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HYDROLOGICAL SURVEY GARNANT SITE

SUPPLEMENTARY VOLUME

January 1985

SIR ALEXANDER GIBB & PARTNERS
CARDIFF OFFICE
124 CATHEDRAL ROAD
CARDIFF

This supplementary volume is in support of the Report prepared by Sir Alexander Gibb & Partners and entitled 'HYDROLOGICAL SURVEY - GARNANT SITE' dated January, 1985.

It contains calculations to support the statements and opinions made in the main report about the hydrology of the site and the hydraulics of the culvert, as well as some old record drawings of the Nant-y-Gath culvert.

This report comprises three parts as follows:

Part 1:	HYDROLOGY CALCULATIONS
Part 2:	RECORD DRAWINGS OF NANT-Y-GATH CULVERT
Part 3:	HYDRAULIC CALCULATIONS OF CULVERTS

HYDROLOGY CALCULATIONS

Nant-y-Gath catchment

AREA = 1·18 km^2
SAAR = 1775 mm
S1085 = 99 m/km
URBAN = 0·0
MSL = 1·70 km

MS-2day = 90·5 mm (5·1% of SAAR)
r = 0·21

Calculate design storm duration D

$$Tp^{(0)} = 283\,(S1085)^{-0.33}\,(1+URBAN)^{-2.16}\,SAAR^{-0.54}\,MSL^{0.23}$$

$$= 283 \times 0.2195 \times 1 \times 0.01760 \times 1.130$$
$$= 1.235 \text{ hr}$$

Choose data interval of about $Tp^{(0)}/5$, ie. $\Delta T = 0.25$ hr

Then $Tp^{(0.25)} = 1.235 + \dfrac{0.25}{2} = 1.36$

Design storm duration D:

$$D = \left(1 + \frac{SAAR}{1000}\right) Tp^{(0.25)} = (1 + 1.775)\,1.36$$
$$= 3.77$$

Rounded to nearest odd multiple of 0·25 we have:

D = 3·75 hr.

Calculate storm depths, P

MS-2day = 90·5 mm
r = 0·21

∴ MS-3·75hr / MS-2day = 36·8% (Interpolation in Table II.3.7)

Hence MS-3·75hr = ·368 × 90·5
= 33·3 mm.

Areal reduction factor for D = 3·75 hr and AREA = 1·18 km^2 is ARF = 0·975, (Extrapolation of Fig. II.5.1).

Storm return period	MT/MS	MT	P = MT. ARF
2	0.76	25.3	24.7
8	1.14	38.0	37.0
17	1.35	45.0	43.8
35	1.56	51.9	50.6
81	1.86	61.9	60.4
140	2.09	69.6	67.9
300	2.46	81.9	79.9
520	2.84	94.6	92.2

Calculate percentage runoffs, PR

$$PR = SPR + 0.25\,(CWI - 125) + 0.45\,(P-40)^{0.7}$$

For SAAR = 1775 mm, design CWI = 125.

From soil mapping, site inspection and literature survey assessed standard percentage runoff is:

$$SPR = 62.0$$

$$\therefore PR = \begin{cases} 62.0 & \text{if } P \leq 40 \text{ mm} \\ 62.0 + 0.45(P-40)^{0.7} & \text{if } P > 40 \text{ mm} \end{cases}$$

Flood return period	Storm return period	P	PR
2.33	2	24.7	62.0
5	8	37.0	62.0
10	17	43.8	63.1
20	35	50.6	64.3
50	81	60.4	65.7
100	140	67.9	66.6
250	300	79.9	67.9
500	520	92.2	69.2

Apply short-cut method to calculate flood peaks, Q

Peak response runoff, $q = RC \cdot \frac{PR}{100} \cdot \frac{P}{D} \cdot AREA$

From Flood Studies Supplementary Report No.9:

$D/Tp^{(0.25)} = 3.75/1.36 = 2.76 \Rightarrow RC = \cdot 407$

Hence $q = \cdot 407 \times \frac{1}{3.75} \times 1.18 \times \frac{PR}{100} \times P = 0.1281 \frac{PR}{100} \cdot P$

Baseflow allowance:

$ANSF = 0.00033(CWI - 125) + 0.000030\ SAAR$
$= 0.0533$ cumecs/km^2

$\therefore$ Baseflow $= ANSF \cdot AREA$
$= 0.063$ cumecs.

Flood return period	$\frac{PR}{100}$	P	q	baseflow	Q=q+baseflow
2.33	0.620	24.7	1.962	0.063	2.03
5	0.620	37.0	2.939	0.063	3.00
10	0.631	43.8	3.540	0.063	3.60
20	0.643	50.6	4.168	0.063	4.23
50	0.657	60.4	5.083	0.063	5.15
100	0.666	67.9	5.793	0.063	5.86
250	0.679	79.9	6.950	0.063	7.01
500	0.692	92.2	8.173	0.063	8.24

The flood peak estimates (Q) are shown in Fig.3 as a flood frequency curve, labelled 'Nant-y-gath".

Nant-Maen catchment

AREA = 0·44 km^2
SAAR = 1750 mm
S1085 = 111 m/km
URBAN = 0·0
MSL = 0·82 km

MS-2day = 89·3 mm (5·1% of SAAR)
r = 0·21

Calculate design storm duration, D

$$T_p^{(0)} = 283\,(S1085)^{-0.33}(1+URBAN)^{-2.16}\,SAAR^{-0.54}\,MSL^{0.23}$$

$$= 283 \times 0{\cdot}211 \times 1 \times 0{\cdot}0177 \times 0{\cdot}955$$
$$= 1{\cdot}01 \text{ hr}$$

Choose data interval of about $T_p^{(0)}/5$, ie. $\Delta T = 0{\cdot}2$ hr

Then $T_p^{(0.2)} = 1{\cdot}01 + \frac{0{\cdot}2}{2} = 1{\cdot}11$

Design storm duration D:

$$D = \left(1 + \frac{SAAR}{1000}\right) T_p^{(0.2)} = (1 + 1{\cdot}750)\,1{\cdot}11$$
$$= 3{\cdot}05$$

Rounded to nearest odd multiple of 0·2 we have

$$D = 3{\cdot}0 \text{ hr.}$$

Calculate storm depths, P

MS-2day = 89·3 mm
r = ·21

∴ MS-3hr / MS-2day = 33·4% (Interpolation in Table II.3.7)

Hence MS-3hr = ·334 × 89·3
= 29·8 mm.

Areal reduction factor for D = 3·0 hr and AREA = 0·44 km^2
is ARF = 0·98 (Extrapolation of Fig II.5.1)

Storm return period	MT/MS	MT	P=MT.ARF
2	0·75	22·4	21·9
8	1·14	34·0	33·3
17	1·36	40·5	39·7
35	1·58	47·1	46·1
81	1·88	56·0	54·9
140	2·12	63·2	61·9
300	2·51	74·8	73·3
520	2·90	86·4	84·7

Calculate percentage runoffs, PR

$$PR = SPR + 0{\cdot}25(CWI - 125) + 0{\cdot}45(P-40)^{0{\cdot}7}$$

For SAAR = 1750 mm, design CWI = 125·0 (Fig. I.6.62)

From soil mapping, site inspection, and literature survey

assessed standard percentage runoff is: SPR = 60·0

$$\therefore PR = \begin{cases} 60{\cdot}0 + 0{\cdot}45(P-40)^{0{\cdot}7} & P > 40 \text{ mm} \\ 60{\cdot}0 & \text{if } P \leq 40 \text{ mm.} \end{cases}$$

Flood return period	Storm return period	P	PR
2·33	2	21·9	60·0
5	8	33·3	60·0
10	17	39·7	60·0
20	35	46·1	61·6
50	81	54·9	63·0
100	140	61·9	63·9
250	300	73·3	65·2
500	520	84·7	66·4

Apply short-cut method to calculate flood peaks, Q

peak response runoff, $q = RC \cdot \frac{PR}{100} \cdot \frac{P}{D} \cdot AREA$

From Flood Studies Supplementary Report No. 9:

$D / T_p^{(0.2)} = 3.0/1.11 = 2.70 \Rightarrow RC = \cdot 402$

Hence $q = \cdot 402 \times \frac{1}{3.0} \times 0.44 \times \frac{PR}{100} \times P = 0.0590 \frac{PR}{100} \cdot P$

Baseflow allowance:

$ANSF = 0.00033(CWI-125) + 0.000030\ SAAR$
$= 0.0525$ cumecs/km²

∴ Baseflow allowance = ANSF · AREA
= 0.023 cumecs.

Flood return period	PR/100	P	q	baseflow	Q = q + baseflow
2.33	0.600	21.9	0.775	0.023	0.80
5	0.600	33.3	1.179	0.023	1.20
10	0.600	39.7	1.405	0.023	1.43
20	0.616	46.1	1.675	0.023	1.70
50	0.630	54.9	2.041	0.023	2.06
100	0.639	61.9	2.334	0.023	2.36
250	0.652	73.3	2.820	0.023	2.84
500	0.664	84.7	3.318	0.023	3.34

The flood peak estimates (Q) are shown in Fig. 3 as a flood frequency curve, labelled "Nant-Maen".

Minor catchment

AREA = 0.16 km^2
SAAR = 1775 mm
S1085 = 80 m/km
URBAN = 0.0
MSL = 0.32 km

M5-2day = 90.5 mm (5.1% of SAAR)
r = 0.21

Calculate design storm duration, D

$$Tp^{(0)} = 283\ (S1085)^{-0.33}(1+URBAN)^{-2.16}\ SAAR^{-0.54}\ MSL^{0.23}$$

$$= 283 \times 0.2355 \times 1 \times 0.01760 \times 0.769$$
$$= 0.90 \text{ hr}$$

Choose data interval of about $Tp^{(0)}/5$, ie. $\Delta T = 0.2$

Then $Tp^{(0.2)} = 0.90 + \frac{0.2}{2} = 1.00$ hr.

Design storm duration, D:

$$D = \left(1 + \frac{SAAR}{1000}\right) Tp^{(0.2)} = (1 + 1.775)\,1.00 = 2.775$$

Rounded to nearest odd multiple of 0.2 we have:

D = 2.6 hr.

Calculate storm depths, P

M5-2day = 90.5 mm
r = 0.21

∴ M5-2.6hr/M5-2day = 31.2% (Interpolation in Table II.3.7)

Hence M5-2.6hr = .312 × 90.5
= 28.2 mm.

Areal reduction factor for D = 2.6 hr and AREA = 0.16 km^2 is:

ARF = .985

MINOR

Storm return period	MT/M5	MT	P = MT.ARF
2	0·74	20·9	20·6
8	1·14	32·1	31·7
17	1·37	38·6	38·1
35	1·59	44·8	44·2
81	1·89	53·3	52·5
140	2·14	60·3	59·4
300	2·54	71·6	70·6
520	2·93	82·6	81·4

Calculate percentage runoffs, PR

$$PR = SPR + 0.25(CWI - 125) + 0.45(P-40)^{0.7}$$

For SAAR = 1775 mm, design CWI = 125.

From soil mapping, site inspection, and literature survey assessed standard percentage runoff is:

$$SPR = 60.0$$

$$PR = \begin{cases} 60.0 & \text{if } P \leq 40 \text{ mm} \\ 60.0 + 0.45(P-40)^{0.7} & \text{if } P > 40 \text{ mm} \end{cases}$$

Flood return period	Storm return period	P	PR
2·33	2	20·6	60·0
5	8	31·7	60·0
10	17	38·1	60·0
20	35	44·2	61·2
50	81	52·5	62·6
100	140	59·4	63·6
250	300	70·6	64·9
500	520	81·4	66·1

Apply short-cut method to calculate flood peaks, Q

Peak response runoff, $q = RC \cdot \frac{PR}{100} \cdot \frac{P}{D} \cdot AREA$

From FSSR No.9:

$D/T_p^{(0.2)} = 2.6/1.00 = 2.60 \Rightarrow RC = \cdot 390$

Hence $q = \cdot 390 \times \frac{1}{2.6} \times 0.16 \times \frac{PR}{100} \times P = 0.0240 \frac{PR}{100} \cdot P$

Baseflow allowance:

$ANSF = 0.00033(CWI - 125) + 0.000030\ SAAR$
$= 0.0533$ cumecs/km²

Baseflow = ANSF . AREA
= 0.009 cumecs

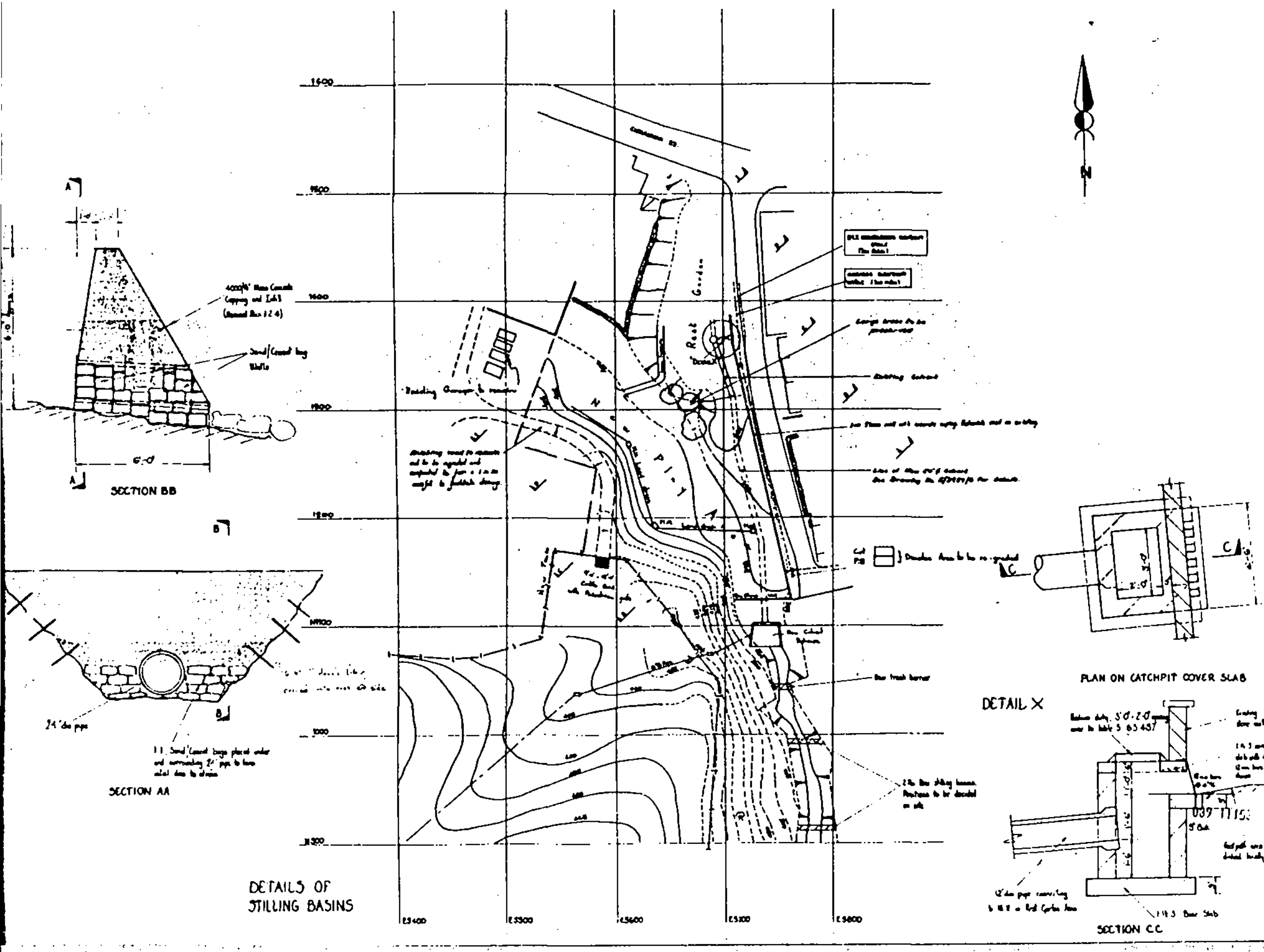
Flood return period	$\frac{PR}{100}$	P	q	baseflow	Q=q+baseflow
2.33	0.600	20.6	0.297	0.009	0.31
5	0.600	31.7	0.456	0.009	0.47
10	0.600	38.1	0.549	0.009	0.56
20	0.612	44.2	0.649	0.009	0.66
50	0.626	52.5	0.789	0.009	0.80
100	0.636	59.4	0.907	0.009	0.92
250	0.649	70.6	1.100	0.009	1.11
500	0.661	81.4	1.291	0.009	1.30

The flood peak estimates (Q) are shown in Fig.3 as a flood frequency curve, labelled "minor catchment".

Calculations: DWR Nov. 1984

Checked: MR Dec 1984

RECORD DRAWINGS OF NANT-Y-GATH CULVERT



Proposed New Contours.
Existing contours not to be regraded.

1) This drawing to be read in conjunction with Engineers drg E/5900-1.

2) For details of original ground level see drg E/5900-2.

Proposed Boundary Line

E				
D			P	
C				
B		Play Area Changed		
A				

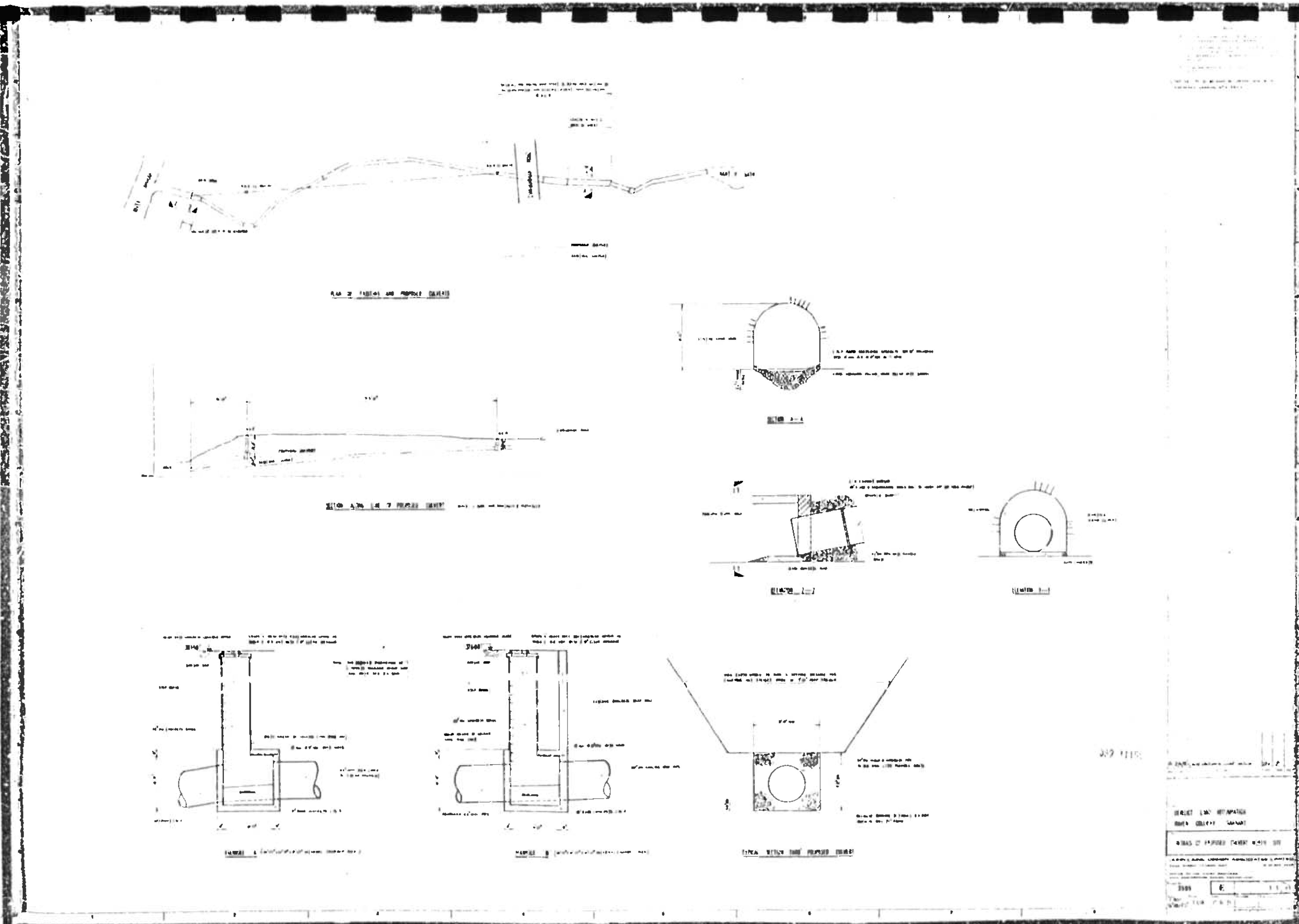
DERELICT LAND RECLAMATION
RAVEN COLLIERY, GARNANT

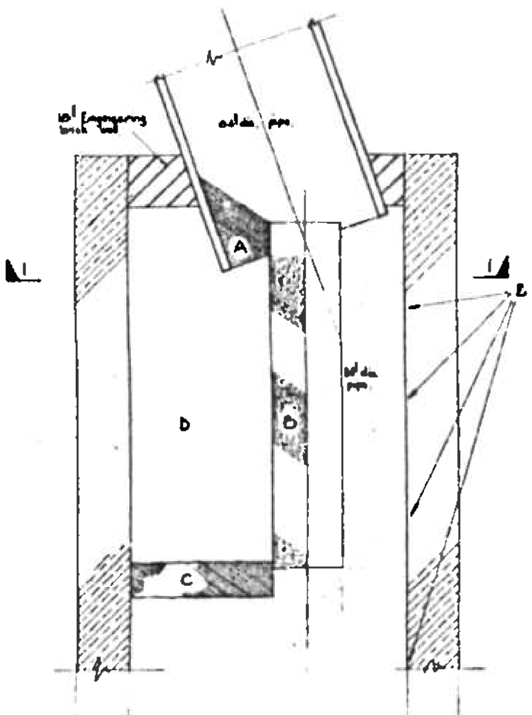
SITE LAYOUT SHOWING PROPOSED NEW CONTOURS, N END SOUTH SITE

OFFICE OF THE CHIEF ENGINEER
CIVIL ENGINEERING DESIGN DEPT

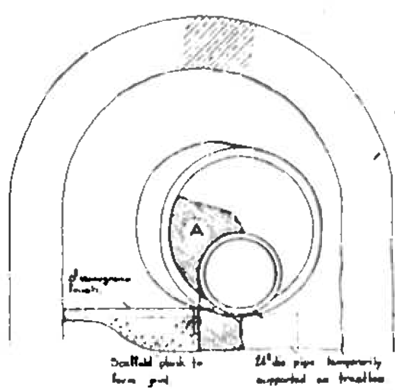
LAING	E/ 5985	/ 5 E

1:500	GW		





Stage 1.



