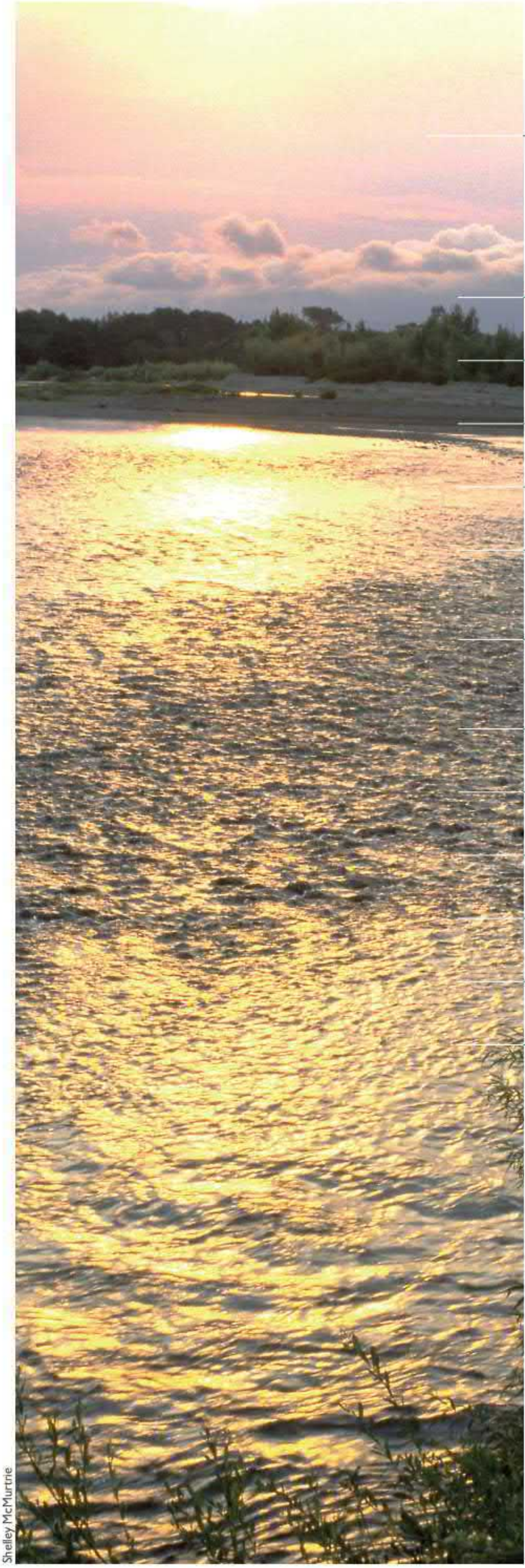


Appendices

Appendix I: Definitions and Useful Numbers

Level Datum

Christchurch Drainage Datum (CDD), formerly known as the Christchurch Drainage Board Datum (CDB Datum), is the reference plane used for drainage purposes in Christchurch. Levels above this plane are stated as 'reduced level' or RL in metres. The Christchurch Drainage Board established this datum in 1878 at 50 feet below the floor of the Cathedral but it has since been redefined as 9.043 m below Lyttelton Vertical Datum (MSL1937).

MSL 1937 is an approximation to mean sea level as determined by short period averaging in 1937. It is used as the level datum for the LINZ Canterbury topographic maps.

MLOS: Mean Level of Sea. Actual mean sea level for a given interval, date and location.

Flows

Litre/second (l/s):	The usual unit for low flows, generally below 1000 l/s.
Cubic metre/second (m^3/s):	Also known as 'cumec'. Used for larger flow calculations.

Pressure and Head

For most purposes, metres head of water is the most useful unit.

1 metre head of water	= 9.806 kPa
1 atmosphere (mean at sea level)	= 10.3 metres water head
	= 101 kPa
	= 1012.8 hPa (for Christchurch)
	(ranges typically from 970 to 1035 hPa)

Standard Tide Levels (2011) to Drainage Datum

Tide State	Scarborough	Ferrymead
Mean Low Water (MLW)	8.30	
Mean Level of Sea (MLOS)	9.15	
Mean High Water (MHW)	10.05	
Pragmatic MHWS*	10.16	10.13
Perigean Mean High Water**	10.37	

* Defined as the 12 percentile exceedence high tide (Goring 1998).

**Perigean is the lunar closest approach to earth = 27.55 day period

Highest recorded tide level at Pages Road was RL10.95 in 1992. Highest recorded tide level at Ferrymead was RL10.81 in 1978, based on records beginning 1968.

Sea Level Rise

International research has determined that the global average rate of sea level rise (SLR) over the last century has been 1.5 mm per year and the current (2011) rate is about 3mm per year (Wikipedia, 2011).

CCC has adopted the current (2011) Ministry for the Environment recommendation to plan for a rise of 0.5m by the year 2100 and check the sensitivity of 0.8m rise by 2100. Recent research however suggests that the median expected sea level rise by 2100 may be 1.0m. For very long term assets such as greenfield subdivisions, or other new major infrastructure or cultural assets, a risk management approach could be adopted which could require consideration of an even higher sea level rise.

An approximate estimate of sea level rise from the datum year of 2000 to any future year can be determined from the formula:

$$\text{Rise (mm)} = 1.5 \times \text{Yrs} + K \times \text{Yrs}^2 \qquad \text{Eqn (App 1-1)}$$

where Yrs = current year – 2000

and K = 0.035 for a 500 mm rise by 2100

or K = 0.065 for an 800 mm rise by 2100

Extreme tide levels

A set of 192 hour tidal sequences in terms of datum year 2011 have been produced for Sumner, Ferrymead, Bridge St, and the Styx for various AEP (Goring, 2011). These tidal sequences are designed to be used for computer catchment model tidal boundaries but they also provide an extreme level in the sequence of the corresponding annual probability. These are summarised in the table below. Levels include tide, storm surge, annual cycle and residual MLOS. To incorporate sea level rise add on the predicted SLR value relative to 2011.

