

MUNICIPAL WORKS DESIGN MANUAL

MUNICIPAL ENGINEERS ASSOCIATION

SECOND EDITION 1984

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MUNICIPAL WORKS DESIGN MANUAL

SECOND EDITION 1984

PREFACE TO THE FIRST EDITION

The Municipal Engineers Association of the Province of Ontario is composed of professional engineers employed by municipalities in their Engineering or Works Departments. In 1973, the Executive of the M.E.A. approved the establishment of a committee to prepare a Municipal Works Design Manual. This manual is the first edition, as a result of the work of that committee.

Many engineers and designers of public works facilities have felt that a practical manual for solving design problems would be of great benefit. Individually, in their own offices, they have prepared such manuals, but these have never been co-ordinated for the use of other designers, whether they are in a municipal engineering office, a consulting engineering office, or students in a design school.

A municipal works designer usually prepares basic guidelines to assist him in design. Many of these have never been put into writing by the individual, and are lost when he retires, or leaves the design field. These quick-cuts and ready references are established from practical experience over many years.

The expenditure of funds for municipal works is a major item. Frequent reference to this manual should ensure that the taxpayer receives adequate benefit for that portion of his tax dollar which is spent on municipal works. Adequate and quality construction can only start when the design is based on sound technical data coupled with practical experience. This manual should be of great benefit to those who are learning municipal works design criteria. It should also help the veteran designer since he will have at hand a reference from which he can quickly and easily extract relevant information concerning design problems. Hopefully, it will help all, in a ready-to-use form.

This manual is not meant to reflect ideal academic thinking on municipal works design practices; rather, it introduces what we believe to be a very practical way to design public works facilities, based upon theory and experience. The reader will find much that is debatable, and alternatives of design criteria are mentioned. It is hoped that this manual will promote discussion and modifications to present practices, and that periodically, the manual will be revised. It is recognized by the Committee that this is the first edition, and its use will indicate whether revision is warranted. The reader is encouraged by the Committee to submit revisions and additions.

..... 2.

These comments should be sent to Mr. W. J. Warwick, P. Eng., Executive Director - Engineering, Borough of Scarborough, 150 Borough Drive, Scarborough, Ontario, M1P 4N7.

The Committee was composed of the following:

W. J. Warwick, Chairman	-	Borough of Scarborough
G. H. Bagshaw (resigned)	-	Regional Municipality of Niagara
Clive Bennett	-	City of Sarnia
A. Bernard	-	Borough of Etobicoke
D. C. Brownlow	-	Borough of Etobicoke
V. S. Doerr (resigned)	-	Town of Mississauga
Donald H. Husson	-	Middlesex County
W. S. Leach	-	City of Port Colborne
G. H. Jackson	-	City of London
R. F. Meadley	-	City of Orillia
W. M. Metcalfe	-	City of Brockville
N. H. Orr	-	Regional Municipality of Ottawa-Carleton
D. Redmond	-	City of Sault Ste. Marie
R. W. Rodman	-	City of Niagara Falls
A. H. Tilt	-	Scarborough Public Utilities Commission
D. Van Beilen	-	City of Brampton
Peter Wong	-	City of Sudbury
R. T. Peacock (resigned)	-	Regional Municipality of Durham

W.J.W.

1977

PREFACE TO THE SECOND EDITION

The Municipal Engineers' Association, at its Annual Meeting held in February, 1981, approved of the preparation of a Second Edition to the Municipal Works Design Manual originally published in 1977.

A new Committee was set up as follows:

*	W. J. Warwick, Chairman	- Scarborough
*	David C. Brownlow	- Etobicoke
	W. E. C. Coulter	- Chatham
	Stephen Cox (Appointed April '83)	- Twp. of Ernestown
	David C. Marett	- Ottawa-Carleton
	F. G. Marks	- Sault Ste. Marie
	S. S. Matharu	- Whitby
	D. J. McDonald	- Counties of Stormont, Dundas and Glengarry
*	R. F. Meadley	- Orillia
*	Noel H. Orr	- Ottawa-Carleton
	J. G. Pavelka	- Hamilton-Wentworth
	Ronald A. Standish	- City of York
	Shafik N. Thomas (Appointed June '81)	- Wallaceburg
*	D. J. Van Beilen	- Brampton
	A. Dagostino (Resigned March '82)	- Sudbury
*	D. C. Redmond (Replaced by F. G. Marks in 1982.)	- Sault Ste. Marie

* Signifies member of First Edition Committee.

The first Edition was well received by Municipal Engineers, Consultants, other Government Agencies, and their staff. It seemed to fill a need that other publications could not fill.

The acceptance of the manual by some Community Colleges as a required text in their Civil Engineering Program is most gratifying. In fact, nearly half of the copies of the First Edition were sold to the Colleges.

In this Second Edition, each chapter has been reviewed, revised where necessary, and metricated. A new chapter has been added: Legislation Relating to Municipal Engineering, and a new section has been added to Chapter 1 - Storm and Sanitary Sewers, relating to Storm Water Management. Lastly, typographical errors which crept into the First Edition have, hopefully, all been corrected.

W.J.W.

January, 1984

1.
STORM and SANITARY SEWERS

CHAPTER NO. 1 - SANITARY AND STORM SEWERS

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CHAPTER NO. 1 - SANITARY AND STORM SEWERS

1-1 INTRODUCTION

The purpose of this chapter is to provide Municipal Designers with a source of information that hopefully can be looked upon as the "first" place to look for design related information and procedures applicable to sanitary and storm works. A bibliography has been included so that more detailed information may be obtained if required.

Following on the works of the first edition, this edition provides a metric update as well as two new sections. The new sections include Culvert Design and an Introduction to Storm Water Management. As with the other topics covered in this chapter volumes have been written on each subject and attempts have been made to précis the information to those areas most applicable to Municipal Designers. As Storm Water Management moves into the "high tech" areas, update and changes in procedures could change quite quickly. Hopefully the information provided will serve as a sufficient grounding to enable the designer to make responsible decisions.

We also highly recommend that the designer obtain a copy of the "Ontario Provincial Standard Drawings". This publication, which should be in every design office, was prepared by a sub-committee of the Municipal Engineers Association and the Provincial Ministries of the Environment and Transportation and Communications. It consists of three binders containing a set of standard drawings and specifications. Copies are obtainable from:

The Head
Highway Standards Section,
Ministry of Transportation and
Communications,
1201 Wilson Avenue,
Downsview, Ontario, M3M 1J8

at $50.00 per complete set.

1.2 HYDRAULICS

1.2.1 Flow Formula

In the design of gravity sewers and open channels, various formula are available to the designer, such as the Kutter, Hazen-Williams, Darcy-Weisbach and Manning. Due to the simplicity and accuracy of the Manning equation it is the formula most used by designers. The Manning formula is written as:

$$V = \frac{1}{n} \cdot R^{2/3} \cdot S^{1/2} \qquad \text{(metric units)}$$

where V = mean velocity

n = roughness coefficient

R = hydraulic radius

$= \frac{\text{Area of liquid}}{\text{wetted perimeter}}$

S = Slope of energy line

= slope of invert for gravity flow

Using the continuity principle:

$$Q = A \cdot V$$

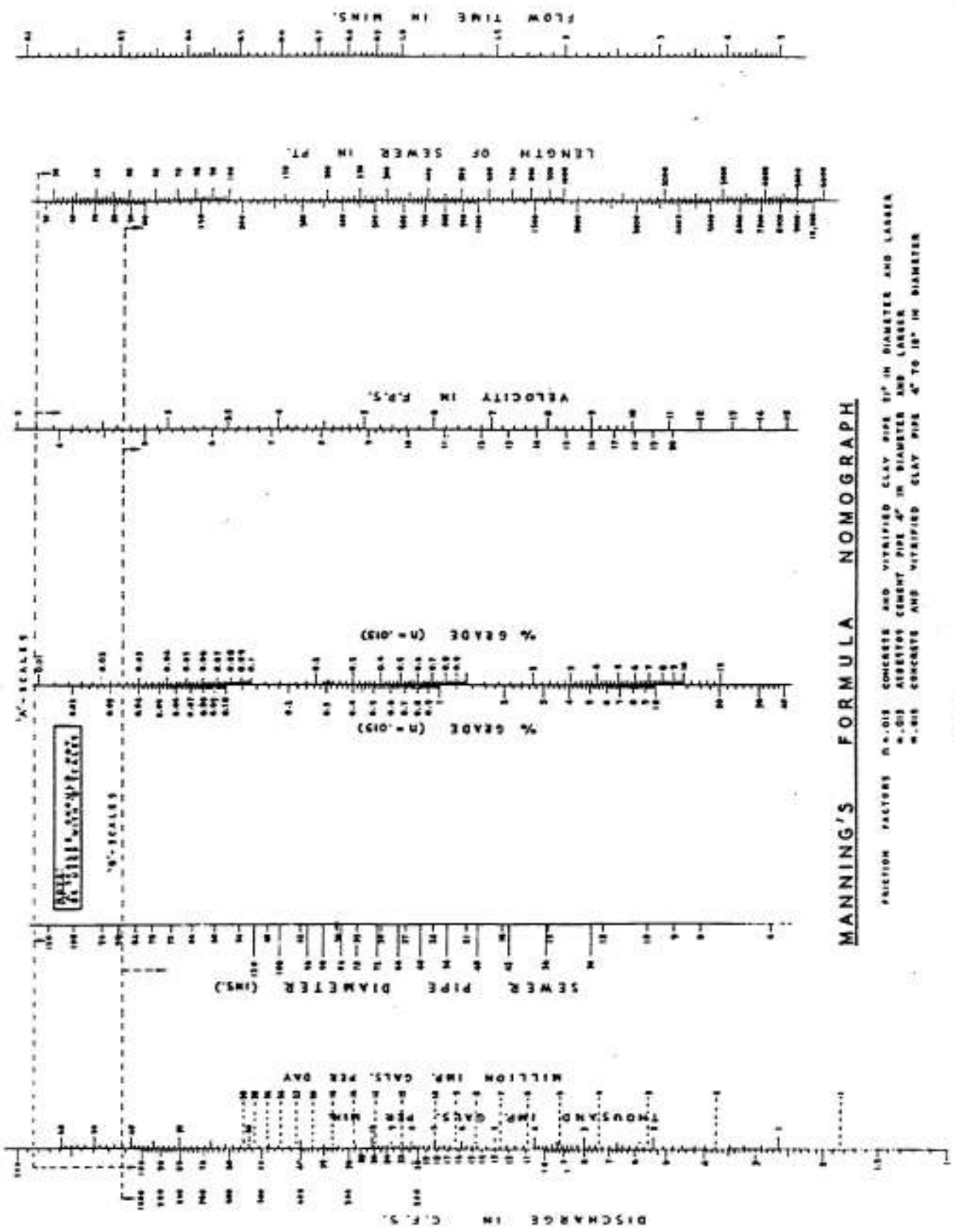
where Q = flow

A = cross sectional area of liquid

With the advent of sophisticated calculators and easy access to computers the designer can calculate the flow capacities in pipes for each circumstance. Also available to calculate the flows are nomographs (Figure 1.2.1), design tables (an example of this is Figure 1.2.2), or the calculation of each condition considered. This is done with the use of tables shown in Figures 1.2.3, 1.2.4 and 1.2.5.

For more charts and tables dealing with flow calculation please turn to the section on Culverts 1.14.

CITY OF LONDON - ENGINEERING STANDARDS



MANNING'S FORMULA NOMOGRAPH

FRICTION FACTORS n = .015 CONCRETE AND VITRIFIED CLAY PIPE 21" IN DIAMETER AND LARGER
n = .013 ASBESTOS CEMENT PIPE 4" IN DIAMETER AND LARGER
n = .015 CONCRETE AND VITRIFIED CLAY PIPE 4" TO 18" IN DIAMETER

FIGURE 1.3.1

DISCHARGE & VELOCITY TABLE

FOR CIRCULAR PIPES

"n" = 0.013

DISCHARGE IN LITRES/SEC

VELOCITY IN METERS/SEC

DIAMETER IN mm

"n" = 0.013

% GRADE	152		203		254		305		381		457		533		610		% GRADE
	Q	V	Q	V	Q	V	Q	V	Q	V							
5.00	35.5	1.95	74.5	2.96	138.7	2.74	223.6	3.09									5.00
4.00	31.8	1.74	66.4	2.11	124.1	2.45	201.5	2.77									4.00
3.50	29.5	1.63	64.1	1.97	115.2	2.28	186.5	2.65	341.9	3.01							3.50
3.00	27.5	1.51	59.3	1.85	107.5	2.12	174.7	2.39	316.8	2.78							3.00
2.50	25.1	1.37	54.2	1.67	97.4	1.94	159.2	2.19	288.9	2.44	488.7	2.98					2.50
2.00	22.5	1.23	48.4	1.49	87.7	1.73	142.7	1.96	259.7	2.27	442.7	2.54	634.5	2.84			2.00
1.80	21.3	1.17	45.9	1.42	83.2	1.64	136.4	1.86	245.4	2.15	399.1	2.45	602.0	2.69	858.4	2.94	1.80
1.60	20.1	1.10	43.3	1.33	78.5	1.55	127.6	1.75	231.4	2.03	376.2	2.29	567.5	2.54	810.3	2.78	1.60
1.50	19.5	1.07	41.9	1.29	76.0	1.50	123.6	1.69	224.0	1.97	364.3	2.22	549.5	2.46	784.4	2.69	1.50
1.40	18.8	1.03	40.5	1.25	73.4	1.45	119.4	1.64	216.4	1.90	351.9	2.14	530.9	2.38	758.0	2.60	1.40
1.30	18.1	0.99	39.0	1.20	70.7	1.40	115.0	1.58	208.4	1.83	339.1	2.07	511.4	2.29	730.4	2.50	1.30
1.20	17.4	0.95	37.5	1.16	68.0	1.34	110.5	1.51	200.4	1.76	325.8	1.98	491.5	2.20	701.7	2.40	1.20
1.10	16.7	0.91	35.9	1.11	65.1	1.28	105.8	1.45	191.8	1.68	312.0	1.90	470.6	2.11	671.9	2.30	1.10
1.00	15.9	0.87	34.2	1.06	62.0	1.22	100.9	1.38	182.9	1.60	297.4	1.81	448.7	2.01	640.6	2.19	1.00
0.98	15.7	0.86	33.9	1.04	61.4	1.21	99.9	1.37	181.1	1.59	294.5	1.79	444.2	1.99	634.2	2.17	0.98
0.96	15.6	0.85	33.5	1.03	60.9	1.20	98.8	1.35	179.2	1.57	291.4	1.78	439.6	1.97	627.6	2.15	0.96
0.94	15.4	0.84	33.2	1.02	60.2	1.19	97.8	1.34	177.3	1.56	288.4	1.76	435.0	1.95	621.1	2.13	0.94
0.92	15.2	0.84	32.8	1.01	59.5	1.17	96.8	1.33	175.5	1.54	285.3	1.74	430.4	1.93	614.4	2.11	0.92
0.90	15.1	0.83	32.5	1.00	58.9	1.16	95.7	1.31	173.5	1.52	282.2	1.72	425.7	1.90	607.7	2.08	0.90
0.88	14.9	0.82	32.1	0.99	58.2	1.15	94.6	1.30	171.6	1.51	279.0	1.70	420.9	1.88	600.9	2.06	0.88
0.86	14.7	0.81	31.7	0.98	57.5	1.14	93.6	1.28	169.6	1.49	275.8	1.68	416.1	1.86	594.1	2.04	0.86
0.84	14.60	0.80	31.4	0.97	56.6	1.12	92.5	1.27	167.6	1.47	272.6	1.66	411.2	1.84	587.1	2.01	0.84
0.82	14.4	0.79	31.0	0.96	56.2	1.11	91.4	1.25	165.6	1.45	269.4	1.64	406.3	1.82	580.1	1.99	0.82
0.80	14.2	0.78	30.6	0.94	55.5	1.10	90.2	1.24	163.6	1.44	266.0	1.62	401.3	1.80	573.0	1.96	0.80
0.78	14.0	0.77	30.2	0.93	54.8	1.08	89.1	1.22	161.6	1.42	262.7	1.60	396.3	1.77	565.8	1.94	0.78
0.76	13.9	0.76	29.8	0.92	54.1	1.07	88.0	1.21	159.5	1.40	259.5	1.58	391.1	1.75	558.5	1.91	0.76
0.74	13.7	0.75	29.4	0.91	53.4	1.05	86.9	1.19	157.4	1.38	255.9	1.56	386.0	1.73	551.1	1.89	0.74
0.72	13.5	0.74	29.0	0.90	52.6	1.04	85.6	1.17	155.2	1.36	252.4	1.54	380.7	1.70	543.6	1.86	0.72
0.70	13.3	0.73	28.6	0.88	51.9	1.02	84.4	1.16	153.0	1.34	248.9	1.52	375.4	1.68	536.0	1.84	0.70
0.68	13.1	0.72	28.2	0.87	51.2	1.01	83.2	1.14	150.8	1.32	245.3	1.49	370.0	1.66	528.2	1.81	0.68
0.66	12.9	0.71	27.8	0.86	50.4	0.99	82.0	1.12	148.6	1.30	241.6	1.47	364.5	1.63	520.4	1.78	0.66
0.64	12.7	0.70	27.4	0.84	49.6	0.98	80.7	1.11	146.3	1.28	238.0	1.45	358.9	1.61	512.5	1.76	0.64
0.62	12.5	0.69	26.9	0.83	48.9	0.96	79.4	1.09	144.0	1.26	234.2	1.43	353.3	1.58	504.4	1.73	0.62
0.60	12.3	0.67	26.5	0.82	48.1	0.95	78.1	1.07	141.7	1.24	230.4	1.40	347.5	1.56	496.2	1.70	0.60
0.58	12.1	0.66	26.1	0.80	47.2	0.93	76.8	1.05	139.3	1.22	226.5	1.38	341.7	1.53	487.9	1.67	0.58
0.56	11.9	0.65	25.6	0.79	46.4	0.92	75.5	1.03	136.9	1.20	222.6	1.36	335.8	1.50	479.4	1.64	0.56
0.54	11.7	0.64	25.1	0.78	45.6	0.90	74.1	1.02	134.4	1.18	218.6	1.33	329.7	1.48	470.7	1.61	0.54
0.52	11.5	0.63	24.7	0.76	44.7	0.88	72.8	1.00	131.9	1.16	214.5	1.31	323.5	1.45	461.9	1.58	0.52
0.50	11.2	0.62	24.2	0.75	43.9	0.87	71.5	0.98	129.5	1.13	210.3	1.28	317.3	1.42	453.0	1.55	0.50
0.48	11.0	0.60	23.7	0.73	43.0	0.85	69.9	0.96	126.7	1.11	206.1	1.26	310.9	1.39	443.8	1.52	0.48
0.46			23.2	0.72	42.1	0.83	68.4	0.94	124.1	1.09	201.7	1.23	304.3	1.36	434.5	1.49	0.46
0.44			22.7	0.70	41.2	0.81	69.9	0.92	121.3	1.06	197.3	1.20	297.6	1.33	424.9	1.46	0.44
0.42			22.2	0.68	40.2	0.79	65.4	0.90	118.5	1.04	192.8	1.17	290.8	1.30	415.1	1.42	0.42
0.40			21.6	0.67	39.2	0.77	63.8	0.87	115.7	1.01	188.1	1.15	283.8	1.27	405.1	1.39	0.40
0.35			20.2	0.62	36.7	0.72	59.7	0.82	108.2	0.95	176.0	1.07	266.4	1.19	379.0	1.30	0.35
0.30					34.0	0.67	55.3	0.76	100.2	0.88	162.9	0.99	245.8	1.10	350.9	1.20	0.30
0.25					31.0	0.61	50.4	0.69	91.5	0.80	148.7	0.91	224.3	1.00	320.3	1.10	0.25
0.20							45.1	0.62	81.8	0.72	133.0	0.81	200.7	0.90	286.5	0.98	0.20

