecs	Design Flow cusecs/acre	Disaster Flow cusecs	Size width high ft	Length not including Fans
"X" Ridge St Deviation	7010	2.06	9500	7 No.9'x10'	196'
Ekibin Waterway	7010	2.08	9500	62 open channel	290' future 805
"X" Station 100	6000	2.34	8800	4 No.10'x12'	480'
Station 156	2700	2.96	4200	4 No.9'x10'	365'

TABLE IV

Design Details - Ridge St Deviation

Location	Entry Lip	Culvert Entrance	Culvert Exit	Exit Lip
Reduced Level. Total Energy	32.5	32.5	Assumed 31.52	30.0
Reduced Level. Invert	26.0	21.55	20.57	24.0
Specific Energy - H_s ft	6.5	10.95	10.95	6.0
Critical Depth - y_c ft	4.3	7.3	7.3	4.0
Flow/ft width - q cusecs	51.2	112	112	45.4
Width ft	137	62.6	62.6	154
Reduced water Level calculated	30.3	28.85	27.87	28.0
Reduced water level model	30.2	29.5*	28.9	28.8

* Sloping culvert walls at entrance.

The layout and ground features demanded considerable obliquity of the approach flow. It was desired to observe if guide vanes, in the sense of commencing the culvert walls at the lip, or some other device would be necessary. However, the obliquity of approach apparently presented no serious problem. There was some separation from the entrance boundary wall on the left bank between the lip and culvert entrance, but not to the extent of causing undue overload or deficiency in any cell. At exit, despite high velocities (15.0 ft/sec) in the culvert, the flow spread reasonably over the exit fan and no further protection except grass downstream is proposed. In fact, the exit fan edge was made square to the culvert to save concrete as no detrimental effect of this curtailment could be observed.

In this design a new device was introduced to minimise the effect of the sudden change of section at the entrance to the culverts. As there are seven culverts with 1 foot thick walls there is a change in width from 69 feet to 63 feet to accommodate the culvert walls. At design flow the depth in the culverts is critical at 7.3 feet. In the wider width just outside, the depth must be sub-critical and is given $y + \frac{v^2}{2g} = 10.95$, the energy at this point and $vy = \frac{7000}{69}$. Thus $y^3 - 10.95\,y^2 + 160.8 = 0$. Solving gives $y = 8.95$. Obviously such a discontinuity, 1.65 ft, is not possible and resolves itself into the formation of a pronounced bow wave.

The difficulty could possibly be resolved by using a sharp edged nose to the piers extending some distance upstream. This, however, presents construction difficulties. Instead blunt nosed piers with a sloping front face were used and the floor level adjusted to give critical depth both at the commencement of the pier as well as in the culvert. In the intervening length the area of flow A is decreased, the top width B remains constant, thus the term $\frac{A}{B}$ is diminished, thus $\frac{Q^2}{2gA}$ increases forcing the water surface down and the extra energy required is supplied by the fall in the floor and a smooth water surface profile is achieved. (Fig. 5).

If the flow is critical at the 69 feet width, the flow per foot width is $\frac{7000}{69} = 101.4$ cusecs. Thus

$$y_c = 3\,\frac{101.4^2}{g} = 6.9 \text{ feet.}$$

Thus the required specific energy $H_s = 10.35$.

Thus the invert level is 32.5 - 10.35 = R.L.22.15, which compares with R.L.21.55 in the culvert. The water level is here 22.15 + 6.9 = R.L.29.05, compared to R.L.30.4 with the vertical profile to the culvert walls.

It will be seen that the depths on the model at the exit to the culvert and exit lip are in excess of the calculated values. The storage demand sets the upstream energy level which is in excess of that which would be in the undisturbed flow. The downstream energy is determined by the level of the undisturbed flow at that point. As there is no means of readily dissipating energy in the culvert, there is excess energy available at the lower end of the culvert leading to an increase in depth. This energy has to be dissipated and will be by turbulence below the outlet fan. In this case, it is not excessive and as the outlet fan did spread the flow quite widely although not completely, there was little concentration of energy across the section. A reasonable grass cover should be adequate protection.

In all these Freeway crossings of the Norman Creek, provision had to be made to carry the stream proper through the culvert in a low flow channel. As the area is completely urbanized, there is always a small flow, largely sullage water. The choice of the level of the culvert floors was determined largely by the necessity to avoid ponding in the culverts and maintaining, always a through flow. The culverts in general were set about 2 feet above the invert of the low flow channel which in this case was four feet wide and taken through the third culvert.

It is clear the local effect of this low flow channel depends on the relative levels of the inverts of the culverts and the low flow channel. The grade of the low flow channel is also determined by the bed levels of the stream at the inlet lip and that at the outlet lip and leaves little choice. The grade of the culvert follows this grade. The slope of the inlet and outlet fans is markedly different and consequently the difference in specific energy compared to that in the low flow channel is very significant in flood flows. However, provided the low flow channel is small compared to the whole cross sectional area of flow, these local changes do not appear to affect the overall performance to a degree necessitating special treatment, e.g., roofing the low flow channel through the inlet and outlet - and the considerable cost involved.

CROSSING AT STATION 100

The siting of this crossing gives rise to considerable problems. The crossing is very oblique and the culverts long. The stream proper, upstream of the crossing, is hard against the very steep right valley bank and then meanders across the flood valley at the site of the Freeway crossing. The cross sectional area of the stream upstream is also relatively large. The Freeway crossing interferes with the confluence of an important tributary system which has to be diverted to below the crossing. It occurs immediately above a local traffic bridge and a major bend. At the same time, it offers the best site for detention storage. The right bank of the valley is high and steep. The high Freeway embankment will form the left bank. The valley grade is steep and so confines the increased flood level to a reach in which there is to be no development. Future development envisages a major intersection at this site and adequate provisions have to be allowed for this development.

TABLE V

Design of culvert Station 100
Design flow 6000 cusecs
Maximum flow 8800 cusecs
Effect of low flow channel ignored

Location	Entry Lip	Culvert Entrance	Culvert Exit	Exit Lip
R.L. Total Energy	46.4	45.3		40.0 [1]
R.L. Invert	37.1	32.0	28.0 [2]	32.0
Specific Energy H_s ft	9.3	13.3		8.0
Critical Depth y_c ft	6.2	8.8		5.3
Maximum flow/ft q cusecs	87	149		69
Width	68	40	40	85
R.L. Water Surface	43.3	40.9		37.3
R.L. Water Surface MODEL	45.8	40.5		36.4

1. Assumed from downstream.
2. Accommodates a sudden fall in culvert floor.

The effect of the low channel through the inlet lip is considerable and its effect is accentuated by the shape of the natural approach channel. The shape of the inlet was modified also for site reasons, but the higher water level can be tolerated. It was maintained moreso as this inlet lip determines the upstream flood level at the disaster flow of 8800 cusecs which produces a depth of 11.7 at the culvert inlet. It is obviously desirable that the change in flood level from 6000 cusecs to 8800 cusecs is as low as possible.

In general the model behaved reasonably well. With modification at the entry lip (battered ends), the fan distributed the flow adequately to the culvert cells with a maximum variation of one foot - which is tolerable.

It was obvious that unless some form of dissipator was introduced, there would be considerable surplusenergy at exit. The floor of the culverts was dropped suddenly 20 feet from the lower end. Later a similar drop was introduced about half way up the outlet fan. It is estimated about 1 ft lb/lb surplus remains, but as the flow is reasonably well distributed across the outlet fan, it is considered that a reasonable grass cover will provide adequate protection below a stone pitched transition from the lip to natural surface.

A preliminary drawing of this culvert is given in Figure 6.

THE DESIGN OF WEIRS

Of some interest should be the design of weirs by this same procedure. The first was built at Clermont, Central Queensland, by necessity of the situation. Clermont is situated downstream of the chosen storage site on Sandy Creek. The river banks are considerably higher than the adjoining land. If the river overtops the banks, the floods sweep away from the river and inundate the town to considerable depths. The town had previously suffered devastating floods - one drowning nearly a hundred residents. Although the demand for storage was urgent, the local Council added the condition that the weir must not cause the flooding to occur at lower flows or to any greater extent. To obtain satisfactory storage a minimum storage level of R.L.842 was determined. The bank level was R.L.850.0, the bed level R.L.822 and the bank full flow was 30,000 cusecs. No traditional weir type could be found to satisfy the requirement. So far as I know this problem has not previously been solved.

The concept of constant energy was applied. The bank full water level wastaken as R.L.850. The area of flow was 4300 sq. ft given an approach velocity of 7 ft/sec. The R.L. of the energy line was thus 850.75.

Assuming rectangular section -

Specific energy	=	8.75 ft
Critical depth	=	5.83 ft
Flow/ft width	=	79 cusecs
Minimum width	=	380 ft.