Location	MLOS 2007	AEP% 50%	20%	10%	5%	2%	1%	0.5%
Sumner	9.153	10.631	10.702	10.745	10.782	10.826	10.856	10.889
Ferrymead	9.332	10.632	10.718	10.768	10.811	10.861	10.894	10.924
Bridge St	9.363	10.688	10.780	10.829	10.869	10.910	10.936	10.958
Styx	9.346	10.705	10.790	10.846	10.896	10.965	11.014	11.056

Tidal Cycle Water Levels

The following formula can be used to approximate tidal cycle water levels over a time period. Note that in reality, attenuation of the low tide levels takes place inland from the coast due to interference by the channel bed.

$$WL = MWL + A.\sin(\omega(t + t_0)) \qquad \text{Eqn (App 1-2)}$$

where MWL = mean water level

A = half amplitude

ω = $2\pi/T$

T = tidal period

= 12.421 hours average

t = time (hours)

t_0 = MWL time origin

River Flood Water Levels

Computer models of the major catchments in Christchurch have been developed by the CCC. Computed flood levels and flood extents in the major rivers for common scenarios may be available on request. Special scenarios or adaptations of the models may be run on a cost recovery basis.

References

Goring, D. 1998. MHWS in the Christchurch region: technical aspects. NIWA client report No.CHC 98/6. National Institute of Water and Atmospheric Research (NIWA), Christchurch.

Goring, D. 2011. Extreme tide profile spreadsheet collection: Sumner, Ferrymead, Bridge St, Styx. Excel spreadsheets (2).

Wikipedia, 2011. "Current sea level rise". Accessed on 28 November 2011. http://en.wikipedia.org/wiki/Current_sea_level_rise

Appendix 2: Financial Comparison of Costs

References in this appendix are to Lu (1969). Two types of comparison predominate as useful decision-making aids. The first type compares two ways of achieving the same benefit, and the second type compares schemes with variable benefits and varying costs. These kind of economic comparisons are usually reserved for large projects such as expensive flood protection schemes, pumping stations, or dams, where projects are staged over several years.

In making comparisons, it is often necessary to compare cashflows at different times, and these are usually compared by means of a discount rate, interest rate, or internal rate of return.

Type 1

Compare two (or more) ways of achieving the same end, when the costs occur at different times:

(a) The simple way is to assume an appropriate interest rate (say between 3 and 10% real) and convert all costs to present value. The present values of each option are then compared.

(b) A more sophisticated method is to determine the discount rate at which the present value of two alternatives become equal. This can be done by trial and error, graphically, or with programmable calculators. This discount rate represents the marginal internal rate of return of the scheme involving the higher early (capital) expenditure.

Type 2

Compare options with different costs and different benefits occurring at different times:

(a) The simple way is to convert the cashflows to present values using an assumed discount rate, and compare either the benefit/cost ratio or the difference between benefits and costs.

(b) The internal rate of return for each option is that which makes the present value of costs of benefits equal for a given i:

$$\sum PV_{benefit(i)} - \sum PV_{cost(i)} = 0 \qquad \text{Eqn (App 2-1)}$$

Again this is done by a trial and error procedure.

The internal rates of return of the options can then be compared and, everything else being equal, the options can be ranked.

Formulae

(a) Single Payment

$$PV = S_n(1+i)^{-n} \qquad \text{Eqn (App 2-2)}$$

Refer to Lu (1969), Appendix A: SPPWF

(b) Series from Year 1 To Year n

$$PV = \frac{A(1-(1+i)^{-n})}{i} \qquad \text{Eqn (App 2-3)}$$

(c) Series from Year n to Year m

$$PV = A(1+i)^{-n}\left(1+\frac{1-(1+i)^{n-m}}{i}\right) \qquad \text{Eqn (App 2-4)}$$

if $i = 0$ then use the following equation:

$$PV = A(m - n + 1) \qquad \text{Eqn (App 2-5)}$$

where i = real interest/discount rate

PV = present value

A = annual amount (at end of year)

S_n = single payment after n years

n = number of years from now to start

m = number of years from now to end

Inflation

The usual method of allowing for inflation is to assume that all items will inflate at the same rate, and base all calculations on current costs. The corresponding discounting (interest) rates should be real.

real interest rate = apparent interest rate - inflation rate   Eqn (App 2-6)

Therefore if the apparent interest rate is 15% and the inflation rate is 10%, then the real interest rate is 5%.

Historically, the real long-term interest rates have been more stable than the apparent interest rates. Although instantaneous real interest rates can sometimes appear to be negative, for market technical reasons, it is usually sound to assume that minimum real returns should lie between 3% and 10% in the long-term.

References

Lu, F. P. S. 1969. *Economic Decision-making for Engineers and Managers.* Whitcoulls, Christchurch.

Appendix 3: Drawing Standards

General Drawing Guidelines

Drawings should be produced on 'A' format to microfilming standard as follows:

Drawing Size	Minimum Letter Height Using Capitals
A2, A3, A4	2.5 mm
A1	3.5 mm

Follow these guidelines for drawings:

- Use upper case letters with adequate spacing, and also adequate space inside enclosed letters. Condensed or extended lettering styles should be avoided.
- Space between lines should be a least half letter height.
- Line thickness on A1 and A2 drawings should be at least 0.25 mm.
- Clear space between parallel lines should be at least 0.75 mm and fine hatching should be avoided.
- Normally plans should be forwarded as full size prints to allow 'as built' plans to be prepared prior to microfilming or digital scanning for record purposes.

Standard scales to be used are as follows:

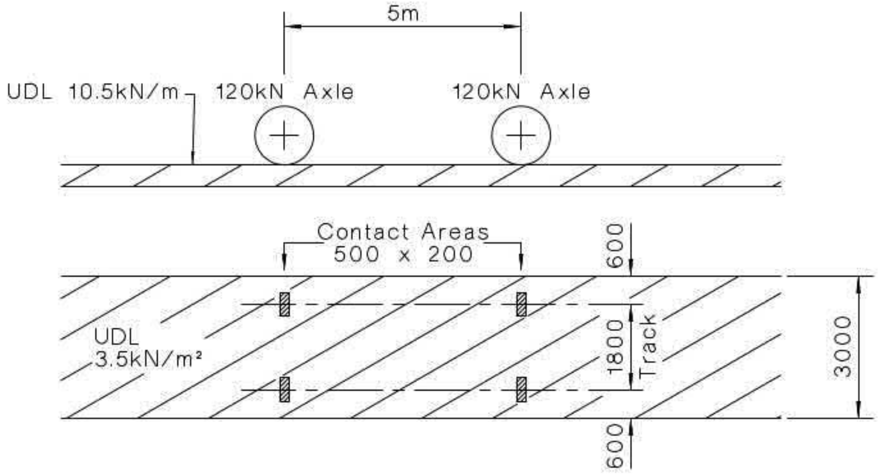
	Scales
Plan	1:200, 1:500, or 1:1000
Longsection	Horizontally: 1:200, 1:500, or 1:1000 Vertically: 1:20, 1:50, or 1:100
Details	1:2, 1:5, 1:10, 1:20, or 1:50