1——1

Stage 1.

A Form Cement/Sand bag wall to direct flow into 24" dia. pipe.

B Form Cement/Sand bag wall under 24" dia. pipe.

C. Form Cement/Sand bag wall to isolate area D from water.

D Invert to be brought up to correct level with 1:1½:3 Cement/Sand/Limestone/Aggregate mix. Finished with 2" layer of Monogrance which must be applied on wet concrete.

E. Next course to bags to be removed into voids under existing culvert wall, and cavity behind wall to be pressure grouted to fill all voids.

Stage 2.

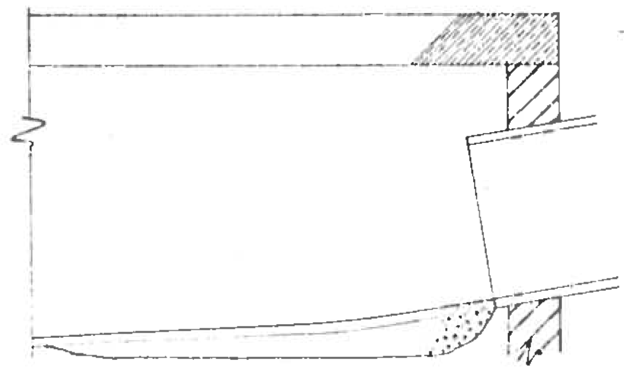
F. 24" dia. pipe moved to divert flow.

G. Cement/Sand bags to direct flow

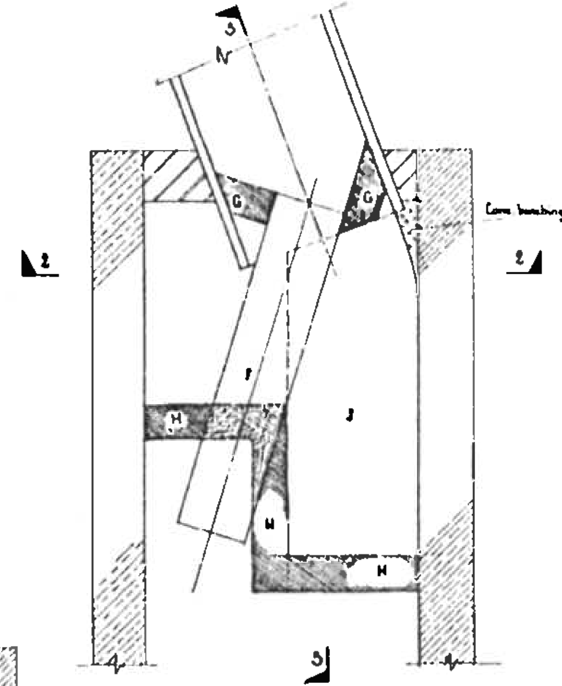
H Cement/Sand bags to isolate area J from water.

J. Refer to note D.

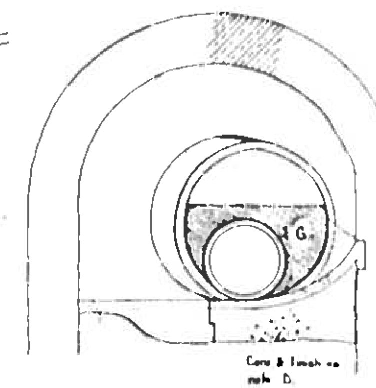
Remove 24" dia. pipe trestles & sandbags.



3——3



Stage 2.



2——2

039 11155

DERELICT LAND RECLAMATION
RAVEN COLLIERY GARNANT

PROCEDURE FOR MAKING GOOD
TO NORTH END OF MAIN CULVERT

5085 E

HYDRAULIC CALCULATIONS OF CULVERTS

SIR ALEXANDER GIBB & PARTNERS
CONSULTING ENGINEERS

Made by: MCC	Date: Jan 1985	Project: N.C.B. GARNANT SITE	Sheet No. (i)
Checked by:	Date:	Contract: HYDROLOGICAL SURVEY	Calc. No.
Revised by:	Date:	Section: INDEX TO CALCULATIONS	File
Checked by:	Date:	Subject:	Job No. 84710A

SIR ALEXANDER GIBB & PARTNERS
CONSULTING ENGINEERS

Made by A.D.J.	Date 17/12/84	Project N.C.B Grannat		Sheet No. 1	
Checked by Mnc	Date 20/12/84	Contract Hydrological Survey		Calc. No.	
Revised by	Date	Section Culvert Analysis		File	
Checked by	Date	Subject		Job No. 84710A	

The First Calculation for each culvert, examines the entrance of the culvert and assesses whether or not the entrance is adequate to allow the maximum flows into the culvert.

The second calculation performed for each culvert evaluates the maximum capacity of each section of culvert assuming that the flow is of normal depth. For a circular section this means that the flow is calculated for depth/diameter = 0.94 (see page G1) and that the hydraulic gradient 'i' is equal to the pipe slope s.

The third calculation is only performed if any of the maximum capacity flows calculated in stage 2 are below the peak flow which the culvert is likely to take. If this is the case, the hydraulic heads – which are required to drive the peak flow through the culvert – are calculated. This is done by working back from the 'throttle point' which is furthest down stream. The culvert is then examined to check if it can safely maintain these pressure heads without over-flowing.

The flow calculations are performed using Mannings Equation i.e.

$$Q = i^{1/2} m^{2/3} / n$$

Q = Flow m^3/s
M = hydraulic mean depth m
i = hydraulic gradient
n = Mannings friction coefficient

SIR ALEXANDER GIBB & PARTNERS
CONSULTING ENGINEERS

Made by A.D.[illegible]	Date 18/12/[illegible]	Project N C B GORUNNT	Sheet No. 2
Checked by MWC	Date 20/a/[illegible]	Contract HYDROLOGICAL SURVEY	Calc. No.
Revised by	Date	Section Culvert Analysis	File
Checked by	Date	Subject	Job No. SL7[illegible]A

The Values of 'n' used were obtained from reference 4 with use also being made of Table 5-6 in reference 2. (Note, References are at the back of the calculations).

To evaluate the head losses due to enlargements, contractions, bends, manholes, gratings, entries and exits, various coefficients and formulae were obtained from References 1-3. A few of the relevant pages from these publications have been included in these calculations.

The peak flows for various storm return periods have been assessed by the Institute of Hydrology for each of the culverts.

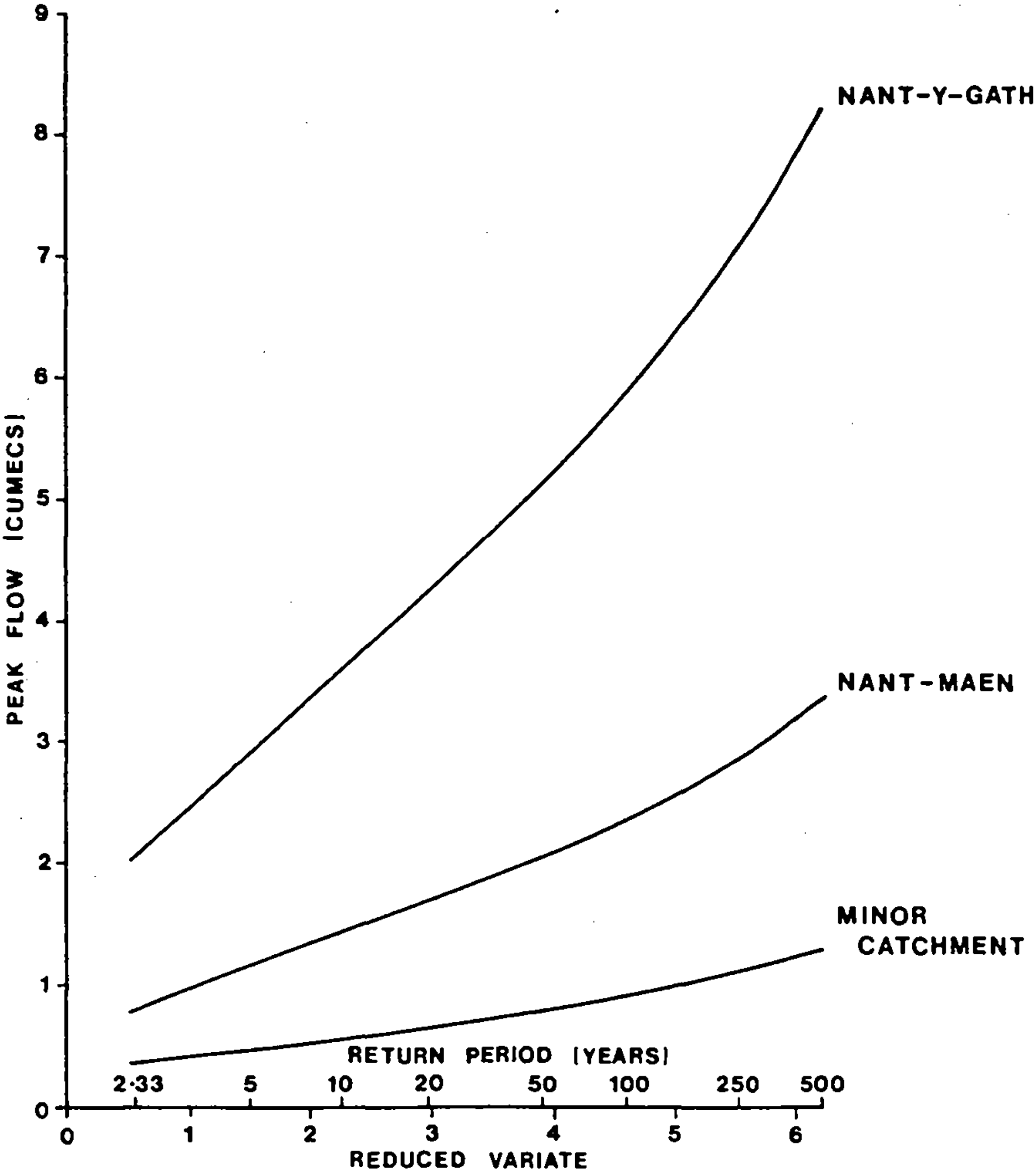


Fig. 3 Flood frequency curves - present condition

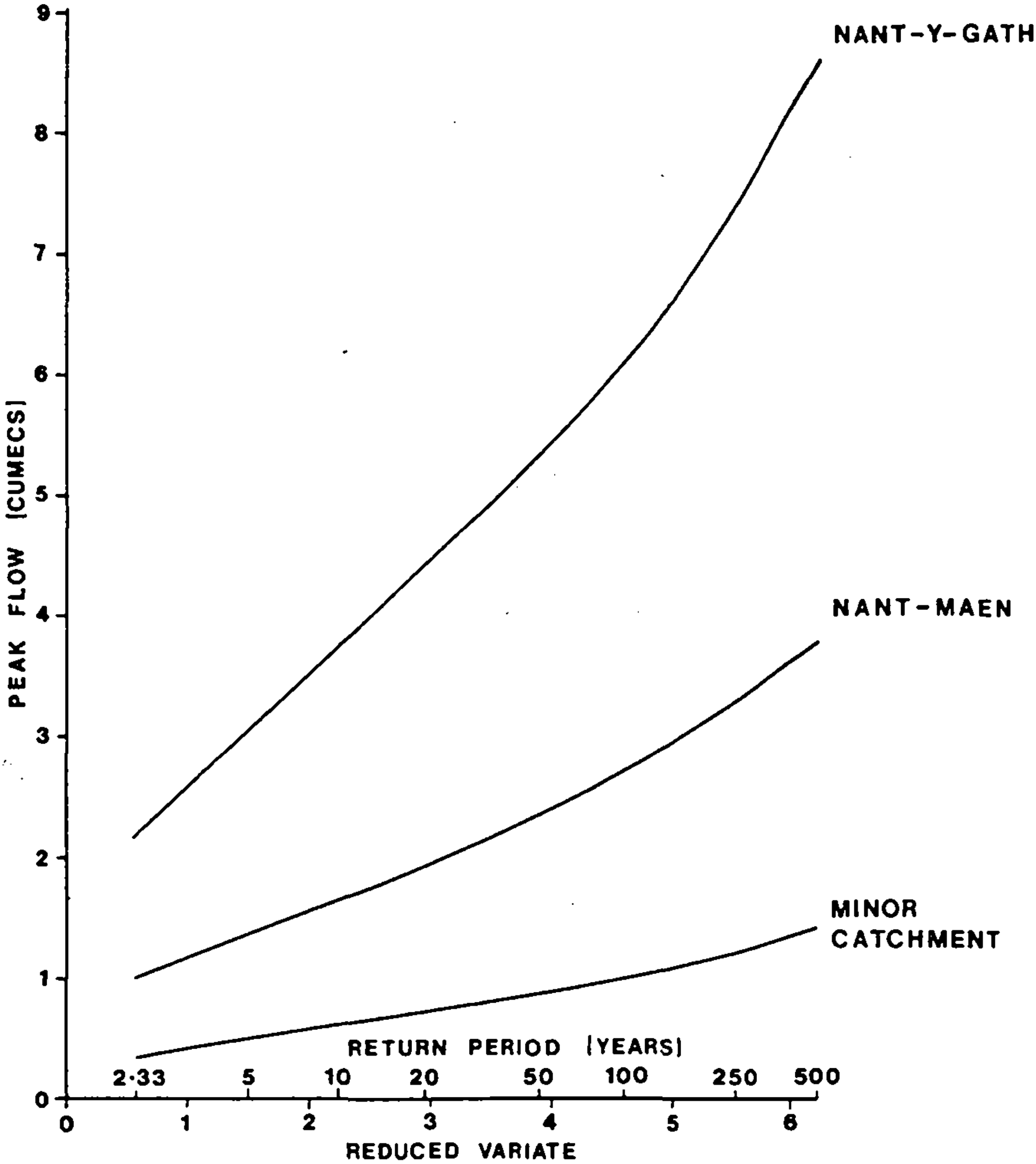


Fig. 4 Flood frequency curves - on restoration.

A theoretical determination of the additional losses is seldom possible, and so experimentally-determined figures must be called on. Since the losses have been found to vary as the square of the mean velocity, they are frequently expressed in the form

$$\text{Head loss} = k\frac{u^2}{2g}$$

and for high Reynolds numbers the value of k is practically constant.

7.5.1 Loss at Abrupt Enlargement

One 'minor loss' which can be subjected to analysis is that at an abrupt enlargement of the cross-section, such as that illustrated in Fig. 7.4. The pipes run full and the flow is assumed steady. Fluid emerging from the smaller pipe is unable to follow the abrupt deviation of the boundary; consequently pockets of turbulent eddies form in the corners and result in the dissipation of energy as heat. By means of a few simplifying assumptions, an estimate of the head lost may be made.

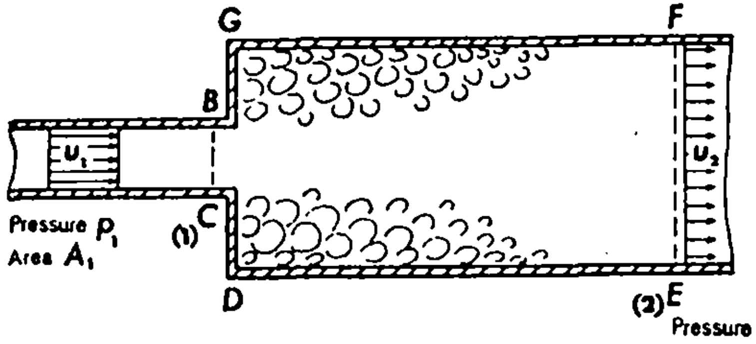


FIG. 7.4

For the high values of Reynolds number usually found in practice, the velocity in the smaller pipe may be assumed sensibly uniform over the cross-section. At section (1) the streamlines are straight and parallel and so the piezometric pressure here is uniform. Downstream of the enlargement the vigorous mixing caused by the turbulence helps to even out the velocity and it is assumed that at a section (2) sufficiently far† from the enlargement the velocity (like the piezometric pressure) is again uniform over the cross-section. For simplicity the axes of the pipes are assumed horizontal. Continuity requires the velocity u_2 to be less than u_1 and the corresponding momentum change requires a net force to act on the fluid between sections (1) and (2). On the fluid in the 'control volume' *BCDEFG* the net force acting towards the right is

$$p_1A_1 + p'(A_2 - A_1) - p_2A_2$$

where p' represents the mean pressure of the eddying fluid over the annular face *GD*. On the grounds that radial accelerations over the annular face *GD* are very

† About 8 times the larger diameter.

small, we assume (with the support of experimental evidence) that p' is sensibly equal to p_1. The net force is thus $(p_1 - p_2)A_2$. Shear forces on the boundaries over the short length between sections (1) and (2) are neglected. From the momentum equation (see Section 4.2) the net force on the fluid equals the rate of increase of momentum in that direction:

$$(p_1 - p_2)A_2 = \rho Q(u_2 - u_1)$$

where ρ represents the density and Q the volume flow rate.

$$\therefore\ p_1 - p_2 = \rho\frac{Q}{A_2}(u_2 - u_1) = \rho u_2(u_2 - u_1) \tag{7.7}$$

From the energy equation for a constant density fluid we have

$$\frac{p_1}{\rho g} + \frac{u_1^2}{2g} + z - h_l = \frac{p_2}{\rho g} + \frac{u_2^2}{2g} + z$$

where h_l represents the loss of total head between sections (1) and (2).

$$\therefore\ h_l = \frac{p_1 - p_2}{\rho g} + \frac{u_1^2 - u_2^2}{2g}$$

and substitution from eqn 7.7 gives

$$h_l = \frac{u_2(u_2 - u_1)}{g} + \frac{u_1^2 - u_2^2}{2g} = \frac{(u_1 - u_2)^2}{2g} \tag{7.8}$$

Since by continuity $A_1u_1 = A_2u_2$ eqn 7.8 may be alternatively written

$$h_l = \frac{u_1^2}{2g}\left(1 - \frac{A_1}{A_2}\right)^2 = \frac{u_2^2}{2g}\left(\frac{A_2}{A_1} - 1\right)^2 \tag{7.9}$$

This result was first obtained by J.-C. Borda (1733–99) and L. Carnot (1753–1823) and is sometimes known as the Borda-Carnot head loss. In view of the assumptions made, eqns 7.8 and 7.9 are subject to some inaccuracy, but experiments show that for coaxial pipes they are within only a few per cent of the truth.

7.5.1.1 Exit loss

If $A_2 \to \infty$, then eqn 7.9 shows that the head loss at an abrupt enlargement tends to $u_1^2/2g$. This happens at the submerged outlet of a pipe discharging into a

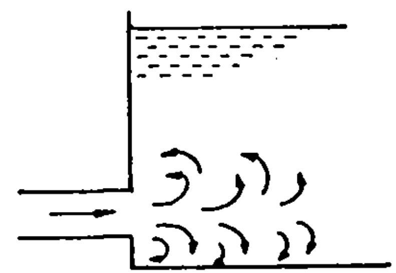


FIG. 7.5

From Ref. 1

5

large reservoir, for example (Fig. 7.5). The velocity head in the pipe, corresponding to the kinetic energy of the fluid per unit weight, is thus lost in turbulence in the reservoir. In such circumstances the loss is usually termed the exit loss for the pipe.