This width is as one would expect considerably wider than the river. It was envisaged the weir would be a bank with flattish upstream and downstream batters. At any level on this bank again the minimum width can be calculated until the flow returns to the natural channel width.

Bank level	H_s ft	y_c ft	q cusecs	width ft
840	10.75	7.2	109	274
838	12.75	8.5	140	214
836	14.75	9.8	173	173.

It will be seen that the required width decreases rapidly as the depth increases and it very quickly reaches the natural width of river. Theoretically these widths are required upstream and downstream. The flow pattern requires curves which conflict upstream and downstream. As the downstream face is always vulnerable, the crest was curved to a circular arc with a downstream centre.

To make it practical an arc was chosen so that the radii at the ends of the requisite length approximately gave the widths required with the slope of the downstream face. The upstream shape was arranged to give radial flow at the ends of the crest. This shape, shown in Slide 22, was subjected to considerable model testing. The results were so promising it was decided to construct the weir despite all the obvious constructional hazards. It was designed by the Department of Local Government; an earth bank protected at the crest and downstream face by a 6" thick R.C. slab and upstream by rock pitching. The construction did give trouble, but for reasons not particularly associated with the design technique. Ultimately, it was successfully completed. So far as it has been possible to observe, it is behaving as predicted. It was completed in 1962 and has been overflowed many times, but the size of the maximum flow is not known. No scour or erosion has taken place downstream.

DEVELOPMENT OF GRASS PROTECTED OVERSHOT WEIRS

Again the very smooth flow associated with these designs was evident in both the model tests and flows over the actual weir. The major portion of the cost of the Clermont weir was in the protection of the earth bank. A series of experiments was carried out to study the feasibility of these weirs protected only by vegetation. After a number of laboratory experiments, four weirs were built. The choice of site was changed.

The widest part of the stream was sought, where it was shallowest - this invariably gave two feet of storage immediately. The banks were 4 - 5 feet high on this elevated bed level and the crest calculated to the local conditions varied from 150 - 250 feet wide.

The first was built at Yuleba near Roma. The cross section and finished plan is shown in Figure 7. The construction cost was $292.00. It stores 5½ million gallons. Five weeks after construction overflow occurred. The owner had, however, managed to obtain a fair cover of sown grass and the weir survived. Since then it has been inundated many times in many ways. In 1969, a sudden storm of over 3" caused an overflow after the dam had been completely exposed for over six summer months. In 1970, overflow took place continuously for twenty eight days, although at times this flow would only be about one inch (Slide 25).

The second weir built at Bungunya in 1969 cost $400.00 and stores over 10 million gallons. It was severely damaged resulting from rain commencing the day formation was completed. It was reinstated for $120.00. A good grass cover was achieved by sprigged kikuyu and it was completely intact after a severe (over 1500 cusecs) and prolonged flood in 1970. The catchment is about 120 square miles.

The other two weirs, at Tara $100.00 and Moonie, $400.00, both failed. Both were almost devoid of vegetation; Tara because no attempt was made to grow. Moonie had the original cover completely removed by invading sheep and then a complete drought - not a single runoff in eighteen months voided any attempt to replant it. Moonie is to be rebuilt and will cost about $100.00, but in addition a plastic reinforcement will be built in.

A further weir also reinforced has been built at the Department of Primary Industries Research Station at Kingaroy.

The object of reinforcement is to reduce the vulnerability of the bank in the early stages of vegetal growth and also that resulting from accidental damage.

These small weirs are subjected to velocities nominally in excess of 16 ft/sec. With a good grass cover, they show no signs of failure and there is every reason to believe that grass protection will be completely adequate in this constant energy structure with considerable cost saving.

SPILLWAYS

Two spillways have been designed to this constant energy principle - that at Lake Kurongbah is a major structure 350 feet long at the crest carrying a maximum flow of 30,000 cusecs. It converges rapidly to a 100 feet wide channel ending in a ski jump spillway (Slide 26). The efficiency of this spillway allowed an extra 1'6" of storage possible. The second is a smaller structure from the cooling water storage at Swanbank Power House.

The design is identical with that of a culvert inlet. The energy line is initially the maximum flood level in the reservoir. The storage level determines the crest length. It can then be converged into any size of channel determined solely if necessary by the cost structure of floor compared with the side retaining walls.

Although only relatively small overflows have occurred, both appear to be performing as predicted.

CONCLUSION

1. It would appear that the concept of constant total energy and compatible specific energy is valid within the limits required for practical design.

2. That the minimum energy condition can be used to provide economic structures for a wide variety of purposes.

3. Although the concept leads to quite different forms which for full development may require some variation of traditional construction requirements, it has been shown that with present methods considerable savings are nevertheless possible.

4. The concept offers practical solutions to previously unsolved problems.

5. The design techniques suggested to make use of this concept allow for much more detailed analysis than is otherwise possible.

6. Essentially the designs minimize the random turbulence normally associated with Civil Engineering structures.

7. Although the design determines where energy is best dissipated, it is still not possible to control with any accuracy the rate of dissipation.

8. The limits to which the techniques can be successfully used are not yet defined, but the operation full size does appear to be more favourable than on a model.

CAPTIONS TO SLIDES (NOW REPRODUCED HERE AS PHOTOGRAPHS)

1. Page 7 - Debris collected at Melton Road Bridge during a small flood October 25th 1970. Photo taken 28th October 1970 by Courier Mail.

2. Page 7 - Debris in the approach to Hedley Avenue Crossing taken 29th October 1970 after Brisbane City Council had cleared debris from the roadway.

3. Page 7 - Nudgee Road Bridge completely free of any debris. Taken 29th October 1970.

4. Page 7 - Melton Road Bridge November 8th 1970. Small flood receding. Note debris packed against bridge. Dragline was clearing debris from bridge deposited by previous flood.

5. Page 7 - Hedley Avenue Bridge November 8th 1970. Again debris packed against bridge.

6. Page 7 - Nudgee Road Bridge November 8th 1970. General view of bridge. No fixed debris anywhere.

7. Page 7 - Nudgee Road Bridge - end of wet season. Grass in good condition - no deposition of debris.

8. Page 8 - Downstream end of a twin 2/7' x 4'6" culvert. The radiused concrete lip is visible in front of the C.D.W. engineer. The heap of silt on which he is standing was almost certainly deposited by later flow in the downstream table drain after the main flow had ceased.

9. Page 9 - The upstream end of the same culvert. The barrels were clear of silt but about 12" had been deposited on the upstream apron by late flow from the newly completed (and therefore highly erodable) table drains. There is no grass cover in the area.

10. Page 8 - The upstream end of a single cell culvert showing the radiused concrete lip on which the C.D.W. engineer is standing. Beyond this a natural lip concentric with the concrete apron lip has formed naturally. The inlet had been filled in for a temporary side track and this had not been removed at the time of the flood.

11. Page 8 - Looking downstream through a single cell culvert. The high level of the bed upstream and downstream compared with the culvert invert level can be seen. These again are higher than designed and some improvement to bed level is yet to be done. Nevertheless the culvert worked and is free of silt.

12. Page 8 - Looking upstream through the same culvert as in 11. The side track across the outlet head started to scour but is still not down to the level of the downstream lip.

13. Page 9 - Site of "Jerrys Downfall" on the Brisbane-Beaudesert road about 16 miles from Brisbane - a floodway crossing 1000 ft wide.

14. Page 9 - A standard M.R.D. (Qld) 7' x 5' 8 cell culvert at Jerry's Downfall set as a minimum energy culvert (November 8th 1970).

15. Page 9 - The same culvert immediately after a major flow (in excess of design) February 24th 1971.

16. Page 9 - Minor scour at the downstream fan of the culvert.

17. Page 9 - Scour round a telegraph pole near Jerry's Downfall culvert.

18. Page 10 - Culvert on Burnett Highway, Shire of Kilkivan looking downstream.

19. Page 10 - Culvert on Burnett Highway entrance fan.

20. Page 12 - Model of culvert for South East Freeway (Brisbane) Ridge Street Deviation.

21. Page 12 - Model of culvert for South East Freeway (Brisbane) Station 100 (No noses).

22. Page 17 - Plan of Clermont Weir.

23. Page 17 - Model of Clermont Weir 5000 cusecs

24. Page 17 - Flow over actual weir approximately 5000 cusecs.

25. Page 18 - Grass weir at Yuleba.

26. Page 19 - Spillway at Lake Kurongbah.

27. A constant energy shape formed naturally below a small bridge.

28. Severe erosion site on a stream bank protected by plastic mesh reinforced grass after four years.

Some movie shots of the flood in the Kedron Brook at Nudgee Road were also shown.