Engineering Drawings

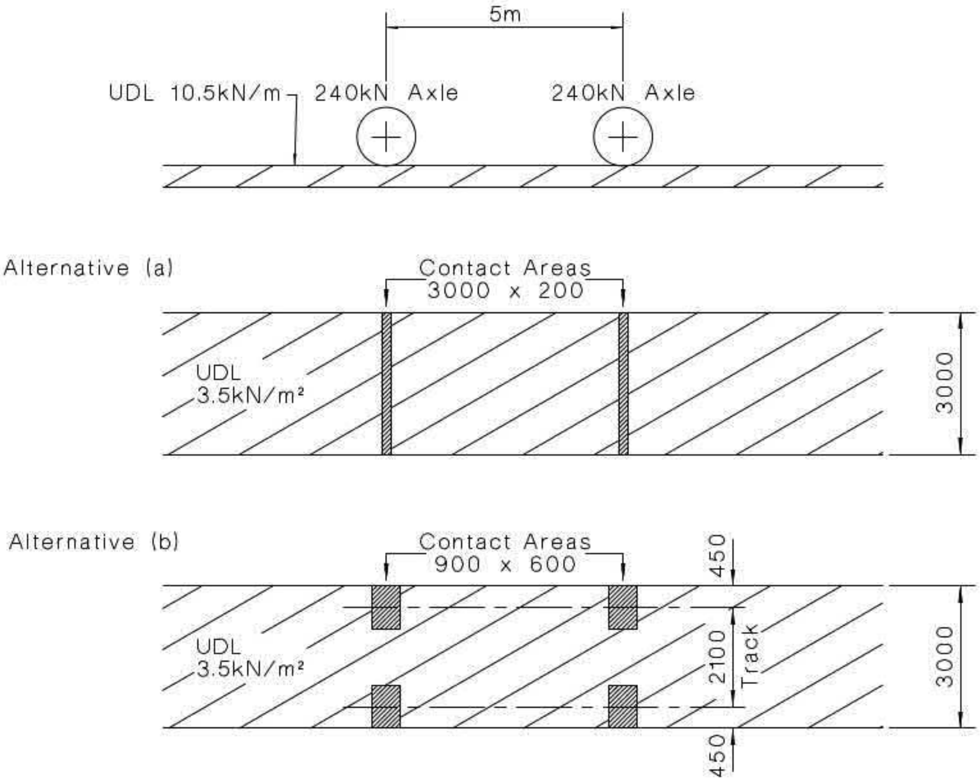
Engineering drawings should show the following:

- positions and offsets of pipelines
- diameter, type, and class of pipe
- pipe bedding type M, C, or H
- longitudinal section including invert levels, gradients, ground levels, manholes, sumps, and existing services
- location of manholes, sumps, etc
- bench marks - maximum spacing 650 m
- north point and locality diagram
- origin of levels
- services legend.

Appendix 4: Traffic Loading HN-HO-72

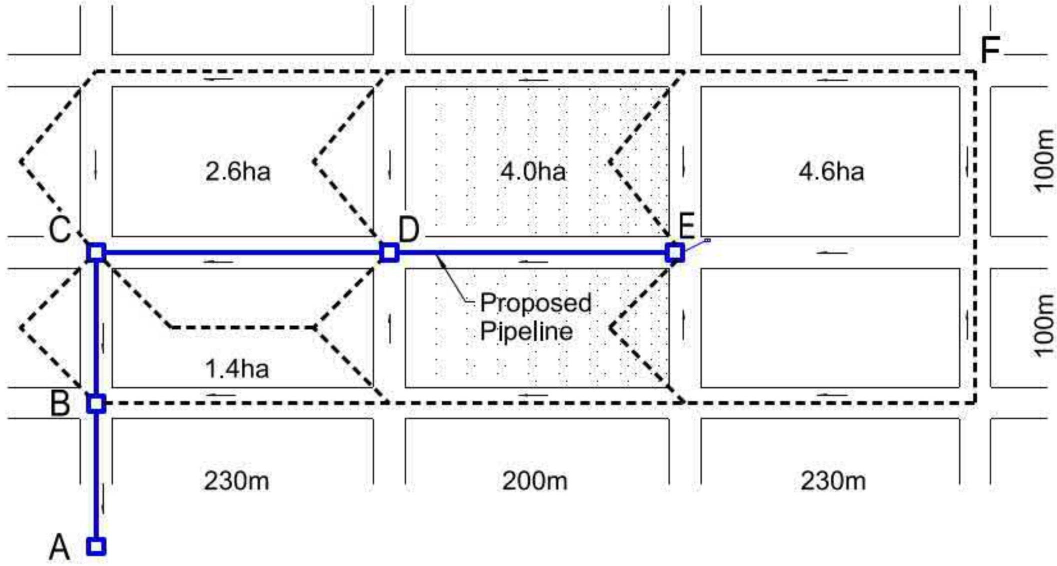


HN Load Element (Normal)



HO Load Element (Overweight)

Appendix 5: Typical Small Urban Catchment Calculation

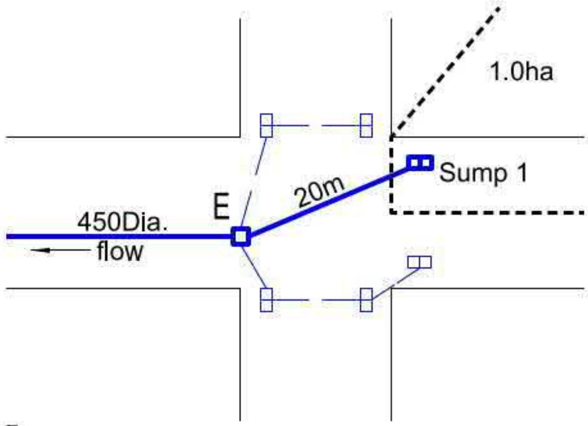


KEY

Industrial Area (Zone B)

Remaining residential (Zone L1)

Catchment boundaries



For roading stormwater design AEP = 20%

Assume sump 1 relieves 1.0 ha of total catchment E.
Tc = 33 minutes (Ex Appendix 5.1, MH E)
$i_{20\%}$ = 22.5 mm/hr (Ex Appendix 10)

Check pipe connection MH E to sump 1:

Q = 2.78 C i A
= 2.78 x 0.38 x 22.5 x 1.0
= 23.8 l/s

Try a 225 mm diameter pipe:

Sf = 0.0030 (Ex Appendix 11)
hf = PipeLength x Sf
= 20 m x 0.0030 = 0.060 m

v = 0.60 m/s (Ex Appendix 11)
hv = $v^2/2g$ = 0.018 m

Pipe entry loss = 0.5 $v^2/2g$ = 0.009 m

Grating loss = $Q^2/30{,}000$
= $23.8^2/30{,}000$
= 0.019 m

TOTAL LOSS = 0.088 m

Water level at MHE = RL 50.49 (Ex Appendix 5.1)
So water level in side channel = 50.49 + 0.09
if sump is drowned = RL 50.58

App 5.1 Backwater Calculation

Nodes and areas here relate to the catchment plan shown in Appendix 5.