7.5.2 Loss at Abrupt Contraction

Although an abrupt contraction (Fig. 7.6) is geometrically the reverse of an abrupt enlargement it is not possible to apply the momentum equation to a control volume between sections (1) and (2). This is because, just upstream of the junction, the curvature of the streamlines and the acceleration of the fluid cause the pressure at the annular face to vary in an unknown way. However, immediately downstream of the junction a vena contracta is formed, after which the stream widens again to fill the pipe. Eddies are formed between the vena contracta and the wall of the pipe, and it is these which cause practically all the dissipation of energy. Between the vena contracta and the downstream section (2)—where the velocity has again become sensibly uniform—the flow pattern is similar to that

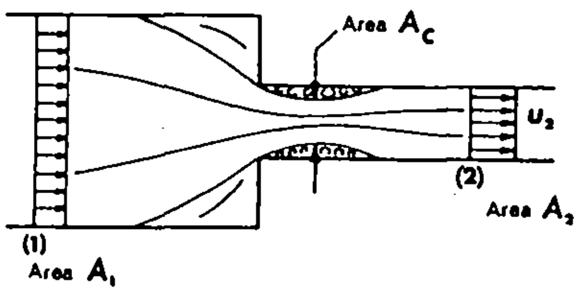


Fig. 7.6

after an abrupt enlargement, and so the loss of head is assumed to be given by eqn 7.9:

$$h_l = \frac{u_2^2}{2g}\left(\frac{A_2}{A_c} - 1\right)^2 = \frac{u_2^2}{2g}\left(\frac{1}{C_c} - 1\right)^2 \qquad (7.10)$$

where A_c represents the cross-sectional area of the vena contracta, and the coefficient of contraction $C_c = A_c/A_2$.

Although the area A_1 is not explicitly involved in eqn 7.10, the value of C_c depends on the ratio A_2/A_1. For coaxial circular pipes and fairly high Reynolds numbers Table 7.1 gives representative values of the coefficient k in

$$h_l = ku_2^2/2g \qquad (7.11)$$

Table 7.1

d_2/d_1	0	0·2	0·4	0·6	0·8	1·0
k	0·5	0·45	0·38	0·28	0·14	0

7.5.2.1 Entry Loss

As $A_1 \to \infty$ the value of k in eqn 7.11 tends to 0·5, and this limiting case corresponds to the flow from a large reservoir into a sharp-edged pipe, provided that the end of the pipe does not protrude into the reservoir. (A protruding pipe, as at Fig. 7.7(*b*) causes a greater loss of head.) For a non-protruding, sharp-edged pipe the loss $0{\cdot}5u_2^2/2g$ is known as the entry loss. If the inlet to the pipe is well rounded, as in Fig. 7.7(*c*) the fluid can follow the boundary without separating from it, and the entry loss is much reduced. A tapered entry also gives a much lower loss than the abrupt entry.

7.5.3 Diffusers

The head lost at an abrupt enlargement (or at the exit from a pipe) can be considerably reduced by the substitution of a gradual, tapered, enlargement usually known as a *diffuser* or *recuperator*. Its function is to reduce the velocity of the fluid gradually, and thus eliminate, as far as possible, the eddies responsible for the dissipation of energy.

The loss of head which does occur in a diffuser depends on the angle of divergence θ and on the ratio of the upstream and downstream areas. One contribution to the loss is made by ordinary pipe friction. This decreases as the angle θ increases since, for a given ratio of areas A_2/A_1, a larger angle gives a smaller length. For all but the smallest angles, however, energy is also dissipated by eddies caused by the separation of the flow from the walls. The loss increases with θ. There is thus an optimum angle for which the sum of the two types of loss is a

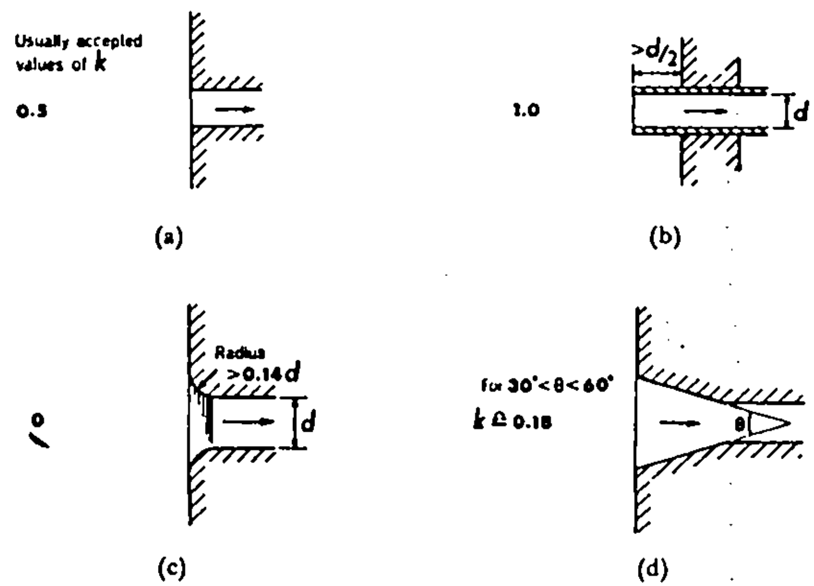


Fig. 7.7

From. Ref. 1

6

7

From. Ref. 2.

TABLE 5-6. VALUES OF THE ROUGHNESS COEFFICIENT n
(Boldface figures are values generally recommended in design)

Type of channel and description	Minimum	Normal	Maximum
A. CLOSED CONDUITS FLOWING PARTLY FULL			
A-1. Metal			
a. Brass, smooth	0.009	0.010	0.013
b. Steel			
1. Lockbar and welded	0.010	0.012	0.014
2. Riveted and spiral	0.013	0.016	0.017
c. Cast iron			
1. Coated	0.010	0.013	0.014
2. Uncoated	0.011	0.014	0.016
d. Wrought iron			
1. Black	0.012	0.014	0.015
2. Galvanized	0.013	0.016	0.017
e. Corrugated metal			
1. Subdrain	0.017	0.019	0.021
2. Storm drain	0.021	0.024	0.030
A-2. Nonmetal			
a. Lucite	0.008	0.009	0.010
b. Glass	0.009	0.010	0.013
c. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
d. Concrete			
1. Culvert, straight and free of debris	0.010	0.011	0.013
2. Culvert with bends, connections, and some debris	0.011	0.013	0.014
3. Finished	0.011	0.012	0.014
4. Sewer with manholes, inlet, etc., straight	0.013	0.015	0.017
5. Unfinished, steel form	0.012	0.013	0.014
6. Unfinished, smooth wood form	0.012	0.014	0.016
7. Unfinished, rough wood form	0.015	0.017	0.020
e. Wood			
1. Stave	0.010	0.012	0.014
2. Laminated, treated	0.015	0.017	0.020
f. Clay			
1. Common drainage tile	0.011	0.013	0.017
2. Vitrified sewer	0.011	0.014	0.017
3. Vitrified sewer with manholes, inlet, etc.	0.013	0.015	0.017
4. Vitrified subdrain with open joint	0.014	0.016	0.018
g. Brickwork			
1. Glazed	0.011	0.013	0.015
2. Lined with cement mortar	0.012	0.015	0.017
h. Sanitary sewers coated with sewage slimes, with bends and connections	0.012	0.013	0.016
i. Paved invert, sewer, smooth bottom	0.016	0.019	0.020
j. Rubble masonry, cemented	0.018	0.025	0.030

TABLE 5-6. VALUES OF THE ROUGHNESS COEFFICIENT n (continued)

Type of channel and description	Minimum	Normal	Maximum
B. LINED OR BUILT-UP CHANNELS			
B-1. Metal			
a. Smooth steel surface			
1. Unpainted	0.011	0.012	0.014
2. Painted	0.012	0.013	0.017
b. Corrugated	0.021	0.025	0.030
B-2. Nonmetal			
a. Cement			
1. Neat, surface	0.010	0.011	0.013
2. Mortar	0.011	0.013	0.015
b. Wood			
1. Planed, untreated	0.010	0.012	0.014
2. Planed, creosoted	0.011	0.012	0.015
3. Unplaned	0.011	0.013	0.015
4. Plank with battens	0.012	0.015	0.018
5. Lined with roofing paper	0.010	0.014	0.017
c. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished	0.014	0.017	0.020
5. Gunite, good section	0.016	0.019	0.023
6. Gunite, wavy section	0.018	0.022	0.025
7. On good excavated rock	0.017	0.020	
8. On irregular excavated rock	0.022	0.027	
d. Concrete bottom float finished with sides of			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Cement rubble masonry, plastered	0.016	0.020	0.024
4. Cement rubble masonry	0.020	0.025	0.030
5. Dry rubble or riprap	0.020	0.030	0.035
e. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
f. Brick			
1. Glazed	0.011	0.013	0.015
2. In cement mortar	0.012	0.015	0.018
g. Masonry			
1. Cemented rubble	0.017	0.025	0.030
2. Dry rubble	0.023	0.032	0.035
h. Dressed ashlar	0.013	0.015	0.017
i. Asphalt			
1. Smooth	0.013	0.013	
2. Rough	0.016	0.016	
j. Vegetal lining	0.030	.....	0.500

From Ref. 3

9.2.1. BASIC COEFFICIENTS K^*_b (CLASS 1)

Basic loss coefficients at a Reynolds number of 10^6 are given in Fig. 9.2.

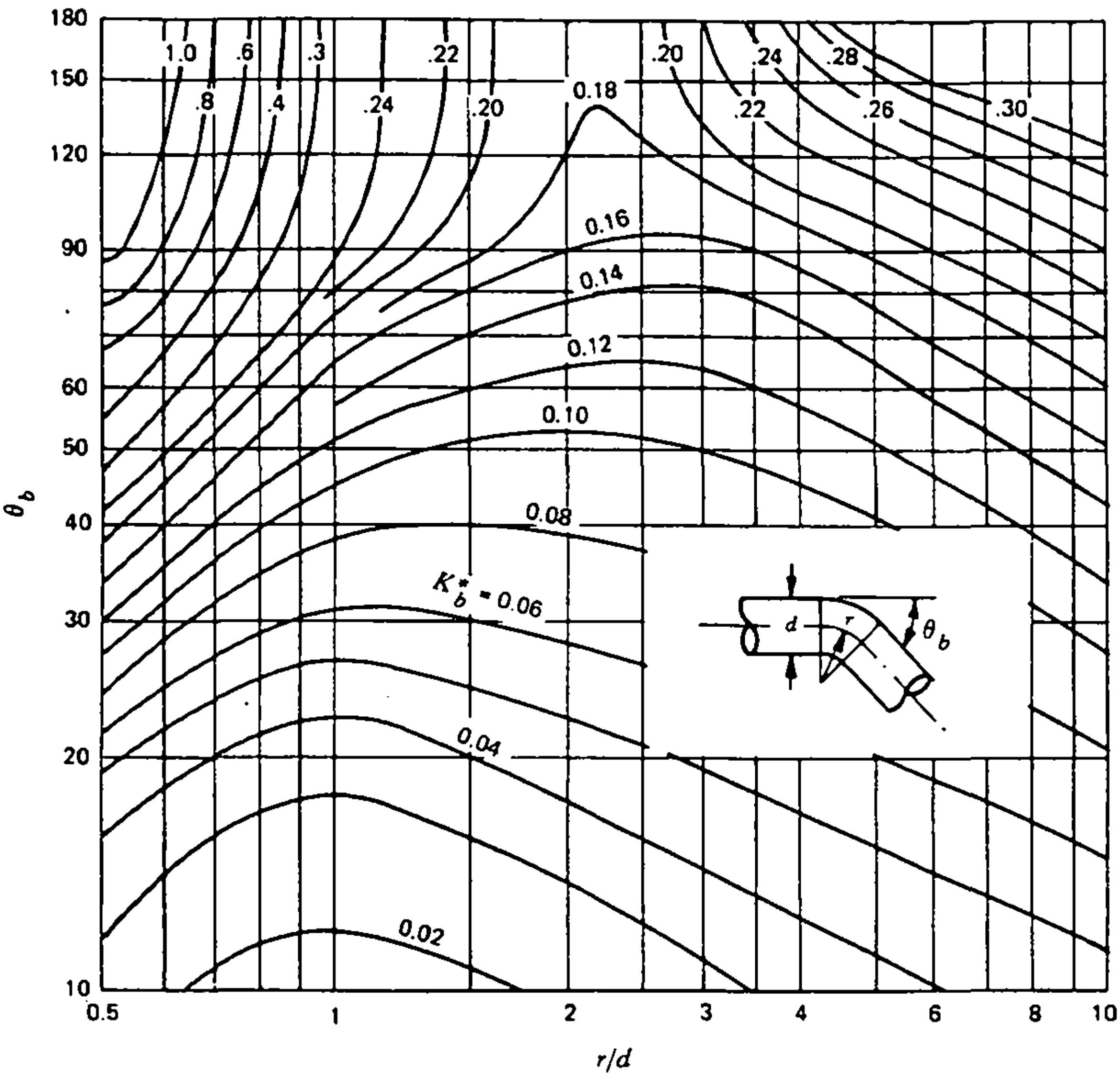


Fig. 9.2. Loss coefficients, K^*_b for circular cross-section bends (Re = 10^6)

9.2.2. REYNOLDS NUMBER CORRECTION C_{Re} (CLASS 2 FOR $Re > 10^6$, CLASS 1 FOR $Re < 10^6$)

The Reynolds number correction C_{Re} to be applied to the basic coefficients K^*_b are given in Fig. 9.3. Only part of the loss due to bends of $r/d < 1$ is strongly Reynolds number dependent. For $r/d > 0.7$ or $K^*_b < 0.4$ use the correction factors for $r/d = 1$ bends (class 2), otherwise the correction factor is given by (class 3):

$$C_{Re} = \frac{K^*_b}{K^*_b - 0.2\,(C_{Re} \text{ from Fig 9.3 for } r/d = 1) + 0.2}$$

9.4. SINGLE MITRE BENDS (CIRCULAR CROSS-SECTION CLASS 1, RECTANGULAR CROSS-SECTION CLASS 2)

9.4.1. BASIC COEFFICIENT K^*_b

Basic loss coefficients, K^*_b, are plotted against angle in Fig. 9.9. for circular and rectangular cross-section mitre bends.

9.4.2. CORRECTION TO BASIC COEFFICIENTS (CLASS 3)

Use the Reynolds number correction from Section 9.2.2. for r/d <0.7 and the outlet pipe length corrections of Fig. 9.4. Roughness corrections should be made using Section 9.2.4. and the inlet pipe limitations as in Section 9.2.6.

9.5. 90° COMPOSITE MITRE BENDS

9.5.1. BASIC COEFFICIENTS K^*_b (CIRCULAR CROSS-SECTION CLASS 1, RECTANGULAR CROSS-SECTION CLASS 2)

Basic loss coefficients K^*_b are plotted against r/d or r/W in Fig. 9.10.

9.5.2. CORRECTIONS TO BASIC COEFFICIENTS (CLASS 3)

Use the Reynolds number corrections from Fig. 9.3. and the outlet pipe corrections from Fig. 9.4. Roughness corrections should be made using Section 9.2.4. and the inlet pipe limitations as in Section 9.2.6.

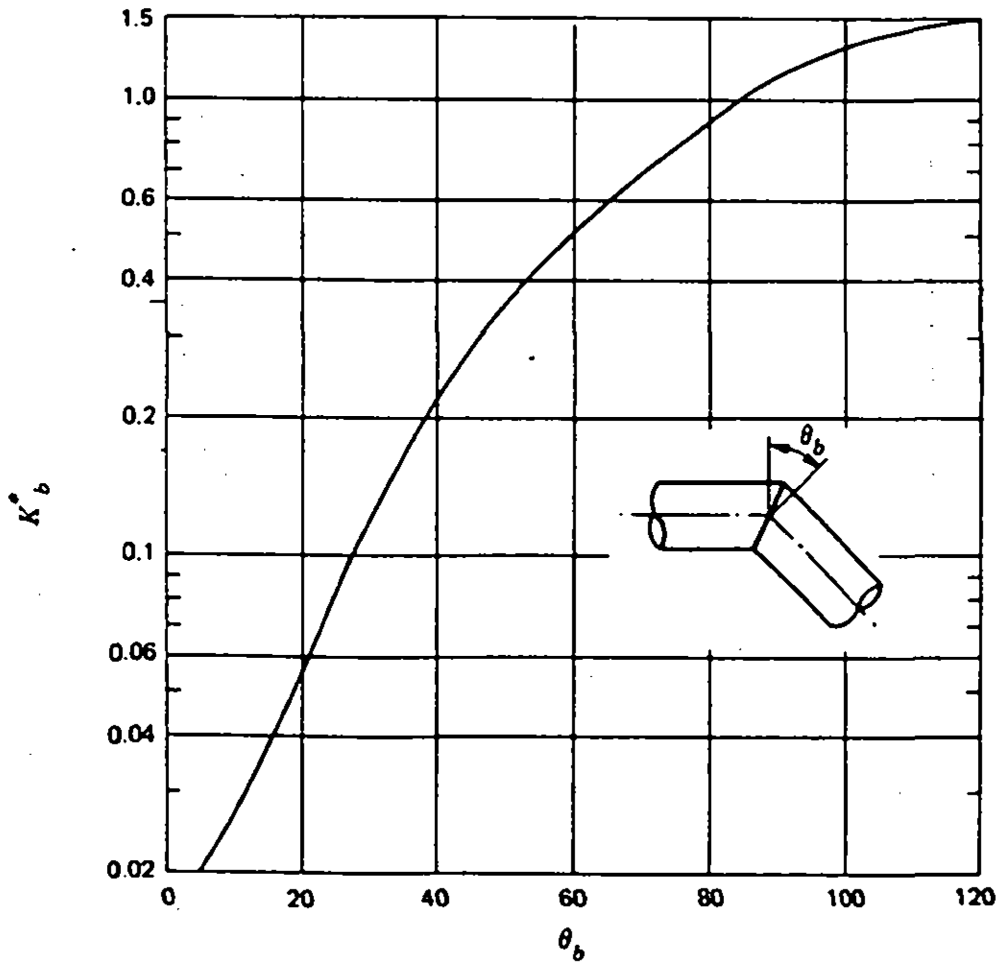


Fig. 9.9. Mitre bend loss coefficients (Re = 10^6)

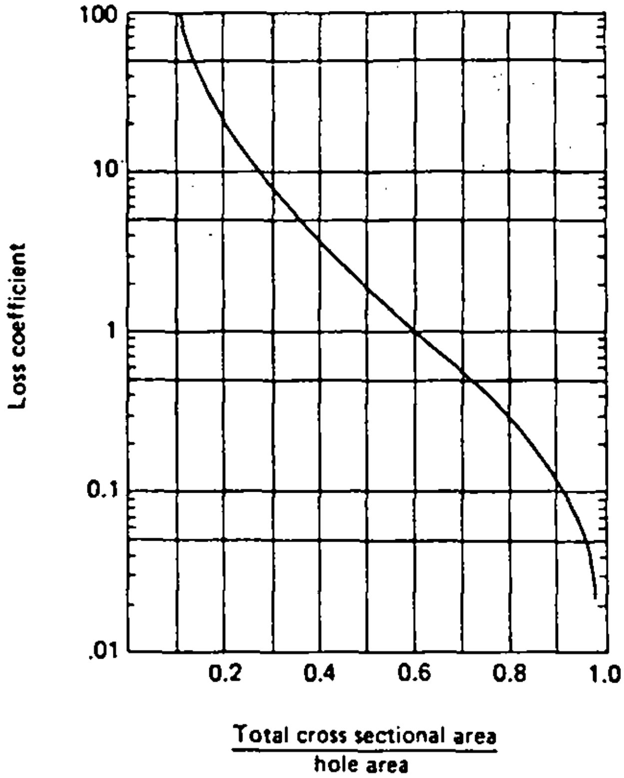


Fig. 14.7. Loss coefficients for round wire screens or netting (Re>500 based on wire diameter and velocity in holes)

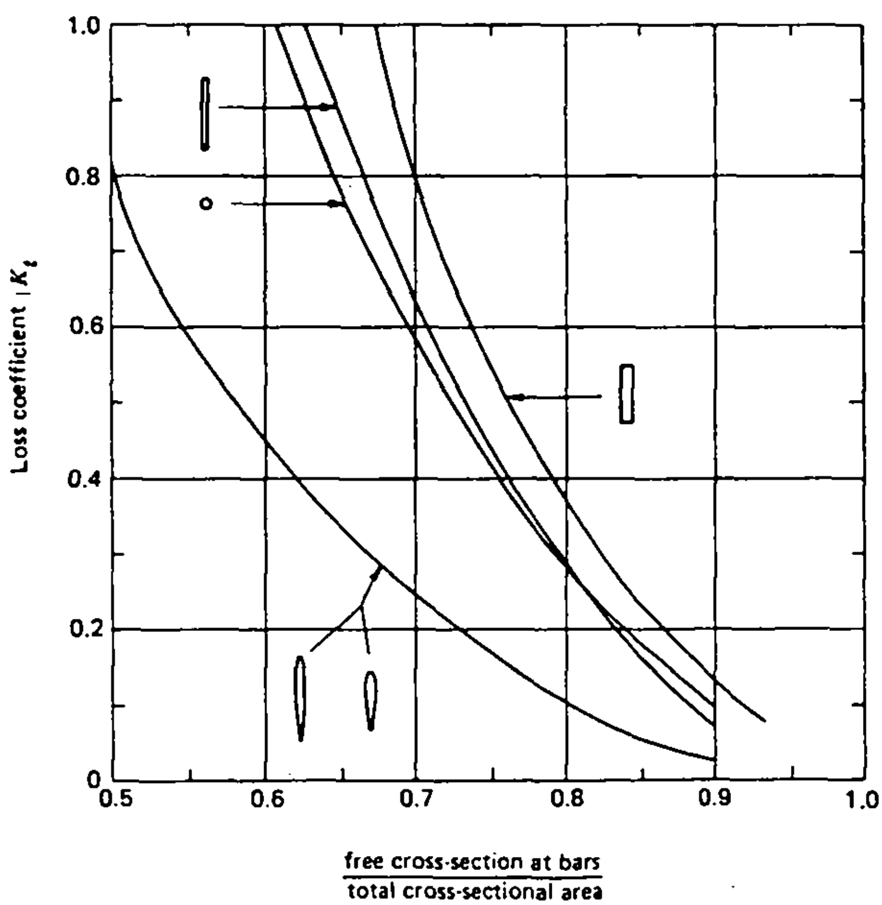


Fig. 14.8. Loss coefficients for trashracks

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Subject Hydraulic Mean Depth of Circular Pipe, Flowing at maximum discharge
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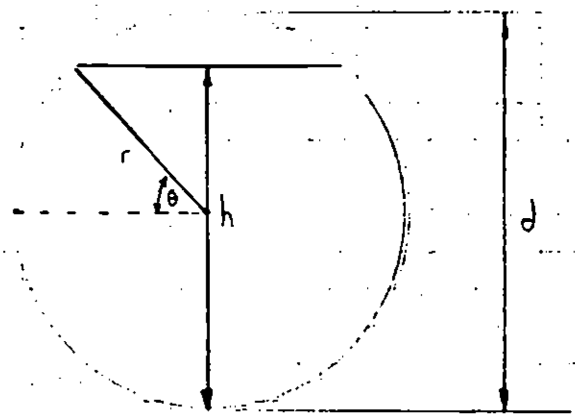
Checked by   Date



r = radius = $\frac{d}{2}$

ref 1 p.344 For Maximum discharge $\frac{h}{d} = 0.94$ $\therefore h = 0.94d$
$= 0.94 \cdot 2r$
$= 1.88r$