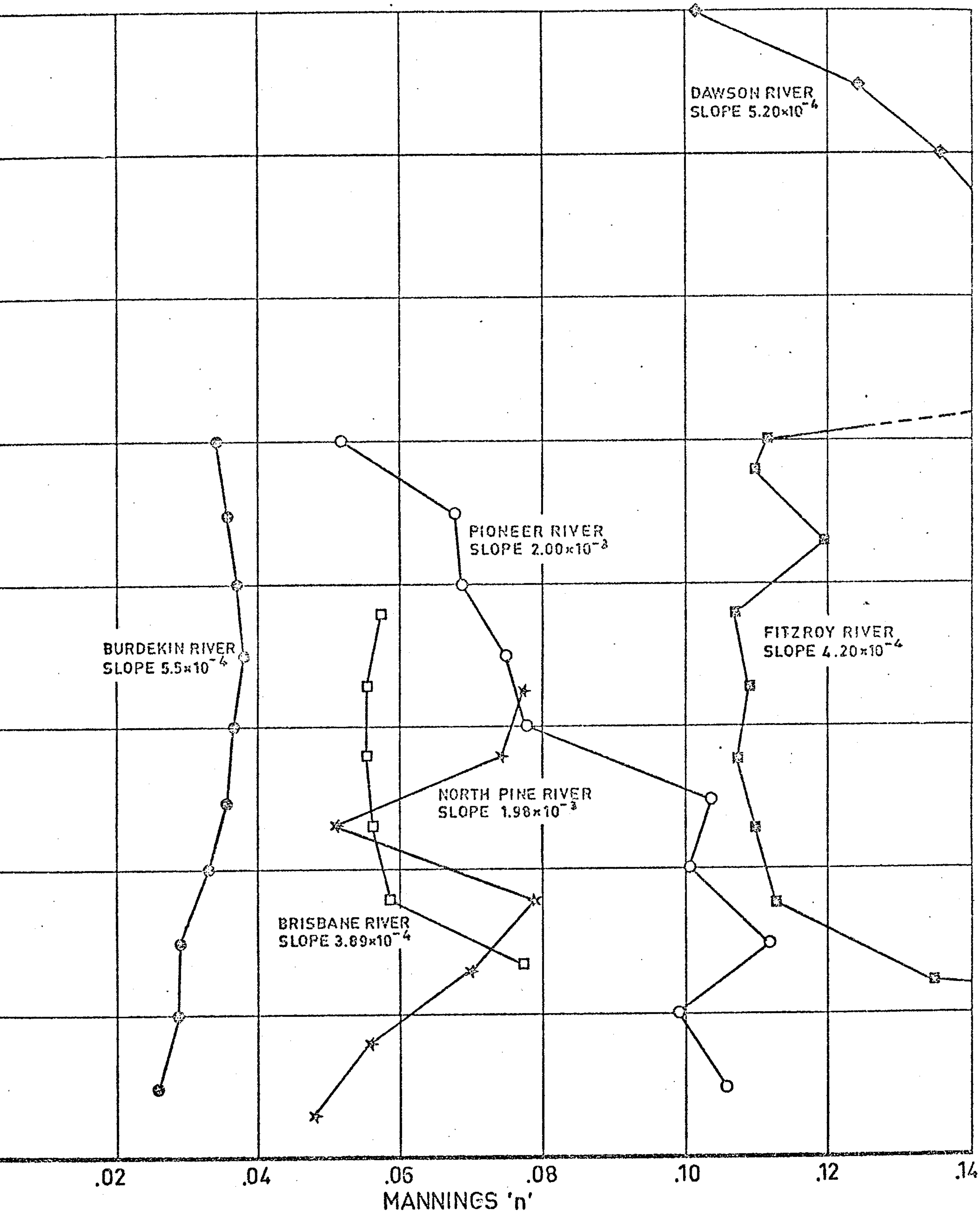


Fig.1 PLOT FOR RIVERS of MANNINGS 'n' vs. DEPTH

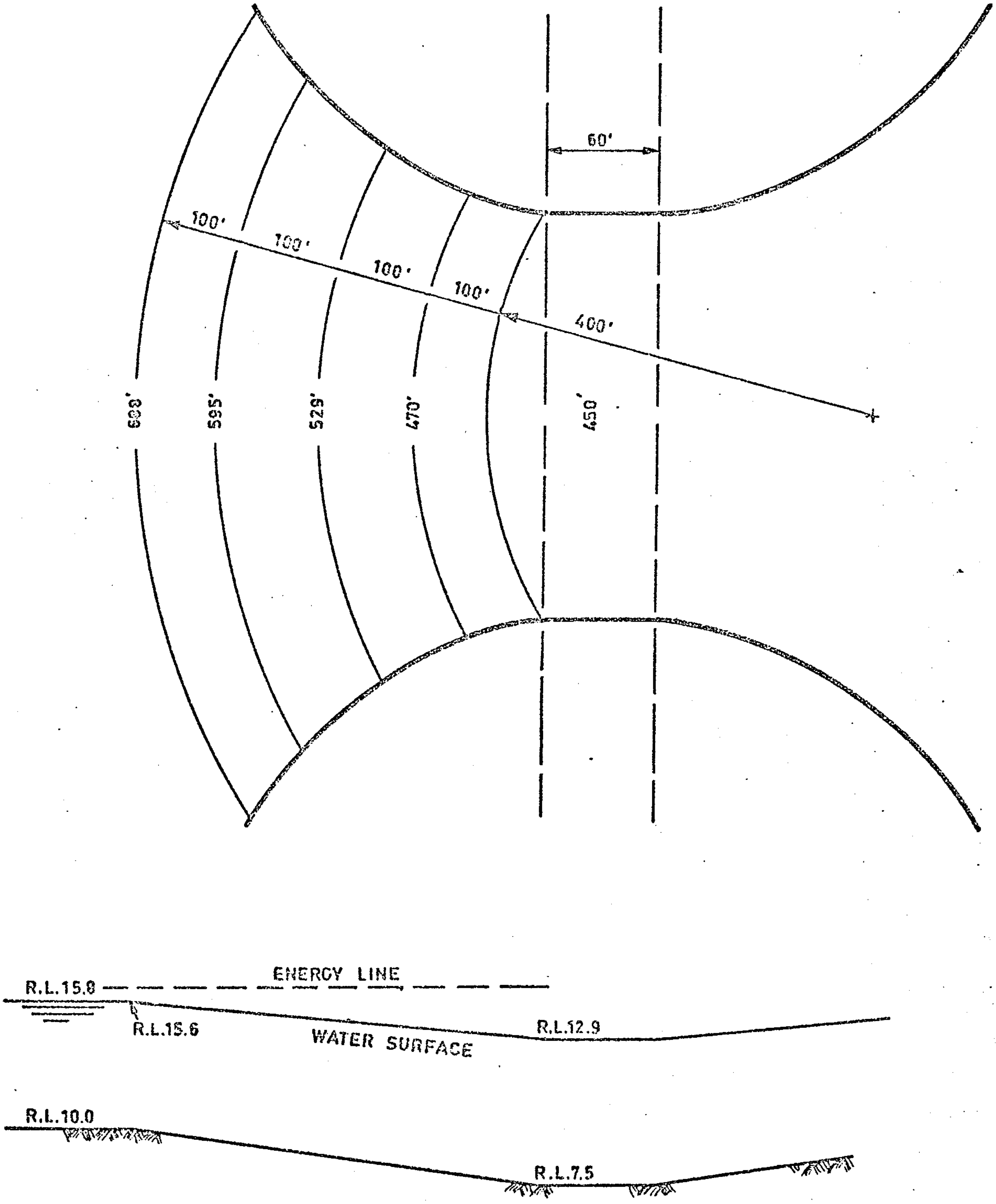
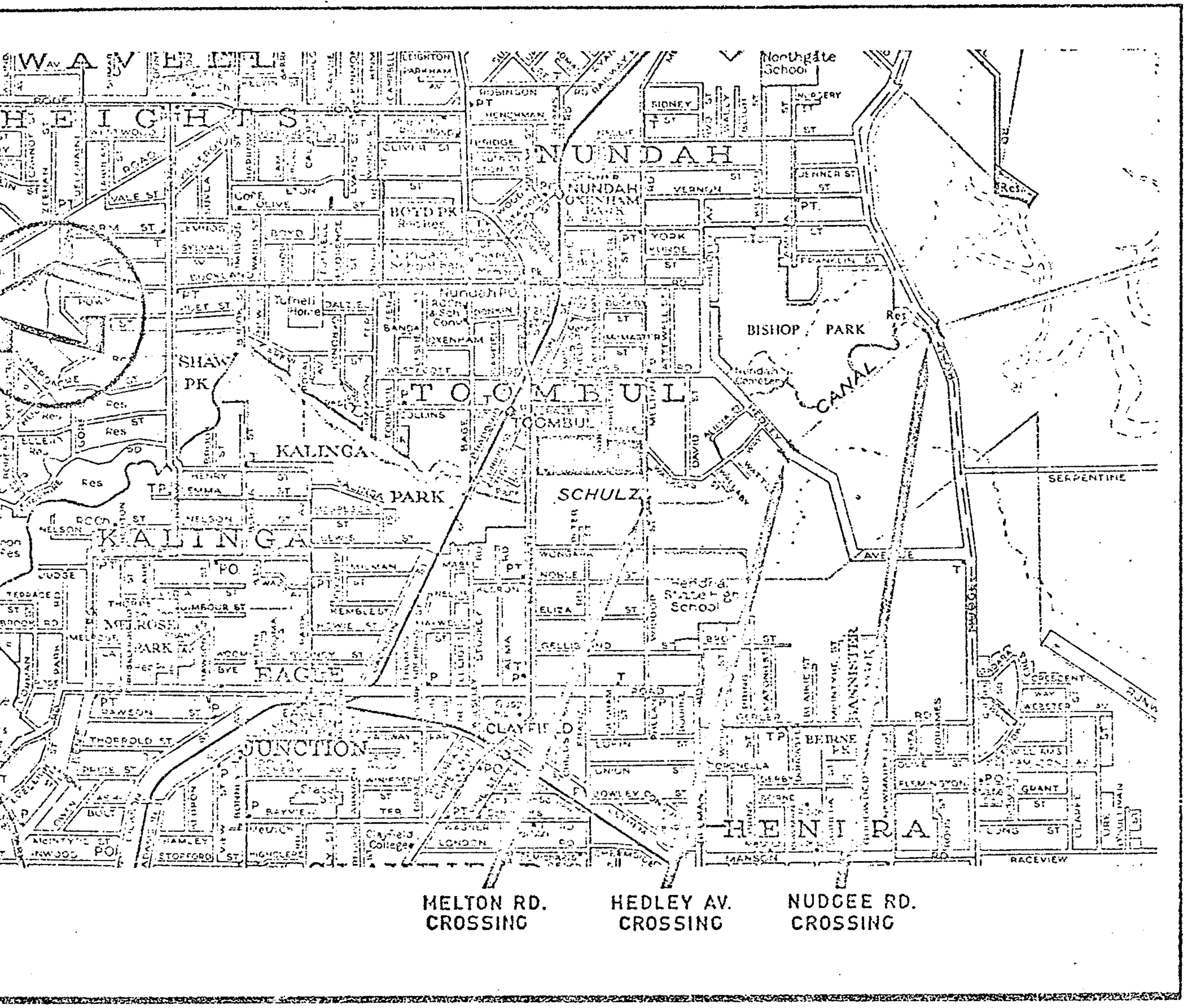