Node Name	Segment Length (m)	Dist (m)	Area (ha)	*1 Land Zone	*2 Runoff Coeft	C.A (ha)	ΣC.A (ha)	*3 Segment Flow Time (min)	*4 Time of Conc (min)	*5 Rainfall Intensity (mm/h)	*6 Design Flow (l/s)	Pipe Size (mm)	*7 Full Pipe Vel (m/s)	Vel. Head (m)	*8 Friction Slope (per 1000)	*9 k	*10 Friction Head Loss (m)	*11 Node Head Loss (m)	Water Level (m)	Total Energy (m)	Remarks
	L	*D*	*A*		*C*			*Ts*	*Tc*	*i*	*Q*	*ID*	*V*	*Hv*	*Sf*		*HLf*	*HLn*	*WL*	*TE*	
MH F		960					Te=	**15**													
	330							**+18**													
MH E		630	4.6	L1	0.38	1.75			33	22.5						0.5		0.012	50.494	50.519	
	200						1.75	**+5**			109	450	**0.69**	0.024	**0.51**		**0.102**				
MH D		430	4.0	B	0.73	2.92			38	21.0									50.380	50.405	
	230						4.67	**+4**			273	600	**0.96**	0.047	**0.96**		**0.221**				
MH C		200	2.6	L1	0.38	0.99			42	19.9						0.9		0.043	50.160	50.207	90° bend
	100						5.66	**+2**			313	675	**0.87**	0.039	**0.71**		**0.071**				
MH B		100	1.4	L1	0.38	0.53			44	19.5									50.046	50.085	
	100						6.19				335	750	**0.76**	0.029	**0.46**		**0.046**				
MH A		0															**Start Water Level =**		50.000	50.029	
	Area Sum		**12.6**	**ha**																	

Notes:

*1 Ex District Plan Maps

*2 Ex Chapter 21, Table 21-5

*3 Time of Entry (Te) (ex C1.21.2.3.3)
Segment FlowTime (Ts) = Segment Lgth/velocity (=L/v)
May require iteration
Begin with 0.3m/s for surface flow
and 0.7m/s for pipe flow (See Table 21-3)

*4 Tc = Sum of flow times to this node

*5 Obtain Rainfall Intensity (i) from App 10 for AEP and Tc

6 Flow (Q) = 2.78 i * Σ(CA) (Eqn 21-1)

*7 Velocity ex App 11

*8 Friction Slope (Sf) ex App 11

*9 k = entry, grating, bend or exit loss factor

*10 Friction Head Loss (HLf) = Sf x L

*11 Node head loss = $kV^2/2g$

Design Storm AEP: 20%

File: DES 001

Eng: JLW

Date: 07/2011

APPENDIX 5

Appendix 6: Soakage and Permeability Field Test Methods

App 6.1 Soakage Pit Soakage Test

Field testing to assess the viability of a soakage pit may be carried out as follows (after E1/VM1):

(a) Bore test holes of 100–150 mm diameter to the depth of the proposed soak pit. If groundwater is encountered in the bore test hole then this depth shall be taken as the depth of the soak pit.

(b) Fill the hole with water and maintain full for at least 4 hours (unless the soakage is so great that the hole completely drains in a short time).

(c) Fill the hole with water to within 750 mm of ground level, and record the drop in water level against time, at intervals of no greater than 30 minutes, until the hole is almost empty, or over 4 hours, whichever is the shortest.

(d) Plot the drop in water level against time on a graph; the soakage rate in mm/hr is determined from the minimum slope of the curve. If there is a marked decrease in soakage rate as the hole becomes nearly empty, the lower rates may be discarded and the value closer to the average can be adopted.

App 6.2 Double Ring Infiltrometer Test Procedure

Equipment	Diameter (mm)	Length (mm)	Thickness (mm)	Finish
Measuring cylinder	300	300 - 500	2*	smooth
Buffer cylinder	500	200		
Driving plate	400		12	**

*A greater thickness may be used if a ground cutting-edge is provided. **Lugs on the underside, to centre the driving plate onto the measuring cylinder.

Tamping Hammer

Suitable for driving the measuring cylinder. Recommended weight 10 kg.

Water Supply

Sufficient for the full test procedure.

Measuring Device

Hook gauge or manometer or automatic flow/stage recorder.

Method

(i) Select a site which is representative of the general soil in the area of the test.

(ii) Press the measuring cylinder into the soil using the driving plate and hammer, as required, to drive the cylinder vertically into the soil to a depth of approximately 100 mm. Note: irregular driving (side to side penetration) will lead to poor bonding between the cylinder wall and the soil.

(iii) Press the buffer cylinder into the soil to a depth of 50 to 100 mm and approximately concentric with the measuring cylinder.

(iv) Fill the buffer cylinder to approximately 50 mm depth and maintain at least 25 mm throughout the test.

(v) Protect the surface of the soil at the bottom of the measuring cylinder with a piece of cloth, then fill the cylinder with water to a depth of approximately 75 mm.

(vi) Having removed the soil-protecting cloth, make a measurement of the water surface elevation using a hook gauge (or manometer or automatic flow/stage recorder). Record the elevation and note the time.

(vii) Make additional hook gauge measurements at intervals (typically at 1, 3, 5, 10, 20, 30, 45, 60, 90, 130 minutes and hourly thereafter). The intervals shall be such that the water level does not fall more than 25 mm between successive measurements. Continue until the rate of intake to the soil is almost constant.

(viii) When the water level in the measuring cylinder has dropped by 25–50 mm, then add sufficient water to return the water surface to (approximately) its initial elevation. Record the elevation just before, and again just after filling. This time interval should be as short as practicable. The assumption made in the theory is that the refilling is instantaneous.

Alternatively a constant head device incorporating a flow measuring device or automatic flow-stage recorder may be substituted.