Area of Flow $= \frac{1}{2}(\pi r^2) + r^2\Theta^c + r^2\cos\theta\sin\theta$

Wetted perimeter $= \pi r + 2r\Theta^c$

$\Theta^c = \arcsin\left(\frac{h - r}{r}\right) = \arcsin\left(\frac{1.88r - r}{r}\right) = 1.076$ radians [or 61.64°]

$\therefore A = \frac{1}{2}\pi r^2 + 1.076r^2 + r^2\cos 1.076 \sin 1.076$

$= r^2\left(\frac{\pi}{2} + 1.076 + 0.4179\right) = 3.065r^2$

$P = \pi r + 2r\Theta = r(\pi + 2 \times 1.076) = 5.294r$

Hydraulic Mean depth $M = \frac{A}{P} = \frac{3.065r^2}{5.294r} = \underline{\underline{0.579r}}$

NANT-Y-GATH CULVERT

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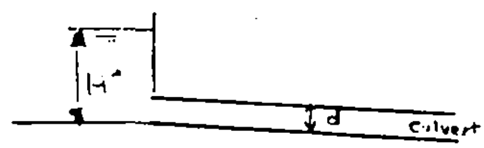
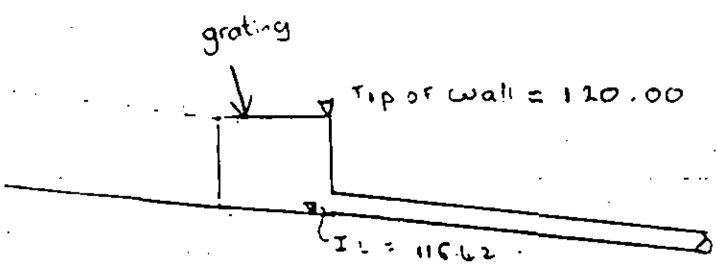
Pipe Entrance

From reference 2 section 17-8 a hydraulically long culvert will run full when H* (see below) 1·2 – 1·5 · diam. If the area around the entrance can contain a head of 1·5 H* (+ an additional head due to grating loss), it can be assumed that the pipe could run full.



d = 1·4 ∴ H* = 1·5 × 1·4 = 2·1m.

An examination of the entrance of the Nant-y-Goth culvert will show that this head could easily be contained. (see below)



i.e. max possible head at entrance to Nant-y-Goth is: 3·58 m.

or $\frac{3.58}{1.4}$ = 2·56 × diameter

O.K.

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RUN P – M Spun Concrete. $n = 0.013$

Pipe Diam. (54") = 1.37 m

pipe slope $S = \dfrac{I.L.\ u/s - I.L.\ d/s}{L} = \dfrac{116.42 - 113.84}{64.8}$

$S = 0.0398$

For steady flow $S = i = 0.038$.

$Q = \dfrac{A \times M^{2/3} \times i^{1/2}}{n}$

$n = 0.013$

see p. G1 $M = A/p = 0.579r = 0.579 \times \frac{1.37}{2} = 0.397$ m

$A = 3.065 r^2 = 3.065 \cdot \left(\frac{1.37}{2}\right)^2 = 1.438$ m²

$Q = \dfrac{1.438 \times 0.397^{2/3} \times 0.0398^{1/2}}{0.013} = 11.9$ m³/sec

Maximum Capacity of pipe P-M = 11.9 m³/sec P-M 11.9 cumecs

RUN M – L

Pipe as above (54" spun Concrete)

$S = \dfrac{I.L.\ u/s - I.L.\ d/s}{L} = \dfrac{113.84 - 113.1}{23} = 0.032$

$S = i = 0.032$

$Q = \dfrac{1.438 \times 0.397^{2/3} \times 0.032^{1/2}}{0.013} = 10.7$ m³/sec

Maximum Capacity of pipe M-L = 10.7 m³/sec M-L 10.7 cumecs

RUN L-K

pipe as above (54" spun concrete)

$S = \dfrac{I.L.\ u/s - I.L.\ d/s}{L} = \dfrac{0.6}{8} = 0.075$

$S = i = 0.075$

$Q = \dfrac{1.438 \times 0.397^{2/3} \times 0.075^{1/2}}{0.013} = 16.4$ m³/sec L-K 16.4 cumecs

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Run K - J

Pipe as before (56" Spun Concrete)

$$S = \frac{I.L\ u/s - I.L\ d/s}{L} = \frac{112.5 - 111.9}{9.5} = 0.063$$

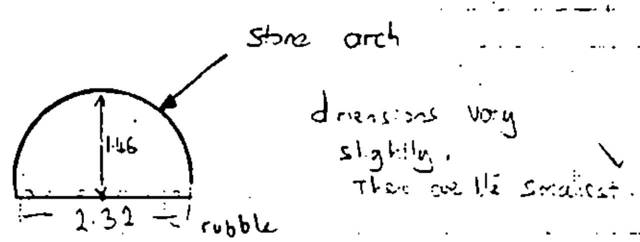
$S = i = 0.063$

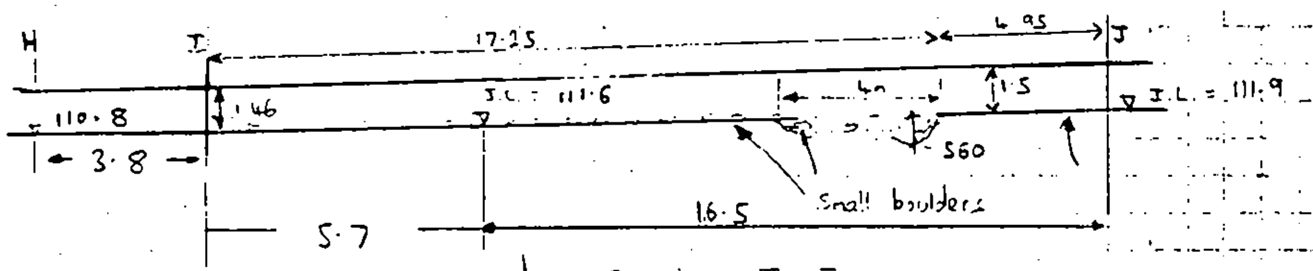
$$Q = \frac{1.638 \times 0.397^{2/3} \times 0.063^{1/2}}{0.013} = 15.0\ M^3/Sec$$

K-J
15.0 cumecs

J - I

Section





Long Section I - J

Although the hole shown obviously disturbs the Flow it should not reduce the capacity of the culvert. Therefore it will be ignored at this stage of the analysis.

(i) Calculate at what depth maximum Flow occurs for this section

n value for Floor covered with rubble = 0.035

n value for pointed stone work = 0.020

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ref 2 p136 Using the Following Formula for 'n' value of Composite Section

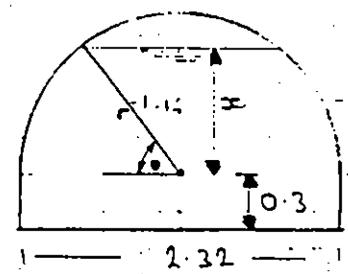
$$n = \left[\frac{\sum_1^N (P_N n_N^{1.5})}{P} \right]^{2/3}$$

$n_1 = 0.035 \quad P_1 = 2.32$

$n_2 = 0.020 \quad P_2 = ? \quad$ Say $\cancel{3.0}$ 3.198

$$n = \left(\frac{2.32 \cdot 0.035^{1.5} + 3.198 \cdot 0.020^{1.5}}{2.32 + 3.198} \right)^{2/3} \doteq \cancel{0.027} \; 0.022$$

Using trial method, Find Q max.



$$Q = \frac{A \cdot m^{2/3} i^{1/2}}{n}$$

Try: $\frac{x}{r} = 0.91 \quad \therefore x = 0.91r = 0.91 \times 1.16 = 1.056$

$\theta^c = 1.1433^c \quad (65.55^\circ)$

Area $A = 2.32 \times 0.3 + r^2\theta + r^2 \sin\theta\cos\theta$
$= 2.32 \times 0.3 + 1.16^2 \times 1.1433 + 1.16^2 \sin 1.1433 \cos 1.1433$
$= 0.696 + 1.538 + 0.508 = 2.742$

Perimeter $P = 2.32 + 2 \cdot 0.3 + 2r\theta$
$= 2.32 + 0.6 + 2 \cdot 1.16 \cdot 1.1433 = 5.572$

$$Q = \frac{A^{5/3}}{P^{2/3}} \times \frac{i^{1/2}}{n} = \frac{2.742^{5/3}}{5.572^{2/3}} \times \frac{i^{1/2}}{n} = 1.707 \frac{i^{1/2}}{n}$$

Try $\frac{x}{r} = 0.90 \quad \theta = 1.12^c$

$A = 0.696 + 1.16^2 \times 1.12 + 1.16^2 \sin 1.12 \cos 1.12 = 2.731$

$P = 2.92 + 2 \times 1.16 \times 1.12 = 5.518$

$$Q = \frac{2.731^{5/3}}{5.518^{2/3}} \cdot \frac{i^{1/2}}{n} = 1.709 \frac{i^{1/2}}{n} \quad \text{MAX}$$

Try $\frac{x}{r} = 0.89 \quad \theta^c = 1.097$

$A = 0.696 + 1.16^2 \times 1.097 + 1.16^2 \sin 1.097 \cos 1.097 = 2.718$

$P = 2.92 + 2 \times 1.16 \times 1.097 = 5.465$

$$Q = \frac{2.718^{5/3}}{5.465^{2/3}} \cdot \frac{i^{1/2}}{n} = 1.706 \frac{i^{1/2}}{n}$$

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Max Q occurs at $x/r = 0.90$

$$Q = 1.709 \frac{i^{1/2}}{n}$$

$n = \cancel{0.027}$ 0.022 (see previous page)

No I.L. is known at I but one is known at a place 5.7m upstream of I. Using this I.L.

$$S = \frac{111.9 - 111.6}{16.5} = 0.018 = i$$

$$Q = \frac{1.709 \times 0.018^{1/2}}{\cancel{0.027}\ 0.022} = \cancel{8.492}\ 10.422$$

Max flow = ~~8.492~~ 10.422 M^3/sec

J - I
10.42 cumecs.

I - H

Section Circular Stonework, Diameter = 1.52m

n value (pointed stonework) = 0.020

Slope use level 5.7m u/s I and level at H

$$S = \frac{111.6 - 110.5}{3.8 + 5.7} = 0.084$$

$S = i = 0.084$.

See p G1 $M = \frac{A}{P} = 0.579r = 0.579 \times \frac{1.52}{2} = 0.440M$

$A = 3.065 r^2 = 3.065 \times \left(\frac{1.52}{2}\right)^2 = 1.770$

$$Q = \frac{1.770 \times 0.44^{2/3} \times 0.084^{1/2}}{0.020} = 14.8\ M^3/sec$$

Max flow = 14.8 M^3/sec

I - H
14.8 cumecs.

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H - G

Section as previous i.e. 1.52 m dia. circular stonework.

Next known level is just U/S of M.H. at E

$$S = \frac{110.8 - 110.69}{16.2} = 0.0068 \quad (\text{From } H \rightarrow E.)$$

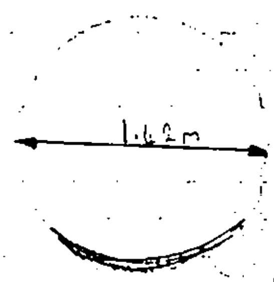
$$Q = \frac{1.77 \times 0.44^{2/3}\ 0.0068^{1/2}}{0.02} = 4.22\ m^3/sec$$

steady state
Max Flow = 4.22 m³/sec

H - G
4.22 cumecs

G - F

SECTION



rusty iron p.p.

Concrete

n Value for rough concrete
n Value for rusty iron = 0.02

Treat section as circular pipe with n value = 0.02

Say Diam = ~~1.38~~ 1.40 to allow for Concrete

Slope (as for H-G) = 0.0068 = i

See p. G1 $M = \frac{A}{P} = 0.579r = 0.579 \times \frac{\cancel{1.38}\ 1.40}{2} = 0.405$

$A = 3.065r^2 = 3.065 \times \left(\frac{\cancel{1.38}\ 1.40}{2}\right)^2 = \cancel{1.459}\ 1.502$

$$Q = \frac{\cancel{1.459}\ 1.502 \times 0.405^{2/3} \times 0.0068^{1/2}}{0.02} = \cancel{3.27}\ 3.39\ m^3/sec$$

steady state 3.39
Max Flow = ~~3.27~~ m³/sec

G - F
3.39 cumecs

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F - E

1.20 Spun Concrete pipe

$n = 0.013$ $\qquad i = S = 0.0068$ (see 4-G, G-F).

$M = 0.579r = 0.579 \times r = 0.347$

$A = 3.065r^2 = 3.065 \cdot \left(\frac{1.2}{2}\right)^2 = 1.103$

$$Q = \frac{1.103 \times 0.347^{2/3} \times 0.0068^{1/2}}{0.013} = 3.46\ M^3/sec$$

Q Max = 3.46 M^3/sec $\qquad$ F-E 3.46 cumecs

E-D

Spun Concrete $n = 0.013$

pipe diam. (52") = 1.32 m

$$\text{pipe slope} = \frac{109.8 - 107.43}{93} = 0.0255$$

$S = i = 0.0255$

$M = 0.579r = 0.579 \times 1.32/2 = 0.382\ m$

$A = 3.065r^2 = 3.065 \cdot \left(\frac{1.32}{2}\right)^2 = 1.335\ m^2$

$$Q = \frac{1.335 \times 0.382^{2/3} \times 0.0255^{1/2}}{0.013} = 8.63\ M^3/sec$$

Qmax (steady state) = 8.63 M^3/sec $\qquad$ E-D 8.63 cumecs

D-C

Pipe section (as above i.e. 52" Spun Concrete

$n = 0.013$

$$\text{Pipe Slope} = \frac{107.43 - 102.87}{22.3} = 0.204$$

$$Q = \frac{1.335 \times 0.382^{2/3} \times 0.204^{1/2}}{0.013} = \cancel{20.2}\ 24.4\ M^3/sec$$

D-C 24.4 cumecs

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Section Culvert analysis
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Checked by [signature] Date 20/12/84
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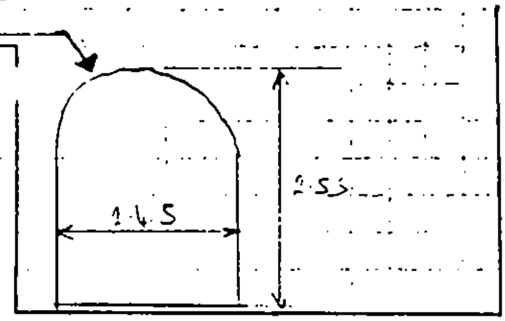
C - A

This is an arch 14 m long of section shown below.

The capacity of this section is obviously much larger than the rest of the culvert. And effectively the culvert outfall is at C



The max peak flow for a 1 in 100 year return period storm for present situation has been evaluated to be 5.85 cumecs by the Institute of Hydrology.

The above calculations have shown that, under uniform flow conditions, a section of the culvert has a capacity lower than the 5.85 m³/sec. Since the actual figures are considerably below 5.85 m³/sec the culvert will 'throttle' at this point and a pressure head will build up to 'drive' the flow (of 5.85 m³/sec) through this section.

The next calculation performed, works back from point E, which is the end of the throttle, and calculates the pressure heads required to drive the flow through this section. The calculation proceeds up the channel to a point where the flow

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is no longer under pressure, and the water surface becomes free.

CALCULATION OF HYDRAULIC HEADS REQUIRED TO DRIVE A FLOW OF $5.85\ M^3/sec$ THROUGH THE CULVERT.

F – E

(pipe Full conditions)

1.20 spun Concrete pipe $n = 0.013$

$$M = \frac{A}{P} = \frac{\pi r^2}{2\pi r} = r/2 = 1.2/4 = 0.3\ M$$

$$A = \pi r^2 = \pi \times \left(\frac{1.2}{2}\right)^2 = 1.131\ m^2$$

Working From I.L at E as datum.

$$i = \frac{H_F - IL_e}{3.09}$$

$$Q = 5.85 = \frac{A m^{2/3} i^{1/2}}{n} \qquad i = \left(\frac{Q\,n}{A\,m^{2/3}}\right)^2$$

$$i = \left(\frac{5.85 \times 0.013}{1.131 \times 0.3^{2/3}}\right)^2 = 0.0225$$

$$\frac{H_F - 110.69}{3.09} = 0.0225$$

$$H_F = 0.0225 \times 3.09 + 110.69 = \underline{110.76}\ M$$

I.L at F = I.L at E + S × L = 110.69 + 0.0058 × 3.09 = 110.71

pressure head at F (H_F) = 110.76 − 110.71 = 0.05 M

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Section Culvert analysis
Job No. 84710A
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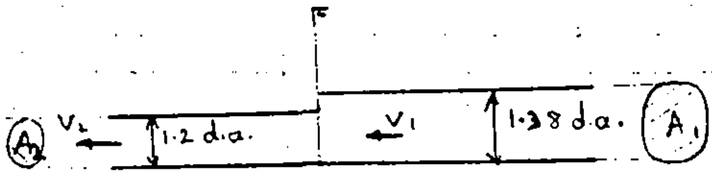
Head loss at junction F



$$V_1 = \frac{Q}{A_1} = \frac{5.85}{1.38^2 \pi/4} = 3.91 \text{ m/s}$$

$$V_2 = \frac{Q}{A_1} = \frac{5.85}{1.2^2 \pi/4} = 5.17 \text{ m/sec}$$

From p200 Table 7.1 ref 1

$$\text{head loss } h = \frac{K V_2^2}{2g} \qquad K \approx 0.1$$

$$\therefore h = \frac{0.1 \times 5.17^2}{2 \times 9.81} = 0.136 \text{ m}$$

Head lost due to the abrupt contraction $\approx$ 0.14 M

$$H^F_{contr.} = 0.14$$

G - F

Section (see p N.G 6) rusty iron pipe with concrete invert

Diameter = ~~1.38~~ 1.40 m n = 0.02

$$M = \frac{r}{2} = \frac{\cancel{1.38}\ 1.40}{4} = \cancel{0.345}\ 0.35$$

$$A = \pi r^2 = \pi \times \cancel{1.38}^{1.4\,2}/(2)^2 = \cancel{1.496}\ 1.54$$

$$i = \left(\frac{Q\, n}{A\, M^{2/3}}\right)^2 = \left(\frac{5.85 \times 0.020}{\cancel{1.496}_{1.54} \times \cancel{0.345}_{0.35}{}^{2/3}}\right)^2 = \cancel{0.025}\ 0.0234$$

$$\therefore i = \frac{H_G - (H_F + H^F_{contr})}{L} = \frac{H_G - 110.76 - 0.14}{6.7} = \cancel{0.025}\ 0.0234$$

$$H_G = \cancel{0.025}\ 0.0234 \times 6.7 + 110.76 + 0.14 = \cancel{111.07}\ 111.057$$

$$I.L. \text{ at } G = I.L_F + S \times L = 110.71 + 0.0065 \times 6.7 = 110.76$$

pressure head at G = ~~111.07~~ 111.057 - 110.76 = ~~0.31M~~ 0.30 m.