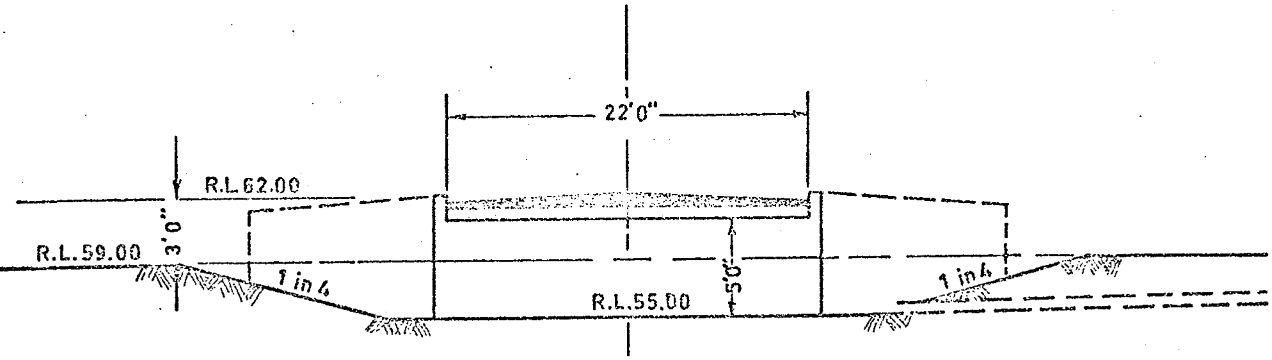
Fig. 2
NUDGEE RD. BRIDGE INLET



LOCALITY MAP

SCALE 1 inch = 30 Chains

Fig 3



CROSS SECTION

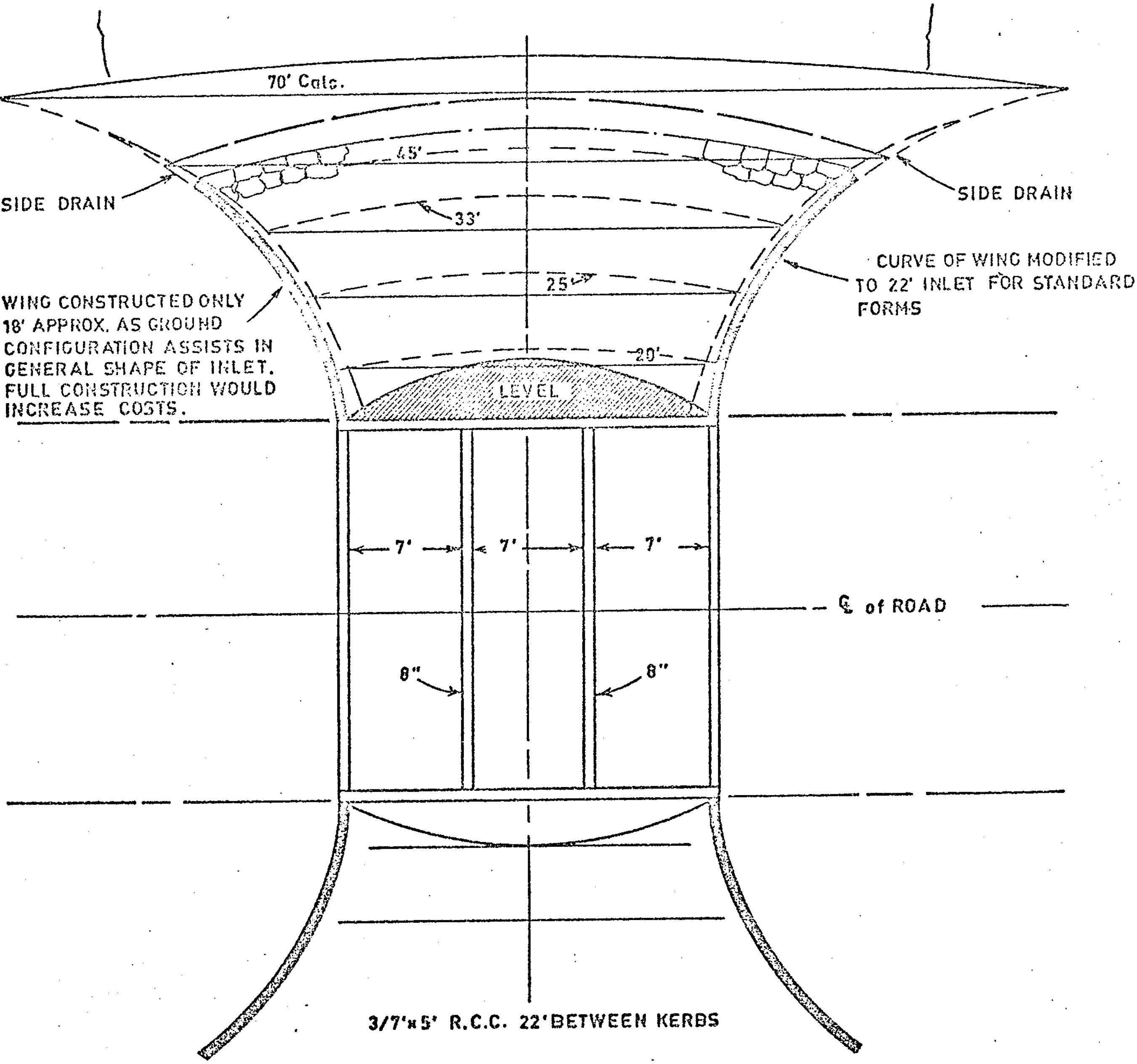


Fig.4