Figure 1.2.2

TABLE FOR SOLVING MANNING'S FORMULA FOR CONDUITS RUNNING FULL

Diameter mm	Area m^2	Hydraulic Radius $R = \frac{D}{4}$ m	$R^{2/3}$	$AR^{2/3}$	Values of $\frac{1}{n} \times R^{2/3} \times A$									
					n= .010	n= .011	n= .012	n= .013	n= 015	n= .021	n= .022	n= .024	n= .025	n= .035
101	0.008	0.025	0.085	0.00068	0.068	0.062	0.057	0.052	0.045	0.032	0.031	0.028	0.027	0.019
127	0.013	0.032	0.101	0.00131	0.131	0.119	0.109	0.101	0.087	0.062	0.060	0.055	0.052	0.037
152	0.018	0.038	0.113	0.00203	0.203	0.185	0.169	0.156	0.135	0.097	0.092	0.085	0.081	0.058
203	0.032	0.051	0.138	0.00442	0.442	0.402	0.368	0.340	0.295	0.210	0.201	0.184	0.177	0.126
254	0.051	0.0635	0.159	0.00811	0.811	0.737	0.676	0.624	0.541	0.386	0.369	0.338	0.324	0.232
305	0.073	0.076	0.179	0.01307	1.307	1.188	1.089	1.005	0.871	0.622	0.594	0.545	0.523	0.373
356	0.0995	0.089	0.199	0.01980	1.980	1.800	1.650	1.523	1.320	0.943	0.900	0.825	0.792	0.566
381	0.114	0.095	0.208	0.02371	2.371	2.155	1.976	1.824	1.581	1.129	1.078	0.988	0.948	0.677
406	0.130	0.1015	0.218	0.02834	2.834	2.576	2.362	2.180	1.889	1.350	1.288	1.181	1.134	0.810
457	0.164	0.114	0.235	0.03854	3.854	3.504	3.212	2.965	2.569	1.835	1.752	1.606	1.542	1.101
508	0.203	0.127	0.253	0.05136	5.136	4.669	4.280	3.951	3.424	2.446	2.334	2.140	2.054	1.467
533	0.223	0.133	0.261	0.0582	5.82	5.291	4.850	4.477	3.880	2.771	2.645	2.425	2.328	1.663
610	0.292	0.1525	0.285	0.0832	8.32	7.564	6.933	6.400	5.547	3.962	3.782	3.467	3.328	2.377
686	0.370	0.1715	0.309	0.11433	11.433	10.394	9.528	8.795	7.622	5.444	5.197	4.764	4.573	3.267
762	0.456	0.1905	0.331	0.15094	15.094	13.722	12.578	11.611	10.063	7.188	6.861	6.289	6.038	4.313
838	0.552	0.2095	0.353	0.19486	19.486	17.715	16.238	14.989	12.991	9.279	8.857	8.119	7.794	5.567
914	0.656	0.2285	0.374	0.24534	24.534	22.304	20.445	18.872	16.356	11.683	11.152	10.223	9.814	7.010
1067	0.894	0.267	0.415	0.37101	37.101	33.728	30.918	28.539	24.754	17.667	16.864	15.459	14.840	10.600
1219	1.167	0.305	0.453	0.52865	52.865	48.059	44.054	40.665	35.243	25.174	24.030	22.027	21.146	15.104
1372	1.478	0.343	0.490	0.72422	72.422	65.838	60.352	55.709	48.281	34.487	32.919	30.176	28.969	20.692
1524	1.824	0.381	0.526	0.95942	95.942	87.220	79.952	73.802	63.961	45.687	45.610	39.976	38.377	27.412
1676	2.206	0.419	0.560	1.23536	123.536	112.31	102.95	95.028	82.357	58.827	56.153	51.473	49.414	35.296
1829	2.627	0.457	0.593	1.5578	155.78	141.62	129.82	119.83	103.85	74.181	70.809	64.908	62.312	44.509
1981	3.082	0.495	0.626	1.9293	192.93	175.39	160.78	148.41	128.62	91.871	87.695	80.388	77.172	55.123
2134	3.577	0.5335	0.658	2.3537	235.37	213.97	196.14	181.05	156.91	112.08	106.99	98.071	94.148	67.249
2286	4.104	0.5715	0.689	2.8277	282.77	257.06	235.64	217.52	188.51	134.65	128.53	117.82	113.11	80.791
2438	4.668	0.6095	0.719	3.3563	335.63	305.12	279.69	258.18	223.75	159.82	152.56	139.85	134.25	95.899
2591	5.273	0.648	0.749	3.9495	394.95	359.05	329.13	303.81	263.30	188.07	179.52	164.56	157.98	112.84
2743	5.909	0.686	0.778	4.5972	459.72	417.93	383.10	353.63	306.48	218.91	208.96	191.55	183.89	131.35
2896	6.587	0.724	0.806	5.3091	530.91	482.65	442.43	408.39	353.94	252.81	241.32	221.21	212.36	151.69
3048	7.297	0.762	0.834	6.0857	608.57	553.25	507.14	468.13	405.71	289.80	276.62	253.57	243.43	173.88

FIG. 12.3

SQUARE ROOTS OF DECIMAL NUMBERS ($S^{1/2}$ IN MANNING'S FORMULA)

No.	.–0	.–1	.–2	.–3	.–4	.–5	.–6	.–7	.–8	.–9
00001	003162	003317	003464	003606	003742	003873	004000	004123	004243	004359
00002	004472	004583	004690	004796	004899	005000	005099	005196	005292	005385
00003	005477	005568	005657	005745	005831	005916	006000	006083	006164	006245
00004	006325	006403	006481	006557	006633	006708	006782	006856	006928	007000
00005	007071	007141	007211	007280	007348	007416	007483	007550	007616	007681
00006	007746	007810	007874	007937	008000	008062	008124	008185	008246	008307
00007	008367	008426	008485	008544	008602	008660	008718	008775	008832	008888
00008	008944	009000	009055	009110	009165	009220	009274	009327	009381	009434
00009	009487	009539	009592	009644	009695	009747	009798	009849	009899	009950
00010	010000	010050	010100	010149	010198	010247	010296	010344	010392	010440
0001	01000	01049	01095	01140	01183	01225	01265	01304	01342	01378
0002	01414	01449	01483	01517	01549	01581	01612	01643	01673	01703
0003	01732	01761	01789	01817	01844	01871	01897	01924	01949	01975
0004	02000	02025	02049	02074	02098	02121	02145	02168	02191	02214
0005	02236	02258	02280	02302	02324	02345	02366	02387	02408	02429
0006	02449	02470	02490	02510	02530	02550	02569	02588	02608	02627
0007	02646	02665	02683	02702	02720	02739	02757	02775	02793	02811
0008	02828	02846	02864	02881	02898	02915	02933	02950	02966	02983
0009	03000	03017	03033	03050	03066	03082	03098	03114	03130	03146
0010	03162	03178	03194	03209	03225	03240	03256	03271	03286	03302
001	03162	03317	03464	03606	03742	03873	04000	04123	04243	04359
002	04472	04583	04690	04796	04899	05000	05099	05196	05292	05385
003	05477	05568	05657	05745	05831	05916	06000	06083	06164	06245
004	06325	06403	06481	06557	06633	06708	06782	06856	06928	07000
005	07071	07141	07211	07280	07348	07416	07483	07550	07616	07681
006	07746	07810	07874	07937	08000	08062	08124	08185	08246	08307
007	08367	08426	08485	08544	08602	08660	08718	08775	08832	08888
008	08944	09000	09055	09110	09165	09220	09274	09327	09381	09434
009	09487	09539	09592	09644	09695	09747	09798	09849	09899	09950
010	10000	10050	10100	10149	10198	10247	10296	10344	10392	10440
01	1000	1049	1095	1140	1183	1225	1265	1304	1342	1378
02	1414	1449	1483	1517	1549	1581	1612	1643	1673	1703
03	1732	1761	1789	1817	1844	1871	1897	1924	1949	1975
04	2000	2025	2049	2074	2098	2121	2145	2168	2191	2214
05	2236	2258	2280	2302	2324	2345	2366	2387	2408	2429
06	2449	2470	2490	2510	2530	2550	2569	2588	2608	2627
07	2646	2665	2683	2702	2720	2739	2757	2775	2793	2811
08	2828	2846	2864	2881	2898	2915	2933	2950	2966	2983
09	3000	3017	3033	3050	3066	3082	3098	3114	3130	3146
10	3162	3178	3194	3209	3225	3240	3256	3271	3286	3302

FIGURE 1 1.

$R^{2/3}$ IN MANNING'S FORMULA
TWO-THIRDS POWERS OF NUMBERS

No.	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0	.000	.046	.074	.097	.117	.136	.153	.170	.186	201
.1	.215	.229	.243	.256	.269	.282	.295	307	.319	331
.2	.342	.353	.364	.375	.386	.397	.407	418	428	438
.3	.448	.458	.468	.477	.487	.497	.506	515	.525	534
.4	.543	.552	.561	.570	.578	.587	.596	604	.613	622
.5	.630	.638	.647	.655	.663	.671	.679	687	.695	703
.6	.711	.719	.727	.735	.743	.750	.758	765	.773	781
.7	.788	.796	.803	.811	.818	.825	.832	840	.847	855
.8	.862	.869	.876	.883	.890	.897	.904	911	.918	925
.9	.932	.939	.946	.953	.960	.966	.973	980	.987	994
1.0	1.000	1.007	1.013	1.020	1.027	1.033	1.040	1.046	1.053	1.059
1.1	1.065	1.072	1.078	1.085	1.091	1.097	1.104	1.110	1.117	1.123
1.2	1.129	1.136	1.142	1.148	1.154	1.160	1.167	1.173	1.179	1.185
1.3	1.191	1.197	1.203	1.209	1.215	1.221	1.227	1.233	1.239	1.245
1.4	1.251	1.257	1.263	1.269	1.275	1.281	1.287	1.293	1.299	1.305
1.5	1.310	1.316	1.322	1.328	1.334	1.339	1.345	1.351	1.357	1.362
1.6	1.368	1.374	1.379	1.385	1.391	1.396	1.402	1.408	1.413	1.419
1.7	1.424	1.430	1.436	1.441	1.447	1.452	1.458	1.463	1.469	1.474
1.8	1.480	1.485	1.491	1.496	1.502	1.507	1.513	1.518	1.523	1.529
1.9	1.534	1.539	1.545	1.550	1.556	1.561	1.566	1.571	1.577	1.582
2.0	1.587	1.593	1.598	1.603	1.608	1.613	1.619	1.624	1.629	1.634
2.1	1.639	1.645	1.650	1.655	1.660	1.665	1.671	1.676	1.681	1.686
2.2	1.691	1.697	1.702	1.707	1.712	1.717	1.722	1.727	1.732	1.737
2.3	1.742	1.747	1.752	1.757	1.762	1.767	1.772	1.777	1.782	1.787
2.4	1.792	1.797	1.802	1.807	1.812	1.817	1.822	1.827	1.832	1.837
2.5	1.842	1.847	1.852	1.857	1.862	1.867	1.871	1.876	1.881	1.886
2.6	1.891	1.896	1.900	1.905	1.910	1.915	1.920	1.925	1.929	1.934
2.7	1.939	1.944	1.949	1.953	1.958	1.963	1.968	1.972	1.977	1.982
2.8	1.987	1.992	1.996	2.001	2.006	2.010	2.015	2.020	2.024	2.029
2.9	2.034	2.038	2.043	2.048	2.052	2.057	2.062	2.066	2.071	2.075
3.0	2.080	2.085	2.089	2.094	2.099	2.103	2.108	2.112	2.117	2.122
3.1	2.126	2.131	2.135	2.140	2.144	2.149	2.153	2.158	2.163	2.167
3.2	2.172	2.176	2.180	2.185	2.190	2.194	2.199	2.203	2.208	2.212
3.3	2.217	2.221	2.226	2.230	2.234	2.239	2.243	2.248	2.252	2.257
3.4	2.261	2.265	2.270	2.274	2.279	2.283	2.288	2.292	2.296	2.301
3.5	2.305	2.310	2.314	2.318	2.323	2.327	2.331	2.336	2.340	2.345
3.6	2.349	2.353	2.358	2.362	2.366	2.371	2.375	2.379	2.384	2.388
3.7	2.392	2.397	2.401	2.405	2.409	2.414	2.418	2.422	2.427	2.431
3.8	2.435	2.439	2.444	2.448	2.452	2.457	2.461	2.465	2.469	2.474
3.9	2.478	2.482	2.486	1.490	2.495	2.499	2.503	2.507	2.511	2.516
4.0	2.520	2.524	2.528	2.532	2.537	2.541	2.545	2.549	2.553	2.558
4.1	2.562	2.566	2.570	2.574	2.579	2.583	2.587	2.591	2.595	2.599
4.2	2.603	2.607	2.611	2.616	2.620	2.624	2.628	2.632	2.636	2.640
4.3	2.644	2.648	2.653	2.657	2.661	2.665	2.669	2.673	2.677	2.681
4.4	2.685	2.689	2.693	2.698	2.702	2.706	2.710	2.714	2.718	2.722
4.5	2.726	2.730	2.734	2.738	2.742	2.746	2.750	2.754	2.758	2.762
4.6	2.766	2.770	2.774	2.778	2.782	2.786	2.790	2.794	2.798	2.802
4.7	2.806	2.810	2.814	2.818	2.822	2.826	2.830	2.834	2.838	2.842
4.8	2.846	2.850	2.854	2.858	2.862	2.865	2.869	2.873	2.877	2.881
4.9	2.885	2.889	2.893	2.897	2.901	2.904	2.908	2.912	2.916	2.920

FIGURE 1.2.5

1.2.2 "n" Values

Manning's roughness coefficient (n) varies with the type of pipe material, the type of installation, and the material being transported. Sewers have their carrying capacity reduced mainly by house connections, joints which do not line up properly, deposits in the sewer, coatings inside the pipe and solids transported in the sewage. For this reason, sewers are considered to have a roughness coefficient greater than that which would occur if clear water were carried in a straight, well aligned new sewer. "n" values commonly used by sewer designers are:

	Pipe Sizes	
	203 to 457	508 and larger
Asbestos cement pipe	.013	.013
Concrete and vitrified clay pipe	.013 - .015	.013
Asphalt coated - 25% paved invert corrugated metal pipe with 12.5 mm corrugations	.022	.022
Unpaved galvanized corrugated metal pipe with:		
12.5 mm corrugations	.024	.024
50 mm corrugations	.035	.035

1.2.3 Minimum Velocities and Grades

Where adequate grade is available, sewers can be placed at grades which will ensure that material does not settle in the sewer. Where the grade available is limited, sewers are laid at the flattest grades which have been found, through experience, to be acceptable. These are based on the minimum permissible velocities in the sewers of:

Sanitary sewers - 0.6 m/sec
- 0.76 m/sec is sometimes used

Storm sewers - 0.9 m/sec

Minimum grades for different sewer sizes are shown in Figure 1.2.6. Care should be exercised in designing sewers having a grade less than 0.10% due to problems of installing these without too many flat sections.

Many designers place the first one or two lengths of sanitary sewers at steeper grades due to the very low flows which will occur in these sewers. Examples of minimum grades used by different municipalities are:

a) The first or highest leg (or legs) in a sewer system are set at a grade not less than 1% or more than 2%.

b) 1 to 5 units, minimum grade = 0.70%
6 to 8 units, minimum grade = 0.60%
9 to 12 units, minimum grade = 0.50%
13 or more units, minimum grade as obtained from Figure 1.2.6 (example for 203 mm n = .015) = 0.44%

c)

Design Flows m3/s	Minimum Grade (1) %	
0.0028	0.92	
0.0056	0.61	
0.0084	0.48	
0.0112	0.41	The minimum grade is not to be less than those indicated on Figure 1.2.6 for the size of pipe being used.
0.0168	0.32	
0.0224	0.27	

MINIMUM & MAXIMUM SEWER GRADES

Size mm	Minimum Grades - %				Maximum Grades - %					
	0.6 m/s		0.9 m/s		3 m/s		4.6 m/s		6 m/s	
	n=.015	n=.013	n=.015	n=.013	n=.015	n=.013	n=.015	n=.013	n=.015	n=.013
203	0.44	0.40	-	-	11.04	8.29	24.8	18.5	44.19	33.17
254	0.34	0.28	0.75	0.56	8.29	6.23	18.6	14.0	33.16	24.93
305	0.26	0.22	0.58	0.44	6.44	4.85	14.5	10.9	25.78	19.38
356	-	0.18	-	0.36	-	3.96	-	8.91	-	15.84
381	0.20	0.14	0.43	0.32	4.78	3.60	10.8	8.15	19.13	14.39
406	-	0.15	-	0.30	-	3.32	-	7.47	-	13.29
457	0.15	0.12	0.34	0.26	3.76	2.83	8.48	6.36	15.05	11.31
508	-	0.11	-	0.22		2.46	-	5.53		9.83
533	0.12	0.09		0.21		2.31		5.18		9.23
610	0.10	0.08		0.18		1.93		4.33		7.72
686	For velocities of			0.15		1.65		3.74		6.60
762	0.76 m/s the slopes			0.13		1.43		3.23		5.73
838	above must be mult-			0.11		1.26		2.75		5.05
914	iplied by 1.56.			0.10		1.12		2.53		4.50
1067						0.91		2.03		3.66
1219						0.77		1.72		3.06
1372						0.65		1.45		2.62
1524						0.57		1.28		2.27
1676						0.50		1.13		2.00
1829						0.45		1.01		1.78
1981						0.40		0.90		1.60
2134						0.36		0.82		1.45

Figure 1.2.6

Recent studies have indicated that for these minimum flows the effect of pipe size is negligible. Therefore, these minimum grades for partially full pipe are not effected by minimum pipe size used.

d) Calculate the velocity of flow in the partially full sewer using Values of Hydraulic Elements of Circular Sections for Various Depths of Flow - Figures 1.3.2 and 1.3.5. See Section 1.3.6 for examples.

1.2.4 Maximum Velocities and Grades

1.2.4.1 Sewers

Little is known about the desirable maximum grades which should be permitted in sewers. Due to the possibility of erosion of the invert, if abrasive material is carried in the pipe, maximum velocities are usually limited to:

Sanitary sewers 3 to 4.6 m/s

Storm sewers 4.6 to 6 m/s

The grades which will produce these maximum velocities are shown on Figure 1.2.6.

1.2.4.2 Open Channels

The maximum velocities normally permitted in channels where no special provision is made for erosion protection, are as follows:

Soil Type	Maximum Permissible Velocities - m/sec
Fine sand	0.76
Fine gravel	1.5
Course gravel	1.8
Sandy loam	0.76
Silty loam	0.9

Ordinary firm loam	1.1
Stiff clay	1.5
Shales and hardpan	1.8

Where grass is well established, permissible velocities are:

Easily eroded soils	1.2 m/sec
Erosion resistant soils	1.5 m/sec

1.3 DESIGN OF SANITARY SEWERS

1.3.1 Introduction

Sanitary sewage is defined as a combination of (a) the liquid wastes conducted away from residences, commercial buildings and institutions and (b) the liquid wastes from industrial developments and (c) such ground, surface and storm water as may be admitted to or finds its way into the sanitary sewers. The allowance of water into sanitary sewers which does not contain objectionable impurities should not be permitted and such prohibition should be strictly enforced.

The flow rate of sewage, for which sewer capacity is to be provided, is determined from careful considerations of the present and probable future quantities of domestic sewage, commercial and industrial wastes, ground - water infiltration and any other unavoidable contributions.

A sanitary sewer has two main functions; namely (a) to carry the peak discharge for which it is designed and (b) to transport suspended solids so that deposits in the sewer are kept to a minimum - all to a satisfactory point for treatment or ultimate disposal. It is obviously essential that the sewers have adequate capacity for the peak flow and that they function at minimum flow without nuisance.

1.3.2 Design Procedure

1.3.2.1 General

In designing a sanitary sewer or a sanitary sewer system, the designer normally commences by establishing the specific location of the area to be serviced, and obtains maps as required. The area is broken down into units. The existing or proposed land use is determined and a population figure produced. From the population data, the anticipated flow is obtained and allowances included for foundation drains, if these are to be connected, and other infiltration sources.

After the proposed sewer routing is established and the necessary profiles obtained, the designer will proceed to calculate sewer pipe sizes, grades, etc., including the selection of pipe material and bedding. Then, the various appurtenances are designed and added to the plans.

1.3.2.2 Design Flows

One of the first considerations in estimating the sewage flow is to establish a "design period", or a length of time which the proposed sewers will adequately serve the expected population. There is a direct relation between "sanitary sewage and industrial wastes" and "water consumption and population". Therefore, the sewers should be designed for the saturation density of population expected in the design area.

In Canada, when design periods are used, the most common are 25 to 50 years.