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Section Culvert analysis
Job No. 82712A
Checked by [signature] Date 20/12/86
Subject Nant-y-gaih
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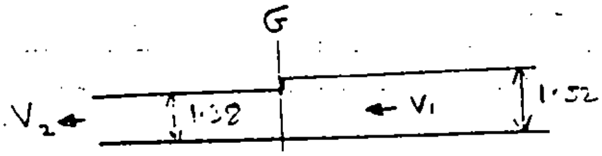
Head loss at junction G



$$V_2 = \frac{Q}{A_2} = \frac{5.85}{1.38^2 \pi/4} = 3.91 \text{ M/sec}$$

From p200 Table 7.1 ref 1

$$\text{head loss } h = \frac{KV_2^2}{2g} \qquad K \doteq 0.09$$

$$h = \frac{0.09 \times 3.91^2}{2 \times 9.81} = 0.07 \text{ M}$$

Head loss due to abrupt contraction at G ≈ 0.07M

$$H^G_{contr.} = 0.07$$

H - G

Section 1.52 dia. circular stone work. $n = 0.02$

$M = r/2 = 1.52/4 = 0.38$ m

$A = \pi r^2 = \pi \times 1.52^2/4 = 1.81 \text{ m}^2$

$Q = 5.85$, $n = 0.02$ $\qquad i = \left(\frac{Q\,n}{A\,M^{2/3}}\right)^2$

$$i = \left(\frac{5.85 \times 0.02}{1.81 \times 0.38^{2/3}}\right)^2 = 0.0152$$

$$i = \frac{H_H - (H_G + H^G_{contr.})}{L} = \frac{H_H - (\cancel{111.07}\ 111.057 + 0.07)}{6.5} = 0.0152$$

$$\therefore H_H = 0.0152 \times 6.5 + \cancel{111.07}\ 111.057 + 0.07 = \cancel{111.24}\ 111.226$$

IL at H = 110.8 $\therefore$ Pressure head $= \cancel{111.24}\ 111.226 - 110.8 = \cancel{0.44}\ 0.43$ M

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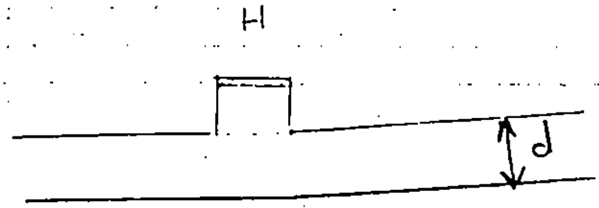
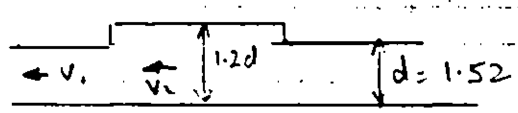
HEAD LOSS AT MANHOLE H.



The Manhole has been constructed by breaking out the top of the circular stonework culvert. There is likely to be a little head loss at the manhole but not very much.

Assume that the losses are equivalent to a sudden enlargement and then contraction, with the larger section being of diameter 1.2d

p 199-201
ref 1.



$$V_1 = \frac{Q}{A} = \frac{5.85}{1.52^2\pi/4} = 3.22\ m^3/sec$$

$$V_2 : 5.85/(1.2\times1.52)^2\pi/4 = 2.24 \qquad A_2/A_1 = d_2/d_1$$

$$h_{en} = \frac{V_2^2}{2g}\left(\frac{A_2}{A_1}-1\right)^2 = \frac{2.24^2}{2g}\left(\frac{(1.2)^2}{(1)^2}-1\right)^2 = \underline{\underline{0.05\ M}}$$

h_{co} (From table 7.1 ref 1) $K \simeq 0.13$

$$h_{co} = \frac{KV_1^2}{2g} = \frac{0.13\times3.22^2}{2\times9.81} = 0.07$$

Total loss at M.H (H_{MX}) $\approx 0.07 + 0.05 = \underline{0.12\ M}$

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I - H

Section (as H-G) 1·52 dia. circular stonework
n = 0·02

See pNG11 $i = 0.0152$

$$i = \frac{H_I - (H_H + H_{MH})}{L} = \frac{H_I - (\cancel{111.24}\ 111.226 + 0.12)}{3.8} = 0.0152$$

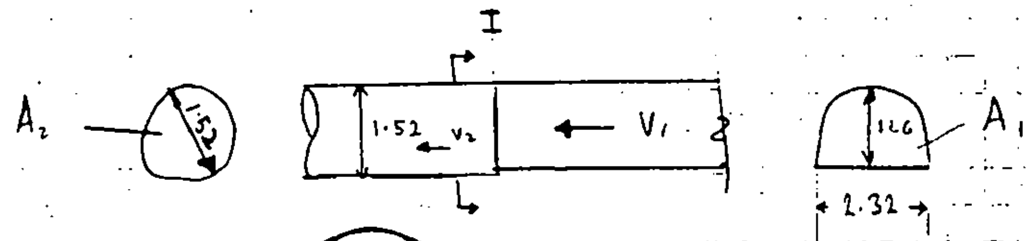
$$\therefore H_I = 3.8 \times 0.0152 + \cancel{111.24}\ 111.226 + 0.12 = \cancel{111.42}\ 111.40$$

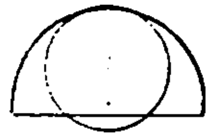
For S see p.NG5 IL at I = IL_H + SxL

$$= 110.8 + 0.084 \times 3.8 = 111.12$$

Pressure head at I = $\cancel{111.42}$ 111.40 - 111.12 = $\cancel{0.3}$ 0.28 M

Junction I

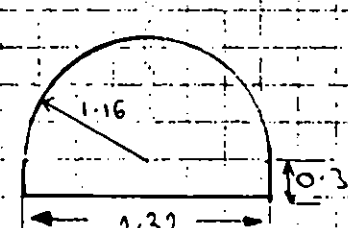




The head loss will be calculated as if for 2 concentric pipes of areas A_2 and A_1

$$A_1 = 0.3 \times 2.32 + \frac{1.16^2 \pi}{2} = 2.81\ m^2$$

$$A_2 = \frac{1.52^2 \pi}{4} = 1.81\ M^2$$



Equiv. diam. For $A_1 = \sqrt{\frac{2.81}{\pi} \times 4} = 1.89$

Table 7.1
ref 1 (p…)

$$\frac{d_2}{d_1} = \frac{1.52}{1.89} = 0.8 \qquad K = 0.14$$

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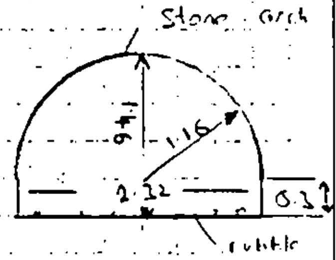
$$\text{head loss } h_I = \frac{K V_2^2}{2g} \qquad V_2 = \frac{5.85}{1.81} = 3.23 M/S$$

$$h_I = \frac{0.14 \times 3.23^2}{2 \times 9.81} = \underline{0.075M}$$

I - J



$n_{stone} = 0.02 \qquad P_1 = 2.32$

$n_{rubble} = 0.035 \qquad P_2 = 0.6 + \pi \times 1.16 = 4.24$

REF 2 p132

$$n = \left[\frac{\sum_1^N (P_N n_N^{1.5})}{P}\right]^{2/3}$$

$$n = \left[\frac{2.32 \times 0.02^{1.5} + 4.24 \times 0.035^{1.5}}{4.24 + 2.32}\right]^{2/3} = 0.030$$

$$A = 2.32 \times 0.3 + 1.16^2 \pi/2 = 2.81 \; m^2$$

$$P = 2.32 + 4.24 = 6.56 \qquad M = \frac{A}{P} = \frac{2.81}{6.56} = 0.428$$

$$i = \left(\frac{Q \times n}{A \times m^{2/3}}\right)^2 = \left(\frac{5.85 \times 0.03}{2.81 \times 0.428^{2/3}}\right)^2 = 0.012$$

$$i = \frac{H_J - (H_I + h_I)}{L} = \frac{H_J - (\overset{111.40}{\cancel{111.42}} + \overset{0.075}{\cancel{0.27}})}{17.25 + 4.95} = 0.012$$

$$H_J = 0.012(17.25 + 4.95) + \overset{111.40}{\cancel{111.42}} + \overset{0.075}{\cancel{0.27}} = \overset{111.74}{\cancel{111.76}}$$

I L at J = 111.9

Since $H_J <$ I.L.$_J$ the flow has returned to open channel.

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The preceding calculations were carried out with the max. flow of 5.85 m³/sec which is the maximum flow in the present condition. The same calculation will now be performed for the 'On Restoration' condition, which gives a maximum flow of 6.1 m³/sec.

Calculation of hydraulic heads required to drive a flow of 6.1 m³/sec through the culvert.

F-E

From p. N-G9 $Q = 6.1$, $M = 0.3m$, $A = 1.131 m^2$, $n = 0.013$

$$i = \left(\frac{Q \cdot n}{A \cdot M^{2/3}}\right)^2 = \left(\frac{6.1 \times 0.013}{1.131 \times 0.3^{2/3}}\right)^2 = 0.0245$$

$H_F = I L_R + l \cdot i = 110.69 + 3.09 \times 0.0245 = \underline{110.77\ m}$

See p N-G9 pressure head at F = $(110.77 - 110.71) = 0.06\,m$

$H_F = 110.77$

Head loss at junction F

See p N-G10 $h_f = \frac{0.1 \times V^2}{2g}$ $V = \frac{6.1}{1.2^2\pi/4} = 5.4\ m/sec.$

$\therefore h_f = \frac{0.1 \times 5.4^2}{2 \times 9.81} = 0.149 \simeq 0.15\ m$

G-F

From p N-G10 $M = \cancel{0.345}\ 0.35\,m$, $A = \cancel{1.96}\ 1.54\,m^2$, $n = 0.02$

$Q = 6.1\ m^3/s$ $i = \left(\frac{Q \cdot n}{A \cdot M^{2/3}}\right)^2 = \left(\frac{6.1 \times 0.02}{\cancel{1.96}\ 1.54 \times \cancel{0.345}\ 0.35^{2/3}}\right)^2 = \cancel{0.0275}\ 0.0254$

$H_G = i \cdot l + H_F + h_f = \cancel{0.0275}\ 0.0254 \times 6.7 + 110.77 + 0.15 = \underline{\cancel{111.10}\ 111.09\ M}$

$\underline{H_G = \cancel{111.10}\ 111.09\ M}$

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see p N-G 10 pressure head at G = ~~111.12~~ 111.09 − 110.76 = ~~0.36~~ 0.33 m

Head loss at junction G

From p N-G 11 $h_g = \dfrac{0.09 \times V^2}{2 \cdot g}$ $V = \dfrac{6.1}{1.38^2 \pi/4} = 4.08$ m/sec

$\therefore h_g = \dfrac{0.09 \times 4.08^2}{2 \cdot 9.81} = \underline{0.076 \text{ M}}$

H - G

From p N-G 11 $M = 0.38$ m, $A = 1.81 m^2$, $n = 0.02$

$i = \left(\dfrac{Q \times n}{A \cdot M^{2/3}}\right)^2 = \left(\dfrac{6.1 \times 0.02}{1.81 \times 0.32^{2/3}}\right)^2 = 0.0165$

$H_H = i \times l + H_G + h_g = 0.0165 \times 5.5 + \cancel{111.12}\ 111.09 + 0.076 = \cancel{111.28}\ 111.27$ m

$H_H = \cancel{111.28}\ 111.27$ m

see p N-G 11 Pressure head at H = ~~111.28~~ 111.27 − 110.8 = ~~0.48~~ 0.47 m

HEAD LOSS AT M.H. at H

See p N.G.12 Using ref 3 Fig's 14.14 ; 14.15 (p 269)

$A_1/A_2 = (1/1.2)^2 = 0.694$ $K_{contr.} = 0.2$ (based on smaller pipe)
$K_{exp.} = 0.13$ ( )

$\therefore h_H = (0.2 + 0.13) \times \dfrac{V^2}{2g} = \dfrac{0.33 \times (6.1/1.52^2 \pi/4)^2}{2 \cdot 9.81} = 0.19$ m

I - H

Section - as (H - G) $\therefore$ $i = 0.0165$

See p N-G 13 $H_I = i \times l + H_H + h_H = 0.0165 \times 3.8 + \cancel{111.28}\ 111.27 + 0.19 = \cancel{111.53}\ 111.52$ m

$H_I = \cancel{111.53}\ 111.52$

Pressure head at I = H_I − I.L.$_I$ = (~~111.53~~ 111.52 − 111.12) = ~~0.41~~ 0.40 M

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Junction I

From p N-G 16 $h_I = \dfrac{0.14 V_I^2}{2g}$ $V_I = \dfrac{6.1}{1.81} = 3.37$ m/sec

$$h = \frac{0.14 \times 3.37^2}{2 \times 9.81} = \underline{\underline{0.081 m}}$$

I - J

see p N-G 16

$A = 2.81 m^2$, $P = 5.56 m$, $M = 0.428 M$

$n = 0.030$ $i = \left(\dfrac{Q \times n}{A \cdot M^{2/3}}\right)^2 = \left(\dfrac{6.1 \times 0.030}{2.81 \times 0.428^{2/3}}\right)^2 = 0.013$

see p N-G 16 $H_J = H_I + h_I + i \times L = \cancel{111.53}$ 111.52 $+ 0.081 + 0.013 \times (17.25 + 4.95)$

$= \cancel{111.9}$ m

111.89 m

I.L at J = 111.9

Since H_J = I.L at J , Flow has just returned to open channel.

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The Capacity of the Nant-Moel Culvert has been calculated as being approximately = 8.0 m³/sec (with an opening at M). However the maximum flow 'On Restoration' is equal to 2.7 m³/sec. The effect of directing the additional 0.7 m³/sec flow down the Nant-y-Gath culvert will now be assessed. This provides a total flow of (6.1 + 0.7) m³/sec through the Culvert.

Calculation of the hydraulic heads required to drive a flow of 6.8 m³/sec through the Culvert.

F - E

See p N-G 9
$M = 0.30$ m
$A = 1.131$ m²
$n = 0.013$

$$i = \left(\frac{Q \cdot n}{A \cdot M^{2/3}}\right)^2 = \left(\frac{6.8 \times 0.013}{1.131 \times 0.3^{2/3}}\right)^2 = 0.030 \ldots$$

$$H_F = IL_E + l \cdot i = 110.69 + 3.09 \times 0.030 = 110.78 \text{ m}$$

See p N-G 9 Pressure head at F = 110.78 − 110.71 = 0.07 m

$H_F = 110.78$

Head loss at junction F

See p.N-G 10 $h_F = \frac{0.1 V_2^2}{2g}$ $V_2 = \frac{6.8}{1.2^2 \pi / 4} = 6.01$ m/s

$$h_F = \frac{0.1 \times 6.01^2}{2 \times 9.81} = 0.184 \text{ m}$$

G - F

From p N-G 10
$M = \cancel{0.365}$ 0.35 m $n = 0.02$
$A = \cancel{1.495}$ 1.54 m² $Q = 6.8$ m³/s

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$$i = \left(\frac{6.8 \times 0.02}{\cancel{1.49}\ 1.54 \times \cancel{0.33}\ 0.35^{2/3}} \right)^2 = \cancel{0.034}\ 0.0316$$

From p N-G10 $H_G = i \times l + H_F + h_F = \cancel{0.034}\ 0.0316 \times 6.7 + 110.78 + 0.184 = \cancel{111.19}\ 111.176\ m$

$$\underline{H_G = \cancel{111.19}\ 111.176\ m}$$

See p N-G10 Pressure head at G $= \cancel{111.19}\ 111.176 - 110.76 = \cancel{0.43}\ 0.42\ m$

Head loss at junction G

See p N-G11 $h_g = \dfrac{0.09\,V^2}{2g}$ $\qquad V = \dfrac{6.8}{1.38^2\pi/4} = 4.55\ m/sec$

$$h_g = \frac{0.09 \times 4.55^2}{2 \times 9.81} = \underline{0.095\ m}$$

H - G

From p N-G11 $M = 0.38\,m$, $A = 1.81\,m^2$, $n = 0.02$

$$i = \left(\frac{6.8 \times 0.02}{1.81 \times 0.38^{2/3}} \right)^2 = 0.020$$

$H_H = i \times l + H_G + h_g = 0.02 \times 6.5 + \cancel{111.19}\ 111.176 + 0.095 = \underline{\cancel{111.41}\ 111.404\ m}$

$$\underline{H_H = \cancel{111.41}\ 111.404\ m}$$

See p N-G11 Pressure head at H $= \cancel{111.41}\ 111.404 - 110.8 = \underline{\cancel{0.61}\ 0.604\ m}$

Head loss at M.H. at H

See p N-G16 $h_H = (0.2 + 0.13)\dfrac{V^2}{2g} = \dfrac{0.33 \times \left(6.8 / 1.53^2\pi/4\right)^2}{2 \times 9.81} = 0.236$

I - H

Section as (H - G) $\therefore$ $i = 0.020$

See p N-G13 $H_I = i \times l + H_H + h_H = 0.020 \times 3.8 + \cancel{111.41}\ 111.404 + 0.236 = 111.72\ m$

$$\underline{H_I = 111.72\,m}$$

Pressure head at I $= H_I - I_L = 111.72 - 111.12 = \cancel{0.61}\ 0.60\ m$

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Section Culvert analysis
Job No. 86710A
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Subject Nant-y-Gath
Revised by   Date
Checked by   Date

Junction I

From pN·G4 $h_I = \frac{0.14\, V_2^2}{2g}$ $V_2 = \frac{6.8}{1.81} = 3.76$ m/s

$$h_I = \frac{0.14 \cdot 3.76^2}{2 \times 9.81} = \underline{0.10\ m}$$

I - J

See pN·G16 $A = 2.81 m^2$, $P = 6.56 m$, $M = 0.428 m$, $n = 0.030$

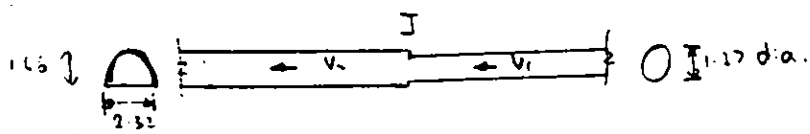
$$i = \left(\frac{Q \cdot n}{A\ m^{2/3}}\right)^2 = \left(\frac{6.8 \times 0.03}{2.81 \cdot 0.428^{2/3}}\right)^2 = 0.016$$

See pN·G14 $H_J = H_I + h_I + i \times l = 111.72 + 0.1 + 0.016 \times (17.25 + 6.95)$

$\underline{H_J = \cancel{112.17\ m}\ 112.18\ m}$

pressure head at J = $\cancel{112.17}$ 112.18 − 111.9 = $\underline{\cancel{0.27}\ 0.28\ m}$

Junction J



$A_1 = \frac{\pi\, 1.37^2}{4} = 1.476\ m^2$

(see above) $A_2 = 2.81 m^2$

$V_1 = 6.8 / 1.476 = 4.61$ m/sec
$V_2 = 6.8 / 2.81 = 2.42$ m/sec

ref 3.
From Fig 16.15 $\frac{A_1}{A_2} = \frac{1.476}{2.81} = 0.525$ K (based on V_1) = 0.28

$$\therefore \text{head loss} = \frac{0.28 \times 4.61^2}{2 \times 9.81} = 0.303\ m$$

ref 3
From Fig 9.9 Also at J there is a bend of 17° $h = 0.05 \frac{V^2}{2g}$

(Using V_1) $h = \frac{0.05 \cdot 4.61^2}{2 \times 9.81} = \cancel{0.05}\ 0.054$ m

Total loss at J ; $h_J = \cancel{0.3}\ 0.303 + 0.054 = \underline{\cancel{0.35}\ 0.357\ m}$

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		Project N C B GARNANT	Sheet No. N-G-21
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Checked by	Date		

K - J

1·37 Concrete pipe

from pN G1

$A = 1{\cdot}37^2 \pi/4 = 1{\cdot}474\ m^2$

$P = \left(\frac{1{\cdot}37}{2}\right) \times 2 \times \pi = 4{\cdot}30\ m$

$n = 0{\cdot}013$

$$i = \left(\frac{Q \times n \times P^{2/3}}{A^{5/3}}\right)^2 = \left(\frac{6{\cdot}8 \times 0{\cdot}013 \times 4{\cdot}3^{2/3}}{1{\cdot}474^{5/3}}\right)^2 = 0{\cdot}015$$

$$H_K = H_J + i \times l + h_J = \overset{112\cdot18}{\cancel{112\cdot17}} + 0{\cdot}015 \times 9{\cdot}5 + 0{\cdot}357 = \overset{112\cdot68}{\cancel{112\cdot66}}\ m$$

Pressure head at K : $H_K - I.L_K = \overset{112\cdot68}{\cancel{112\cdot66}} - 112{\cdot}5 = \overset{0\cdot18}{\cancel{0\cdot16}}\ m$.