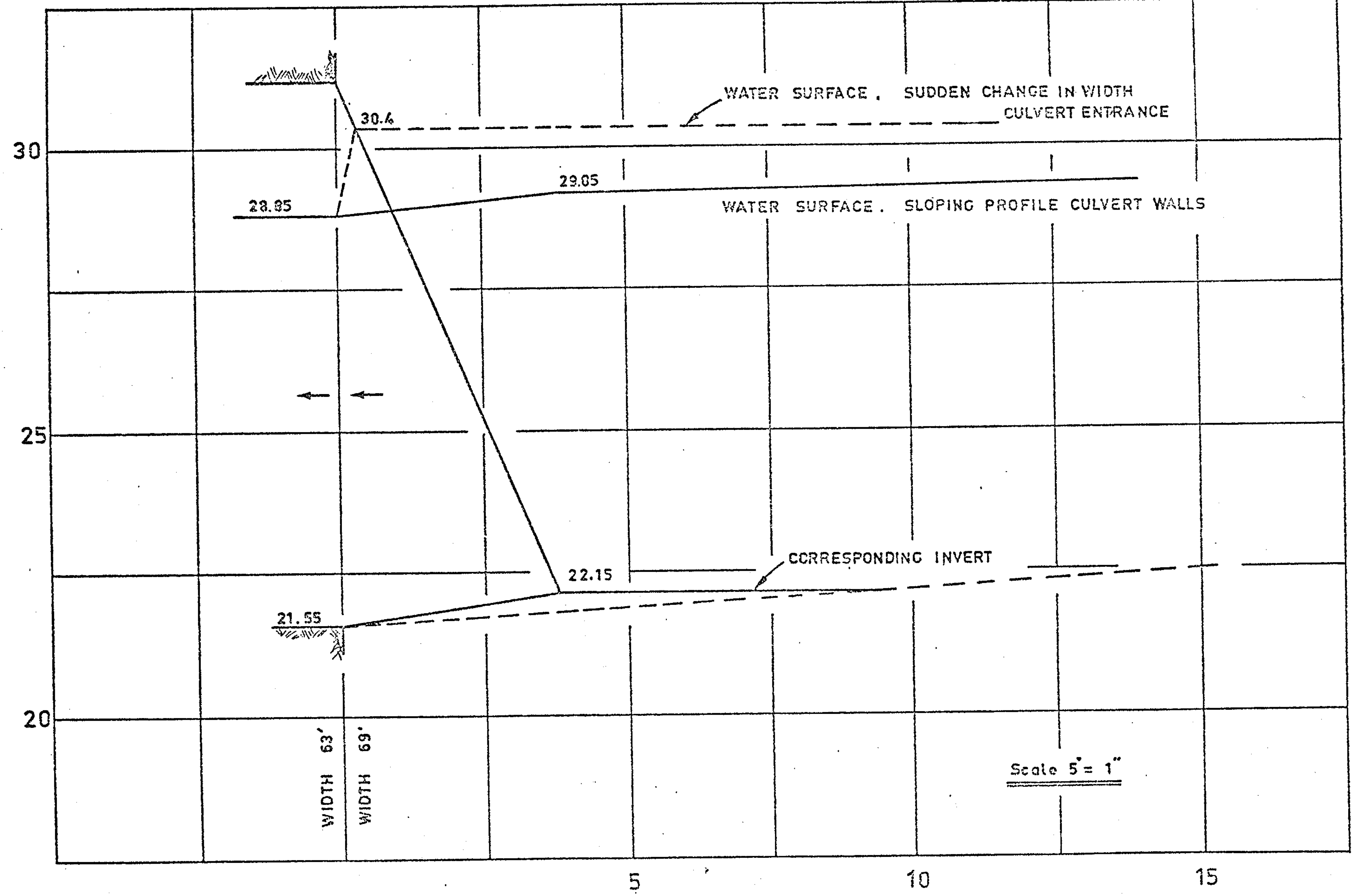
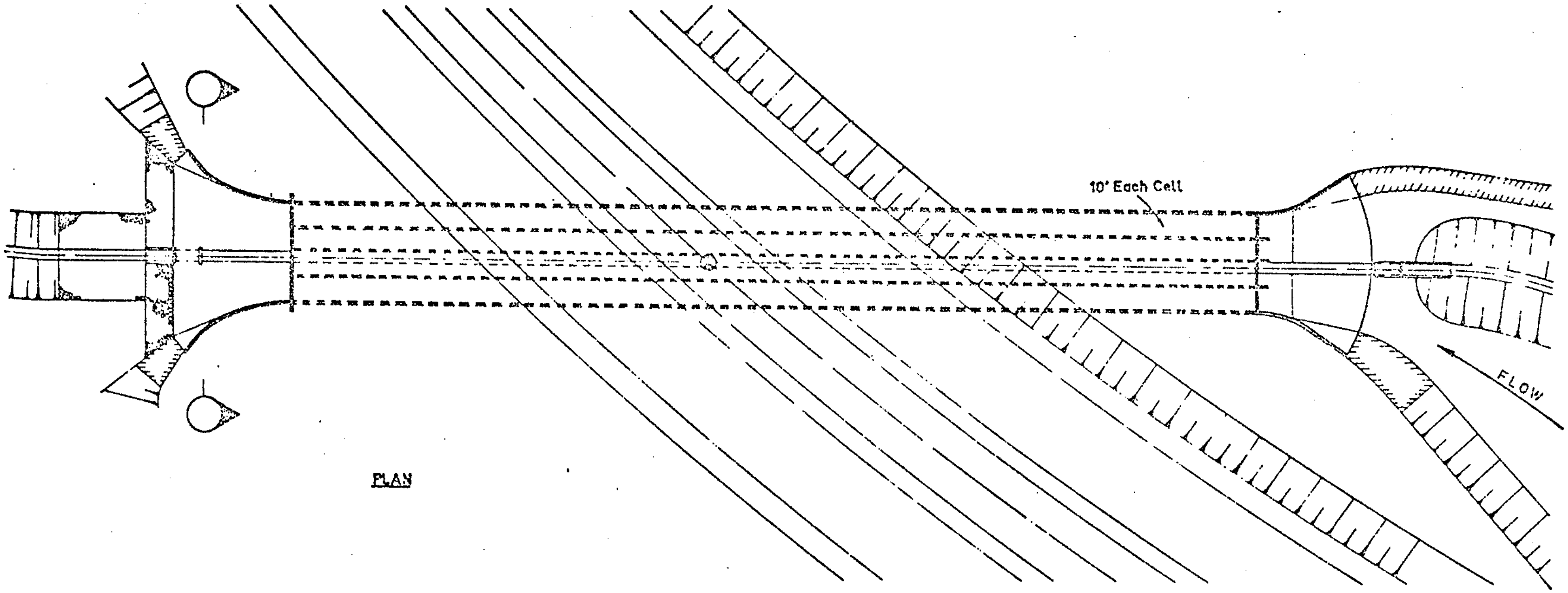
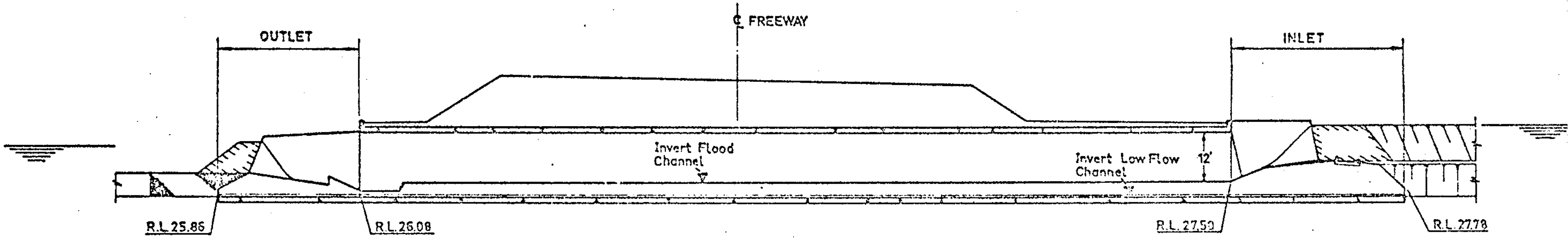


Fig. 5 CULVERT WALL NOSE - S.E. FREEWAY
(CALCULATED LEVELS)