App 6.3 Inverse Augerhole Method

The 'Inverse Augerhole' test method is useful for vertical soakage shaft permeability testing because it is simple, cheap, quick, provides deep permeabilities (so is more realistic than shallow test methods), and because it is of sufficient accuracy.

Permeability is determined by flooding an augerhole and measuring the water level drop over a period of up to 60 minutes. This time period allows initial wetting of the ground, which leads to permeability reducing from an initial value to more stable value (Figure App 6-1). Design permeability is taken as the 60 minute figure (termed k60).

Translation of the site measurement of falling water level to a permeability 'k' figure makes use of the 'Inverse Augerhole Equation' (Eqn App 6-1), or the modified version for use with high water tables (Eqn App 6-2).

These formulae are:

$$k = \frac{1.15 r_0}{t_2 - t_1} \log_{10}\left(\frac{h_1 + r_0/2}{h_2 + r_0/2}\right) \qquad \text{Eqn (App 6-1)}$$

$$k = \frac{1.15\ r_0}{t_2 - t_1} \log_{10}\left(\frac{h_1}{h_2}\right) \qquad \text{Eqn (App 6-2)}$$

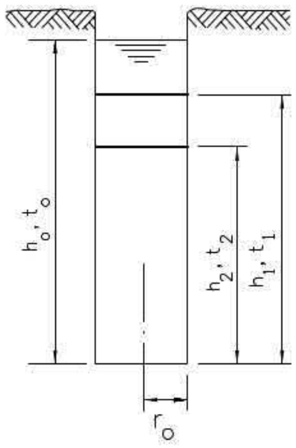


Figure App 6-1A: Inverse augerhole method, with a low watertable.

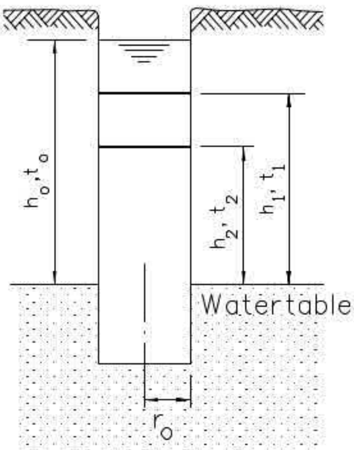


Figure App 6-1B: Inverse augerhole method, with a high watertable.

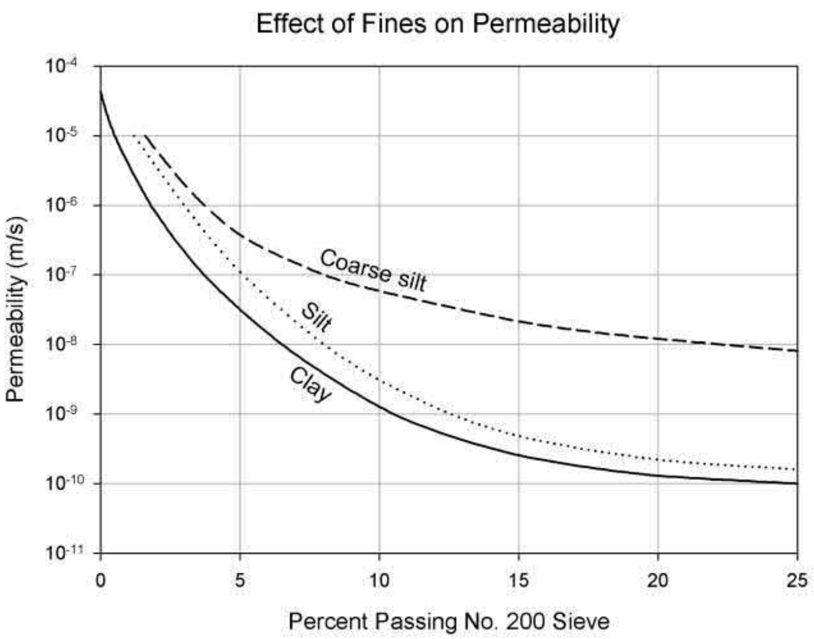
App 6.4 Soil Permeability

The following table gives indicative ranges of soil permeability.

Soil Type	Minimum (m/s)	Maximum (m/s)
Silt, loess	10^{-9}	10^{-5}
Silty sand	10^{-7}	10^{-3}
Clean sand	$5x10^{-6}$	10^{-2}
Clean gravel	10^{-3}	1

The great range in permeability shown in the above table (up to 10^{4}) for various soil types relates to the particle grading variability that can occur for that soil. Maximum permeability occurs in uniformly graded soils with low consolidation and lack of finer particles. Minimum permeability occurs in well-graded soils, high in fine particles and well consolidated.

The figure below shows a graphed example of the effect of fines on permeability.



Appendix 7: Particle Settling Velocity

The calculations for settling velocities (V_s) versus particle diameter in the table below are based on the following values for water at 15 °C:

viscosity = 0.00114 Pa·s

water density, ρ_W = 999 kg/m^3

The following conditions also apply:

1) Effective specific gravity (SG): Decrease in SG with smaller size relates to increasingly non spherical shape and ragged nature of finer particles.
2) Settling velocity (V_s): Settling velocities $\geq$ 0.0040 m/s are from the Heywood tables (Heywood 1962). Settling velocities < 0.0040 m/s are from Stokes Law.
3) Typical sedimentation efficiency: Decrease with smaller size is due to turbulence arising from thermal effects, wind, and flow.

Site conditions may result in a considerable decrease in the efficiency percentage shown. Under very still conditions and uniform flow without short-circuiting, the achievable particle settling size equates to settling velocity:

$$V_s = Q/A_s$$

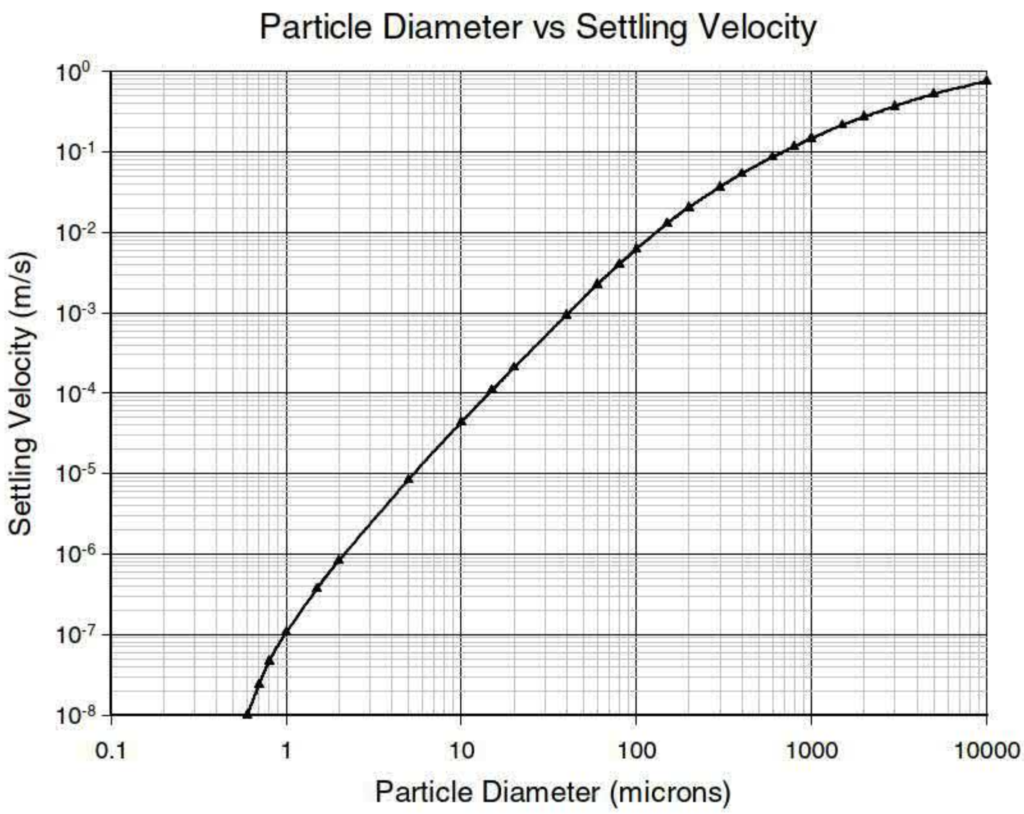
where: Q = through flow (m^3/s)

A_s = pond surface area (m^2)

Diameter (microns)	Diameter (mm)	Effective Specfic Gravity (SG)[1]	V_s (m/s)[2]	Typical Sedimentation Efficiency[3] (%)
10000	10	2.65	0.75	
5000	5.0	2.65	0.52	
3000	3.0	2.65	0.37	
2000	2.0	2.62	0.27	
1500	1.5	2.60	0.21	
1000	1.0	2.58	0.15	
800	0.8	2.56	0.12	
600	0.6	2.55	0.085	
400	0.4	2.53	0.053	
300	0.30	2.50	0.036	
200	0.20	2.46	0.020	100
150	0.15	2.43	0.013	97
100	0.10	2.38	0.0061	95
80	0.08	2.35	0.0040	94
60		2.30	0.0022	92
40		2.22	9.3×10^{-4}	90
20		2.07	2.1×10^{-4}	84
15		2.00	1.1×10^{-5}	80
10		1.90	4.3×10^{-5}	76
5		1.70	8.4×10^{-6}	70
2		1.43	8.2×10^{-7}	65
1		1.22	1.1×10^{-7}	62
0.7		1.10	2.4×10^{-8}	60

Shaded values are from Lawrence & Breen (1998). 1, 2, 3 refer to the numbered notes in the text above.

The figure below has been generated from the table of settling velocities versus particle diameter on the preceding page.

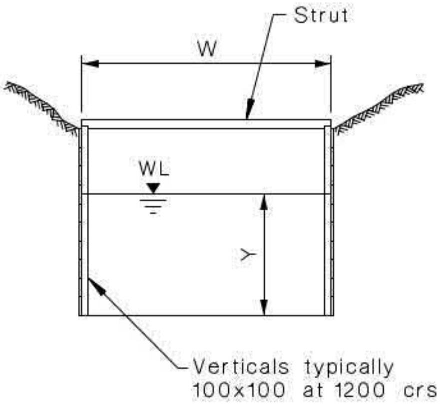


References

Heywood, H. 1962. Uniform and Non-uniform Motion of Particles in Fluids. Proceedings of the Symposium on the Interaction Between Fluids and Particles, IChemE.

Lawrence, I. & Breen, P. F. 1998. *Design Guidelines: Stormwater Pollution Control Ponds and Wetlands.* First Edition. Cooperative Research Centre (CRC) for Freshwater Ecology, Canberra.

Appendix 8: Lined Drain Flow Nomograph



This nomograph is for determining the capacity of existing timber-lined drains. The construction of new timber-lined drains is no longer considered acceptable.

This nomograph applies when water level is below the top strut.

Discharge is given by:

$$Q = 8.1(y-0.06)W^{2} S^{1/2} \qquad \text{Eqn (App 8-1)}$$

Where Q = Discharge (m^2/s)