Manhole at K

Area = 1·474 m²

Assume that the area of flow in the Manhole ≏ 2·2 m²

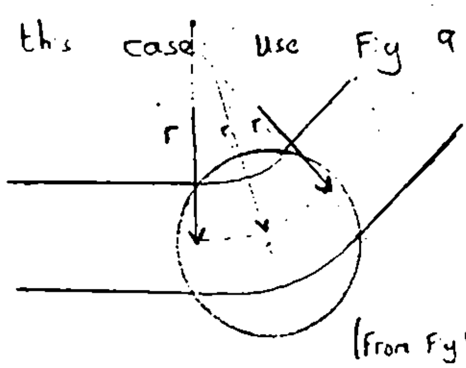
From Ref 3. Fig 14.14 / ·15 $\frac{A_1}{A_2} = \frac{1{\cdot}474}{2{\cdot}2} = 0{\cdot}67$

Enlargement coefficient K (based on V_1) = 0·15
Contraction " " = 0·21

$V_1 = \frac{6{\cdot}8}{1{\cdot}474} = 4{\cdot}6 \qquad \therefore h = \frac{(0{\cdot}15 + 0{\cdot}21)\, 4{\cdot}6^2}{2 \times 9{\cdot}81} = 0{\cdot}39\ m$

Also at K is a 45° bend

In this case use Fig 9.2 From ref D



$r \approx 2{\cdot}5$

$r/d = \frac{2{\cdot}5}{1{\cdot}37} = 1{\cdot}82$

(From Fig 9.2, K = 0·09) Say 0·1

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Checked by	Date		

$$h_{bend} = \frac{0.1 \times 4.6^2}{2 \times 9.81} = 0.11 m$$

$$h_K = 0.39 + 0.11 = \underline{0.5 M}$$

L - K

Section as K-J $\therefore$ $i = 0.015$
$l = 8m$

$$H_L = H_K + h_K + i \times l = \overset{112.68}{\cancel{112.66}} + 0.5 + 0.015 \times 8 = \overset{113.30}{\cancel{113.25}} m$$

Pressure head L = $\overset{113.30}{\cancel{113.25}} - 113.1 = \overset{0.20}{\cancel{0.15}} m$

$$\underline{H_L = \overset{113.30}{\cancel{113.25}} m}$$

Bend at L

20° bend at L.

Ref 3 From Fig 9.9 — $K = 0.06$ $V = 4.6 m/sec$ [section as L - J]

$$\therefore h_L = \frac{0.06 \times 4.6^2}{2 \times 9.81} = 0.065 m$$

L - M

Section as L-J (54" Concrete pipe)

$\therefore i = 0.015.$
$l = 23m$

$$H_M = H_L + h_L + i \times l = \overset{113.30}{\cancel{113.25}} + 0.065 + 0.015 \times 23 = \overset{113.71}{\cancel{113.66}}$$

$H_M = 113.66 < I.L._M (113.84)$

$\therefore$ Flow has returned to open channel

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Section Culvert analysis
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Revised by ______ Date ______
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CALCULATION OF FLOW VELOCITIES IN DAMAGED SECTION I-J

FLOW OF $5.85\ m^3/sec$

For this flow the flow becomes open-channel in the section from I-J.

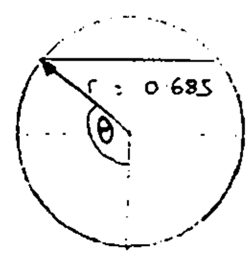
The velocity for the full section $= \dfrac{Q}{A_{full}}$

See pN-G 14 $= \dfrac{5.85}{2.81} = \underline{\underline{2.08\ m/s}}$

The velocity of the flow approaching the section from section J-K is now calculated.

Section J-K is 1.37m diameter concrete pipe

$Q = 5.85\ m^3/sec$
$n = 0.013$
$A = r^2(\theta - \sin\theta\cos\theta)$
$P = 2r\theta^c$
See pN-G 3 $i = 0.063$



$$Q = \frac{A^{5/3} \times i^{1/2}}{P^{2/3} \times n} = \frac{\{0.685^2(\theta - \sin\theta\cos\theta)\}^{5/3} \times 0.063^{1/2}}{(2 \times 0.685 \times \theta^c)^{2/3} \times 0.013} = 5.85$$

$$= \frac{0.283 \times 0.251\ (\theta - \sin\theta\cos\theta)^{5/3}}{0.0160\ \theta^{2/3}} = \frac{4.439\ (\theta - \sin\theta\cos\theta)^{5/3}}{\theta^{2/3}}$$

$\therefore$ For this pipe $Q = 4.439\dfrac{(\theta - \sin\theta\cos\theta)^{5/3}}{\theta^{2/3}}$

Try $\theta = 1.474$. $Q = \dfrac{4.439(1.474 - \sin 1.474\cos 1.474)^{5/3}}{1.474^{2/3}}$

$Q = 5.847 \approx 5.85\ m^3/sec$

θ O.K.

$$V = \frac{Q}{A} = \frac{5.85}{0.685^2(1.474 - \sin 1.474\cos 1.474)} = \underline{\underline{9.05\ m/sec.}}$$

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Checked by [illegible] Date 20/12/81 Subject Nant-y-Gath
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Flow of 6.1 m^3/sec

Full bore velocity of section I-J

$$V = \frac{Q}{A_{full}} = \frac{6.1}{2.81} = \underline{2.17 \text{ m/sec}}$$

Velocity of flow in J-K

try $\Theta = 1.495$. $\quad Q \doteq \frac{4.439\,(1.495 - \sin 1.495 \cos 1.495)^{5/3}}{1.495^{2/3}}$

$$= 6.09 \approx 6.1$$

$$\therefore \Theta \simeq 1.495$$

$$V = \frac{Q}{A} = \frac{6.1}{0.685^2(1.495 - \sin 1.495 \cos 1.495)} = \underline{9.16 \text{ m/sec}}$$

Flow of 6.8 m^3/sec

Full bore velocity I-J $= \frac{6.8}{2.81} = 2.4$ m/s

Now J-K is also full bore.

$$\therefore V = \frac{6.8}{\pi \times 0.685^2} = 4.6 \text{ m/s}$$

Check for a lower return period storm say 1 in 10

Calculation of flow velocities in damaged section for 1 in 10 yr storm

Q at present = 3.6 cumecs

Q on restoration = 3.75 cumecs.

The flow which causes the damage is the foul flow from J-K.

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Sheet No. N·625

Project N C R GARNANT

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Section Culvert Analysis

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Subject Nant-y-Gath

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See p. N·623 $Q_{\text{s-u}} = \dfrac{4.439(\theta - \sin\theta\cos\theta)^{5/3}}{\theta^{2/3}}$

Try $\theta = 1.275$ $Q = \dfrac{4.439(1.275 - \sin 1.275 \cos 1.275)^{5/3}}{1.275^{2/3}}$

$= 3.75 \quad \therefore \theta = 1.275$ O.K.

$V = \dfrac{Q}{A} = \dfrac{3.75}{0.685^2(1.275 - \sin 1.275 \cos 1.275)} = \underline{8.0\ \text{m/s}}$

Try $\theta = 1.259$ $Q = \dfrac{4.439(1.259 - \sin 1.259 \cos 1.259)^{5/3}}{1.259^{2/3}}$

$= 3.600 \quad \therefore \theta = 1.259$ O.K.

$V = \dfrac{3.6}{0.685^2(1.259 - \sin 1.259 \cos 1.259)} = \underline{7.93\ \text{m/s}}$

NANT MAEN CULVERT

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Made by A.D.J.	Date 11/12/84	Project N.C.B. Garnant	Sheet No. N-M 1
Checked by Mue.	Date 21/12/84	Contract	Calc. No.
Revised by	Date	Section Culvert analysis	File
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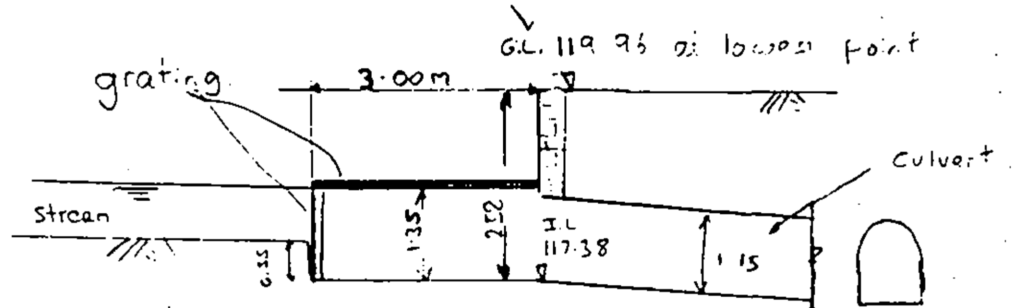
PIPE ENTRANCE.



(width of grating = 2.7m)

Section through culvert entrance

Referring to p. N.G.1. The static head required for a culvert to run full = 1.5 dia.

For this culvert this gives a depth = $1.5 \times 1.15 = 1.725$m

The Maximum Flow Q for 1in100 yr. storm 'on inspection'
$= 2.7\ m^3/sec$.

If an extreme situation the vertical part of the grating becomes largely blocked, the flow into the culvert will be through the top of the grating.

The grating is of size $3.0 \times 2.7\ m^2$.
The bars are of circular section of diameter of 60mm. There is a clear spacing of 125mm between the bars.

The velocity of flow through the bars

$= \frac{Q}{A} = \frac{2.7}{3.0 \times 2.7} = \frac{1}{3.0}$ m/sec

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ref 3. Fig 14.8

Referring the reference 3 Fig 14.8 'Loss coefficients for trash racks'.

$$\frac{\text{Free c.s.A}}{\text{Total c.s.A}} = \left(\frac{105}{105+60}\right) = 0.636$$

head loss $h = \frac{0.97 V^2}{2g} = \frac{0.97 \times (1/3)^2}{2 \times 9.81} = 5.5 \times 10^{-3} m$

h is negligible.

Since h is very small, the entrance should be able to maintain pipe full conditions so long as it can contain 1.5 d = 1.725 m, which it apparently can.

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Made by A.D.J.	Date 12/12/84	Section Culvert analysis	Job No. 31710A
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Revised by	Date	Project N.C.B. GARNANT	Calc. No.
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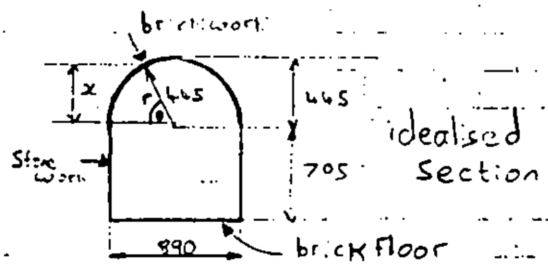
P - Q

length = 12m

n Value for stone/brick-work in good repair = 0.018



Find max. discharge for the above section

Try $x/r = 0.87$ $\theta^c = 1.0552$

$A = 0.705 \times 0.89 + 0.445^2 (1.0552 + \sin 1.0552 \cos 1.0552) = 0.9216$

$P = 0.89 + 2 \times 0.705 + 2 \times 0.445\ \theta^c = 3.239$

$$Q = A \times \left(\frac{A}{P}\right)^{2/3} i^{1/2}/n = \frac{A^{5/3}}{P^{2/3}} \frac{i^{1/2}}{n} = \frac{0.9216^{5/3}}{3.239^{2/3}} \frac{i^{1/2}}{n} = 0.39856 \frac{i^{1/2}}{n}$$

$x/r = 0.86$ $\theta^c = 1.03527$

$A = 0.6275 + 0.198 (1.0352 + \sin 1.0352 \cos 1.0352) = 0.9194$

$P = 2.3 + 0.89 \times 1.0352 = 3.22139$

$$Q = \frac{0.9194^{5/3}}{3.22139^{2/3}} \frac{i^{1/2}}{n} = \underline{\underline{0.39855 \frac{i^{1/2}}{n}}}$$ Maximum Flow

$x/r = 0.85$ $\theta^c = 1.0160$

$A = 0.6275 + 0.198 (1.016 + \sin 1.016 \cos 1.016) = 0.9173$

$P = 2.3 + 0.89 \times 1.016 = 3.204$

$$Q = \frac{0.9173^{5/3}}{3.204^{2/3}} \frac{i^{1/2}}{n} = 0.39847 \frac{i^{1/2}}{n}$$

Max. Q at $x/r = 0.86 \approx 0.399 \frac{i^{1/2}}{n}$

Slope (using slope from Q - m)

$$S = \frac{IL_o - IL_m}{L} = \frac{117.38 - 113.91}{40.5} = 0.086$$

for steady flow $S = i = 0.086$

$$Q = 0.399 \frac{i^{1/2}}{n} = \frac{0.399 \cdot 0.086^{1/2}}{0.018} = \underline{\underline{6.5\ m^3/sec}}$$

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P - M

Section - 900mm Spun Concrete pipe. (some cracked)

The culvert has a 50° bend at one point.

An an allowance is made for the bend

by increasing the 'n'-value

$n = 0.017$

From p G1 $M = 0.579\,r$ $r = 0.45$ $Q = A\dfrac{M^{2/3}\,i^{1/2}}{n}$

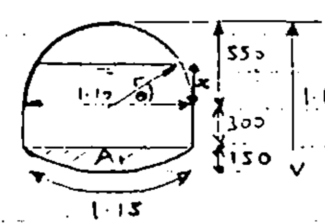
$A = 3.065\,r^2$

see p N.M $i = s = 0.086$

$$Q = \frac{(3.065 \times 0.45^2).(0.579 \times 0.45)^{2/3} \times 0.086^{1/2}}{0.017} = 4.37$$

Max. Flow M-P = $\underline{4.37\ m^3/sec}$

M - L



$A_r = 0.11\,m^2$

idealised section

$A = 0.11 + 1.1 \times 0.3 + r^2\theta^c + r^2 \sin\theta^c \cos\theta^c$

$P = 1.15 + 2 \times 0.3 + 2r\theta^c$

$r = 0.55$

$\therefore A = 0.44 + 0.3025\,(\theta^c + \sin\theta^c \cos\theta^c)$

$P = 1.75 + 1.1\,\theta^c$

$Q = \dfrac{A^{5/3}}{P^{2/3}}\,\dfrac{i^{1/2}}{n}$

Try $x/r = 0.86$ $\theta^c = 1.03527$

$A = 0.44 + 0.3025(1.0352 + \sin 1.0352 \cos 1.0352) = 0.8859$

$P = 1.75 + 1.1 \times 1.0352 = 2.8888$

$$Q = \frac{0.8859^{5/3}}{2.8888^{2/3}}\,\frac{i^{1/2}}{n} = 0.4029\,\frac{i^{1/2}}{n}$$

$x/r = 0.88$ $\theta^c = 1.0759$

$A = 0.44 + 0.3025(1.0759 + \sin 1.0759 \cos 1.0759) = 0.8919$

$P = 1.75 + 1.1 \times 1.0759 = 2.9335$

$$Q = \frac{0.8919^{5/3}}{2.9335^{2/3}}\,\frac{i^{1/2}}{n} = 0.4033\,\frac{i^{1/2}}{n}$$ MAX

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Sheet No. N-MC

Project N.C.R. Gaemont

Calc. No.

Contract

File

Made by ADJ Date 12/12/84

Section Culvert Analysis

Job No. 24710A

Checked by MW Date 21/12/84

Subject Nont-Moon

Revised by Date

Checked by Date

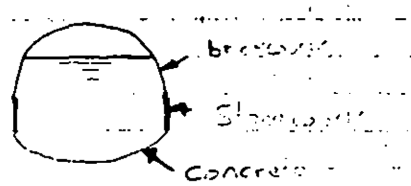
$$x/r = \cancel{0.9}\ 0.89 \qquad \theta^c = \cancel{1.1198}\ 1.0973$$

$$A = 0.44 + 0.3025\,(1.0973 + \sin 1.0973 \cos 1.0973) = 0.8947$$

$$P = 1.75 + 1.1 \times 1.0973 = 2.9570$$

$$Q = \frac{0.8947^{5/3}}{2.9570^{2/3}}\,\frac{i^{1/2}}{n} = \cancel{0.4030}\ 0.4032\,\frac{i^{1/2}}{n}$$

$$Q_{Max} = 0.4033\,\frac{i^{1/2}}{n}$$



n for brickwork / stonework = 0.018

n " concrete = 0.015

$P_1 = 1.15 \quad n_1 = 0.015$

$P_2 = 1.78 \quad n_2 = 0.018$

For 2 p 132

$$n = \left(\frac{1.15 \times 0.015^{1.5} + 1.78 \times 0.018^{1.5}}{2.93}\right)^{2/3} = 0.0168 \simeq 0.017$$

Use $n = 0.017$

gradient s (From M-G)

$$s = \frac{I.L._M - I.L._G}{L} = \frac{113.91 - 112.1}{29.5} = 0.061$$

For steady flow $i = s = 0.061$

$$Q = \frac{0.4033 \times 0.061^{1/2}}{0.017} = \cancel{5.85}\ 5.86\ m^3/sec$$

L - I

The section shown is the smallest that exists. In places the bottom has been gouged out. The culvert also curves through this section.

To allow for the head losses caused by the bends a slightly higher than normal value will be used for 'n'.

$A = 0.508 - 0.269\,(\theta^c - \sin\theta^c \cos\theta^c)$
$P = 2.669 - 1.038\,\theta^c$

Try $x = 200$ $\theta^c = 0.4835$

$A = 0.508 - 0.269\,(0.4835 - \sin 0.4835 \cos 0.4835) = 0.4837$

$P = 2.669 - 1.038 \cdot 0.4835 = 2.168$

$$Q = \frac{0.4837^{5/3}}{2.168^{2/3}} \frac{i^{1/2}}{n} = 0.1810 \frac{i^{1/2}}{n} \ m^3/sec$$

Try $x = 215$ $\theta^c = 0.417$

$A = 0.508 - 0.269\,(0.417 - \sin 0.417 \cos 0.417) = 0.4956$

$P = 2.669 - 1.038 \cdot 0.417 = 2.236$

$$Q = \frac{0.4956^{5/3}}{2.236^{2/3}} \frac{i^{1/2}}{n} = \underline{0.1814 \frac{i^{1/2}}{n} \ m^3/sec} \quad * \text{ Max}$$

$x = 225$ $\theta^c = 0.3667$

$A = 0.508 - 0.269\,(0.3667 - \sin 0.3667 \cos 0.3667) = 0.4994$
$P = 2.669 - 1.038 \cdot 0.3667 = 2.288$

$$Q = \frac{0.4994^{5/3}}{2.288^{2/3}} \frac{i^{1/2}}{n} = 0.1810 \frac{i^{1/2}}{n}$$

$$\underline{Q_{max} = 0.1814 \frac{i^{1/2}}{n} \ m^3/sec}$$

Gradient as before for M-G. $S = 0.061$

$i = S = 0.061$

$n_{stonework} = 0.018$
$n_{concrete} = 0.015$
allowing losses at bends. say $n = 0.018$

$$Q = \frac{0.1814 \cdot 0.061^{1/2}}{0.018} = 2.49 \ m^3/sec$$

$\underline{\text{Max } Q = 2.49 \ m^3/sec}$

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Section Culvert Analysis
Job No. 86710A
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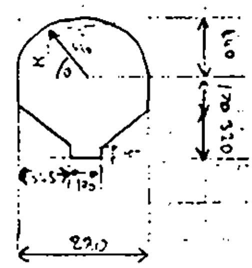
I - H

In the section I-H there are two sections, the flow will be calculated using the smallest.

Find Max. discharge from Section sliscon.

idealised Section.



$A = 0.32 \times 0.19 + 2\left(\frac{0.365 \cdot (0.32 - 0.1)}{2}\right)$
$+ (0.17 \times 0.88) + 0.44^2 \times \theta^c$
$+ 0.44^2 \sin\theta^c \cos\theta^c$

$= 0.2863 + 0.1936(\theta^c + \sin\theta\cos\theta)$

$P = 0.19 + 2 \times 0.1 + 2 \times \sqrt{0.365^2 + 0.22^2} + 2 \times 0.17$
$+ 2 \times 0.44 \times \theta^c$

$= 1.548 + 0.88\theta^c$

Try $x/r = 0.87$ $\therefore \theta^c = 1.0552$

$A = 0.2863 + 0.1936(1.055 + \sin 1.055 \cos 1.055) = 0.5736$
$P = 1.548 + 0.88 \times 1.0552 = 2.4766$

$Q = \frac{A^{5/3}}{P^{2/3}} \times \frac{i^{1/2}}{n} = \frac{0.5736^{5/3}}{2.4766^{2/3}} \times \frac{i^{1/2}}{n} = 0.2163\frac{i^{1/2}}{n}$

Try $x/r = 0.89$ $\theta^c = 1.0973$

$A = 0.2863 + 0.1936(1.0973 + \sin 1.0973^c \cos 1.0973) = 0.5773$
$P = 1.548 + 0.88 \times 1.0973 = 2.5137$

$Q = \frac{0.5773^{5/3}\, i^{1/2}}{2.5137^{2/3}\, n} = \underline{0.2165\frac{i^{1/2}}{n}}$ * Max. Flow

Try $x/r = 0.91$ $\theta^c = 1.1433$

$A = 0.2863 + 0.1936(1.1433 + \sin 1.1433 \cos 1.1433) = 0.5807$
$P = 1.548 + 0.88 \times 1.1433 = 2.5541$

$Q = \frac{0.5807^{5/3}}{2.5541^{2/3}} \frac{i^{1/2}}{n} = 0.2163\frac{i^{1/2}}{n}$

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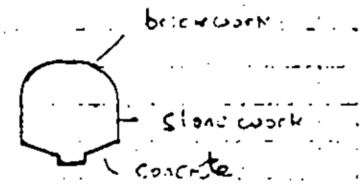
grad. as M-F $S = 0.061$

$i = S = 0.061$



n_2 Stonework / brickwork $= 0.018$ (n_2)
n_1 Concrete $= 0.015$ (n_1)

$$n_{combined} = \left(\frac{n_1^{1.5}P_1 + n_2^{1.5}P_2}{P_1+P_2}\right)^{2/3}$$

$P_1 = 0.191 + 0.609 + 2 \lambda = \frac{+0.2}{1.208}$ ~~1.002~~

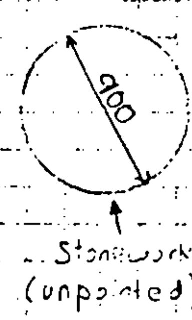
$P_2 = P - P_1 = 2.514 - \cancel{1.002}\ 1.208 = \cancel{1.512}\ 1.306$

$$n = \left(\frac{0.015^{1.5} \times 1.208 + 0.018^{1.5} \times 1.306}{2.514}\right)^{2/3} = \cancel{0.0168}\ 0.0166 \simeq 0.017$$

$$Q = \frac{0.2165 \times 0.061^{1/2}}{0.017} = 3.14\ m^3/sec$$

<u>M-G</u>

900 Circular Stonework (unpointed)



from p G1
$A = 3.065 r^2 = 3.065 \times 0.45^2 = 0.6207$
$M = 0.579 r = 0.579 \times 0.45 = 0.2606$
$n = 0.02$
$S = (\text{as F} - \text{M-G}) = 0.061$

$$Q = \frac{0.6207 \times 0.2606^{2/3} \times 0.061^{1/2}}{0.02} = \underline{\underline{3.13\ m^3/sec}}$$

<u>G-F</u>

Section as above.