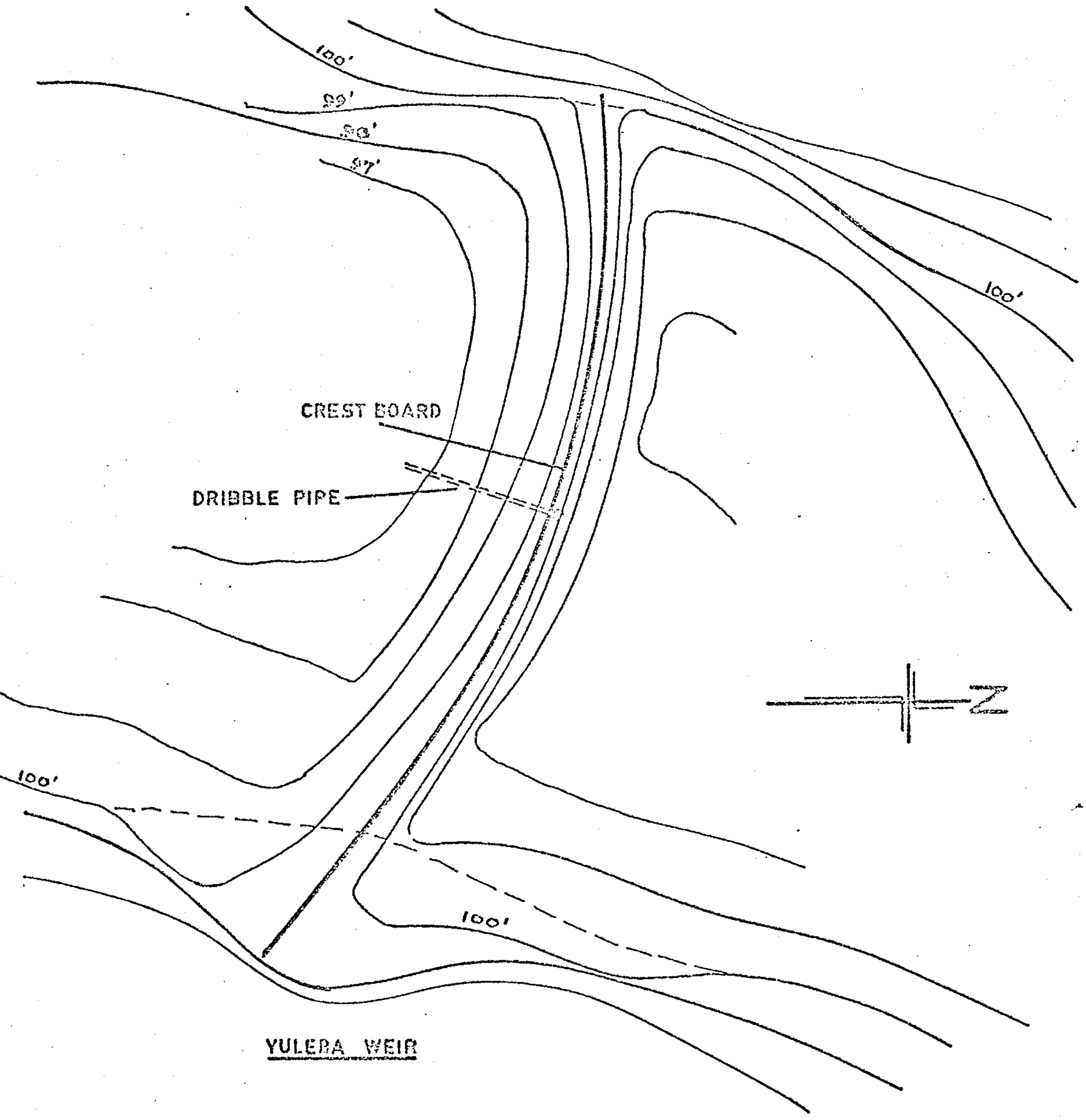


PLAN



LONGITUDINAL SECTION

Fig. 6


100'
99'
98'
97'
100'
CREST BOARD
DRIBBLE PIPE
100'
100'
N
YULERA WEIR

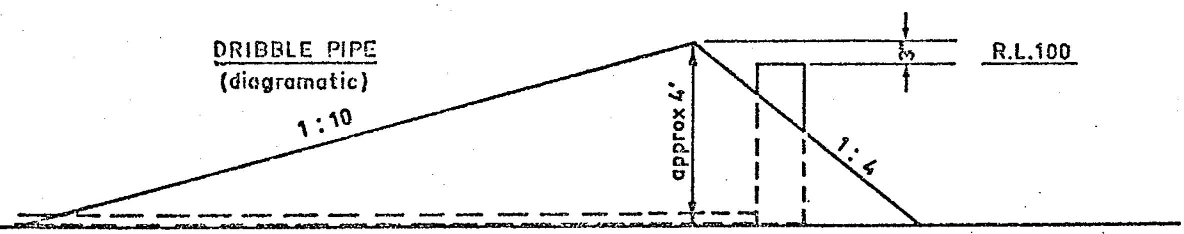
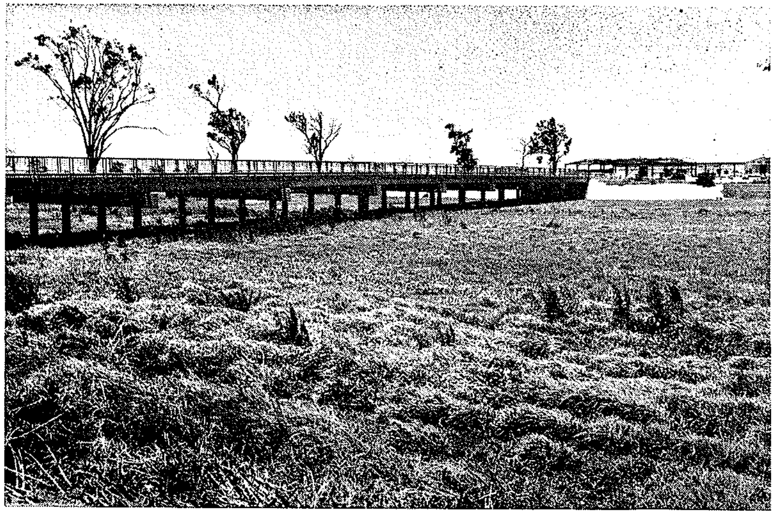

DRIBBLE PIPE
(diagramatic)
1 : 10
approx 4'
3"
R.L.100
1 : 4

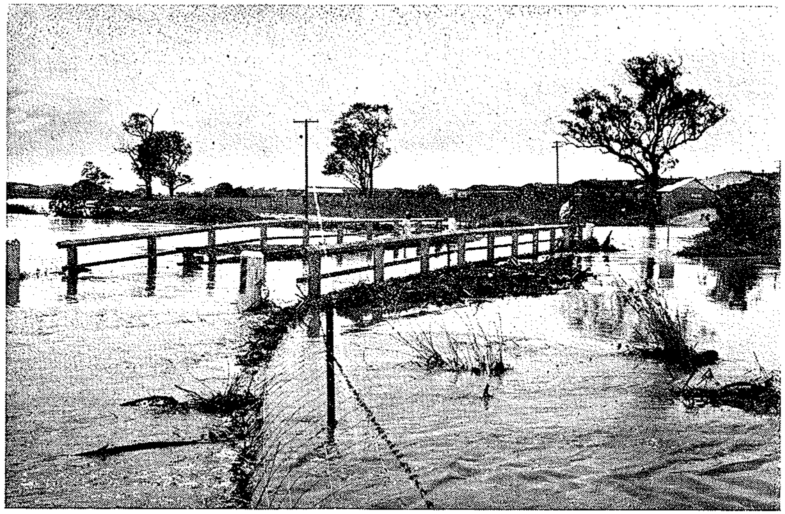
Fig. 7

Slide 1. Debris at Melton Road Bridge during a small flood October 25th, 1970. (Taken 28th October, 1970, by C.M.)

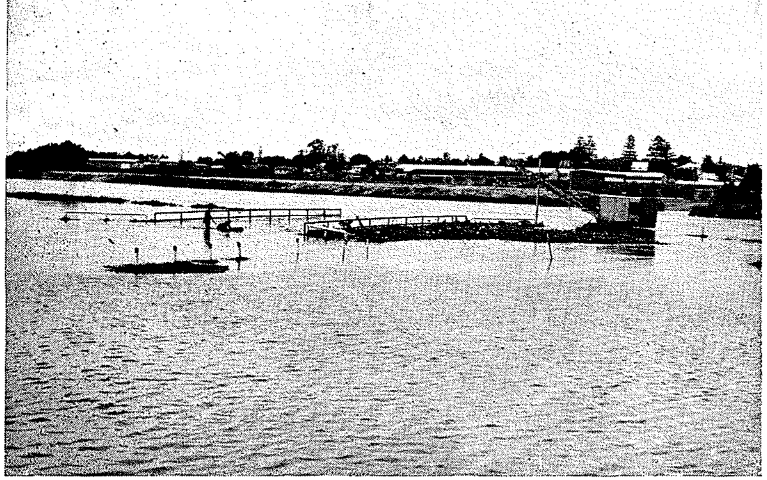
Slide 2. Debris at Hedley Avenue Crossing taken 29th October, 1970, after B.C.C. had cleared debris from roadway.



Slide 3. Nudgee Road Bridge completely free of any debris. Taken 29th October, 1970.

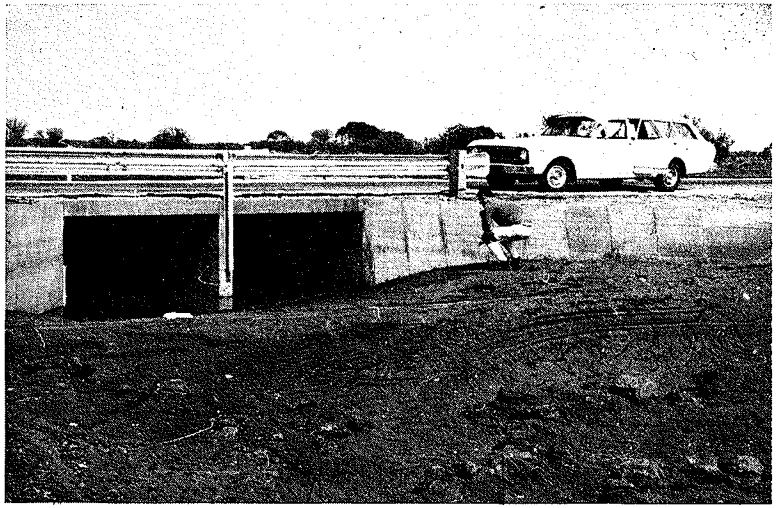


Slide 4. Melton Road Bridge 8th November, 1970. Small flood receding. Note debris packed against bridge.