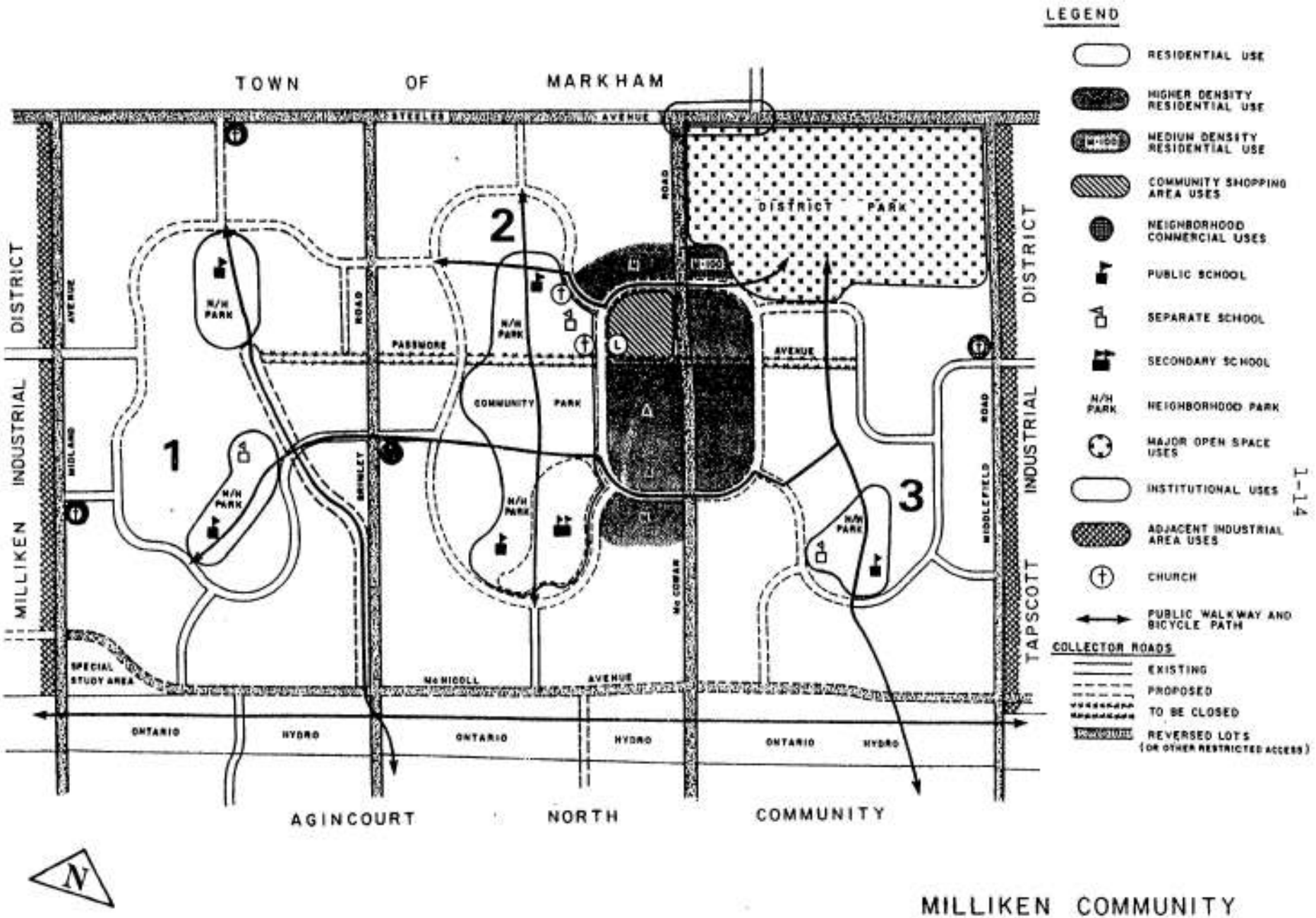
There are many methods used to predict population growth. The most common is a graphical extension of past records into the future. However, the best method is by use of "Official Plans". These plans indicate the future land use locations and will indicate the areas of designation such as single family, semi-detached, multiple, apartments, industrial, commercial, parkland and schools, etc. The Official Plan acts as an extremely useful guide in predicting population and sanitary sewer capacity requirements. (See Figure 1.3.1 for an example.)

Population densities commonly used in sanitary sewer design are shown in the following table:

Area Type	Persons Per ha
Single Family	8-- 14
Semi-Detached	12 - 20
Multi-Family (row housing)	24 - 40
Apartments	160 - 240
Commercial	8 - 12
Institutional	8 - 12

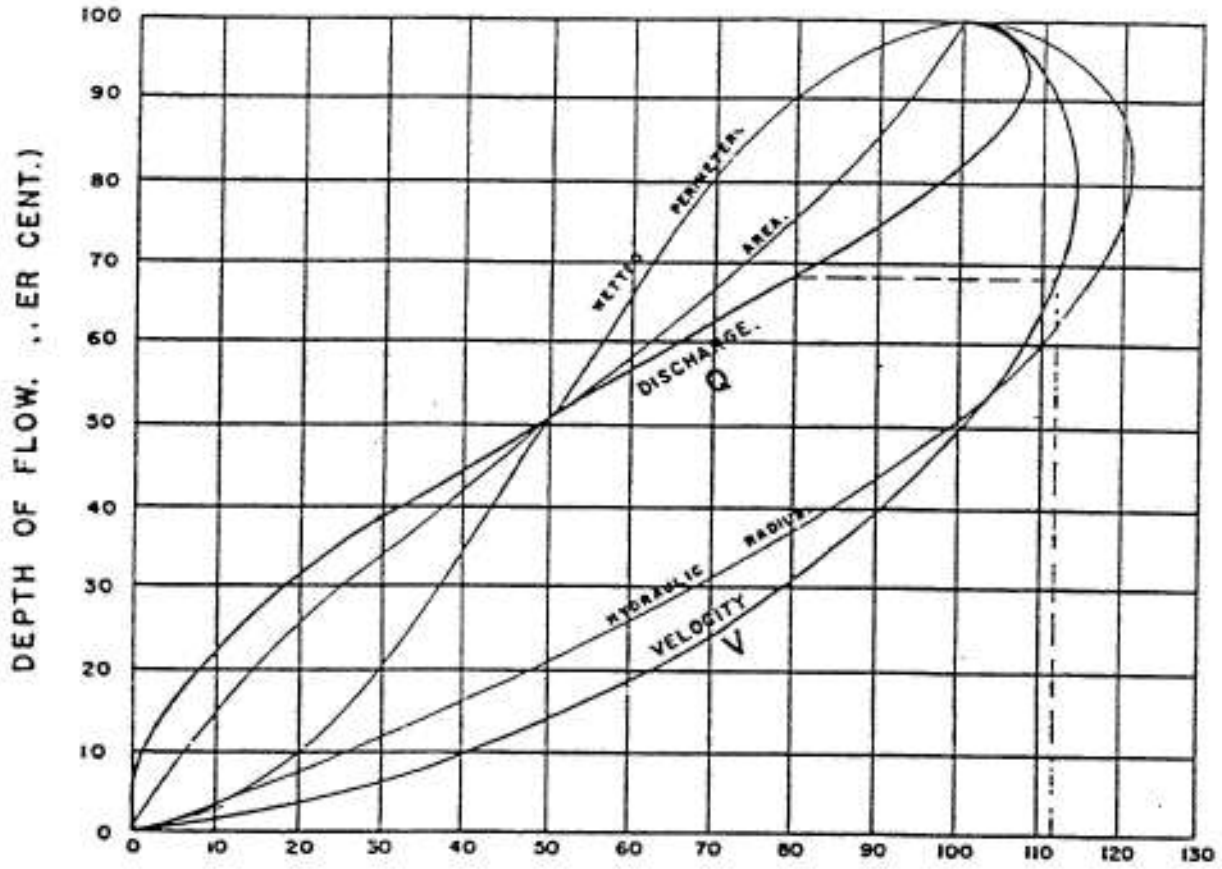
When there is doubt about the validity of the above for a specific location, the designer should check on the criteria by sampling the population in a similar area.

Sanitary sewage flow varies throughout the day, week, season and according to water consumption. The extremely low flow will occur during the midnight to 6:00 a.m. period and peak periods occur during the daylight hours but primarily at meal times and bed times.



MILLIKEN COMMUNITY
SECONDARY PLAN
Figure 1.3.1

VALUES OF HYDRAULIC ELEMENTS
OF CIRCULAR SECTION FOR VARIOUS
DEPTHS OF FLOW.



HYDRAULIC ELEMENTS
PER CENT OF VALUE FOR FULL PIPE.(approx.)

EXAMPLE -
Given: Discharge = 340.8L/sec through a pipe which has capacity flowing full of 426L/sec at velocity 2.13m/sec
Find: V. for Q = 340.8L/sec
Since percentage of full discharge = $\frac{340.8}{426}$ = 80%
enter chart at 80% of value for full section of Hydraulic Elements, find V = 112.5%x2.13m/sec = 2.4m/sec

Figure 1.3.2

The ratio of the maximum daily flow at the end of the design period to the minimum daily flow at the beginning of the design period will range from 1.3 to more than 5 depending on the rate of growth of the areas.

Sanitary sewer design is almost always based on peak flow rates expected and then the flow velocities are checked at the average and minimum flow rates to determine whether settling of solids will occur or not. This is accomplished by calculating the hydraulic elements for various depths of flow (See Figure 1.3.2). An example of this calculation is given later in this chapter.

The peaking factor used by most municipalities is 250% for main sanitary sewers serving stable populations to 500% for growing or unzoned areas.

The average daily sewage flow used by most municipalities is 450 litres per day per capita.

As can be seen from the Official Plan, the number of persons per acre is readily available and this is very helpful in calculating the sanitary coefficient for a future area.

Referring to Figure 1.3.1, the accumulated net acreage of the three neighbourhoods is 254 ha and the accumulated projected population is 27,700 giving an average net density of 109.1 persons her ha.

The additional information available is, flow per persons, 450 litres per day, peaking factor of 250%, and one half of the flow occurring in an 8 hour period.

$$\frac{109.1 \times 4501 \times 2.5}{2 \times 8 \times 3600} = 2.15 \text{ l/s per ha}$$

Another rule of thumb that is used by designers when Official Plans are prepared, is given by the following table for estimating minimum sanitary coefficients. (includes infiltration allowance)

Residential	1.05L/sec.ha
Commercial and Industrial	1.75L/sec.ha
Schools and Multiple Family	2.45L/sec.ha
Apartments(150 units per ha)	3.5 L/sec.ha
High density Apartments (redevelopment areas)	7.0 L/sec.ha

The contribution from industrial uses must be examined carefully when the user is known e.g. a brewery can discharge higher flows than 1.75L/sec.ha indicated above while a large warehouse with few employees will contribute less.

1.3.3 Areas to be Serviced

A map of the areas to be drained to the proposed sewer system should be prepared. This will show existing and/or proposed streets. It is also useful to have contours included which will aid in carrying out a routine study.

Where sewers are proposed on existing streets, it is usual practice to obtain profiles of the road during early design stages. The basement elevations of all existing houses and buildings must be shown on the profile so that the new sewer will be of sufficient depth to provide for gravity drainage when connected.

In undeveloped areas, the sewer patterns can usually be laid out by the use of contours.

The drainage area is broken down into sub-areas based on the length of sewer between each manhole. In many cases, the size of these areas can cover more than one length of sewer depending on:

- the minimum sewer size permitted
- the density of population in each sub-area
- the size of pipe at any given point, since larger pipes require less increase in sizes for given population increments
- the different land uses encountered in each sub-area

This breakdown of areas may require some trial and error on the part of a new designer. With experience, this operation becomes more trial and less error. Some adjustments in the sub-areas may be required during the actual sewer design.

1.3.4 Other Sources of Flow

1.3.4.1 Infiltration

Sanitary sewers must be sized to allow for infiltration which is simply the entry of ground or other non-sewage water into to the system. This is over and above the design figures for sanitary sewage obtained from population counts.

This type of infiltration usually results from the quality of workmanship in the installation of sewers and drains (both on public road allowances and private property) as well as manufacturing tolerances in pipe gaskets and joints, connections, etc.

Common factors used for design for infiltration are 0.14 to 0.28 L/sec.ha.

1.3.4.2 Foundation Drains

Some municipalities permit the connection of foundation drains (weeping tiles) into sanitary sewers. With rising costs in sewage treatment, this practice is becoming less prevalent.

In existing developed areas with no storm sewers, it may be necessary to permit the connection of foundation drains. If this happens, provision must be made in the sanitary sewer design to accommodate the additional flow.

A common coefficient used by a number of municipalities is 0.071L/sec per house foundation drain or 92.9 M^2 of lot coverage.

1.3.5 Design Sheets

The calculation of the sewage flows and the recording of the sewer size, grade and capacity is usually done on a standardized design sheet. These vary according to the method of design used and the information the designer wishes to record on the design sheet.

Figure 1.3.3 is an example of a design sheet where sewage flows are assigned to different land uses. The sewage flow is obtained for each land use by multiplying the design coefficient for the land use by the number of acres of that land use. These land use flows are added together to obtain the total flow for the length of sewer.

Figure 1.3.4 is an example of a design sheet which totals the number of people and area tributary to a sewer. From this, the total peak sewage flow is calculated, and an allowance for infiltration is added to obtain the design flow.

1.3.6 Flow Calculations and Problems

The following two problems are typical of the requirements the designer should meet in the design of a sanitary sewer.

BOROUGH OF SCARBOROUGH
WORKS DEPARTMENT

SANITARY SEWER DESIGN

18.05-025

DATE _____

CALCULATED BY: _____

CHECKED BY: _____

STREET _____

FROM _____

TO _____

DESIGN COEFFICIENTS

Single Family.....0.015 c.f.s./acre
Commercial or Industrial....0.025 c.f.s./acre
Schools or Multiple Family..0.035 c.f.s./acre
Apartments - 60 u.p.a. .....0.050 c.f.s./acre
Apartments - 100 u.p.a. ....0.084 c.f.s./acre
Apartments - 120 u.p.a. ....0.100 c.f.s./acre

M.H.	to M.H.	AREA (acres)						CUMULATIVE AREA SERVED (acres)	MAXIMUM CUMULATIVE FLOW (c.f.s.)	TYPE OF PIPE No	PIPE DIAMETER (in.)	MINIMUM GRADE (ft/ 100 ft.)	CAPACITY AT MIN. GRADE c.f.s.	LENGTH OF SEWER (ft.)	AVERAGE DEPTH OF CUT (ft.)	APPURTENANCES	
		Single Family	Comm. or Ind.	Schools or Mult. Family	Apts. 60 u.p.a.	Apts. 100 u.p.a.	Apts. 120 u.p.a.									M.H.'S	OTHER

Figure 1.3.3

POPULATION DENSITIES:-

SINGLE FAMILY LOTS		6 PEOPLE
DUPLEX LOT		8 "
APTS: 1 BR.		3½ "
2 BR.		5 "
3 BR.		6½ "

CITY OF LONDON
CITY ENGINEER'S DEPARTMENT
SANITARY SEWER DESIGN SHEET

PROJECT _____
FILE NO _____ SHEET _____ OF _____
SANITARY AREA _____
DESIGNED BY _____ CHECKED BY _____
DATE _____

SEWER LOCATION			AREA				POPULATION					SEWAGE FLOW			SEWER DESIGN					PROFILE				
AREA NO	STREET	FROM	TO	DEF OR LOOPS	DIMENSIONS	Δ A ACRES	Σ A ACRES	PER ACRE	OR LOT	PER LOT	Δ POP.	Σ POP.	INFILT. C.F.S.	SEWAGE C.F.S.	TOTAL C.F.S.	SIZE ins.	S %	n	CAP. F.P.S.	VELOCITY F.P.S.	LOSSES IN M.H.	FALL IN SEWER	LENGTH feet	INVERT ELEV. U.S.	INVERT ELEV. D.S.

PROPORTIONAL VALUES FOR VELOCITIES & DISCHARGES IN CIRCULAR SEWERS & PIPES RUNNING PARTLY FULL

PROP. DEPTH	PROP. AREA	PROP. VELOCITY	PROP. DISCHARGE
.03	.0087	.1839	.0016
.04	.0134	.2221	.0030
.05	.0187	.2569	.0048
.06	.0245	.2892	.0071
.07	.0308	.3194	.0098
.08	.0375	.3481	.0130
.09	.0446	.3752	.0167
.10	.0520	.4012	.0209
.11	.0598	.4260	.0255
.12	.0680	.4500	.0306
.13	.0764	.4730	.0361
.14	.0851	.4953	.0421
.15	.0941	.5168	.0486
.16	.1033	.5376	.0555
.17	.1127	.5578	.0629
.18	.1224	.5775	.0707
.19	.1323	.5965	.0789
.20	.1424	.6151	.0876
.25	.1955	.7007	.1370
.30	.2523	.7761	.1958
.35	.3119	.8430	.2629
.40	.3735	.9022	.3370
.45	.4364	.9544	.4165
.50	.5000	1.0000	.5000
.55	.5636	1.0398	.5857
.60	.6265	1.0724	.6718
.65	.6881	1.0998	.7564
.70	.7477	1.1198	.8372
.75	.8045	1.1335	.9119
.80	.8576	1.1397	.9775
.85	.9059	1.1374	1.0304
.90	.9480	1.1243	1.0658
.95	.9813	1.0950	1.0745
1.00	1.0000	1.0000	1.0000

Figure 1.3.5

1.3.6.1 Problem

1.3.6.1.1 Given:

(a) Area to be serviced has been determined to be:
- 6 ha of single family residential
- 1.2 ha of multi-family residential
- 0.8 ha of neighbourhood commercial

(b) Minimum pipe size 203 mm

(c) N = .013

(d) Minimum velocity 0.76 m/s

(e) Minimum grade 1.00%

1.3.6.1.2 Required:

(a) The size and grade of a sanitary sewer to service the area.

(b) Check to determine if the actual velocity meets the minimum requirements.

1.3.6.1.3 Solution:

Determination of flows (from Sec. 1.3.2.2)

Single family residential

6 ha x 1.05 = 6.3 L/sec

Multi-family residential

1.2 ha x 2.1 = 2.5 L/sec

Commercial

0.8 ha x 1.75 = 1.4 L/sec

Total Flow 10.2 L/sec

From Figure 1.2.2 an 203 mm diameter concrete pipe @ 1.00% flowing full has a capacity of 34.2 L/sec and a velocity of 1.06 m/s. This size meets our requirements for flow and now must be checked to see if the velocity at the design flow of 10.2 L/s is above the minimum of 0.76 m/s.

To do this, we make use of the chart "Values of Hydraulic Elements of Circular Section for Various Depths of Flow" in Figure 1.3.2. We would get satisfactory results from using the table entitled "Proportional Values for Velocities and Discharges in Circular Sewers and Pipes Running Partly Full" in Figure 1.3.5 the designer may use either the chart or the table and his results will be similar. Many designers find the table easier to work. For this problem we will use the chart in Figure 1.3.2.

Capacity Actual Q_{Act} 10.2L/sec

Capacity Full Q_{Cap} 34.2L/sec

$$\frac{Q_{Act}}{Q_{Cap}} = \frac{10.2}{34.2} \times 100 = 29.8\%$$

Entering the chart vertically at 29.8% we read depth of flow at 39% and the proportional is 90%. The velocity at full flow is 1.06 m/s, therefore, the velocity at design flow "Q_{Act}" is 1.06 x .90 = 0.95 m/s which is in excess of the minimum velocity permitted. The design requirements are met and 203 mm diameter at 1.00% grade is acceptable.

1.3.6.2 Problem

1.3.6.2.1 Given:

(a) An undeveloped area to be serviced contains 256 ha.

(b) The proposed ratio of development is:

(i) Single family residential 40%
(ii) Semi-detached residential 20%
(iii) Multi-family residential 20%
(iv) Apartment 150 units per ha 15%
(v) Commercial 5%

(c) The maximum available grade is 0.5%

(d) The minimum acceptable velocity is 0.9 m/sec

1.3.6.2.2 Required:

(a) The size and grade of a trunk sanitary sewer to service this area

(b) Does the actual velocity meet the minimum velocity requirement?

1.3.6.2.3 Solution:

Determination of flows (from sec 1.3.2.2)

Single family	256 ha x .40 x 1.05 =	107.52L/S
Semi-detached	256 ha x .20 x 1.05 =	53.76L/S
Multi-family	256 ha x .20 x 2.45 =	125.44L/S
Apartments	256 ha x .15 x 3.5 =	134.4L/S
Commercial	256 ha x .05 x 1.75 =	22.4L/S
	Total Flow	443.52L/S

From charts similar to that on Figure 1.2.2, the following pipe selection and grades will meet the capacity requirement.

Grade	Size	Q_{Cap}	V_{Cap}
0.50	610mm	453L/S	1.55m/s -(see Figure 1.2.2
0.30	686mm	480.3L/S	1.3m/s
0.20	762mm	519.4L/S	1.14m/s

We must now check the velocity at actual (design) flow for the three pipe selections.

610mm pipe $\frac{Q_{Act}}{Q_{Cap}} = \frac{433.52}{455.0} \times 100 = 96\%$

We will use the Chart in Figure 1.3.2. Entering the chart vertically at 96% we read depth of flow is 79% and proportional velocity is 113%. Velocity at design flow is velocity at full flow x 113% or 1.55 x 1.13 = 1.75 m/s.

686mm pipe $\frac{Q_{Act}}{Q_{Cap}} = \frac{433.52}{480.3} \times 100 = 90\%$

Using Figure 1.3.2 as before, we have depth of flow 75% and proportional velocity is 112% therefore, velocity at design flow is 1.3 x 1.12 = 1.46m/s

762mm pipe $\frac{Q_{Act}}{Q_{Cap}} = \frac{433.52}{519.4} = 83\%$

Depth of flow is 70%. Proportional velocity is 113%, therefore, velocity at design flow is 1.14 x 1.13 = 1.29m/s. All three pipe selections meet the requirements for capacity and minimum velocity. The designer must now select the size of pipe and grade to suit the conditions of available grade and economics of the system. In calculating the pipe sizes required in the design of a sanitary sewer computation sheets, as shown in Figure 1.3.3, are convenient.