There is a step in I.L of 0.43 m just U/s of F.

$$S = \frac{I.L_G - (I.L_F + 0.43)}{L} = \frac{112.13 - (111.49 + 0.43)}{11} = 0.017$$

$$Q = \frac{0.6207 \times 0.2606^{2/3} \times 0.017^{1/2}}{0.02} = \underline{\underline{1.65\ m^3/sec}}$$

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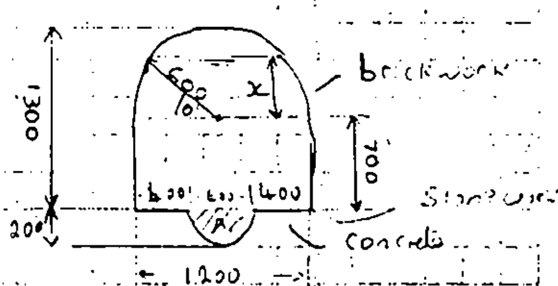
F – E

At F there is an inspection chamber which has a small hole [illegible] the pipe. There is a bend of 20° at F and a step of 430 mm just U/S of F.

If it is (conservatively) assumed for the steady state case that the step compensates for the losses at the bend and manhole.

$$\text{Slope } S = \frac{IL_F - IL_E}{L} = \frac{111.48 - 108.55}{30.6} = 0.0957$$

(The shape of this Channel is similar to that on page N-M2
∴ assume max flow occurs at $x/r = 0.86$)



$$A = \frac{0.2^2\pi}{2} + 0.7 \times 1.2 + 0.6^2 \times \theta^c + 0.6^2 \sin\theta^c \cos\theta^c$$
$$= 0.9028 + 0.36(\theta^c + \sin\theta^c \cos\theta^c)$$

$$P = \pi \times 0.2 + 0.4 + 0.4 + 2 \times 0.7 + 2\theta^c \times 0.6$$
$$= 2.828 + 1.2\theta^c$$

$$\theta^c = \arcsin 0.86 = 1.035$$

$$\therefore A = 0.9028 + 0.36(1.035 + \sin 1.035 \cos 1.035) \quad 1.433$$

$$P = 2.828 + 1.2 \times 1.035 = 4.070$$

$$Q = \frac{A^{5/3}}{P^{2/3}} \frac{S^{1/2}}{n}$$

Use $n = 0.017$ (see previous combination of stonework and concrete).

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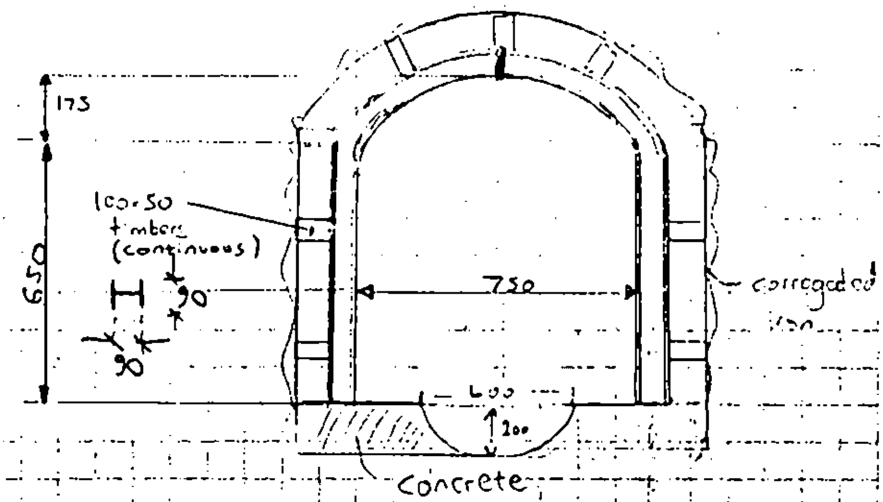
$$Q = \frac{1.423^{5/3}}{4.07^{2/3}} \times \frac{0.0957^{1/2}}{0.017} = \underline{12.8\ M^3/sec}$$

E-D

This section has a number of steel portal frames forming constrictions to the flow. Behind the portals are timbers and corrugated iron sheets which support the original stonework/brickwork.



For this section any assumptions of uniform flow open channel flow are obviously very inadequate. In order to calculate the capacity of this section running without pressure head, it will be assumed that the culvert is running full in this section. A head loss will be calculated for each piece which will be subtracted from the static

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Job No. 8171DA
Checked by: [signature] Date: 21/12/84
Subject: Non - Moc
Revised by: Date:
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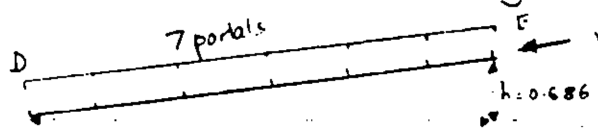
head available for driving the flow



l = 7.17m

S (as for E-D) = 0.0957

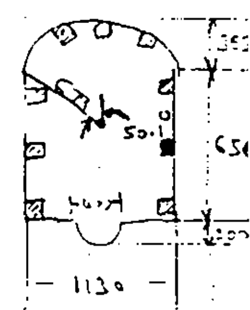
drop in static head from E to D = 0.0957 × 7.17 = 0.686m

Head loss through end constriction = $KV^2/2g$

total head loss due to constrictions = $7KV^2/2g$

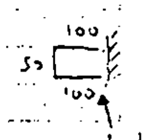
Total head available for driving the flow

$$0.686 - 7\frac{KV^2}{2g}$$



Non - constricted Area $A_N = \frac{0.2^2\pi}{2} + 1.13 \times 0.65$
$+ \left(0.579^2\pi \times \frac{100.2}{360} - 0.314 + 0.565\right) - (0.1 \times 0.05 \times 9)$ timbers
$= 0.978 m^2$

Constricted Area $A_c = A_N - $ (portal area + timbers)
$= 0.978 - \left(2 \times 0.65 \times 0.09 + .534 \times \frac{2\pi \times 100.2}{360} \times 0.09\right)$
portal area
$= 0.978 - 0.201 = 0.777$



Wetted perimeter $P = \pi \times 0.2 + 0.73 + 2 \times 0.65$
$+ 0.679 \times 2 \times \pi \times \frac{100.2}{360} + 9 \times (2 \times 0.1)$ timbers
$= 5.65$

Ref 3
Fig 14.3
p263

Using the loss coefficient for a sharp edged orifice

$$\frac{A_c}{A_N} = \frac{0.777}{0.978} = 0.79 \quad K = 0.38$$

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Project N.C.B. GARNANT
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Section Hydraulic Analysis
Subject Nant - Moel Culvert

Made by A.D.J. Date 13/12/84
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Total head loss due to portals = $\frac{7 \cdot 0.38\, v^2}{2g}$

$h = 0.685 - \frac{7 \cdot 0.38 \cdot v^2}{2g}$

$V = \frac{m^{2/3} i^{1/2}}{n}$ but $i = \frac{h}{l}$ (for this case).

n for corrugated iron $= 0.025 = n_2$
n for concrete $= 0.015 = n_1$
n for wood $= 0.015 = n_3$

$n = \left(\frac{n_1^{1.5} P_1 + n_2^{1.5} P_2 + n_3^{1.5} P_3}{P_1 + P_2 + P_3} \right)^{2/3}$

$P_1 = 1.36$
$P_2 = 2.49$
$P_3 = 1.8$

$n = \left(\frac{0.015^{1.5} \times 1.36 + 0.025^{1.5} \times 2.49 + 0.015^{1.5} \times 1.8}{5.65} \right)^{2/3}$

$= 0.020$

$M = \frac{A}{P} = \frac{0.978}{5.65} = 0.173$ m

$V = \frac{0.173^{2/3} \left(\frac{0.685 - \frac{7 \cdot 0.38 v^2}{2g}}{7.17} \right)^{1/2}}{0.020}$

Squaring gives $V^2 = \frac{0.173^{4/3} \left(\frac{0.685 - 7 \cdot 0.38 v^2 / 2g}{7.17} \right)}{0.020^2}$

$= \frac{0.0964 (0.686 - 0.136 v^2)}{4.0 \times 10^{-4} \times 7.17} = 33.61 (0.686 - 0.135 v^2)$

$= 23.0 - 4.57\, v^2$

$\therefore v^2 + 4.57 v^2 = 23.0 \quad \therefore v^2 = \frac{23.0}{1 + 4.57} = 4.13$

$\therefore$ **V = 2.03 m/sec**

Hence Flow $= VA_N = 2.03 \times 0.978 =$ **1.99 m³/sec**

Max Flow = 1.99 /sec

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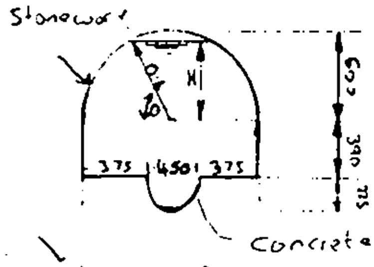
D - C



From Similar Sections Max Flow occurs at about $x/r : 0.88$.
And $n = 0.017$ $\theta^c = 1.0758$

Area A $= \frac{0.225^2\pi}{2} + 1.2 \times 0.39 +$

$0.6^2(1.0758 + \sin 1.0758 \cos 1.0758) = 1.085\ M^2$

$P = 0.225 \times \pi + 2 \times 0.375 + 2 \times 0.39 + 2 \times 1.0758 \times 0.6 = 3.528\ M$

S (as for F-B) $= 0.0957 = i$

$\therefore Q = \frac{1.085^{5/3}}{3.528^{2/3}} \cdot \frac{0.0957^{1/2}}{0.017} = 9.0\ M^3/sec$

C - B

1.2m Spun concrete pipe

There is a step at C down to the pipe of 0.15m

S (without step) $= 0.0957$
l C-B $= 10m$ $\therefore h = 10 \times 0.0957 = .957$
$h - 0.15 = .957 - .15 = .807$

$S = i = \frac{.807}{10.0} = 0.0807$

P 64
$M = 0.579r = 0.347\ M$
$A = 3.065r^2 = 1.103\ M^2$
n (precast concrete pipe) $= 0.013$

$\therefore Q = \frac{A M^{2/3} i^{1/2}}{n} = \frac{1.103 \times 0.347^{2/3} \times 0.0807^{1/2}}{0.013} = \underline{\underline{11.9\ M^3/sec}}$

B - A

Section as above

$i = S = \frac{IL_B - IL_A}{L} = \frac{108.55 - 107.93}{13.2} = 0.047$

$Q = \frac{1.103 \times 0.347^{2/3} \times 0.047^{1/2}}{0.013} = \underline{\underline{9.08\ m^3/sec}}$

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Checked by: mee Date: 21/12/84
Subject: Nant-Moen.
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The preceding calculations have shown that - under assumed uniform flow conditions - certain sections of the culvert have a capacity below the 2.7 m^3/sec flow required (see Fig 4. 'On Restoration graph)

The next calculation works back from the throttle section E-D and calculates the pressure heads required to drive the flow through the culvert. The calculation proceeds to a point where the flow again becomes open-channel.

CALCULATION OF HYDRAULIC HEADS REQUIRED TO DRIVE A FLOW OF 2.7 M^3/sec THROUGH THE CULVERT

E - D

See pages N-M15 - N-M18

Total head available for driving the flow

$$= \text{Pressure head } (h) + \left(0.685 - 7\frac{K v^2}{2g}\right)$$

where $K = 0.38$

$$i = \frac{\left(h + 0.685 - \frac{7 \times 0.38 v^2}{2g}\right)}{7.17}$$

length l : 7.17 m

$$Q = A \times \frac{M^{2/3} i^{1/2}}{n}$$

$M = 0.173$ m
$n = 0.020$
$A = 0.978$ m^2
$V = \frac{Q}{A} = \frac{2.7}{0.978} = 2.76$ m/s

$$i = \left(\frac{Q \times n}{A \times M^{2/3}}\right)^2 = \left(\frac{2.7 \times 0.02}{0.978 \times 0.173^{2/3}}\right)^2 = 0.0316$$

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$\therefore$ Since $i = \dfrac{h + 0.686 - 7 \times 0.38\, V^2/2g}{7.17} = 0.0316$

$h = 0.0316 \times 7.17 - 0.686 + \dfrac{7 \times 0.38 \times 2.76^2}{2 \times 9.81} = 0.573\,m$

F-E

The section E-D which contains the portals is almost the same as the section F-E but without the portals. Since the head loss at exiting portal has been calculated, it is assumed that any loss due to a section change from (F-E) to (E-D) (at E) is accounted for by the losses in the portals.

For $\theta = \pi/2$

See p N-M9 $A = 0.9028 + 0.36(\theta' + \sin\theta\cos\theta) = 1.468$
$P = 2.828 + 1.2\,\theta' = 4.713$

$i = \left(\dfrac{Q \times n \times P^{2/3}}{A^{5/3}}\right)^2$

$n = 0.017$ $\quad i = \left(\dfrac{2.7 \times 0.017 \times 4.713^{2/3}}{1.468^{5/3}}\right)^2 = 0.0046$

pressure head at F h_F = pressure head at E + $i \times l$ − change in static head from E-F
$l_{11} = 7.36\,m$
$= 0.573 + 0.0046 \times 7.36 - 0.0937 \times 7.36 = -0.0975$

$\therefore$ Since h_F is negative the flow has returned to open channel.

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The flow returned to open-channel in the section E-F but another throttle section occurs from G-F. The calculation recommences from point F.

G-F

Pipe full conditions

$A = \pi r^2 = \pi \times 0.45^2 = 0.636\ m^2$

$P = 2\pi r = 2 \times \pi \times 0.45 = 2.827\ m$

see p N-M8 $n = 0.020$

$$Q = \frac{A^{5/3} \times i^{1/2}}{P^{2/3} \times n} \quad \therefore i = \left(\frac{Q \times n \times P^{2/3}}{A^{5/3}}\right)^2$$

$$i = \left(\frac{2.7 \times 0.020 \times 2.827^{2/3}}{0.636^{5/3}}\right)^2 = 0.053$$

Using I.L. at F as datum. (remembering 0.43m step)

$$i = \frac{H_G - I.L_F}{l} = \frac{H_G - (111.48 + 0.43)}{11} = 0.053$$

$\therefore H_G = 0.053 \times 11 + 111.48 + 0.43 = \underline{112.49\ m}$

Pressure head at G = 112.49 - 112.1 = 0.39 m

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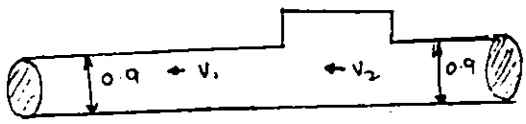
Head loss at junction G

M.H. G



$\longleftarrow V_1 = \dfrac{2.7}{0.9^2 \pi/4}$: 4.244

This manhole is not a full separate construction but is an inspection chamber knocked into the original section. Assuming that the increased area of flow is equivalent to a pipe of diam 1.1 d = 1.1 × 0.9 = 0.99 m.

ref 3. Fig 14.14 & 14.15

$A_2/A_1 = \dfrac{1}{1.1^2} = 0.826$

Exp + contr. loss $K = (0.08 + 0.08) = 0.16$

$\therefore h = \dfrac{K V_{pipe}^2}{2g} = \dfrac{0.16 \times 4.24^2}{2g} = 0.15\,M$

Total head loss = 0.15

H - G

Section as G - F $\therefore i = 0.053$

$i = \dfrac{H_H - (H_G + h_g)}{l} = \dfrac{H_H - (112.49 + 0.15)}{3.6} = 0.053$

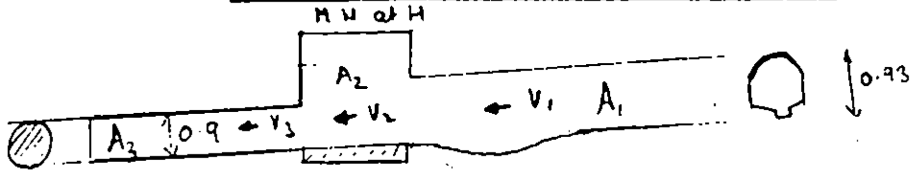
$\therefore H_H = 0.053 \times 3.6 + 112.49 + 0.15 = 112.84$

(See p NM4) gradient M - G = 0.061 $\therefore$ I.L.H = I.L$_G$ + 0.061 × 3.6
= 112.1 + 0.061 × 3.6
112.3

$\therefore$ Pressure head at H = 112.84 - 112.3 = 0.51 M

Head loss at junction H

MH at H



$A_1 =$ (From p NM 7) $0.2863 + 0.1936\,(\theta' + \sin\theta\cos\theta)$ where $\theta = \pi/2$
$= 0.590$

$A_3 = 0.636$.

Assume area of flow in MH = 1 M². 1 [ ] A_2 1

See p 199.201 ref

Enlargement loss $\dfrac{V_2^2}{2g}\left(\dfrac{A_2}{A_1} - 1\right)^2 = \dfrac{2.7^2}{2g}\left(\dfrac{1}{0.59} - 1\right)^2 = 0.18\,M.$

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see p 269 Fig 14.14 ref. 3.

Contraction loss For $\frac{A_3}{A_2} = \frac{0.636}{1}$ $K = 0.24$

$\therefore h = 0.24 \frac{V_3^2}{2g} = \frac{0.24 \times (2.7/0.636)^2}{2g} = 0.22$

$\therefore$ Head loss at H $(h_H) \simeq 0.18 + 0.22 = 0.40$

I - H

Sections shown on page N.M 7.

$A = 0.2863 + 0.1936(\theta^c + \sin\theta\cos\theta)$ For $\theta = \pi/2$ $A = 0.590$

$P = 1.548 + 0.88\,\theta$. For $\theta = \pi/2$ $P = 2.93$

See p N.M 8

$n = 0.017$

$i = \left(\frac{Q \cdot n \cdot P^{2/3}}{A^{5/3}}\right)^2 = \left(\frac{2.7 \times 0.017 \times 2.93^{2/3}}{0.59^{5/3}}\right)^2 = 0.0512$

$i = \frac{H_I - (H_H + h_H)}{L} = \frac{H_I - (112.8 + 0.4)}{6.3} = 0.0512$

$\therefore$ $\underline{H_I = 113.56\ m}$

(see grad. M.G p. NM4)

I.L at I = I L$_H$ + S $\times$ 6.3 = 112.3 + 0.061 $\times$ 6.3 = 112.684

pressure head at I = 113.55 - 112.68 = $\underline{\underline{0.87}}$

L - I

For section see page N.M 5

From N.M 5 $A = A_1 + A_2 + A_3 = 0.0724 + \cancel{0.21}^{\,0.27} + 0.166 - 0.259(\theta^c - \cos\theta\sin\theta^c)$

for $\theta^c = 0$ $A = \cancel{5.1.8\ m^2}$ $0.508\ m^2$

$P = \cancel{2.469}\ 2.669 - 1.038\theta^c = \cancel{2.469}\ 2.669$ M

For the open channel analysis of L - I, a higher than usual n value was taken for the section to allow for the loss at M.H.K and for the bends. A slightly more detailed break down of the losses will now be made

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Loss at bend J . 30° bend

see ref 3. p149. Fig 9.9

For 30° Mitre bend $h = 0.12 \frac{v^2}{2g} = 0.12 \frac{\cancel{6.0}^{2}\ 5.3}{2g} = \cancel{0.222}\ 0.173$ m

$V = \frac{2.7}{\cancel{0.448}\ 0.508} = \cancel{6.0}\ 5.3$ m/sec

head loss at bend J = ~~0.222~~ 0.173 m.