Slide 5. Hedley Avenue Bridge 8th November, 1970.

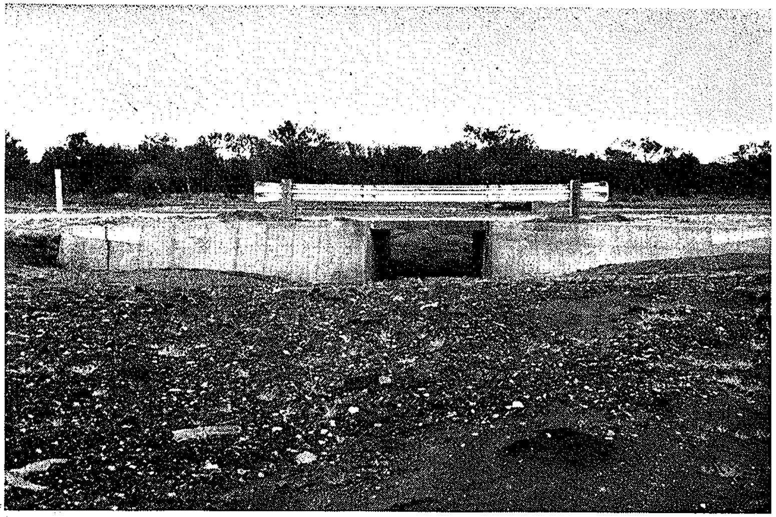

Slide 6. Nudgee Road Bridge 8th November, 1970. General view of bridge. No fixed debris.


Slide 7. Nudgee Road Bridge—end of wet season. Grass in good condition—no deposition of debris.


Slide 8. Downstream end of a twin 2/7' x 4' 6" culvert.

Slide 9. The upstream end of the same culvert.

Slide 10. The upstream end of a single cell culvert showing the radiused concrete lip on which the C.D.W. engineer is standing.

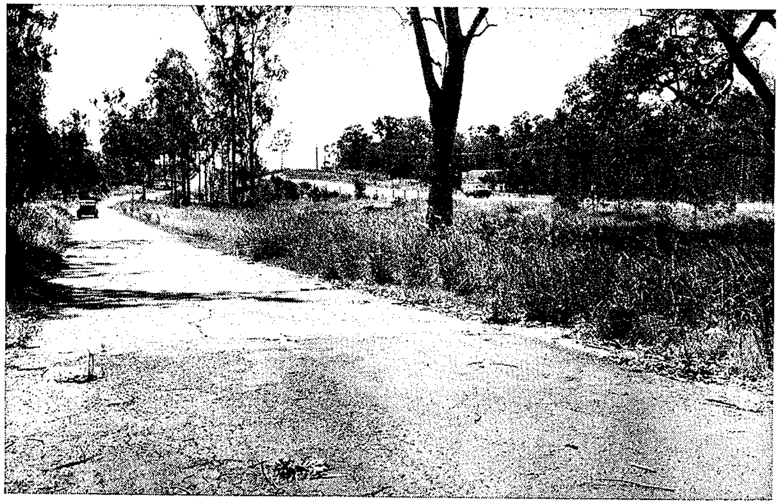


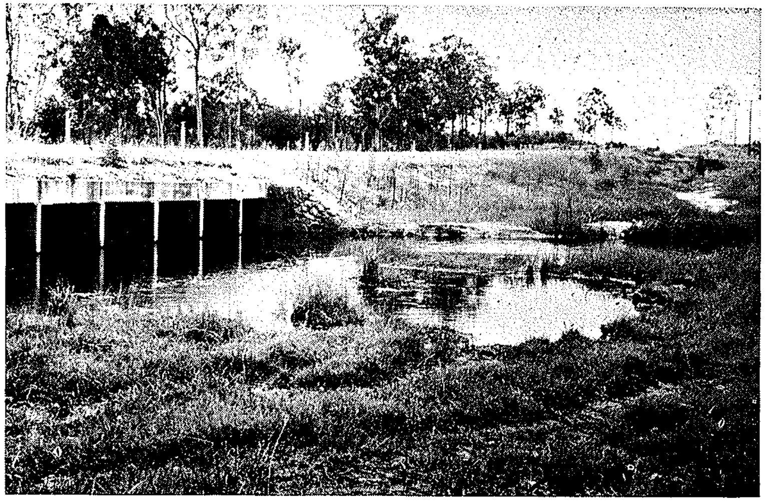
Slide 11. Looking downstream through a single cell culvert.

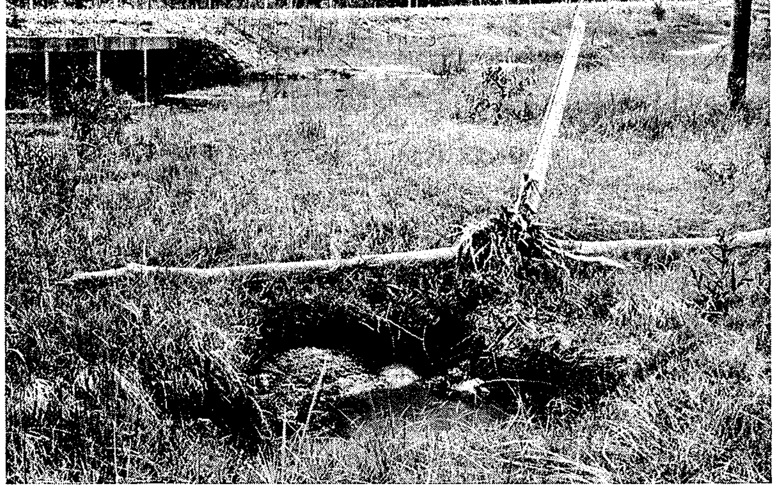
Slide 12. Looking upstream through the same culvert as in 11.


Slide 13. Site of "Jerry's Downfall" on the Brisbane-Beaudesert road about 15 miles from Brisbane—a floodway crossing 1000 ft. wide.

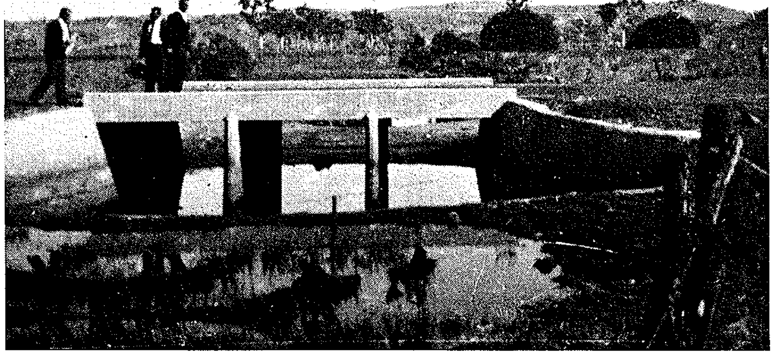

Slide 14. A standard M.R.D. (Qld.) 7' x 5' 8" cell culvert at "Jerry's Downfall" set as a minimum energy culvert (8th November, 1970).


Slide 15. The same culvert immediately after a major flow (in excess of design) 24th February, 1971.


Slide 16. Minor scour at the downstream fan of the culvert.

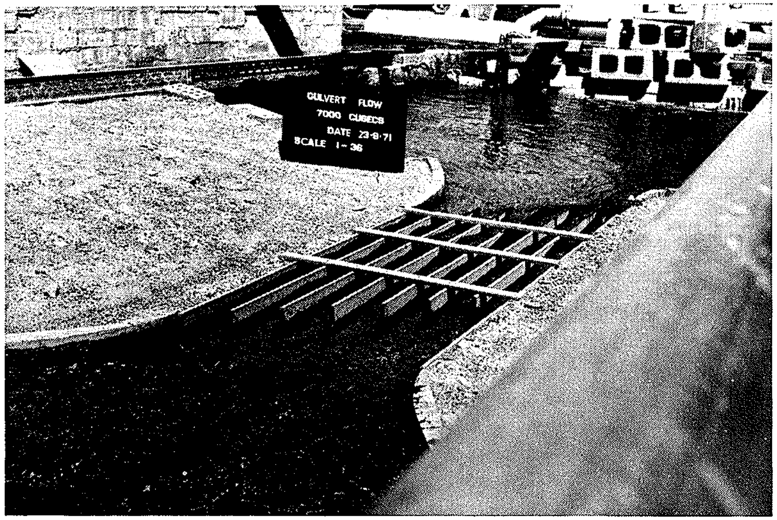


Slide 17. Scour round a telegraph pole near "Jerry's Downfall" culvert.