1.3.7 Designing to Avoid Sulphide Problems

During low flows in sanitary sewers, severe sulphide problems can arise due to sluggish flows or full flows which continue for a period of time out of contact with air. The condition often applies to large trunk sanitary sewers. These gases will deteriorate concrete, steel and cast-iron pipes and, of course, cause odor problems.

The finding of sulphide concentrations is closely related to the original design of the sewer. Severe sulphide conditons should be avoided by proper design but, where lower concentrations are common, these can be minimized or eliminated by treatment.

Sulphide gases are normally insignificant if the velocity that will generate same exists only for a few hundred meters in a section of the system. However, if this same velocity occurs for kilometers, then the accumulation of sulphide may be sufficient to cause serious odor problems, corrosion of pipes and manholes, increase of chlorine demand and increased difficulty of treatment of the sewage at the plant.

1.4 DESIGN OF STORM SEWERS

1.4.1 Rational Method

The rational method is the most widely used design procedure for determining the quantity of storm water runoff. The formula is:

$$Q = 0.0028\ ACi$$

A - is the area drained in hectares

C - is the runoff coefficient which depends on the characteristics of the area drained

i - is the average rainfall intensity in mm./hour

Q - is the peak runoff rate in cubic metres per second c.m.s.

1.4.2 Runoff Coefficient

The runoff coefficient is a ratio of the amount of rainwater which is carried away by the storm sewer to the total rainfall.

It is determined, for a given area,by obtaining a composite average based on portions of each type of surface within the area drained. The runoff coefficients used for this analysis are:

Character of Surface	Runoff Coefficients
Pavement	
Asphaltic and Concrete	0.70 to 0.95
Brick	0.70 to 0.85
Roofs	0.75 to 0.95
Lawns, sandy soil	
Flat, 2 percent	0.05 to 0.10
Average, 2 to 7 percent	0.10 to 0.15
Steep, 7 percent	0.15 to 0.20
Lawns, heavy soil	
Flat, 2 percent	0.13 to 0.17
Average, 2 to 7 percent	0.18 to 0.22
Steep, 7 percent	0.25 to 0.35

The range of coefficients, classified with respect to the general characteristic of the tributary area, which are currently used is as follows:

Description of Area	Runoff Coefficients
Business	
Downtown	0.70 to 0.95
Neighbourhood	0.50 to 0.70
Residential	
Single Family	0.35 to 0.75
Multi-units, detached	0.40 to 0.75
Multi-units, attached	0.60 to 0.75
Residential (suburban)	0.25 to 0.55
Apartment	0.50 to 0.70
Industrial	
Light	0.50 to 0.80
Heavy	0.60 to 0.90
Parks, cemeteries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad Yard	0.20 to 0.35
Unimproved	0.10 to 0.30

The coefficients in these two tables are applicable to storms of 5 to 10 year frequencies. Less frequent, higher intensity storms will require the use of higher coefficients. The coefficients are based on the assumption that the design storm does not occur when the ground surface is frozen.

1.4.3 Rainfall Intensity

The rainfall intensity is based on:

- the rainfall intensity frequency curve for the municipality in which the sewers are being designed.
- the storm frequency used, which determines the degree of protection to be provided.
- the length of time it takes water to flow from the most remote part of the watershed to the sewer being designed. This is called the time of concentration.

These are determined as outlined below.

1.4.3.1 Rainfall Intensity-Duration-Frequency Curves

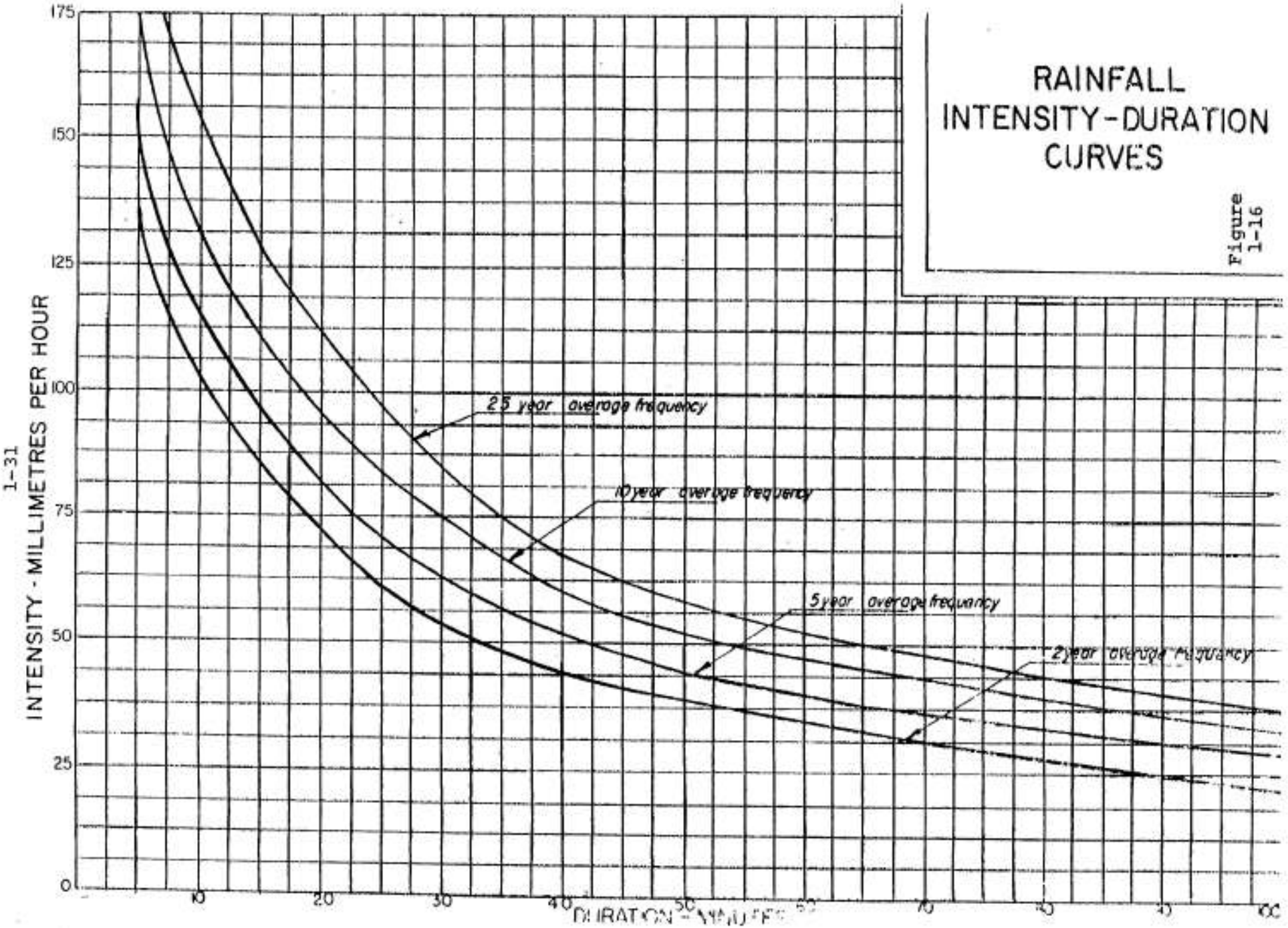
Rainfall intensity-duration-frequency curves are a plot of the peak rainfall rates for storms having different return frequencies.

A typical example of this type of curve is shown on Figure 1.16. Environment Canada has prepared this type of curve for 42 locations within Ontario. They have also prepared a booklet outlined "Atlas of Rainfall Intensity - Duration-Frequency Data for Canada" by J. P. Bruce, sub-titled "Climatological Studies Number 8". Rainfall intensity-duration-frequency curves can be obtained for any area within the Province by:

- Interpolating from the rainfall maps in "Atlas for Rainfall Intensity-Duration - Frequency Data of Canada" the total rainfall for different times and return periods. These are converted to rates of rainfall and a set of curves can then be plotted.

RAINFALL INTENSITY-DURATION CURVES

Figure 1-16



- Obtain from Environment Canada in Downsview, Ontario, the list of locations for which they have prepared curbs. Check the number of years of record on which they are based. From this, select the locations in your area which you believe will be sufficiently accurate for your use. Obtain this curve from Environment Canada.
- Check with a municipality in the area and, if they have found their design curve has worked well for them, use it.

1.4.3.2 Storm Frequency

The return period, or storm frequency, selected for a design determines the degree of protection afforded by a given storm sewer system. The range for frequencies used in Ontario is as follows:

- Residential Areas - 1 to 10 years are used with 2 and 5 years most frequently reported.
- High value general and commercial areas 5 to 10 years are most frequently used.
- Large watershed having a tributary area of over 130 hectares require special designs. The Ministry of Natural Resources require for these areas that flows from a 100-year storm, a "Hazel" or "Timmins" storm, be allowed.

  These designs require special considerations and are beyond the scope of this manual. Those who are in-

volved in the design of systems of this size should contact the Ministry directly to determine their design criteria.

1.4,3.3 Inlet Time & Time of Concentration

The peak flow occurs in a storm sewer when all the areas tributary to the sewer are contributing flows. When this occurs, it is referred to as the time of concentration. This time is made up of two other times, the inlet time and the time of flow in the sewer.

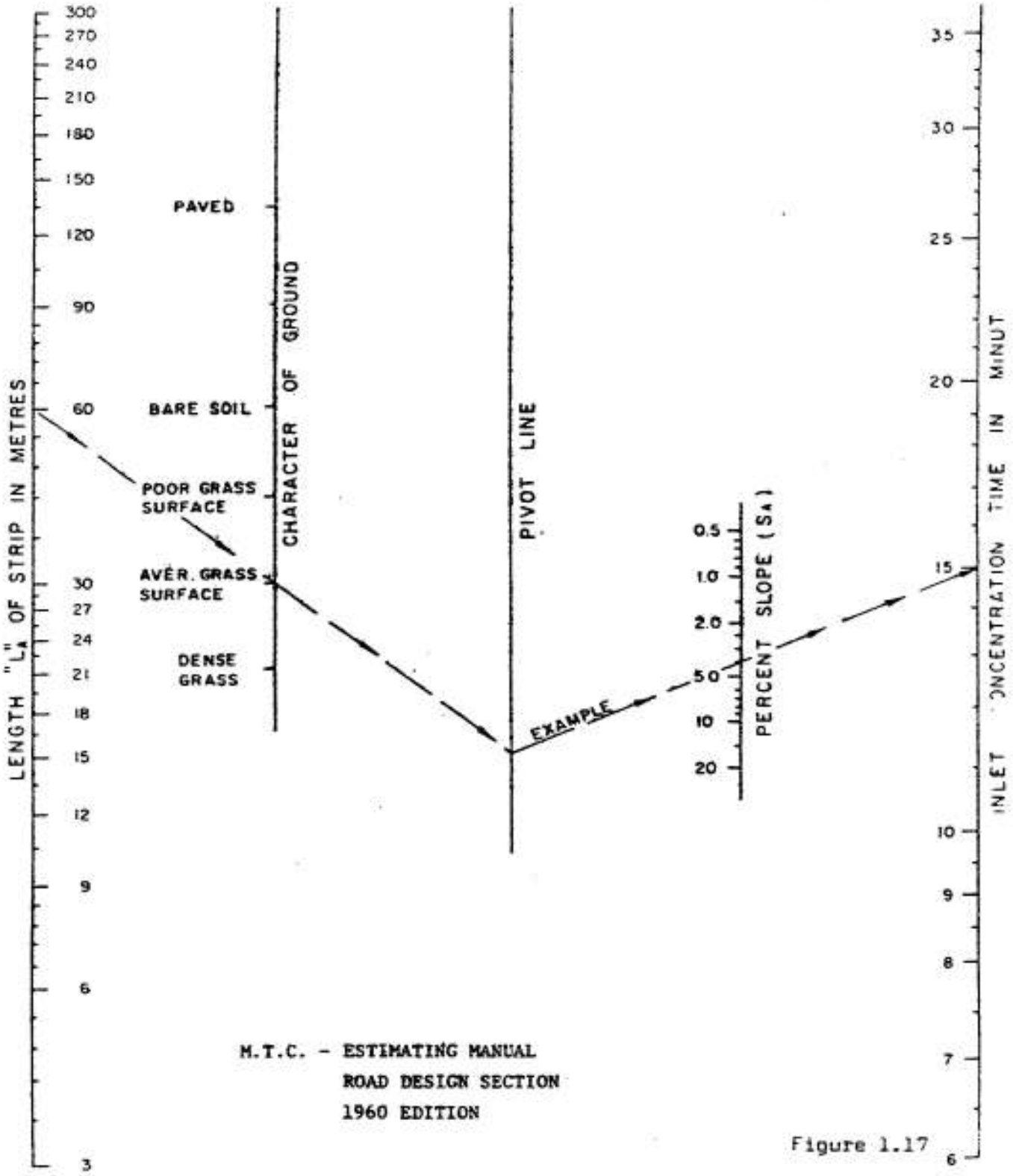
The inlet time is the time required for rainwater to flow from the most remote part of the watershed to the first inlet in the system. Inlet times used for design, vary from 5 to 30 minutes, with 5 to 15 minutes most commonly used. In densely developed areas, where impervious surfaces shed their water directly to storm sewers through closely spaced inlets, an inlet time of 5 minutes often is reported. In well developed districts with relatively flat slopes, an inlet time of 10 to 15 minutes is common. In flat residential districts with widely spaced street inlets, inlet times of 20 to 30 minutes are customary.

Inlet times can be established by adding the overland flow and gutter flow times for typical inlets. These flow times can be obtained from Figures 1.17 and 1.18.

The inlet time is used to obtain the rainfall intensity from the rainfall-intensity-duration-frequency curve. This is used in the rational formula to determine the runoff rate to be used in the design of the first length of storm sewer.

For the second and subsequent lengths of sewer, the design is based on the time of concentration. This is the inlet time plus the time it takes the storm water to flow from the inlet to the length of sewer being designed.

OVERLAND FLOW TIME (Tc)



M.T.C. - ESTIMATING MANUAL
ROAD DESIGN SECTION
1960 EDITION

Figure 1.17

ENTRY TIME FOR GUTTER FLOW - AFTER CARON

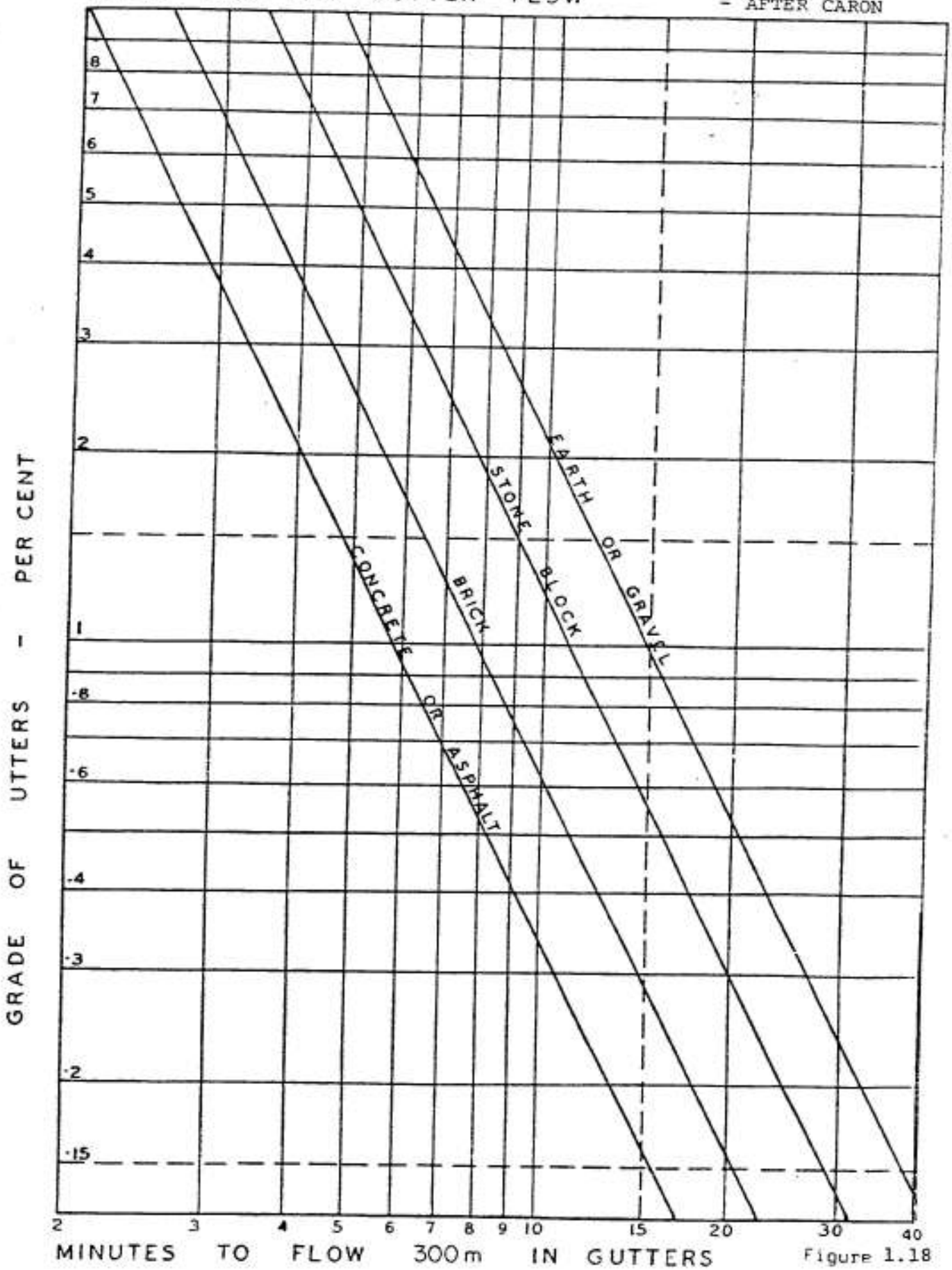


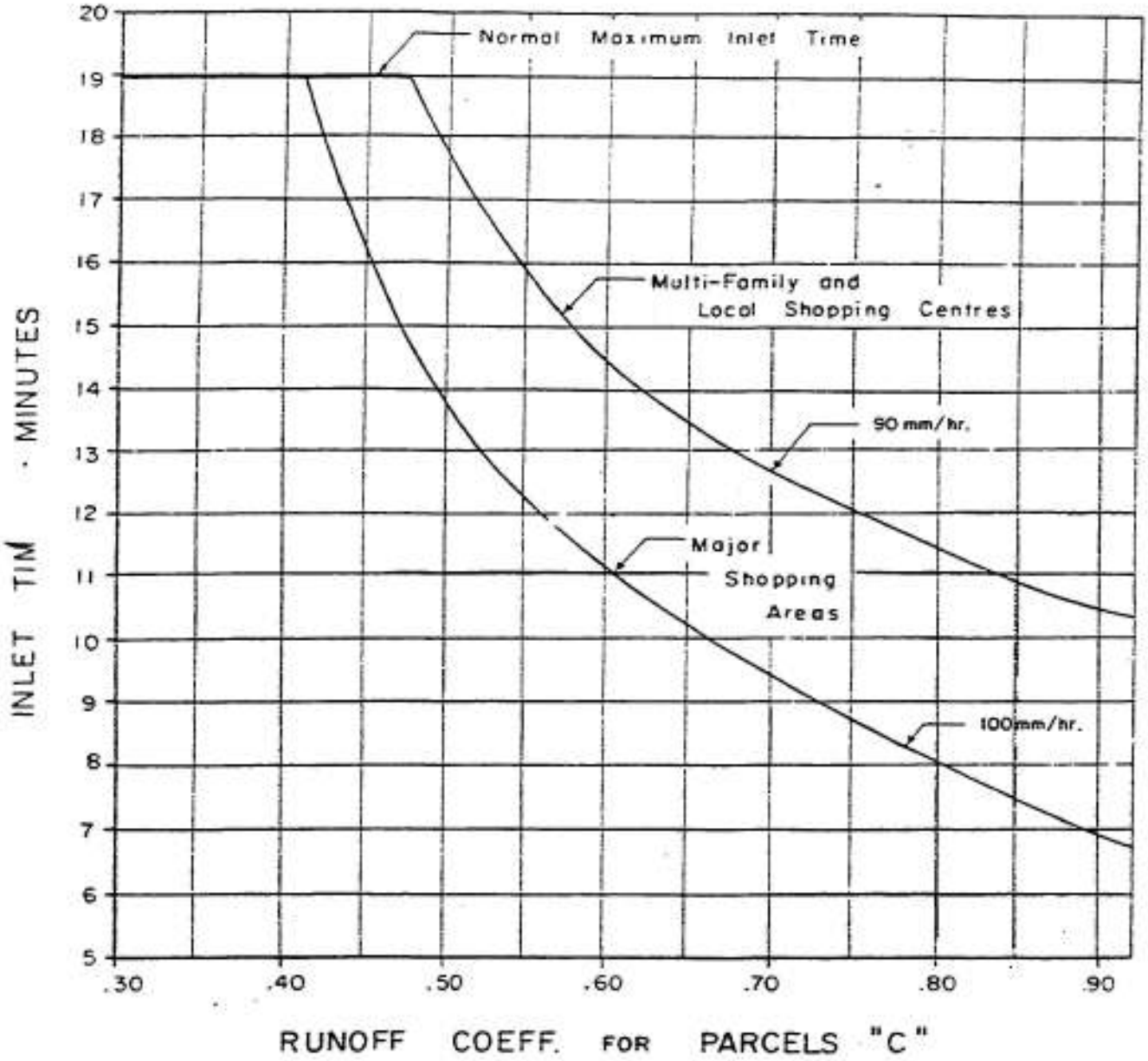
Figure 1.18

Many designers use a graph to obtain the time to be used for the design of the first length of storm sewer. The 90 mm/hr curve on Figure 1.19 is used in the City of London. The inlet time used for single family and semi-detached areas is 19 minutes. For multi-family and commercial areas, the time used varies with the runoff coefficient and the degree of protection desired.