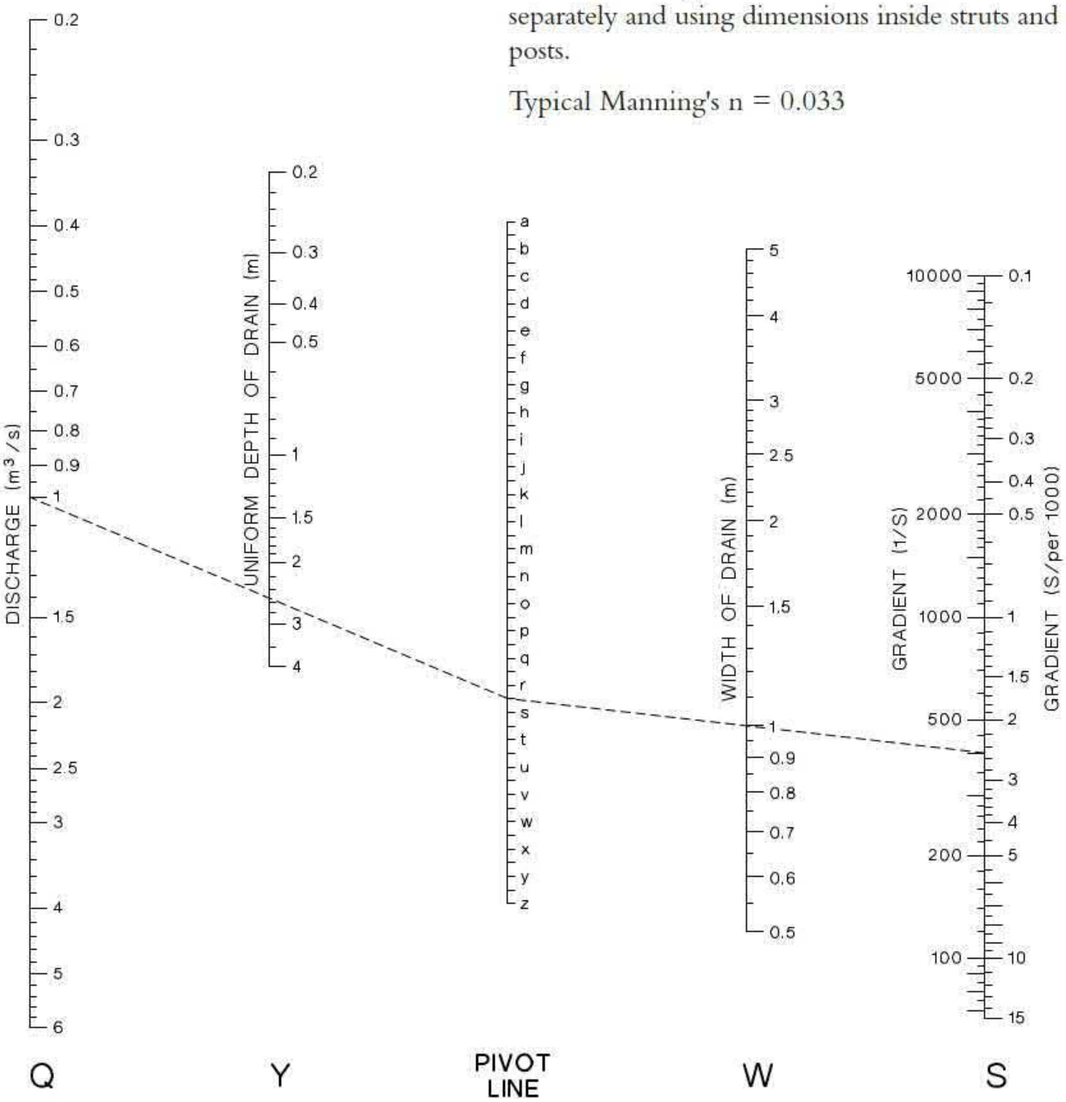
Y = Uniform depth (m)

W = Width outside verticals (m)

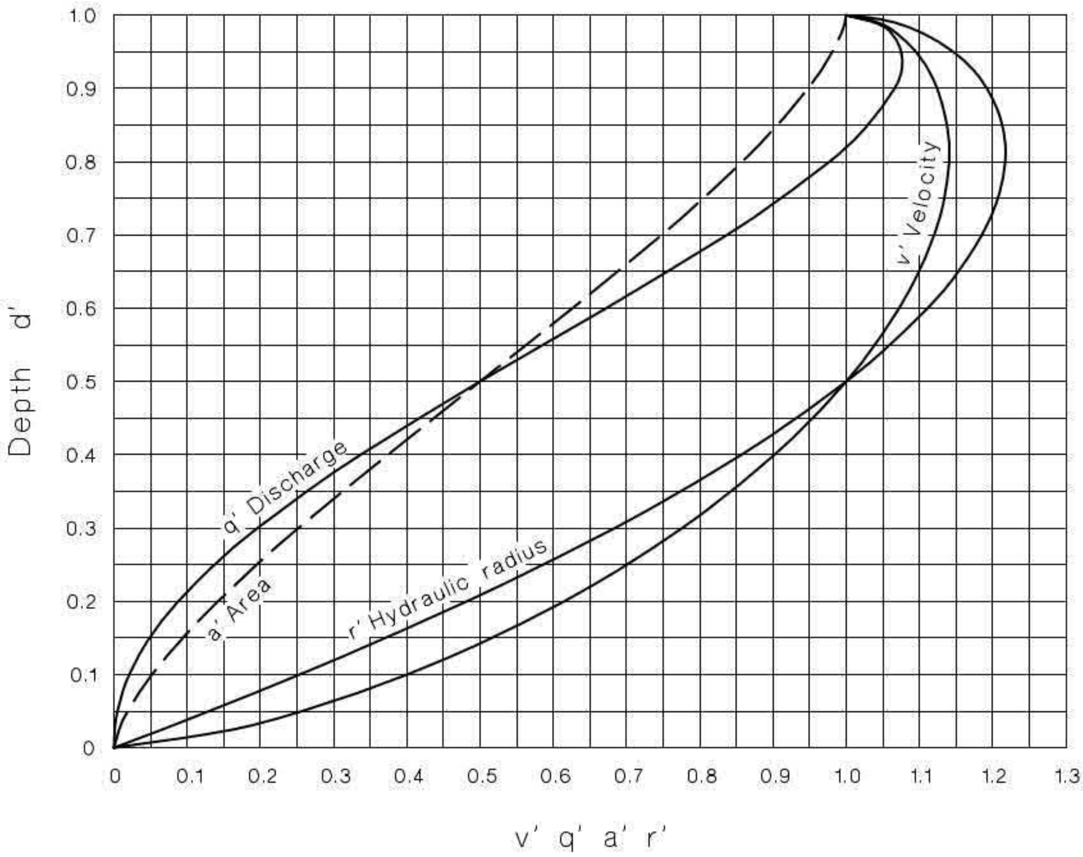
S = Hydraulic gradient (dimensionless)

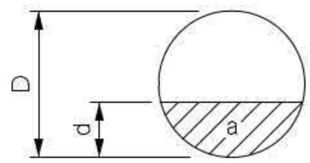
When water level reaches strut, use Manning's formula, treating above and below struts separately and using dimensions inside struts and posts.