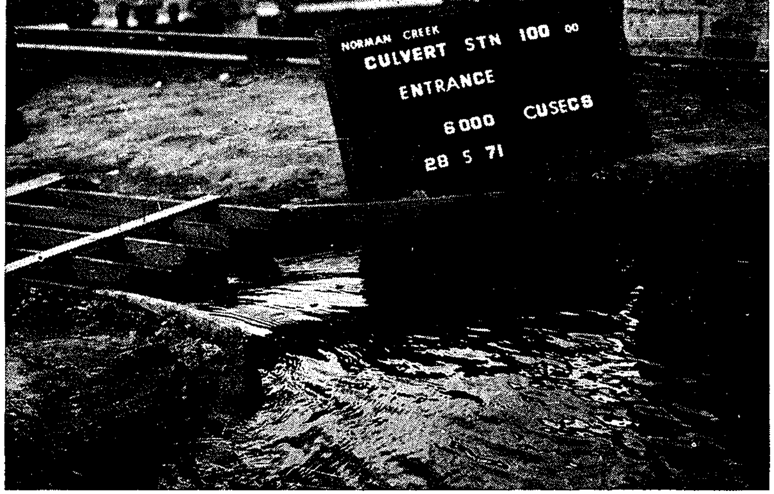


Slide 18. Culvert on Burnett Highway, Shire of Kilkivan looking downstream.

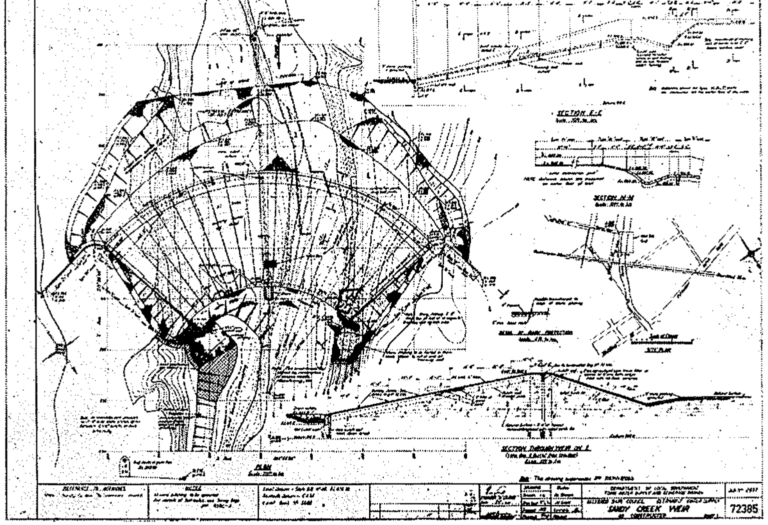
Slide 19. Culvert on Burnett Highway entrance fan.



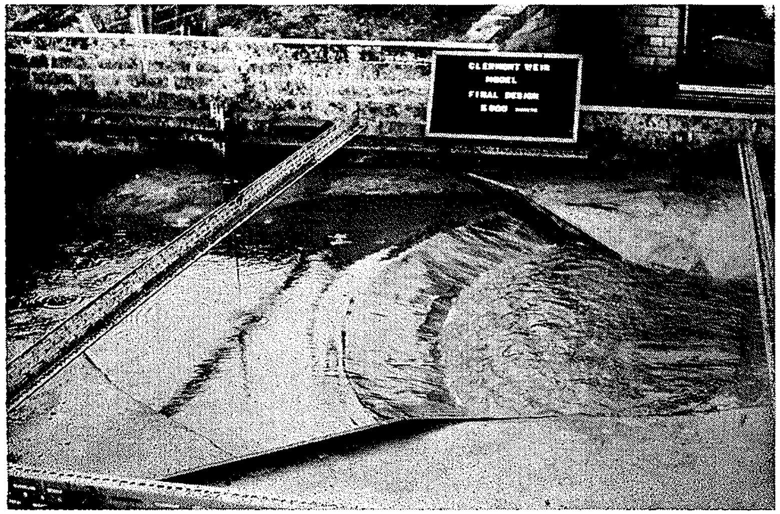
Slide 20. Model of culvert for South East Freeway (Brisbane) Ridge Street Deviation.



Slide 21. Model of culvert for South East Freeway (Brisbane) Station 100 (No noses).



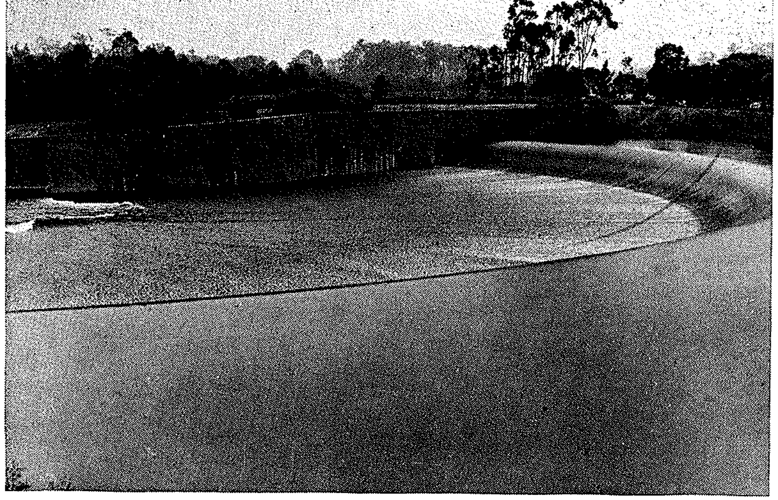
Slide 22. Plan of Clermont Weir.

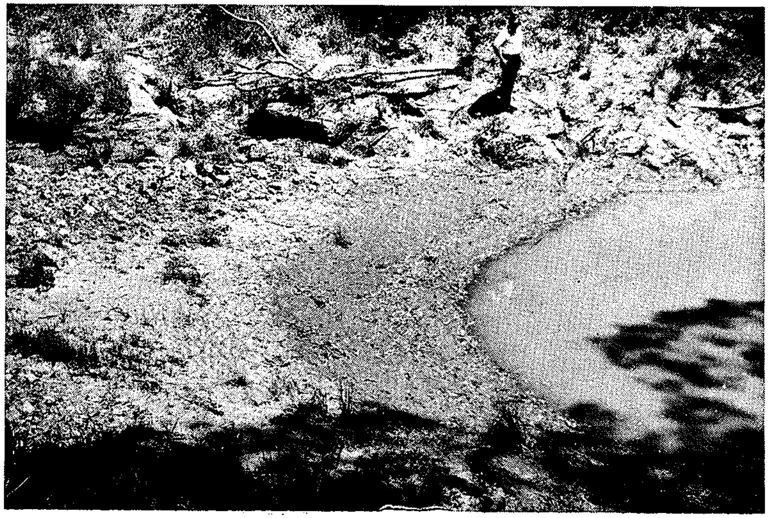


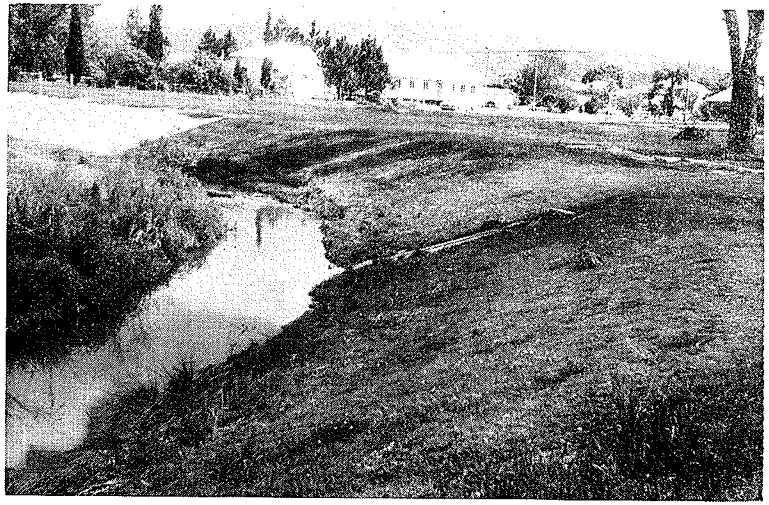
Slide 23. Model of Clermont Weir 5000 cusecs.

Slide 24. Flow over actual weir approximately 5000 cusecs.


Slide 25. Grass weir at Yuleba.


Slide 26. Spillway at Lake Kurongbah.


Slide 27. Natural shape.


Slide 28. Reinforced grass.