The 90 mm/hr. curve has been used successfully on multi-family and local shopping centres. For major shopping areas, the 100 mm/hr. curve is recommended. High value areas are treated as special cases.

These curves were prepared by calculating the storm water runoff using the rainfall rates shown, typical designs, and the procedure outlined in the National Building Code. This Q was used in the Rational Formula along with the AC values for the typical designs to determine the i. The i was then used on the rainfall intensity-duration-frequency curve to determine the time. These times, rainfall rates and runoff coefficients were used to plot the curves.

When designing a storm sewer outlet for a large shopping centre or multi-family development, the question arises as to what time of concentration should be used. A number of municipalities use a curve similar to Figure 1.19 for all parcels up to a certain size. For these parcels, the time of concentration is taken directly from the curve. There is no exact limit to the size of parcel for which this method can be used. A number of municipalities use this approach for areas up to 3.5 hectares.



INLET TIMES FOR VARIOUS RUNOFF COEFFICIENTS

90 MM/Hr.	curve is used for multi-family and local shopping centres
100 MM/Hr.	curve is used for major shopping areas

High Value areas are treated as special cases

City of London curve

FIGURE 1.19

For larger areas, a 3.5 hectare portion of the site is marked off on the highest part of the site. The time of concentration from Figure 1.19 is used for this portion. Then, a time flow is calculated from this 3.5 hectare portion to the outlet sewer. The time of concentration used for the outlet sewer is the time of concentration for the 3.5 hectare portion plus the time of flow to the outlet sewer.

1.4.4 Drainage Areas

The first step in a storm sewer design is to determine the drainage areas. This is done in the following manner.

1.4.4.1 Exterior Watershed Limits and Drainage Areas

Obtain a map showing the major watercourse in the area. Plot on the area for which sewers will be designed. Draw on the watercourses and swales which pass through the design area to the top of their drainage areas. Draw on the drainage limits of the watercourses and swales which pass through the design area. If there is more than one watercourse and/or swale passing through the design area, draw the drainage limits separating these. If the drainage area is not too large, it is usually desirable to plot the property ownerships on the watershed plan.

The exterior drainage areas can now be defined by following the watershed limits. These frequently have to be adjusted to allow for the way these areas are most likely to develop. An example of this is that total property ownerships normally develop at one time. If part of the property is outside the drainage limits, but can be conveniently drained by the main drainage system within the property,

this is usually done and the drainage limits adjusted accordingly.

1.4.4.2 Interior Drainage Areas

(a) Sewer Layout

The watercourses and swales are plotted lightly on a plan of the area being designed. Recognizing the natural drainage pattern and the road grades the sewer system is laid out. The storm sewer system has to start at each location where a major watercourse or swale will have flows entering the design area. Storm sewers can be constructed to these locations. Alternatively, an inlet can be constructed at these locations to pick up the surface drainage. The storm sewers can follow the street pattern to drain the upstream watercourse when the upstream areas develop.

If weeping tile around houses and building foundations are to be connected to the storm sewers, the storm sewers are to be located in front of each parcel in the drainage area. If they are not, then the storm sewers can start at first catch basins on the streets.

(b) Sewer Drainage Limits

From the sewer layout and with a knowledge of the surface grading, the areas to be drained to each sewer can be drawn on the drainage plan. The exterior drainage areas are to be added to the interior drainage areas of the sewers into which they drain.

1.4.5 Runoff Co-efficients

Runoff coefficients are to be shown on the drainage plan for each drainage area within the total watershed drained by the sewers to be designed. If the lands are developed, runoff coefficients are to be assigned according to the existing land uses.

If the lands are undeveloped, the zoning

on the lands is to be determined and coefficients assigned accordingly. If the lands are not zoned, the Official Plan is to be checked. From this, the proposed land uses can be established and runoff coefficients assigned which will provide for the most likely future use of the land.

When considering the upstream lands outside the design area but within the watershed, one should keep in mind that land values are going up significantly throughout the Province. With this, land is being used more intensively. Therefore, when assigning runoff coefficients, one should be careful to give serious thought to the most likely time the land will develop and the type of development which will likely occur. It has generally been found that higher runoff coefficients should be used than that which is currently used for a similar type of development.

1.4.6 Inlet Times & Times of Concentration

Where sewers in the design area have only abutting lands tributary to them, the inlet time is obtained in the manner outlined in Section 1.4.3.3. Where sewers have areas outside the design area tributary to them, then the time of concentration has to be determined. This is done by sketching out a road pattern on the upstream undeveloped lands. This is to be consistent with the zoning or the Official Plan land uses. A storm sewer is plotted from the top of the watershed, along the most likely location of the main sewer to the inlet location in the design area. The drainage areas which would be tributary to this sewer are plotted, and a preliminary design done of the main storm sewer. This will provide the time of concentration for the sewers in the design area which will have these upstream areas connected to them in future.

Care should be taken not to assume an inlet time and a time of flow for these undeveloped lands unless very short times of flows are assumed. Many municipalities have had to spend considerable sums to correct situations where adequate provisions for upstream lands were not provided for in the design of storm sewers. This has

usually resulted from too low 'C' values and too long a time of flow being assigned to upstream undeveloped lands.

1.4.7 Sewer Size and Grade Selection

1.4.7.1 Storm Sewer Design Sheets

When designing storm sewers, it is best to use a standard design sheet. These have the advantage of presenting the design information in a clear, concise and organized manner. Figures 1.20 and 1.21 represent two types of design sheets commonly used. Figure 1.20 permits the use of any 'C' value. Figure 1.22 uses four standard land uses and 'C' values.

Some designers prefer design sheets with three columns for the slope of the sewer. One shows the slope to overcome the friction in the sewer, another shows additional slope required to overcome the head loss in the downstream manhole. The third shows the total grade of the sewer.

RUNOFF COEFF 'C'
PARKS & PLAYGROUNDS (.20 MIN.)
RESIDENTIAL SINGLE & DUPLEX (.35 ")
ROW HOUSING (.50 ")
APARTMENT (.60 ")
COMMERCIAL & INDUSTRIAL (.70 ")
DOWNTOWN BUILT, PAVED (.90 ")

CITY OF LONDON
CITY ENGINEERS DEPARTMENT
STORM SEWER DESIGN SHEET

PROJECT _____
FILE No _____ SHEET ____ OF ____
DRAINAGE AREA _____
DESIGNED BY _____ CHECKED BY _____
DATE _____

SEWER LOCATION			AREA		TOTAL AREA HECTARES	A × C					RAINFALL INTENSITY			Q cms	SEWER DESIGN							PROFILE			
STREET	FROM	TO	APPROX. AREA DIMENSIONS (m)	AREA HECTARES		RUNOFF COEFF 'C'	INCR. A × C	TOTAL A × C	TOTAL LAT. A × C	TOTAL A × C	TIME LINES—MIN. BEGIN	TIME LINES—MIN. ACCUM.	INTENSITY mm/hr		DIA. PIPE mm	SLOPE %	n	ACT. CAP. c.m.s.	VELOCITY m/s	LENGTH m	TIME OF FLOW MIN	GROUND EL. AT M.H.	FALL IN SEWER	INVERT EL. U.S.	INVERT EL. D.S.

CALCULATED BY _____ DATE _____

CHECKED BY _____ DATE _____

CITY OF SCARBOROUGH
WORKS DEPARTMENT

STORM SEWER DESIGN SHEET

Proposed - Sewer
As Constructed - Sewer
——— Separation between fields.
—·—·— New field beginning with implied decimal.
- - - - - - - - Position of implied decimal.

AREA CODE ☐☐☐ TT ☐ UNIT ☐ STREET NAME ☐☐☐☐☐☐☐☐☐☐☐☐☐☐☐☐☐☐☐☐ PAGE ☐

FROM _____ TO _____

FROM M.H.	TO M.H.	CITY USE ONLY			AREAS AND COEFFICIENTS								SEWER			TIME	LEG. AC.	ACCUM AC.	I	Q	CAP.	VEL	TIME	TOTAL TIME
		FROM	TO	REV	AREA	CO	AREA	CO	AREA	CO	AREA	CO	LENGTH	SIZE	GRADE									

1.4.7.2 Cover Over Storm Sewers

Section 1.6.2 outlines the minimum cover normally used for sewers. There are two additional factors to consider in the design of storm sewers.

One is that inlets or catch basins are placed in low spots with leads to the storm sewer. In these cases, the storm sewer has to be deep enough to drain these low spots.

In Section 1.4.6, it was pointed out that a preliminary design was needed on upstrem undeveloped lands. This was to obtain the time of concentration. In areas which do not have steep grades, the preliminary design also provides the cover required to provide for the upstream lands. In flat areas, greater care is required in the preparation of the preliminary design to establish proper cover. The depth of cover may not be established by the main sewer line. Rather it may be established by a lateral sewer which picks up a low or flat area to the side of the main line. Also, in flat sewers, the head losses in manholes can equal or exceed the fall in the sewer between manholes. Therefore, head losses should be provided for in the preliminary design.

1.4.7.3 Calculations

See the Hydraulics Section for recommended flow formula, nomographs, calculated tables, 'n' values and minimum and maximum velocities.

1.4.7.4 Time of Concentration Downstream of Junction Manholes

As stated previously, the time of concentration is the inlet time plus the time of flow in the sewer to the point in question. This time is

normally based on the sewer which starts at the most remote part of the watershed.

When two sewers meet, the time of concentration downstream of the junction manhole can be determined in three ways. These are:

- based on the sewer from the most remote portion of the watershed.
- based on the sewer with the largest flow
- based on an adjusted time obtained from proportioning the times by the flows in the two sewers.

The first two methods are probably satisfactory for small systems. However, once the system is larger, as in the case of a medium size subdivision, an adjustment to the time of concentration is recommended where the flow in a lateral is relatively large and has a different time of concentration. The adjustment is normally only made when the lateral flow is 50% or larger, than the trunk sewer flow.

The adjusted time of concentration is determined by the following formula (3):

$$t_{cx} = \frac{t_{ct}\, Qt + t_{cL} Q_L}{Q_t + Q_L}$$

t_{cx} = adjusted time of concentration in min.

t_{ct} = time of concentration in the trunk sewer in min.

t_{cL} = time of concentration in the lateral sewer in min.

Q_t = flow in the trunk sewer

Q_L = flow in the lateral sewer

The adjusted time of concentration is used downstream of the junction manhole.

1.5 CAPACITIES OF SEWERS

There are four basic flow conditions encountered in sewer design. They are: the sewer is flowing full, the sewer is flowing partially full, the sewer is surcharged and, the sewer has a submerged outlet. The capacities of the sewers with these flow conditions are all based on the capacity of the sewer flowing full with minor adjustments in each case. These adjustments are as follows:

1.5.1 Capacity Flowing Full

This is the basic condition for all sewer designs. The capacity is normally determined, using Manning's Formula. The capacity obtained is for the sewer flowing full without surcharging.

The designer should keep in mind that while the sewer has capacity to carry the flow calculated by Manning's Formula, it will not be able to carry this flow unless proper allowance for head losses in manholes, and junction chambers, are provided. This is particularly true when working with larger diameter storm sewers on fairly flat grades. See section 1.7 for methods of calculating head losses.

1.5.2 Capacity Flowing Partially Full

The capacity of a sewer flowing partially full is obtained by first determining the capacity of the sewer flowing full as noted in Section 1.5.1, then multiplying the capacity by the partial flow factor $\frac{Q}{Q_f}$ obtained from a hydraulic element graph such as Figure 1.3.2. The velocity, area, and hydraulic radius for partially full flows can be determined similarly.

Head losses at manholes and junctions normally do not effect the capacity of partially full sewers.

1.5.3 Hydraulic Grade Line

When considering sewers that are surcharged or where the sewer has a submerged outlet the design is based on the consideration of the piezometric/hydraulic grade line (HGL). The hydraulic grade line is defined as the line established by the height to which water would rise in open tubes located at all possible points along the pipe. Detailed discussion and derivation of the HGL is beyond the scope of this manual. For further details see the references listed at the end of the chapter.

For design purposes several rules of thumb are useful.

1. For open channels, pipes flowing partially full and pipes flowing full but not surcharged, the HGL generally follows the surface of the water.

2. In existing systems where flooding occurs, the HGL is the line defined by the height of the water in basements or surcharges in manholes, etc.

3. When designing storm sewers, the designer may want to check his proposed design against higher potential flows.

The HGL can be computed quickly using the Manning's or Hazen Williams equations or nomographs. The results will be approximately the same for each if n = 0.013 or c = 100 for the respective equations. For example: refer to Figure 1.5.1.

The problem is "given"—a proposed design giving size, slope, capacity and velocity is determined and the obvert elevations set. Then an overload is applied to the system.

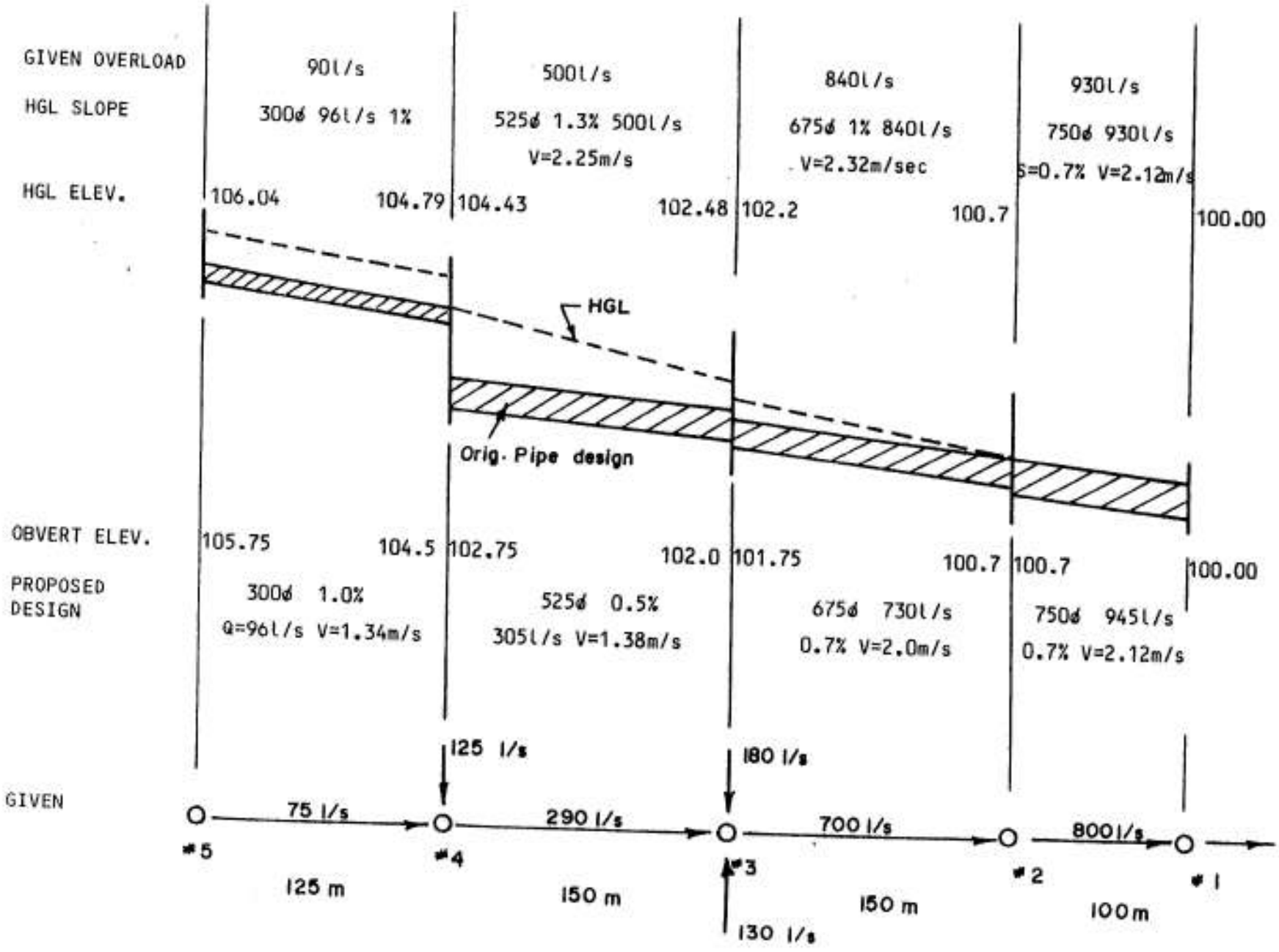

GIVEN OVERLOAD
90l/s
500l/s
840l/s
930l/s
HGL SLOPE
300ø 96l/s 1%
525ø 1.3% 500l/s
V=2.25m/s
675ø 1% 840l/s
V=2.32m/sec
750ø 930l/s
S=0.7% V=2.12m/s
HGL ELEV.
106.04
104.79
104.43
102.48
102.2
100.7
100.00
HGL
Orig. Pipe design
OBVERT ELEV.
105.75
104.5
102.75
102.0
101.75
100.7
100.7
100.00
PROPOSED DESIGN
300ø 1.0%
Q=96l/s V=1.34m/s
525ø 0.5%
305l/s V=1.38m/s
675ø 730l/s
0.7% V=2.0m/s
750ø 945l/s
0.7% V=2.12m/s
125 l/s
180 l/s
GIVEN
75 l/s
290 l/s
700 l/s
800l/s
#5
#4
#3
#2
#1
125 m
150 m
150 m
100m
130 l/s

Fig. 1.5.1.

This overload may occur due to the effect of higher intensity storms, or in an existing system because of flooding, etc.

To calculate the HGL work from the outlet up the system until the required capacity exceeds the pipe capacity. In this case the section from MH #2-3 is overloaded. Using Manning's nomograph calculate the slope to which the existing 675 ϕ pipe would have to be laid to carry the overload capacity of 840 l/sec. The nomograph shows the 675 dia. pipe would have to be laid at a slope of 1% to carry the capacity of 840 l/sec. Starting at the obvert of MH #2--where the HGL is known to be at the water surface, since the pipe is not surcharged but running full, the obvert elevation is 100.7. The HGL would follow a line of 1% slope from there to MH #3, elev. 102.2.

The jump in elevation from 102.2 to 102.48 occurs because of head losses in the manhole. Refering to Section 1.7, Figure 1.7.2 :

$$\frac{Qu}{Qo} = \frac{500}{840} = 0.60 \quad \therefore \quad K_L = 1.0$$

$$H_L = K_L \frac{Vo^2}{29} = 1.0 \times \frac{2.32^2}{2 \times 9.8} = 0.28m$$

hence the jump. Similarly from MH #3 to MH #4 the surcharge

flow is 500 l/sec in the 525 dia. pipe. The required slope for the 525 dia. pipe to carry this flow is 1.3%. Starting at MH #3 at 102.48 the HGL follows a grade line of 1.3% to MH #4. The elevation at this point is 104.43. At MH #4 head losses occur and can be calculated as above. The section of 300 dia. pipe from MH #4 to MH #5 has sufficient size and slope to carry the required flow of 90 l/sec; however, the HGL is higher than obvert level due to the downstream surcharge conditions. Thus the HGL at MH #4 steps from 104.43 to 104.79 because of manhole head losses then follows a grade line of 1% to MH #5 finishing at an elevation of 106.04.

The significance of the HGL occurs when there are buildings, etc. connected to the system which are below the elevation of the hydraulic grade line. They will experience flooding during the peak periods of the storm and in fact act as unplanned temporary holding tanks during these times.

The calculation of the HGL as shown above can be used under the following conditions.

1.5.4 Capacity Of Surcharged Sewers

There are three conditions where a designer may be involved with sewers which are surcharged. These are: when

determining the maximum capacity of the sewer system, when checking the flows that occurred in a sewer when flooding took place, or when it is desirable to design all or part of a system with the sewers surcharged.

The conditions are approached as follows:

1.5.4.1 Maximum Capacity Of Sewer System

The length of the system, for which the maximum capacity is to be established, is determined. On the profile of this length of sewer the highest HGL [piezometric grade line (pressure or surcharge line)] which can be permitted is established. The upper limit may be set by, having to stay a safe distance below basement floors on properties abutting the sewer, by the limit of flooding permissible on adjacent low lands, or by lateral sewers.