Typical Manning's n = 0.033



Appendix 9: Part Full Pipe Flow Nomograph





$$d' = \frac{d}{D} = \frac{\text{Actual depth}}{\text{Pipe diameter}}$$

$$v' = \frac{v}{V} = \frac{\text{Actual velocity}}{\text{Full pipe velocity}}$$

$$q' = \frac{q}{Q} = \frac{\text{Actual flow}}{\text{Full pipe flow}}$$

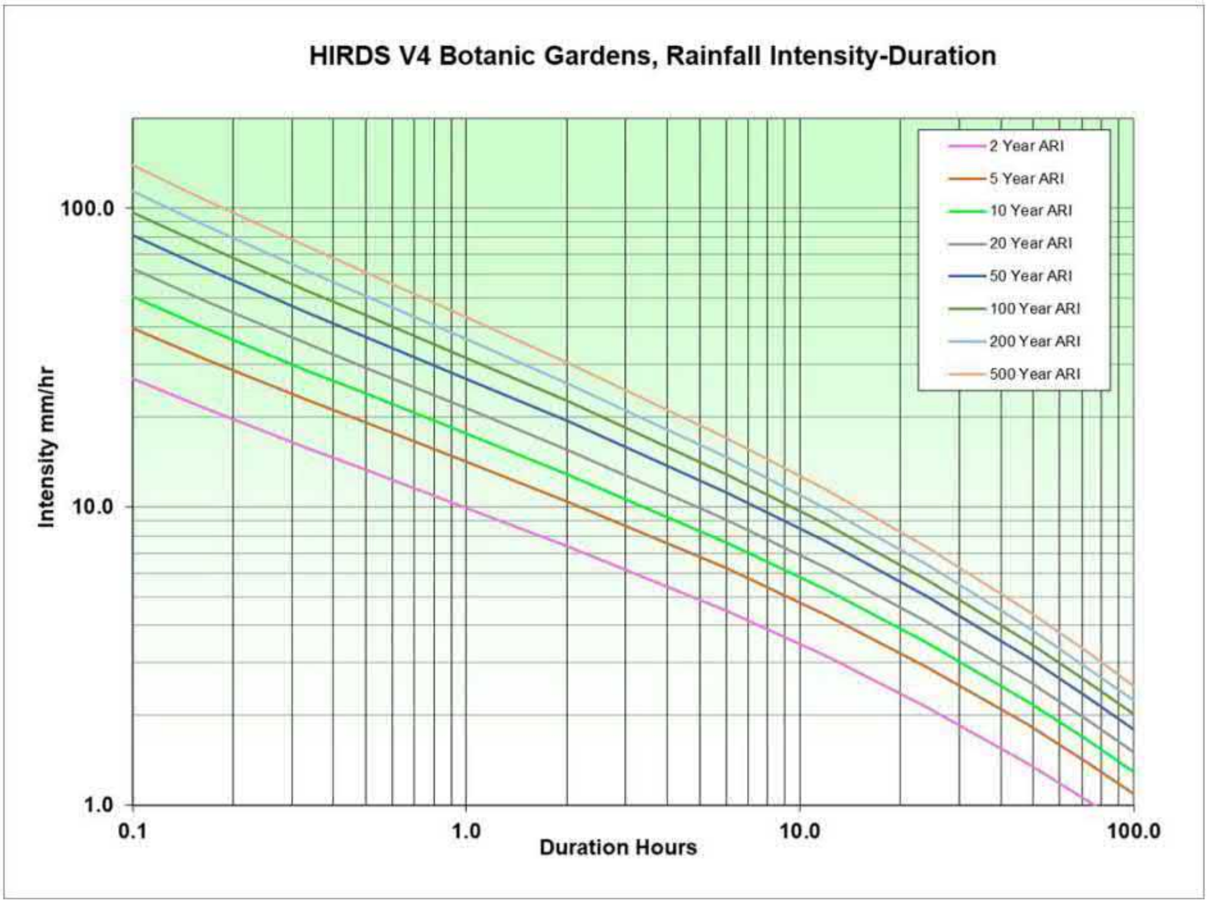
$$a' = \frac{a}{A} = \frac{\text{Area of liquid}}{\text{Area of pipe}}$$

$$r' = \frac{r}{R} = \frac{\text{Actual hydraulic radius}}{\text{Full pipe hydraulic radius}}$$

Appendix 10: Current Botanic Gardens Rainfall Intensities (mm/hr)

Based on NIWA HIRDS Version 4. For quick estimates only of Christchurch City
No Allowance made here for increases due to climate change. Refer to: https://hirds.niwa.co.nz/.

		Duration												
		5min	10min	20min	30min	1h	2h	6h	12h	24h	48h	72h	96h	120h
AEP	ARI	Rainfall Intensity mm/h												
50.00%	2	29.25	21.26	15.74	13.27	9.93	7.39	4.48	3.15	2.14	1.38	1.04	0.84	0.71
20.00%	5	43.29	31.15	22.83	19.13	14.17	10.44	6.23	4.34	2.91	1.86	1.40	1.13	0.94
10.00%	10	55.09	39.38	28.67	23.93	17.60	12.88	7.60	5.26	3.50	2.23	1.67	1.34	1.12
5.00%	20	68.45	48.61	35.16	29.24	21.37	15.54	9.08	6.24	4.13	2.61	1.94	1.56	1.30
3.33%	30	77.07	54.53	39.30	32.61	23.75	17.21	9.99	6.84	4.51	2.84	2.11	1.69	1.41
2.50%	40	83.57	58.98	42.39	35.12	25.51	18.43	10.66	7.28	4.79	3.01	2.23	1.78	1.49
2.00%	50	88.83	62.56	44.88	37.14	26.92	19.41	11.19	7.63	5.01	3.14	2.32	1.86	1.55
1.67%	60	93.27	65.58	46.97	38.83	28.10	20.23	11.63	7.92	5.19	3.24	2.40	1.92	1.60
1.25%	80	100.53	70.50	50.36	41.57	30.01	21.55	12.33	8.38	5.47	3.41	2.52	2.01	1.67
1.00%	100	106.38	74.45	53.08	43.76	31.53	22.59	12.89	8.74	5.70	3.55	2.62	2.09	1.74
0.50%	200	125.68	87.42	61.94	50.88	36.43	25.94	14.66	9.87	6.40	3.96	2.91	2.31	1.92
0.40%	250	132.25	91.80	64.91	53.26	38.05	27.05	15.23	10.24	6.62	4.09	3.01	2.39	1.98
0.20%	500	153.58	105.95	74.45	60.86	43.22	30.53	17.02	11.37	7.31	4.49	3.29	2.60	2.15



Appendix 11: Pipe Flow Nomograph

From Ministry of Technology Research Paper No. 4, Tables for the hydraulic design of storm drains, sewers, and pipelines.

$ks = 0.61 \times 10^{-3}$ for 375ϕ and larger
$\quad = 1.52 \times 10^{-3}$ for 300ϕ and smaller

$$V = -2\sqrt{2gDi}\ .\ \log\left[\frac{ks}{3.7D} + \frac{2.51\nu}{D\sqrt{2gDi}}\right]$$

where V = velocity (m/s)
g = gravitational acceleration (9.81 m/s^2)
D = pipe diameter (m)
i = hydraulic gradient (m/m)
ks = effective roughness (m)
ν = kinematic viscosity of fluid
= 1.113×10^{-6} m^2/s