Loss at M.H. K

As with M.H. at G, this M.H. is an inspection cover knocked out of the top of the section. Assuming an increased flow area of 1.2.

ref 3. Fig 14.15 Fig 14.14

Expansion loss for $A_1/A_2 = 1/1.2 = 0.08\ V_1^2/2g$
Contraction loss " " = " $= 0.08\ V_1^2/2g$

Where $V_1 = Q/A_{culvert} = 2.7/\cancel{0.448}\ 0.508 = \cancel{6.02}\ 5.31$ M/sec
head loss $= (0.08 + 0.08)\ \cancel{6.02}^2\ 5.31^2/2g = \cancel{0.29}\ 0.23$ M

Total loss $= \cancel{0.29}\ 0.23 + \cancel{0.222}\ 0.173 = \cancel{0.51}\ 0.40$ m

Using $n = 0.017$ (stonework & concrete)

$$i = \left(\frac{Q \cdot n \cdot P^{2/3}}{A^{5/3}}\right)^2 = \left(\frac{2.7 \times 0.017 \times \cancel{2.469}\ 2.669^{2/3}}{\cancel{0.448}\ 0.508^{5/3}}\right)^2 = \cancel{0.102}\ 0.075$$

$H_L = H_J$ + (loss at bend + loss at M.H$_K$) + $i \times l$
$= 113.55 + \cancel{0.51}\ 0.40 + \cancel{0.102}\ 0.075 \times 14.8 = \cancel{115.57}\ 115.05$

(Calculate pressure head at M.)

L - M.

For section see page N-M 3.

p N-M 3

$A = 0.44 + 0.3025\ (\theta + \sin\theta\cos\theta)$ when $\theta = \pi/2$
$= 0.44 + 0.3025 \cdot \pi/2 = 0.915\ M^2$

$P = 1.75 + 1.1 \cdot \pi/2 = 3.48$ m

$$i = \left(\frac{Q \cdot n \cdot P^{2/3}}{A^{5/3}}\right)^2 = \left(\frac{2.7 \times 0.017 \times 3.48^{2/3}}{0.915^{5/3}}\right)^2 = 0.015$$

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Sheet No. N M 15

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Section Culvert analysis

Job No. 91 710A

Checked by MLC — Date 21/12/84

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Head loss due to section change

at L.

$$V_2 \leftarrow A = \overset{0.508}{\cancel{0.48}} \qquad A_1 = 0.915 \leftarrow V_1$$

ref 3
Fig 16.16
p 269

$$A_2/A_1 : \overset{0.508}{\cancel{0.48}} / 0.915 = 0.56 \therefore \quad K = 0.38.$$

$$h = 0.38 \times \frac{(2.7/\overset{0.508}{\cancel{0.48}})^2}{2 \times g} = \overset{0.55}{\cancel{0.7}} \text{ M}$$

$$h_L = \overset{0.55}{\cancel{0.7}}.$$

$$H_M = i \times L + H_L + h_L = 0.015 \times 49 + \overset{115.05}{\cancel{115.57}} + \overset{0.55}{\cancel{0.7}} = \underline{\underline{\overset{115.67}{\cancel{115.34}}}}$$

$$\text{Pressure head at } M = \overset{115.67}{\cancel{115.34}} - 113.91 = \overset{1.76}{\cancel{2.43}}$$

Since Cover level at M = 115.58 the culvert is at this point overflowing ∴ Inadequate

N.B. The heads calculated (e.g. H_M) are based on an I.L. datum. This means that the excess pressure head is calculated from H_M - I.L. Therefore the actual head at a point M = soffit level + pressure head

---

The estimated flow - under present conditions - is 2.38 M/sec. The above analysis will now be carried out to see if the culvert is adequate in its present condition.

The calculation will not be repeated for the section E-F since this proved adequate for the 2.7 m³/sec flow.

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CALCULATION OF HYDRAULIC HEADS REQUIRED TO DRIVE A FLOW OF 2·38 m^3/sec THROUGH CULVERT

G - F pipe Full conditions

From p. N.M 14 $A = 0{\cdot}636\ m^2$, $P = 2{\cdot}83\ m$

$n = 0{\cdot}020$

$$i = \left(\frac{Q \times n \times P^{2/3}}{A^{5/3}}\right)^2 = \left(\frac{2{\cdot}38 \times 0{\cdot}02 \times 2{\cdot}83^{2/3}}{0{\cdot}636^{5/3}}\right)^2 = 0{\cdot}041$$

Using I.L at F as datum (remembering 0·43 m step)

$H_G = i \times l + H_F + 0{\cdot}43 = 0{\cdot}041 \times 11 + 111{\cdot}48 + 0{\cdot}43 = \underline{112{\cdot}36}$

Pressure head at G = 112·36 − 112·1 = $\underline{0{\cdot}26\ m}$

Head loss at junction G

See p. N.M 15

$h = \dfrac{0{\cdot}16\ V^2}{2g}$ $V = 2{\cdot}38 / 0{\cdot}9^2\pi/4 = 3{\cdot}74\ m/sec$

$\therefore h_G = \dfrac{0{\cdot}16 \times 3{\cdot}74^2}{2 \times 9{\cdot}81} = \underline{0{\cdot}11\ m}$

H - G

Since section as G-F. $i = 0{\cdot}041$

$H_H = (H_G + h_G) + i \times l = 112{\cdot}36 + 0{\cdot}11 + 0{\cdot}041 \times 3{\cdot}6 = \underline{112{\cdot}62\ m}$

From p. N.M 15 I.L H = 112·3.

$\therefore$ pressure head at H = 112·62 − 112·3 = $\underline{0{\cdot}32\ m}$

Head loss at junction H

$V_{in\ M.H.} = Q/A = Q/\infty = 0$

See p NM15 Enlargement loss = $\dfrac{2{\cdot}38^2}{2g}\left(\dfrac{1}{0{\cdot}59} - 1\right)^2 = 0{\cdot}14\ m$

See p NM16 Contraction loss = $\dfrac{0{\cdot}24\ V_3^2}{2g}$ $V_3 = \dfrac{2{\cdot}38}{0{\cdot}636} = 3{\cdot}74$

$= \dfrac{0{\cdot}24 \times 3{\cdot}74^2}{2 \times 9{\cdot}81} = 0{\cdot}17\ m$

Total head loss at H = 0·14 + 0·17 = 0·31 m

$\underline{h_H = 0{\cdot}31\ m}$

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I - H

See p N-M16

$$i = \left(\frac{Q \times 0.017 \times 2.93^{2/3}}{0.59^{5/3}}\right)^2 \quad \text{For } Q = 2.38$$

$$= 0.04.$$

$$\therefore H_I = i \times l + H_H + h_H = 0.04 \times 6.3 + 112.62 + 0.31$$

$$\underline{H_I = 113.18\ M}$$

See p N-M16 pressure head at I = 113.18 − 112.68 = 0.5 M

L - I

See p N-M16 A = ~~0.12~~ 0.508 m^2, P = ~~2.57~~ 2.669 m, n = 0.017

See p N-M17 Loss at bend I = $0.12\frac{V^2}{2g} = \frac{0.12 \times (2.38/\cancel{0.42}\ 0.508)^2}{2 \times 9.81} = \cancel{0.17}\ 0.13\ M$

See p N-M17 Loss at M.H$_H$ = $0.16\frac{V^2}{2g} = \frac{0.16 \times (2.38/\cancel{0.42}\ 0.508)^2}{2 \times 9.81} = \cancel{0.23}\ 0.18\ M$

$$i = \left(\frac{Q\,n\,P^{2/3}}{A^{5/3}}\right)^2 = \left(\frac{2.38 \times 0.017 \times \cancel{2.57}\ 2.67^{2/3}}{\cancel{0.42}\ 0.508^{5/3}}\right)^2 \simeq \cancel{0.08}\ 0.06$$

$$H_L = H_I + \text{loss}_I + \text{loss}_H + i \times l$$

$$= \cancel{113.8}\ 113.18 + (\cancel{0.17}\ 0.13 + \cancel{0.23}\ 0.18) + \cancel{0.08}\ 0.06 \times 16.8 = \cancel{115.38}\ 114.38$$

L - M

See p N-M17 A = 0.915, P = 3.68, n = 0.017

$$i = \left(\frac{Q \cdot n \cdot P^{2/3}}{A^{5/3}}\right)^2 = \left(\frac{2.38 \times 0.017 \times 3.68^{2/3}}{0.915^{5/3}}\right)^2 = 0.0116 \simeq 0.012.$$

Head loss due to section change at L

See p N-M18

$$h = \frac{0.38\,V^2}{2g} = \frac{0.38 \times (2.38/\cancel{0.42}\ 0.508)^2}{2 \times 9.81} = \cancel{0.55}\ 0.43\ m$$

$$\therefore h_L = \cancel{0.55}\ 0.43$$

$$H_M = i \times l + H_L + h_L = 0.012 \times 6.9 + \cancel{115.38}\ 114.38 + \cancel{0.55}\ 0.43 = \cancel{116.0}\ 114.86$$

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Pressure head at M: $\cancel{116.2}$ 114.86 − 113.91 = $\cancel{2.07}$ 0.96

The actual head at M = soffit at M + pressure head at M
= I.L at M + 1.0 + $\cancel{2.07}$ 0.96
= 113.91 + 1.0 + 0.96 = $\cancel{117.0}$ 115.87

C.L at M = 115.58 ∴ Culvert overflowing.

Since the culvert is overflowing at point M with the flow of 2.38 m³/sec, a calculation will be performed to find out what the safe capacity of the culvert is.

Try a flow of 2.0 m³/sec

Calculation of Hydraulic heads required to drive a flow of 2.0 m³/sec through the culvert.

G-F

Pipe Full conditions

See p N.M.14
A = 0.636
P = 2.827
n = $\cancel{0.02}$ 0.02

$$i = \left(\frac{Q \cdot n \cdot P^{2/3}}{A^{5/3}}\right)^2$$

$$i = \left(\frac{2.0 \times 0.02 \times 2.827^{2/3}}{0.636^{5/3}}\right)^2 = 0.029$$

H_G = i × l + I.L.F + 0.43 = 0.029 × 11 + 111.48 + 0.43 = 112.23

Pressure head at G = 112.23 − 112.1 = 0.13 m

Head loss at junction G

See p.N.M.15. Total head loss = $\dfrac{0.16\,V^2}{2g} = \dfrac{0.16 \times (2.0/0.636)^2}{2 \times 9.81} = 0.08$ m

$h_g = 0.08$ m

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H - G

Section as G - F ∴ $i = 0.029$

See p N.M15 $H_H = i \times l + H_G + h_g = 0.029 \times 3.6 + 112.23 + 0.08 = \underline{112.4\ m}$

Pressure head at H = $(112.4 - 112.3) = 0.1\ m$.

Head loss at junction H

See p N.M15 Enlargement loss = $\frac{2.0^2}{2g}\left(\frac{1}{0.59} - 1\right)^2 = 0.098$

See p N.M15 Contraction loss $\frac{0.24 \cdot (2.0/0.636)^2}{2 \cdot g} = 0.121$

Total head loss at H = $0.098 + 0.121 = \underline{\underline{0.22\ m}}$

$h_H = 0.22\ m$

I - H

See p N.M16 $A = 0.590\ m^2$
$P = 2.93\ m$
$n = 0.017$

$i = \left(\frac{Q \cdot n \cdot P^{2/3}}{A^{5/3}}\right)^2 = \left(\frac{2.0 \times 0.017 \times 2.93^{2/3}}{0.59^{5/3}}\right)^2 = 0.028$

See p N.M15 $H_I = i l + H_H + h_H = 0.028 \times 6.3 + 112.4 + 0.22 = 112.80$

See p N.M16 Pressure head at I = $H_I - IL_I$
$= 112.8 - 112.68 = \underline{0.12\ m}$

L - I

From p N.M15 $A = \cancel{0.615}\ 0.508\ m^2$
$P = \cancel{2.483}\ 2.669\ m$
From p N.M16 $n = 0.017$

$i = \left(\frac{Q \cdot n \cdot P^{2/3}}{A^{5/3}}\right)^2 = \left(\frac{2.0 \times 0.017 \times \cancel{2.483}\ 2.669^{2/3}}{\cancel{0.615}\ 0.508^{5/3}}\right)^2 = \cancel{0.056}\ 0.041$

Loss at bend J

See p N.M15 $h_J = 0.12 \frac{V^2}{2g} = \frac{0.12 \times (2.0/\cancel{0.615}\ 0.508)^2}{2 \times 9.81} = \cancel{0.12}\ 0.10\ m$

Loss at M.H. K

$h_K = 0.16\ V^2/2g = \frac{0.16\ (2.0/\cancel{0.615}\ 0.508)^2}{2 \times 9.81} = \cancel{0.16}\ 0.13\ m$

See p N.M17 $H_L = H_I + i \times l + h_J + h_K = 112.8 + \cancel{0.056}\ 0.041 \times 14.8 + \cancel{0.12}\ 0.10 + \cancel{0.16}\ 0.13 = \cancel{113.97}\ \underline{113.64}\ m$

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L-M

See p N-M17

$A = 0.915\ m^2$
$P = 3.48$
$n = 0.017$

$$i = \left(\frac{Q \cdot n \cdot P^{2/3}}{A^{5/3}}\right)^2 = \left(\frac{2.0 \times 0.017 \times 3.48^{2/3}}{0.915^{5/3}}\right)^2 = 0.0082$$

Head loss at L due to change in section

See p N-M18 $h_L = \dfrac{0.38 \times (2.0/0.508)^2}{2 \times 9.81} = \cancel{0.37}\ 0.30\ m$

see p N-M8 $H_M = i \times L + H_L + h_L = 0.0082 \times 4.9 + \cancel{113.91}\ 113.64 + \cancel{0.37}\ 0.30 = \cancel{114.34}\ 113.98\ m$

Pressure head at M $= \cancel{114.34}\ 113.98 - 113.91 = \cancel{0.43}\ 0.07\ m$

See p N-M21 Actual head at M $= 113.91 + 1.0 - \cancel{?}\ 0.07 = \cancel{115.34}\ 114.98\ m$.

C.L at M $= 115.58 > \cancel{115.34}\ 114.98\ m$ ∴ O.K

Head loss at Manhole M

$A = \dfrac{0.9^2 \pi}{4} = 0.636\ m^2$

Say Area of Flow in Manhole $= 1.1 \times 1.1\ m^2 \approx 1.2\ m^2$

$V_1 = 2.0/0.636 = 3.15\ m/sec$
$V_2 = 2.0/1.2 = 1.67\ m/sec$
$V_3 = 2.0/0.914 = 2.19\ m/sec$

ref 3. p 269 Figs. 14.14, 14.15

Fig 14.15 Expansion $\dfrac{A_1}{A_2} = \dfrac{0.636}{1.2} = 0.53$ $K = 0.24$ (based on V_1)

14.14 Contraction $\dfrac{A_2}{A_3} = \dfrac{0.914}{1.2} = 0.762$ $K = 0.11$ (based on V_3)

$$h_M = \frac{0.24 \times 3.15^2}{2 \times 9.81} + \frac{0.11 \times 2.19^2}{2 \times 9.81} = 0.15\ m$$

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P-M

See p N-M3

Section = 900 dia. Concrete pipe.

N.B. In the calculation performed on page N-M 3 the n value was increased to 0.017 to account for the losses in the bends. Now a more detailed calculation will be made..

For a concrete pipe $n = 0.013$

$A = \pi r^2 = \pi \times 0.45^2 = 0.636\ m^2$

$P = 2\pi r = 2\pi \times 0.45 = 2.827\ m$

loss at 50° bend $= \dfrac{0.35V^2}{2g}$

See ref 3. p169 Fig 9.9.

$V = 2.0 / 0.635 = 3.146\ m/sec$ $\therefore$ $h = \dfrac{0.35 \times 3.16^2}{2 \times 9.81} = 0.176\ m$

$h_{50^\circ} = 0.176$

$i = \left(\dfrac{Q \times n \times P^{2/3}}{A^{5/3}} \right)^2 = \left(\dfrac{2.0 \times 0.013 \times 2.827^{2/3}}{0.635^{5/3}} \right)^2 = 0.0122$

$H_P = H_M + h_{50^\circ} + i \times l + h_n = 114.36 + 0.176 + 0.0122 \times 29.2 + 0.15$
$= 115.02\ m$

$IL_P = IL_M + (\text{dist from M-P}) \times \text{Slope}(n\text{-}0)$
$= 113.91 + 29.2 \times 0.086 = \underline{116.4\ M}$

Since $H_P < IL_P$ the flow has returned to open-channel $\therefore$ O.K.

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Catchment Area of Nant-Maen Culvert. 0.44 KM²

1 in 100 yr Flow = 2.36 m^3/sec of which 0.023 = baseflow

Runoff q = 2.334 m^3/sec from 0.44 KM^2

∴ Area of catchment to be diverted to Nant-y-Goth in order to reduce the flow in the Nant-Maen culvert to 2.0 m^3/sec = Ad.

$$Ad = \left(\frac{2.334 - 2.0}{2.334}\right) \times 0.44 = \underline{0.063\,KM^2} \quad (\text{before})$$

which represents 14.3 % of the catchment

For On Restoration situation

Peak Flow = 2.7 − 0.023 = 2.677 m^3/sec

$$Ad = \left(\frac{2.677 - 2.0}{2.334}\right) \times 0.44 = \underline{\cancel{0.253}\ 0.127\,KM^2} \quad * \quad (\text{after})$$

This Represents 28.6 % of the catchment *

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Revised by:	Date:	Section: Culvert analysis	File:
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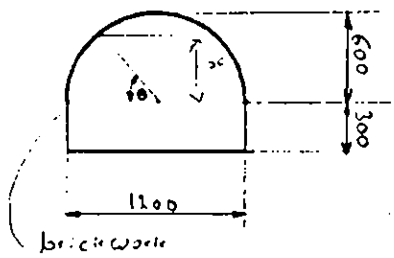
Brick Arch Under road

(Not part of main culvert)

$A = 1.2 \times 0.3 + r^2(\theta + \sin\theta \cos\theta)$

$= 0.36 + 0.36(\theta + \sin\theta\cos\theta)$



$P = 1.2 + 2 \times 0.3 + 2r\theta = 1.8 + 1.2\theta$

$n = 0.02$

(Approx) Max flow at $x/r = 0.86$ $\theta = 1.035$

$A = 0.891\ m^2$

$P = 3.042\ m$

$$i = \frac{I.L.\ u/s - I.L.\ d/s}{l} = \frac{104.68 - 104.36}{11} = 0.029$$

$$Q = \frac{A^{5/3}\, i^{1/2}}{n\, P^{2/3}} = \frac{0.891^{5/3} \cdot 0.029^{1/2}}{0.02 \times 3.042^{2/3}} = \underline{3.35\ m^3/sec}$$

Greater than peak flows therefore O.K.

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Subject Nant-Moor
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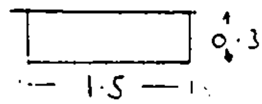
Sleeper Culvert Under Railway



$$i = \frac{IL\ u/s - IL\ d/s}{l}$$

$$= \frac{362.2 - 362.18}{4.5} = 0.0044$$

$n = 0.02$

$A = 1.5 \times 0.3 = 0.45\ m^2$

$P = 3.6\ m$

$$Q = \frac{0.45^{5/3} \times 0.0044^{1/2}}{3.6^{2/3} \times 0.02} = 0.373\ m^3/sec$$

(This area is known to flood)

MINOR CATCHMENT CULVERT

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Culvert Entrance

Referring to the criteria given on p. N-G1.
For a culvert to run full, the head at the entrance above I.L. required = 1.5 × diameter. An examination of the entrance of the minor catchment culvert shows that it could run full.

Pipe capacity

Section 2 No 440 mm diameter spun concrete pipes
length = 11 m and I.L. u/s = 164.96
I.L. d/s = 164.66

From Page G1 for Max. discharge in a circular pipe

$M = 0.579\,r = 0.579 \times .440/2 = 0.127\ m$
$A = 3.065\,r^2 = 3.065 \times \left(\frac{.44}{2}\right)^2 = 0.148\ m^2$
n (spun concrete pipe) = 0.013
$i = \frac{164.96 - 164.66}{11} = 0.027$

$$Q = \frac{A\,M^{2/3}\,i^{1/2}}{n} = \frac{0.148 \times 0.127^{2/3} \times 0.027^{1/2}}{0.013} = 0.47\ m^3/sec$$

Capacity of 2 pipes = 2 × 0.47 = $\underline{\underline{0.94\ m^3/sec}}$

Under Present Conditions the 1 in 100 yr Flow = 0.93 m^3/sec
On Restoration the 1 in 100 yr Flow = 1.0 m^3/sec

Hydraulic head required to drive a flow of 1.0 m^3/sec through the culvert

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Entry loss for each pipe $= \dfrac{0.5\, V_{pipe}^2}{2g} = \dfrac{0.5 \times \left(\dfrac{0.5}{0.22^2\pi}\right)^2}{2 \times 9.81} = 0.276\text{ m}$

Required hydraulic gradient $= i = \left(\dfrac{Q \times n}{A \times M^{2/3}}\right)^2$

A (pipe full) $= \pi r^2 = \pi \times 0.22^2 = 0.152$

$M = \dfrac{\pi r^2}{2\pi r} = \dfrac{r}{2} = 0.22/2 = 0.11$

$$i = \left(\frac{0.5 \times 0.013}{0.152 \times 0.11^{2/3}}\right)^2 = 0.035$$

$S = 0.027$

$\therefore$ additional head (above pipe fall) required to drive flow through pipe $= (i - S) \times l$

$= (0.035 - 0.027) \times 11 = 0.088$

$\therefore$ head above soffit at pipe entrance

$= 0.276 + 0.088 = 0.364\text{ m}.$

The pipe entrance should be able to contain this flow. Therefore culvert is O.K.

N.B. The above analysis assumes that the pipe is clean.

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