In the case of lateral sewers, it is usually best to first determine what the piezometric grade line will be in the main sewer with the maximum permissible flow. Then check the lateral sewers to see if the surcharge will

(b) (Cont'd.)

obtain the details of the storm to be used. Also, the policies related to the application of the regional storm should be obtained. These requirements frequently will control the critical dimensions of the open channel.

(c) Protection Against Flooding of Basements - is a major factor where foundation weeping tiles are connected to storm sewers. In these cases, the degree of protection to be provided for the buildings is first established. Often, a 25-year storm frequency is recommended. Then a check is made to ensure that when the storm sewers are carrying the flows from a 25-year storm, that the surcharging of the storm sewer system does go above the basement floor elevations. This often means that the H.W.L. in the open channel, when carrying a 25-year storm, cannot be above the obvert of the connecting storm sewers, as this would cause unacceptable backing up in the storm sewers.

1.12.1.6 Aesthetics

In many municipalities, trapezoidal channels constructed on straight lines, are acceptable in industrial and commercial areas but not in residential areas. In residential areas, long straight channels, which look like canals, are not acceptable. Some allowance for aesthetic treatment has to be made. The extent to which this is done varies with municipalities. Factors that are commonly considered in trying to make the channel look more natural are:

drainage of the soil.

(c) Grass Along Channels - is not cut too short, in order to prevent drying out of the ground and for protection against erosion during high flows. In locations where the grass can be left fairly long, short grass varieties are sometimes selected to reduce or eliminate the need for cutting. Grassed side slopes are usually 3:1 or flatter for ease of cutting.

1.12.1.5 Depth

There are a number of factors which affect the decision on the depth of the channel. Each factor has to be checked to see which will establish the depth required.

(a) Basic Channel Dimensions - are established by placing the high water level (H.W.L.) at the crown of the connecting storm sewers and the invert of the main channel at the invert of the connecting storm sewers. The bottom width of the channel is then calculated. If the connecting storm sewers are small, some municipalities assume a depth below the H.W.L. of 2 to 3 metres. These basic dimensions are checked against the requirements of acceptable maximum velocities, available right-of-way, regional storm permissible, H.W.L.'s protection to buildings with weeping tile connected to storm sewers, and other factors related to the particular site.

(b) Regional Storm Requirements - as set out by the Ministry of Natural Resources is frequently the controlling factor. It is recommended that, prior to commencing the design of any open channel, the Chief Engineer of the Conservation Branch of the Ministry of Natural Resources be contacted to

the job is not large enough or suitable for the use of concrete. Concrete-lined channels are usually limited to installations carrying high velocities, in permanent locations, within a limited space.

1.12.1.4 Maintenance

(a) Sufficient room is normally provided at the top of each side for maintenance vehicles. This is frequently taken to be a minimum of about 6 m.

(b) Dry Weather Flow Channels are usually provided in the bottom of channels. These are provided to carry the natural flow which occurs in channels during dry weather periods. They confine these low flows to specific locations thereby permitting the remainder of grass-lined channels to drain and be used for maintenance purposes. They also help carry the silt, suspended in the flow, to the outlet of the channel. This helps avoid silting of the channel. In "Vee" channels, the dry weather flow channel is the bottom of the "V." In other channels, they are a small channel constructed in the bottom of the main channel. (See Figure 1.70.) The size of the dry weather flow channel is usually taken as the same size as the natural channel which existed before improvements. If none existed previously, or it was dry for much of the season, it is still a good design feature, even if it is small. Flat-bottomed channels become overgrown and messy. The dry weather flow channel collects the silt and debris in it to be washed out during the next storm.

The freeboard on dry weather flow channels can be about 150 mm on gabion or concrete-lined channels, to about 300 mm on grass-lined channels. The depth of freeboard varies with the ease of

1.12.1.2 Safety

Safety is an emotional factor. To some, an open watercourse is the same as a street. It has its potential dangers, but children and others have to learn to live with them. Others feel they should be fenced for the protection of all.

Many municipalities have accepted grass-lined channels with 3:1 side slopes without fences. However, fences are frequently used where there are concrete or gabion-lined channels with higher velocities, which would be difficult for a child to crawl out.

1.12.1.3 Linings

Earth channels are normally only used for temporary works or for farm drains. They silt up quickly, become overgrown, and generally lose much of their carrying capacity in 5 to 10 years.

Grass-lined channels are generally a first choice where room is available. They can only be used for lower velocities, generally less than 1.5 to 1.8 m.p.s. They are economical, and can be aesthetically pleasing.

Riprap-lined channels present problems due to the size of stones required. If small stones are used, children and teenagers frequently throw them down the sides, exposing the filter layer or bare ground. If stones which are too large to be moved are used, the thickness (generally 1.5 times the thickness of the largest stones), becomes quite expensive. For these reasons, riprap channels are not used too often.

Gabion-lined channels are generally used where velocities are higher than can be permitted on grass-lined channels; where it is desirable to permit drainage to pass through the lining; where some movement is anticipated. Therefore, a flexible material is needed and/or

1.12 OPEN CHANNELS

1.12.1 Basic Considerations

1.12.1.1 Open Channels vs. Piped Storm Sewers

Most municipal engineers have to decide at some time whether large storm water flows should be carried in a storm sewer or an open channel. Some engineers calculate the cost of constructing and maintaining storm drainage systems carrying various flows in storm sewers and in open channels. These two cost curves vs. flows are plotted on a graph. These graphs show storm sewers most economical for lower flows and open channels for larger flows. The point at which open channels become more economical is taken by some engineers to be about 14-15 m^3/s.

This change point is not constant. There are a number of factors which will affect whether this point should be higher or lower. The major factors are:

- The limitations on flooding permitted during a "Regional Storm." These requirements of the Ministry of Natural Resources often make it desirable to go to open channels at lower flows.
- Desire to preserve natural ravines and watercourses.
- Need for open space.
- Land costs.
- Limitations on available lands.
- Safety for children and adjacent vehicular traffic.

Once the decision is made to consider or to build an open channel, a number of factors related to open channels are reviewed before the type of channel is selected. These factors are outlined in the following subsections.

1.11.4.3 Designed Filter Layers

Where it is essential that the riprap and filter system not fail, the filter layer(s) is selected on the basis of well established criteria. This is usually done where the riprap is large, where there is wave action, at gabion weirs across watercourses, and at gabion retaining walls. Filter layers (12) are selected on the following criteria:

Criterion 1:

The 15% size (D_{15}) of a filter material must be not more than 4 to 5 times the 85% size (D_{85}) of a protected soil. This will prevent piping of the soil.

Criterion 2:

The 15% size (D_{15}) of a filter material should be at least 4 to 5 times the 15% size (D_{15}) of a protected soil. This will permit water to pass through the filter.

This can be expressed as follows:

$$\frac{D_{15}\ \text{(of filter)}}{D_{85}\ \text{(of soil)}} < 4 \text{ to } 5 < \frac{D_{15}\ \text{(of filter)}}{D_{15}\ \text{(of soil)}}$$

Additional information on the design and application of filter layers can be found in "Seepage, Drainage and Flow Nets" (12).

Filter layers have a smaller gradation than riprap. However, they have a sufficient number of particle sizes which are too large to pass through the voids in the riprap. These particles block the large voids in the riprap and leave smaller voids between them and the riprap. These are in turn blocked by the smaller sizes in the filter layer. The same blocking of voids occurs between the filter layer and the soil being protected.

If the size of the riprap is large, and the grain size of the soil to be protected is small, two or more filter layers may be required. This occurs when a filter material is not available which can be placed without segregation, and has the wide range of sizes required to block all the voids from the large ones in the riprap down to the size which would prevent the passing of the adjacent soil.

1.11.4.2 Selection of Filter Layer Material

(a) Pit Run Gravel - which is reasonably well graded, is generally used below riprap adjacent to headwalls and bridges, and along watercourses. It functions well when:
- There is little or no wave action, which tends to draw the finer soil out through the riprap.
- The riprap is not normally large.
- In these installations, the voids in the riprap tend to fill in and grass and weeds grow in the fill material.

(b) Crushed Gravel - which is uniformly graded and on the coarse side (i.e., MTC Granular 'A'), is frequently used where some additional care is required.

- Its thickness is to be 1.5 times (or more) the diameter of the largest stone.
- It is to be placed over a filter blanket or bedding of graded gravel.

(b) Care should be taken in preparing drawings and specifications for riprap since the primary cause of riprap failure is the undersizing of individual stones in the maximum size range, and a general tendency for the riprap actually placed to be smaller than specified.

1.11.3.3 Gabions

Frequently, it is desirable to place riprap in protective wire baskets. This is often done when the stone, if not protected, would be dislodged by people climbing over it, or be thrown around by vandals. They are also used where the stone protection is to be in the form of retaining walls, groynes, or form part of the aesthetics of the design.

Technical information on the use of gabions is available from the various manufacturers of the wire baskets.

1.11.4 Filter Layers

1.11.4.1 Background

Filter layers are placed between the riprap and the soil to prevent the action of the water from washing the soil through the riprap. This is called the piping requirement. When this happens, a void is created behind the riprap, which, in turn, settles. As the settlement continues, a failure of the riprap occurs. Filter layers also permit water to pass from the soil out through the riprap. This is referred to as the permeability requirement.

Soil Type	Maximum Permissible Velocities - m.p.s. (8)
Ordinary firm loam	1.06
Stiff clay	1.52
Shales and hardpan	1.83

Grass cover - is frequently used where velocities are higher than permitted by the above table. The velocities which grass cover permits are as follows (9):

- Grass mixtures will permit velocities up to:
  1.22 m.p.s. on easily eroded soils, and
  1.52 m.p.s. on erosion resistant soils.
- Kentucky Bluegrass, which has good cover and is properly maintained, can permit velocities up to:
  1.52 m.p.s. on easily eroded soils, and
  2.13 m.p.s. on erosion resistant soils.

Where adequate protection cannot be obtained by the soil itself, or grass cover, riprap may be used.

1.11.3 Riprap Selection

1.11.3.1 Use of Graphs

Where water flows rapidly from a sewer outlet, concrete lined channel, bridge, or other water carrying installation, the size of riprap needed to prevent erosion can readily be selected by using graphs as prepared by the U.S. Bureau of Reclamation (10) or nomographs as prepared by the State of California (11). When the specific gravity of the stone is unknown, it is common practice to assume it to be 2.65.

1.11.3.2 Riprap Specifications

(a) Riprap specifications usually state that:
- It is to be well graded.
- Not more than 40% of the rock pieces are to be smaller than the size obtained from use of charts or nomographs.

1.11 RIPRAP

1.11.1 Purpose

Riprap is usually stone, crushed rock, or broken concrete. It is placed along rivers, creeks, open channels, and at headwalls, bridges, or other locations where water flows or wave action occurs. Its function is to act as a buffer between the water and the adjacent soil to prevent the action of the water from eroding the soil.

Riprap is required wherever the flow or wave action of the water exceeds the ability of the abutting soil to withstand the action of the water.

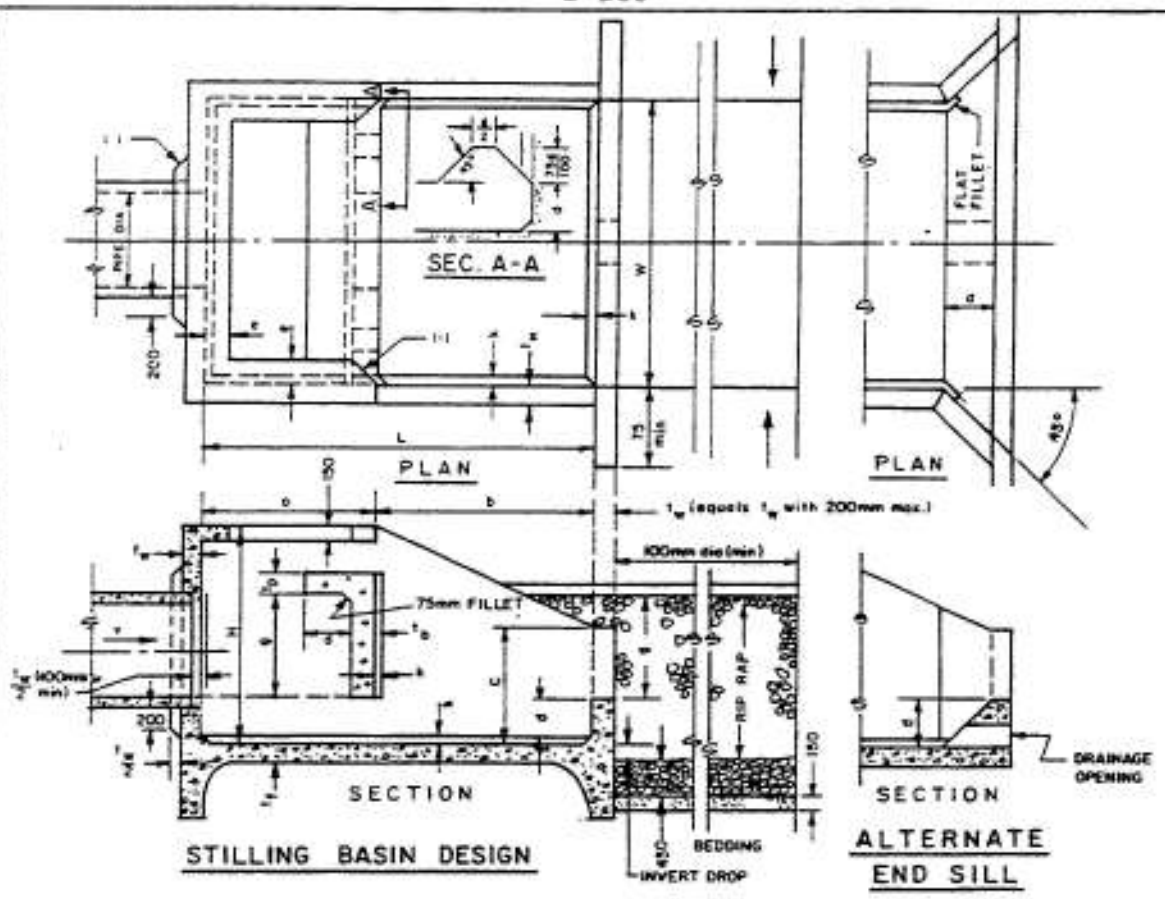
In some cases, the water action is significantly greater than the ability of the abutting soil to withstand. In other cases, it is desired to place the abutting soil at a slope steeper than it could normally stand in a saturated state. In these situations, a filter layer or layers are placed beneath the riprap to prevent the finer soil from washing out through the voids in the riprap.

Riprap with a filter layer can also be used to hold banks in place, and permit water from the bank to pass through it while keeping the soil in place.

1.11.2 Maximum Permissible Velocities for Soils

The maximum average velocities which usually will not cause erosion of the soils are as follows:

Soil Type	Maximum Permissible Velocities - m.p.s. (8)
Fine sand	0.75
Fine gravel	1.52
Coarse gravel	1.83
Sandy loam	0.75
Silty loam	0.91



* ALL DIMENSIONS IN mm

Q	e	k	c	t_w	t_f	t_b	t_p	Recommended Riprap size
100	230*	75	$c=\frac{150}{300}W$	200	200	230	200	240
200	300	300	$c=\frac{175}{300}W$	250	280	250	200	300
300	355	150	$c=\frac{125}{300}W$	300	300	300	200	340
400	405	150	$c=\frac{125}{300}W$	300	300	300	200	375
SUGGESTED MINIMUM THICKNESS OF CONCRETE IS 150mm								

SUGGESTED CONCRETE DIMENSIONS

H = 3/4 W L = 4/3 W
e = 7/12 W b = L - a
d = 1/6 W g = 3/8 W

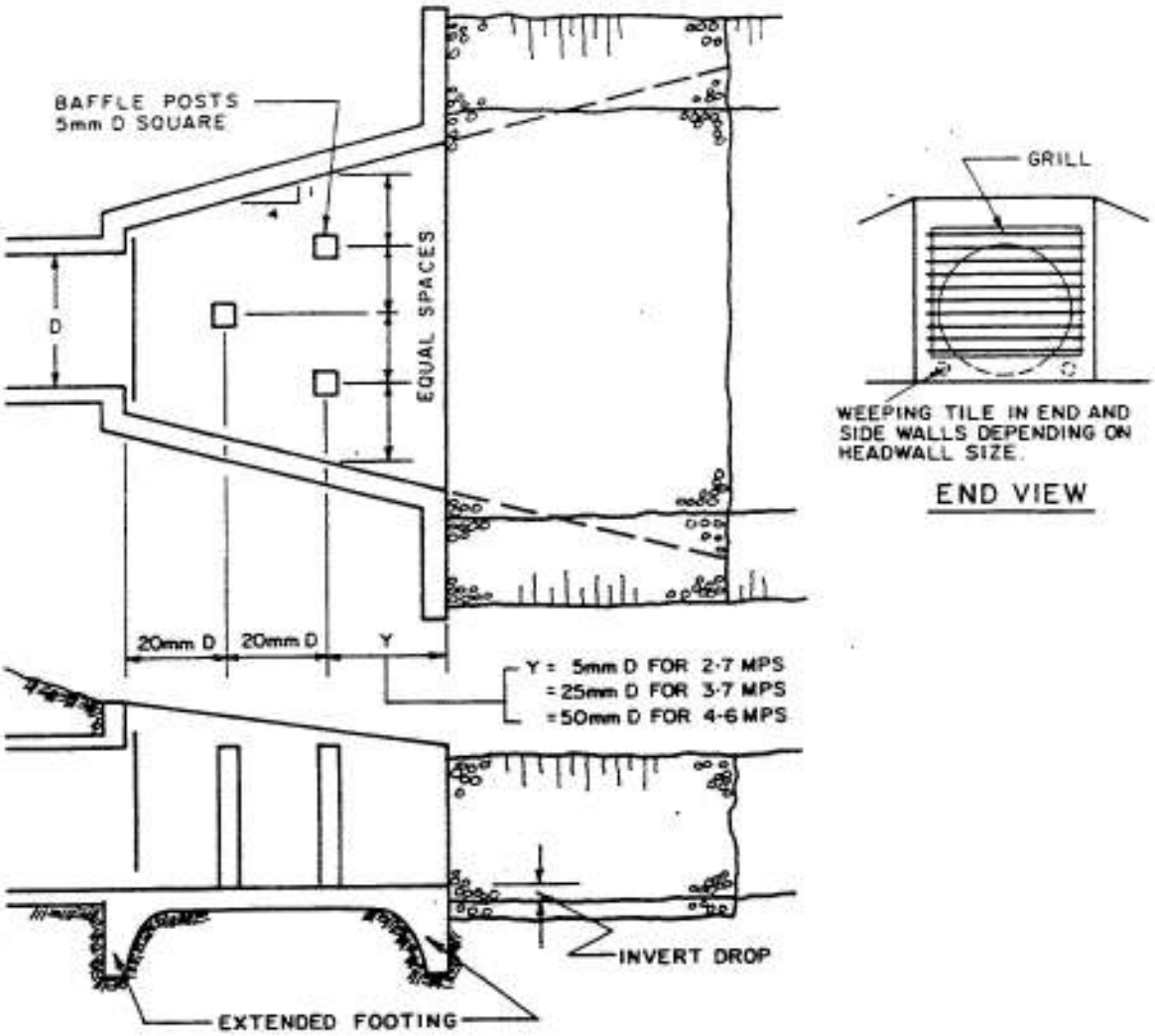
NOTES

1. TYPE IV HEADWALL CAN BE USED FOR: - VELOCITIES UP TO 10 METRES PER SECOND (M.P.S)
   DISCHARGES UP TO 11.3 CUBIC METRES PER SECOND (C.M.S)
   FOR LARGER DISCHARGES MULTIPLE BASINS CAN BE PLACED SIDE BY SIDE.
2. THE DESIGN IS BASED ON THE INCOMING QUANITY OF WATER NOT ITS VELOCITY (EXCEPT THAT IT SHOULD NOT EXCEED 10 M.P.S.
3. THE BOTTOM OF THE BAFFLE SHOULD BE PLACED AT THE SAME LEVEL AS THE INVERT OF THE UPSTREAM PIPE.
4. THE MAXIMUM INCOMING SEWER GRADE SHOULD BE 28% (15°). FOR GREATER SLOPES USE A HORIZONTAL OR SLOPING PIPE (UP TO 28%) FOR TWO OR MORE DIAMETERS LONG JUST UPSTREAM FROM THE HEADWALL.
5. THE NOTCHES IN THE BAFFLE FACILITATE THE CLEANING OF SILT FROM THE BASIN IF SILTING IS NOT CONSIDERED A PROBLEM THEY CAN BE DELETED
6. FOR BEST POSSIBLE PERFORMANCE THE ALTERNATIVE END SILL AND 45° END WALLS ARE RECOMMENDED
7. THE RIPRAP USED SHOULD HAVE MOST OF THE STONES EQUAL TO OR LARGER THAN THE SIZE IN THE TABLE
8. THIS TYPE OF HEADWALL IS SUBJECTED TO LARGE DYNAMIC FORCES, VIBRATION AND SCOURE. THEREFORE THE PLACING OF KEYS AND REINFORCING STEEL MUST BE SUFFICIENT TO RESIST THESE CONDITIONS
9. CONCRETE TO BE AIR - ENTRAINED
10. ALL EXPOSED CORNERS TO BE CHAMFERED.
11. WEEPING TILE TO EXTEND FROM SEWER BEDDING THROUGH THE HEADWALL
12. GRILL TO BE PROVIDED ON ALL HEADWALLS.
13. INVERT DROP VARIES WITH AVAILABILITY OF DEPTH AND SIZE OF HEADWALL, NORMALLY A MINIMUM OF 150mm TO 230mm
14. FOR HEADWALLS AT CREEKS AND RIVERS - EXTEND RIPRAP OR GABION PROTECTION TO CREEK OR RIVER PROTECTION IS TO PROVIDE A SMOOTH HYDRAULIC FLOW FOR HEADWALL DISCHARGE AND CREEK OR RIVER FLOWS

FIGURE 1-66

HEADWALL - TYPE IV

SEWER FLOWS UP TO 10 MPS



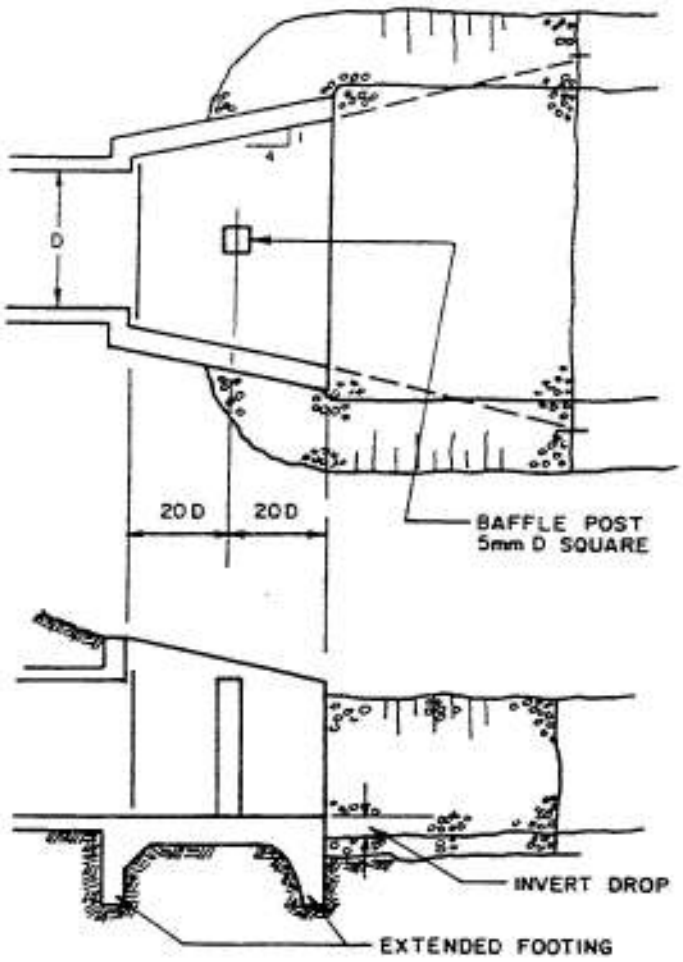
NOTES

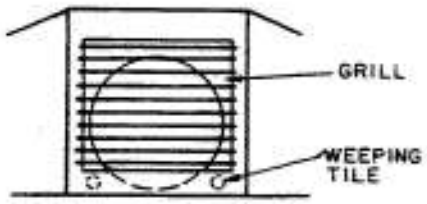
1. CONCRETE TO BE 20MPa WITH 5-7% AIR-ENTRAINMENT.
2. ALL EXPOSED CORNERS TO CHAMFERED (25mm MIN.)
3. WEEPING TILE TO EXTEND FROM SEWER BEDDING THROUGH THE HEADWALL.
4. GRILL TO BE PROVIDED ON ALL HEADWALLS.
5. INVERT DROP VARIES WITH AVAILABILITY OF DEPTH AND SIZE OF HEADWALL, NORMALLY A MINIMUM OF 150 TO 225mm.
6. RIPRAP TO BE PLACED IN OUTLET CHANNEL
   - ON BOTTOM AND SIDES UP TO DESIGN WATER LEVEL.
   - DOWNSTREAM TILL THE PROJECTION OF THE SIDE WALLS MEET THE CHANNEL SIDE SLOPES AT HALF THE DESIGN WATER DEPTH OF FLOW.
7. FOR HEADWALLS AT CREEKS AND RIVERS - EXTEND RIPRAP OR GABION PROTECTION TO CREEK OR RIVER. PROTECTION IS TO PROVIDE A SMOOTH HYDRAULIC FLOW FOR HEADWALL DISCHARGE AND CREEK OR RIVER FLOWS.

FIGURE 1-65

HEADWALL - TYPE III

SEWER FLOWS 2·7 TO 4·6MPS





END VIEW

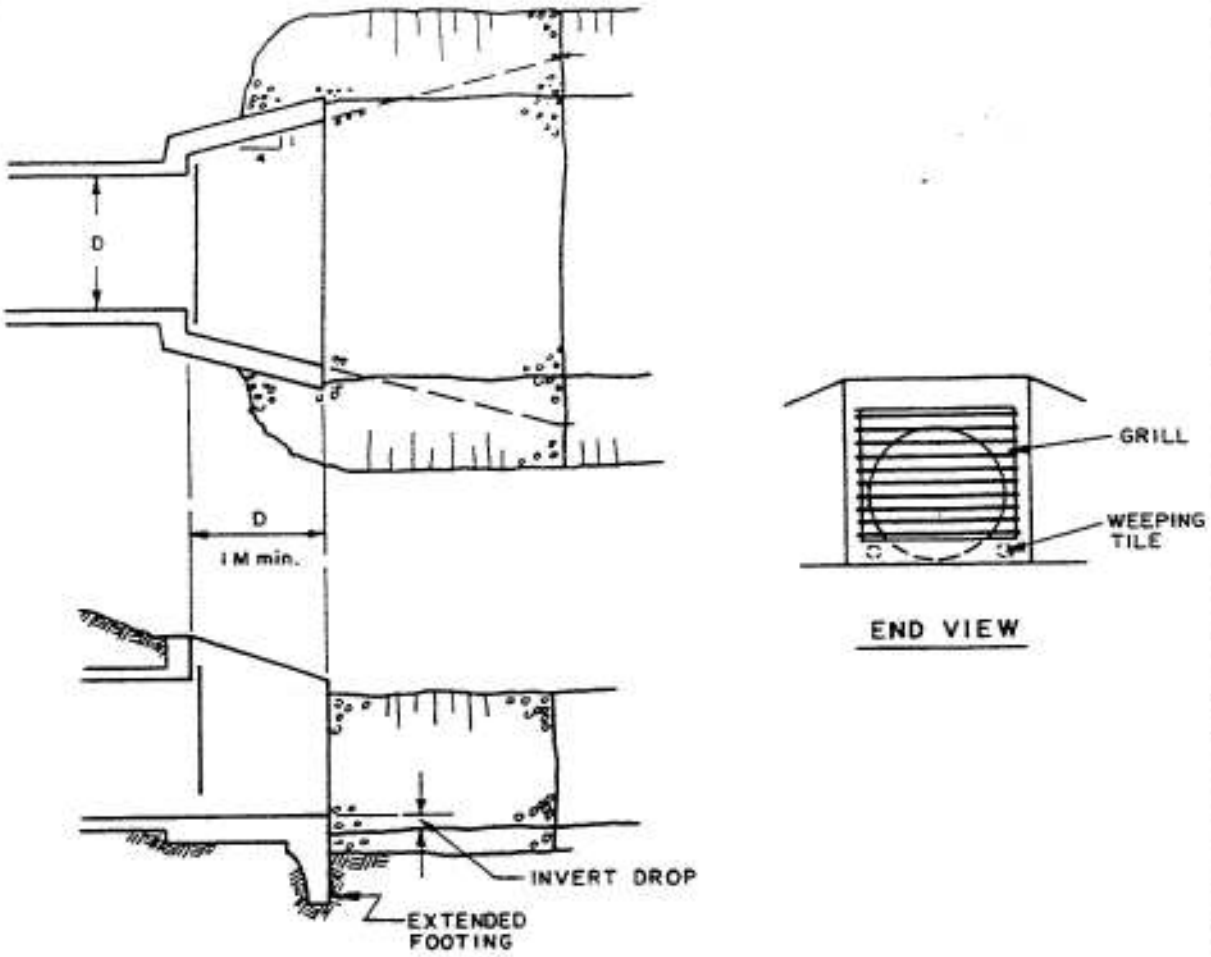
NOTES

1. CONCRETE TO BE 20MPa WITH 5-7% AIR-ENTRAINMENT.
2. ALL EXPOSED CORNERS TO CHAMFERED (25mm MIN.)
3. WEEPING TILE TO EXTEND FROM SEWER BEDDING THROUGH THE HEADWALL.
4. GRILL TO BE PROVIDED ON ALL HEADWALLS.
5. INVERT DROP VARIES WITH AVAILABILITY OF DEPTH AND SIZE OF HEADWALL, NORMALLY A MINIMUM OF 150 TO 225mm.
6. RIPRAP TO BE PLACED IN OUTLET CHANNEL
   - ON BOTTOM AND SIDES UP TO DESIGN WATER LEVEL.
   - DOWNSTREAM TILL THE PROJECTION OF THE SIDE WALLS MEET THE CHANNEL SIDE SLOPES AT HALF THE DESIGN WATER DEPTH OF FLOW.
7. FOR HEADWALLS AT CREEKS AND RIVERS - EXTEND RIPRAP OR GABION PROTECTION TO CREEK OR RIVER. PROTECTION IS TO PROVIDE A SMOOTH HYDRAULIC FLOW FOR HEADWALL DISCHARGE AND CREEK OR RIVER FLOWS.

FIGURE 1-64

HEADWALL - TYPE II

SEWER FLOWS 2·1 TO 2·7MPS



NOTES

1. CONCRETE TO BE 20MPa WITH 5-7% AIR-ENTRAINMENT.
2. ALL EXPOSED CORNERS TO CHAMFERED (25mm MIN.)
3. WEEPING TILE TO EXTEND FROM SEWER BEDDING THROUGH THE HEADWALL.
4. GRILL TO BE PROVIDED ON ALL HEADWALLS.
5. INVERT DROP VARIES WITH AVAILABILITY OF DEPTH AND SIZE OF HEADWALL, NORMALLY A MINIMUM OF 150 TO 225mm.
6. RIPRAP TO BE PLACED IN OUTLET CHANNEL
   - ON BOTTOM AND SIDES UP TO DESIGN WATER LEVEL.
   - DOWNSTREAM TILL THE PROJECTION OF THE SIDE WALLS MEET THE CHANNEL SIDE SLOPES AT HALF THE DESIGN WATER DEPTH OF FLOW.
7. FOR HEADWALLS AT CREEKS AND RIVERS - EXTEND RIPRAP OR GABION PROTECTION TO CREEK OR RIVER. PROTECTION IS TO PROVIDE A SMOOTH HYDRAULIC FLOW FOR HEADWALL DISCHARGE AND CREEK OR RIVER FLOWS.

FIGURE 1-63

HEADWALL - TYPE I

SEWER FLOWS UP TO 2-1 MPS

When the velocity increases the headwall structure must be able to withstand the additional thrust and vibration. The use of endwalls, which project into the channel sides and/or an additional footing below the structure should be provided to resist these forces.

cast in to the headwall. Anchor bolts using lead sleeves often pull out, and are not recommended.

1.10.3.5 Invert Drops

Silt and sand are carried in suspension in storm water. The amount carried will vary with the character of the drainage area and the velocity of flow. When the velocity of flow is slowed down by the headwall, the amount of silt and sand which the water can carry in suspension, is decreased. This results in silt and sand deposits downstream of the headwall. These deposits are significant when the outlet watercourse is relatively flat. In these cases, the deposits build up just downstream of the headwall causing ponding and the collection of debris in the headwall. By dropping the invert of the downstream channel 150 mm to 225 mm below the invert of the headwall, the headwall can remain relatively free of ponding and debris. If large amounts of silt and sand are carried by the storm water, only regular removal of the deposits will keep the outlet clear.

1.10.3.6 Keys (Extended Footings)

Frequently, there is settling or erosion at the end of the headwall. To prevent undermining of the structure, a key, extended footing, or cut off wall is constructed at the end of the headwall apron.

animals from gaining access in to the sewer. To prevent access to the storm sewer, bars in the grill are normally placed no further than 130 mm apart. Grills can cause a significant resistance to flow at the headwall. This is caused by debris collecting on the bars and reducing the effective waterway through the grill.

This problem is reduced by:

- placing most of the bars in the grill horizontally, thereby reducing the amount of debris held by the bars,
- having the thinnest portion of the bar directed against the flow thereby providing for a larger waterway area.
- regular maintenance to clear material which impedes the flow.

Normally little energy is required to push the water through the grill. However, when the velocity in the storm drain is high, care should be exercised in the selection of the bar dimensions. The following energy losses are based on bars in the grill being horizontal with a spacing of 130 mm on centre and only one or two vertical bars in the direct line of the flow.

Dimension of bar at right angles to the flow	Velocity of the storm water	Energy Loss
6 mm	3.1 m.p.s.	0.1 m
9	6.1	0.3 m
13	6.1	0.4 m
25	6.1	1.4 m

Grills are often attached to the headwall by bolts

various design conditions. The designs have been proven by successful installations in many municipalities.

1.10.3 Design Details

1.10.3.1 Concrete

Concrete in headwalls is subjected to normal freezing and thawing cycles during the winter, and to attack by the salts contained in road runoff. It should be the same high quality as used in concrete curbs and gutters and sidewalks and should have a minimum strength of 20 MPa with 5 - 7% air-entrainment and low slump.

1.10.3.2 Chamfers

All exposed corners should be chamfered 25 mm or more depending on the size of the structure.

1.10.3.3 Weeping Tile

Often there is water migrating through the trench backfill material. This water should drain through the headwall, thereby reducing any pressure which might build up behind the headwall. To accomplish this, weeping tiles are placed on each side and at the bottom of the sewer outlet and extended through the headwall. On larger structures, they are also placed in the side or wing walls.

1.10.3.4 Grills

In urban areas, grills are normally placed over storm sewer outlets at headwalls to prevent children and

1.10 HEADWALLS

1.10.1 Purpose

Headwalls are placed at the end of storm sewers to provide a transition from the storm sewer to the open channel, watercourse, river or other receiving body such as a lake. Where the velocity of flow in the sewer is high, the headwall is designed to reduce the velocity to that which will not cause erosion in the receiving watercourse. The headwall is also designed to resist the undermining and erosion action of the water at the outlet and thereby ensuring a permanent and functional structure at the end of the storm sewer.

Where a headwall is required at the inlet of a storm sewer or a large storm drain the design would be similar to a culvert headwall. The reader should refer to the section on culverts in this Chapter for design requirements.

1.10.2 Recommended Headwall Designs

The type of headwall selected is normally based on the velocity of flow in the outlet sewer and the site conditions.

Where the flow in the sewer is low, the structure required is small. OPSD-804.03 is an example of this type of structure. (4)

As velocity increases, the requirements for slowing the flow are also increased.

Headwalls Types I, II, III and IV, shown on Figures 1.63 to 1.66 are designs which have been developed to meet

(e) Flexible Sewer Pipe Installation

General bedding and backfilling concepts developed for rigid sewer pipe may also be applied to flexible pipe, with some exceptions. The general pipe terminology for installation is shown in Fig.9-6.

Detailed information on bedding of flexible pipe is contained in the various ANSI and ASTM specifications or manufacturers literature. The design engineer should consult the applicable specification or literature for design information.

(i) Bedding, Haunching and Initial Backfill

The bedding requirements for flexible thermoplastic sewer pipe are given in ASTM D 2321 for Class I, II and III material (Table 9-2). Haunching and initial backfill requirements are also given, including minimum compaction recommendations. By referring to Tables 9-3 and 9-5, it is possible to estimate the E' value obtained for the expected compaction or to determine the compaction requirements needed to develop the E' value which will keep the pipe deflection within allowable limits.

Similarly, bedding and initial backfill requirements for thermosetting reinforced plastic pipe are given in ASTM D 3839.

(c) Plastic Sewer Pipe Installation

At the present time plastic sewer pipe is not manufactured in a wide range of stiffnesses or wall thicknesses. To enable the pipe to support different heights of fill it is necessary to construct the embedment around the pipe so as to develop the required soil-structure interaction. In general this is measured by the E' value for the installation. Only by increasing the E' value is it possible to support increasingly higher fills and increased loads.

(i) Bedding, Haunching and Initial Backfill

The bedding requirements for flexible thermoplastic sewer pipe are given in ASTM D 2321 for Class I, II and III material (Table 9-2). Haunching and initial backfill requirements are also given, including minimum compaction recommendations. By referring to Tables 9-2 and 9-3 it is possible to estimate the E' value obtained for the expected compaction or to determine the compaction requirements needed to develop the E' value which will keep the pipe deflection within allowable limits.

Similarly, bedding and initial backfill requirements for thermosetting reinforced plastic pipe are given in ASTM D 3839, and a guide for estimating both initial and long-term deflection is included in the appendices.

(d) Design of Corrugated Steel Sewer Pipes

Corrugated steel pipe is manufactured in a variety of gauges, corrugation depths, and corrugation spacings.

The longitudinal seam formed by bolting or riveting curved sheets together for corrugated steel pipe may have to be checked for crushing strength for heavy backfill loads. Tables of seam strengths for various metal gauges and bolt or rivet sizes and spacing can be found in manufacturers' handbooks. Corrugated steel pipe is not used in sanitary sewers without a protective coating which may or may not be asbestos bonded. To improve hydraulic properties the pipe may have a paved invert (25% of circumference) or may have the interior completely paved.

Corrugated steel pipe sewers may be designed for a limiting deflection using the Modified Iowa Formula (Eq.9.5) or by the manufacturer's handbook figures. A design deflection not greater than 5% is commonly used.

and soil stiffness. The primary controlling elements of deflection are the load and the soil stiffness.

Sewer pipe stiffness generally is a relatively minor factor by comparison with soil stiffness. The soil stiffness depends on the type of soil and its density as placed in the sewer pipe embedment zone. In consideration of design requirements, the engineer should evaluate the options of different types of flexible sewer pipes, different embedment soils, and different embedment material compaction with placement requirements.

The choice of factor of safety should be influenced by soil characteristics, the degree of compaction likely to be obtained, any available field tests, and practical experience.

(iii) Field Deflection of Flexible Plastic Pipe

As previously discussed, the pipe stiffness is determined in a standard parallel-plate loading test for an arbitrary deflection of the nominal diameter. This is a measure of the inherent strength of plastic pipe.

The pipe stiffness and soil stiffness as measured by E' must develop sufficient field strength that the deflection of the pipe under load will not exceed the recommended permissible deflection.

Approximate field deflections can be calculated using the Modified Iowa Formula (Eq.9.5). This formula can be simplified to permit a calculation of approximate deflection based on pipe stiffness as determined by Eq 9.8:

$$\Delta X = \frac{D_L K_b W_c}{0.149PS + 0.061E'} \qquad (9.8)$$

(Note: If W_c = prism load for flexible pipe, $D_L \sim 1.$)

Values of bedding factors for use in this formula may be found under the "General Methods" subsection of this section.

The solution of Equation 9.8 requires that various factors be determined. It is desirable, where possible, to establish anticipated long-term field deflections based on well documented empirical data.

The stiffness factor or EI is used in the Modified Iowa Formula (Eq.9.5) to determine approximate field deflections under earth loads. It is the engineer's responsibility to establish the acceptable field deflection limit, and to design the installation accordingly. The manufacturer should be consulted for recommended field installation deflection limits.

Specifications also may require that some types of plastic sewer pipe withstand extreme deflections, such as 40% of the original diameter in a parallel-plate loading test without evidence of splitting, cracking, or breaking. There is no corresponding load requirement. These extreme deflection tests are instantaneous and are intended for production quality control during pipe manufacturing. Although these extreme deflections (observed in ASTM 2412 testing) are limiting in terms of duration and extent of pipe distortion, they can be important to the design engineer in selecting sewer pipe materials of construction.

(ii) Design Relationships

Structural design of flexible plastic pipe requires definition of the critical deflection limit for the specific pipe considered. The critical deflection limit for flexible plastic pipe is commonly based on structural performance characteristics and potential modes of failure. The requirements of serviceability, such as effect on carrying capacity, passage of cleaning equipment, velocity and differential deflection at fittings, manholes and joints, also may be considered. Maximum strain in the pipe wall, discussed above, also should be considered.

The design basis is expressed as follows:

$$\text{Design Deflection Limit} = \frac{\text{Critical Deflection Limit}}{\text{Factor of Safety}}$$

In structural design for a flexible sewer pipe system, the design deflection limit should not be more than the maximum long-term deflection anticipated under load. In the determination of projected long-term deflection, the design element considerations are load (soil, superimposed dead load, and superimposed live load); pipe stiffness;

where E = modulus of elasticity, in pascals; I = $t^3/12$; r = mean radius of pipe, in millimeters and t = wall thickness, in millimeters.

Minimum required pipe stiffness values are stated in plastic sewer pipe specifications. Table 9-12 lists the ASTM Specifications for the various types of plastic pipe and the corresponding pipe stiffness values.

The stiffness factor (SF) is the pipe stiffness multiplied by the quantity of $0.149r^3$.

$$SF = EI = \frac{F}{\Delta Y} 0.149r^3$$

$$= 0.149r^3 \ (PS) \qquad (9.7)$$

Table 9-5. Stiffness Requirements for Plastic Sewer Pipe Parallel-Plate Loading*

Material ** (1)	ASTM Specification (2)	Nominal diameter,d, in inches (3)	Required Stiffness at 5% deflection, in pounds per square inch (4)
PVC	D2729 (PVC-12454)	2	59
		3	19
		4	11
		5	9
		6	8
	D 2729 (P.V.C.-13364)	2	74
		3	24
		4	13
		5	12
		6	10
	D 3033		
	SDR 41	6-15	28
	SDR 35	4-15	46
	D 3034		
	SDR 41	6-15	28
	SDR 35	4-15	46

*ASTM 2412
**Other plastic pipe materials are not listed, in that insufficient data is currently available.
NOTE: 1 in. = 25.4 mm, 1 psi = 6.89 Kpa

material characteristics. Cohesive soils are generally less suitable because of the importance of proper moisture content and the difficulty of obtaining proper compaction in a limited work space.

If, under embankment conditions, the material placed around the sewer pipe is different from that used in the embankment, or if for construction reasons fill is placed around the sewer pipe before the embankment is built, the compacted backfill should cover the pipe by at least 0.3 m and extend at least one diameter to either side.

(b) Design of Plastic Sewer Pipe

A wide range of physical properties is available from various plastic sewer pipe materials. Many have ASTM or ANSI specifications.

Thermoplastic pipe materials (PE, and PVC) are all affected by temperature. Specified test requirements are applicable at 23°C (73.4°F). At higher temperatures the plastic becomes less rigid but impact strength is increased. Lower temperatures result in increased brittleness but greater pipe stiffness. Pipe being installed at low temperatures requires careful handling.

(i) Laboratory Load Test

The standard test to determine "pipe stiffness" or load deflection characteristics of plastic pipe is the parallel-plate loading test. This test is conducted in accordance with ASTM D 2412, "Standard Test Method for External Loading Properties of Plastic Pipe by Parallel-Plate Loading."

In the test a short length of pipe is loaded between two rigid parallel flat plates as they are moved together at a controlled rate. Load and deflection are noted.

The parallel-plate loading test determines the pipe stiffness (PS) at a prescribed deflection (ΔY), which for convenience in testing is arbitrarily set at 5%. This is not to be considered a limitation on field deflection. The pipe stiffness is defined as the value obtained by dividing the force (F) per unit length by the resulting deflection in the same units at the prescribed percentage deflection, and is expressed in newtons per millimeter:

$$PS = \frac{F}{\Delta Y} = \frac{EI}{0.149r^3} \qquad (9.6)$$

[a]ASTM Designation D-2487, USBR Designation E-3.
[b]LL = Liquid limit.
[c]Or any borderline soil beginning with one of these symbols (i.e., GM-GC, GC-SC)
[d]For ±1% accuracy and predicted deflection of 3%, actual deflection would be between 2% and 4%.
NOTE: Values applicable only for fills less than 50 ft. (15 m). Table does not include any safety factor. For use in predicting initial deflections only, appropriate Deflection Lag Factor must be applied for long-term deflections. If bedding falls on the borderline between two compaction categories, select lower E' value or average the two values. Percentage Proctor based on laboratory maximum dry density from test standards using about 12,500 ft-lb/cu ft (598,000 J/m³) (ASTM D-698, ASSHO T-99, USBR Designation E-11). 1 psi = 6.9 Kpa.

Table 9-4 Values of Bedding Constant K_b

Bedding Angle, in Degrees (1)	B_b (2)
0	0.110
30	0.108
45	0.105
60	0.102
90	0.096
120	0.090
180	0.083

NOTE: 1 deg = 0.017 rad

It should be noted that the E' values in Table 9-3 are average values. Using them results in a 50% chance that the actual deflections will be higher than calculated. A conservative approach would be to use 75% of the E' values given to calculate maximum deflections.

The performance of the flexible sewer pipe in retaining its shape and integrity is largely dependent on the selection, placement, and compaction of the envelope of soil surrounding the structure. For this reason as much care should be taken of the bedding and initial backfill as is used in the design of the sewer pipe. The backfill

Table 9-3 Bureau of Reclamation Average Value of E' for Iowa Formula (for Initial Flexible Pipe Deflection)

Soil type-pipe bedding material (Unified Classification System[a]) (1)	E' for Degree of Compaction of Bedding, in pounds per square inch			
	Dumped (2)	Slight <85% Proctor <40% relative density (3)	Moderate 85%-95% Proctor 40%-70% relative density (4)	High >95% Proctor >70% relative density (5)
Fine-grained Soils (LL 50)[b] Soils with medium to high plasticity CH, MH, CH-MH	No data available, consult a competent soils engineer: Otherwise use E' = 0			
Fine-grained Soils (LL 50). Soils with medium to no plasticity CL, ML, ML-CL, with less than 25% coarse-grained particles	50	200	400	1,000
Fine-grained Soils (LL 50) Soils with medium to no plasticity CL, ML, ML-CL, with more than 25% coarse-grained particles	100	400	1,000	2,000
Coarse-grained Soils with fines GM, GC, SM, SC[c] contains more than 12% fines				
Coarse-grained Soils with Little or No Fines GW, CP, SW, SP[c] contained less than 12% fines	200	1,000	2,000	3,000
Crushed Rock	1,000	3,000	3,000	3,000
Accuracy in Terms of Percentage Deflection[d]	±2	±2	±1	±0.5

Lag factors over 2.5 have been recorded in dry soil. Recommended values of this factor range from 1.25 to 1.50. Bedding requirements for flexible sewer pipe installation are discussed below.

Values of the bedding constant K_b depending on the width of the sewer pipe bedding, are shown in Table 9-4.

The passive resistance of the soil at the sides of the pipe greatly influences flexible pipe deflection. This passive resistance is expressed as the Modulus of Soil Reaction, E'. It is generally recognized as being related to the degree of compaction of the soil and to the type of soil. Extensive laboratory and field tests by the U.S. Bureau of Reclamation have resulted in the establishment of an empirical relationship between Modulus of Soil Reaction values, degree of compaction of bedding and type of bedding material (Table 9-3). (See Table 9-2 for soil symbol definition).

In the deflection formula (Eq.9.5), the first term in the denominator, EI (stiffness factor), reflects the influence of the inherent stiffness of the sewer pipe on deflection. The second term, $0.061\ E'r^3$, reflects the influence of the passive pressure on the sides of the pipe. With flexible pipes, the second term is normally predominant.

Empirical data on long-term deflection in different burial conditions generally can be obtained from the manufacturer. When such design data are not available, the approximate long-term deflection of flexible sewer pipe can be calculated using the Modified Iowa Formula developed by Spangler and Watkins:

$$\Delta X = \frac{D_L K_b W_c r^3}{EI + 0.061\ E' r^3} \qquad (9.5)$$

where ΔX = horizontal deflection, in millimeters; K_b = bedding factor; D_L = deflection lag factor; W_c = load, in newtons per linear millimeter; r = mean radius of pipe, in millimetres; E = modulus of tensile elasticity in pascals; I = moment of inertia per length, in millimeters to fourth power per millimeter; and E' = modulus of soil reaction, in pascals.

For small deflections, the vertical deflection ΔY may be assumed to approximately equal the horizontal deflection ΔX in Eq. 9.5. Much research is being conducted on the application of Eq. 9.5 to pipe having a low ratio of pipe stiffness to soil stiffness, where the vertical deflection may not be assumed to approximately equal the horizontal deflection. The result of this research is beyond the scope of this chapter but should be reviewed by the designer as a part of the design process.

Eq. 9.5 was developed primarily for flexible corrugated steel sewer pipe under embankments but has been shown to be applicable to most flexible pipe materials.

Flexible sewer pipes that are to support a fill should not be placed directly on a cradle or pile bents. If such supports are necessary, they should have a flat top and be covered with a compressible earth cushion. In those instances where flexible pipe is to be encased in concrete, the pipe manufacturer should be consulted. Flexible pipes should not be encased in concrete.

Recommended values for E' in Eq. 9.5 have been developed and are given in Table 9-3.

The deflection lag factor, empirically determined, compensates for the time consolidation characteristics of the soil, which may permit flexible sewer pipes to continue to deform for some period after installation. Long-term deflection will be greater with light or moderate degrees of compaction of sidefills when compared to values for heavy compaction. The better the compaction, the lower the initial deflection, and the greater the magnitude of the long-term lag factor.

$H/B_c = 6.63$: $C_c = 10.0$; $x = 0.856$; and $N = 0.575$

From Eq. 9.16:

$$q = \frac{0.5 \times 0.33}{10.0} (6.63 + 0.25) = 0.114$$

and from Eq. 9.15:

$$L_f = \frac{1.431}{0.575 - (0.856 \times 0.114)} = 3.01$$

The required three-edge bearing strength at ultimate load is:

$$\frac{29{,}100 \times 1.5}{3.01} = 14{,}500 \text{ lb/ft } (211{,}700 \text{ N/m}), \text{ or } 3{,}360 \text{ D} (175 \text{ D N/m/mm diam.})$$

Therefore ASTM C76 Class V pipe should be used.

1.9.6 Flexible Sewer Pipe Design

Several types of flexible pipe are available for use as sewer pipe material. Among the most common are polyvinyl chloride (PVC) pipe, polyethylene (PE) pipe and corrugated steel pipe (CSP).

(a) General Methods

Flexible sewer pipes under earth fills and in trenches derive their ability to support load from their inherent strength plus the passive resistance of the soil as the pipe deflects and the sides of the sewer pipe move upward against the soil sidefills. Proper compaction of the soil sidefills is important to the long term structural performance of flexible sewer pipe. This type of pipe fails by excessive deflection and by collapse, buckling, cracking or delamination rather than by rupture of the sewer pipe walls as in the case of rigid pipes. The extent to which flexible pipe deflects is most commonly used to judge performance and as a basis for design. The amount of deflection considered permissible is dependent on physical properties of the pipe material used and project limitations.

The limiting buckling stress for flexible pipes takes into account the restraining effect of the soil structure around the pipe and the properties of the pipe wall. Equations for the critical stress in the pipe wall can be found in manufacturers' handbooks for the various types of pipe.

can be achieved in locations with appropriate soil conditions, trench width control, and light superimposed loads. Care must be taken to prevent point loading of sewer pipe bells; the excavation of bell holes will prevent such loading. Existing soil should be shovel sliced or otherwise compacted under the haunching of the sewer pipe to provide some uniform support. In poor soils, granular bedding material is generally a more practical, cost effective installation.

Since the field conditions with Class D bedding can approach the load conditions for the three-edge bearing test, the bedding factor for Class D bedding is 1.1.

(d) Sample Calculations:

Example 9-5. Refer to Example 9-1, which describes the backfill load on a 24 in. diameter pipe with 14 ft. of cover; the load was found to be 4,720 lb/ft. If vitrified clay pipe is to be specified and a factor of safety of 1.5 is selected, the design load will be 4,720 x 1.5 or 7,080 lb/ft (103,300 N/m).

The crushing strength requirement of 24-in. diameter extra-strength clay sewer pipe (ASTM C700) based on the three-edge bearing method is 4,400 lb/ft. Dividing this into 7,080 lb/ft (the casing load), the minimum required bedding factor of 1.61 is obtained. Fig. 9-8 indicates that a Class B bedding is required for this installation.

Example 9-6. Refer to Example 9-3. If a 24 in. diameter reinforced concrete sewer pipe (one line of reinforcement near center of wall) and a factor of safety of 1.50 based on the minimum ultimate test strength are selected, the design load will be 1.50 x 5,750 or 8,620 lb/ft (125,900 N/m).

If the minimum ultimate test strength of the pipe is 6,000 lb/ft (ASTM C76, Class IV-3,000 D), the required bedding factor will be 8,620/6,000 or 1.44. According to Fig. 9-19, this installation will require a Class C bedding.

Example 9-7. Assume that a 48 in. diameter pipe is bedded on an unreinforced concrete cradle, with p equal to m equal to 0.5 and k equal to 0.33. Using a factor of safety of 1.5 based on the ultimate three-edge bearing strength of the pipe:

spring line of the sewer pipe and across the full breadth of the trench. The haunching material beneath the sides of the arch must be compacted so as to be unyielding. Crushed stone in the 5-mm to 20-mm size range is the preferred material. The top half of the sewer pipe is covered with a cast-in-place plain or reinforced concrete arch having a minimum thickness of 100 mm or one-fourth the inside pipe diameter but not to exceed 380 mm, and having a minimum width equal to the outside sewer pipe diameter plus 200 mm.

The bedding factor for Class A concrete arch-type bedding is 2.8 for plain concrete; up to 3.4 for reinforced concrete with p equal to 0.4%; and up to 4.8 for p equal to 1.0%.

(ii) Class B Bedding

The sewer pipe is bedded in carefully compacted granular material. The granular bedding has a minimum thickness of one-eighth the outside sewer pipe diameter but not less than 100 mm or more than 150 mm, between the barrel and the trench bottom, and covering the full width of the trench.

The haunch area of the sewer pipe must be fully supported; therefore, the granular material should be shovel sliced or otherwise compacted under the pipe haunch. Both haunching and initial backfill to a minimum depth of 300 mm over the top of the sewer pipe should be placed and compacted.

The bedding factor for Class B bedding is 1.9.

(iii) Class C Bedding

The sewer pipe is bedded in compacted granular material. The bedding has a minimum thickness of one-eighth the outside sewer pipe diameter, but not less than 100 mm or more than 150 mm, and shall extend up the sides of the sewer pipe one-sixth of the pipe outside diameter. The remainder of the sidefills, to a minimum depth of 150 mm over the top of the pipe, consists of lightly compacted backfill.

The bedding factor for Class C bedding is 1.5.

(iv) Class D. Bedding

For this class of bedding the bottom of the trench is left flat, as cut by excavating equipment. Successful Class D installations of rigid conduits

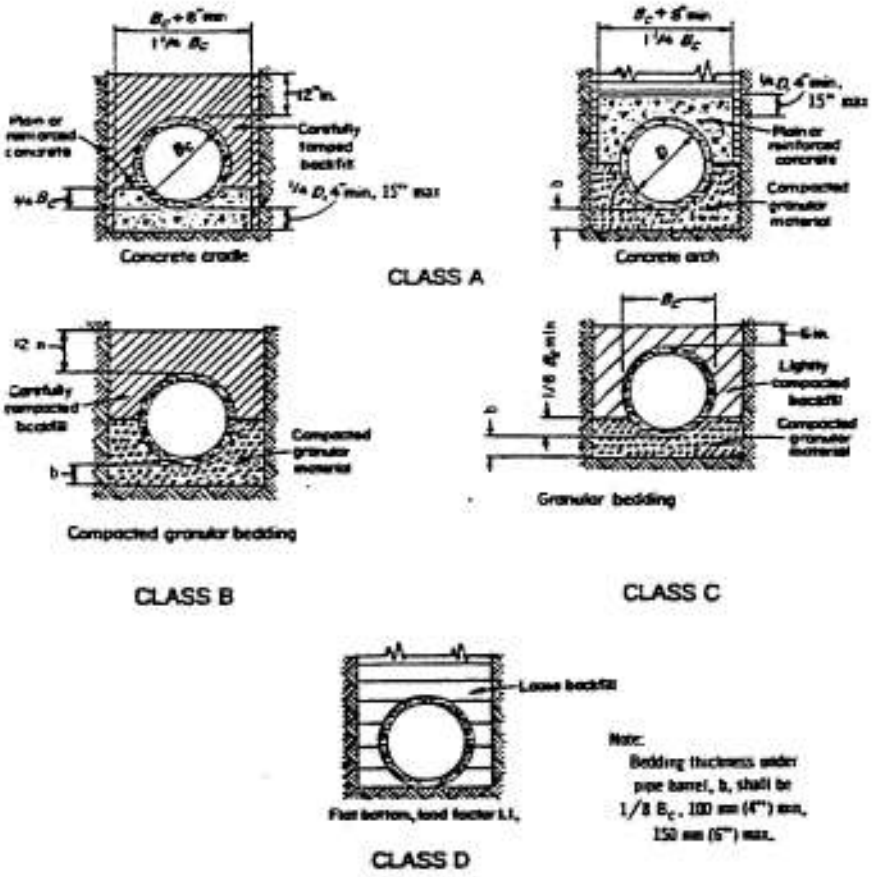


Fig.9-8. Classes of bedding for rigid sewer pipes in trench. NOTE: In rock trench, excavate at least 15 cm (6 in.) below bell of pipe except where concrete cradle is used (in. x 25.4 = mm).

In the case of reinforced concrete sewer pipe, laboratory strengths are divided into two categories: The load that will produce a 0.3-mm crack, and the ultimate load the sewer pipe will withstand. According to ASTM C 76, the ultimate strength load in three-edge bearing is 1.50 to 1.25 times greater than the load that will produce a 0.3-mm crack, and the factor of safety used in design depends on which load is being considered.

Asbestos cement, non-reinforced concrete, and clay sewer pipe class designations indicate the ultimate strength in three-edge bearing directly in newtons per meter (pounds per foot) for any diameter. A Class 1500 asbestos cement sewer pipe, for example, must have a laboratory ultimate strength of 22,000 N/m (1,500 lb/ft) for every diameter in that class.

(c) Rigid Sewer Pipe Installation-Classes of Bedding and Bedding Factors for Trench Conditions

Four classes of beddings are used most often for sewer pipe in trenches (Fig. 9-8). They are described in the following subsections.

(i) Class A - Concrete Cradle

The sewer pipe is bedded in a cast-in-place cradle of plain or reinforced concrete having a thickness equal to one-fourth the inside pipe diameter, with a minimum of 100 mm and a maximum of 380 mm under the pipe barrel and extending up the sides for a height equal to one-fourth the outside diameter. The cradle shall have a width at least equal to the outside diameter of the sewer pipe barrel plus 200 mm. Construction procedures must be executed carefully to prevent the sewer pipe from floating off line and grade during placement of the cradle concrete.

The bedding factor for Class A Concrete Cradle bedding is 2.2 for plain concrete, with lightly tamped backfill; 2.8 for plain concrete with carefully tamped backfill; up to 3.4 for reinforced concrete with 0.4% reinforcement and up to 4.8 with 1.0% reinforcement.

Class A - Concrete Arch

The sewer pipe is bedded in carefully compacted granular material having a minimum thickness of one-eighth the outside sewer pipe diameter but not less than 100 mm or more than 150 mm between the sewer pipe barrel and bottom of the trench excavation. Granular material is then placed to the

It is recommended that the in-place density of Class I and Class II embedment materials be measured by ASTM D2049 by percentage of relative density, and Class III and Class IV measured by either ASTM D2167, D1556 or D2922, by percentage of Standard Proctor Density according to ASTM D698 or AASHTO T99.

An economical indicator of proper bedding and backfill execution on flexible sewer pipes may be by testing the inside pipe deflection by a go/no go mandrell. Mandrell dimensions can be determined by applying the initial deflection to the base inside diameter as determined from the appropriate ASTM pipe standard. The initial deflection does not include the deflection lag factor, as is applied to ultimate long-term design deflection.

1.9.5 Rigid Sewer Pipe Design

(a) General Relationships

The inherent strength of a rigid sewer pipe usually is given by its strength in the three-edge bearing test. Although this test is both convenient and severe, it does not reproduce the actual field load conditions. Thus to select the most economical combination of bedding and sewer pipe strength, a relationship must be established between calculated load, laboratory strength, and field strength for various installation conditions (Fig.9-7).

Field strength, moreover, depends on the distribution of the reaction against the bottom of the sewer pipe and on the magnitude and distribution of the lateral pressure acting on the sides of the pipe. These factors, therefore make it necessary to qualify the term "field strength" with a description of conditions of installation in a particular case, as they affect the distribution of the reaction and the magnitude and distribution of lateral pressure.

(b) Laboratory Strength

Rigid sewer pipe is tested for strength in the laboratory by the three edge bearing test (Fig. 9-7). Methods of testing are described in detail in ASTM Specification C 301 for vitrified clay sewer pipe, C 497 for concrete and reinforced concrete sewer pipe, and C 500 for asbestos cement sewer pipe.

The minimum strengths required for the three-edge bearing tests of the various types of rigid sewer pipe are stated in the ASTM Specifications for the pipe.

Steel sheeting may be used and reused many times, and the relative economy of this type of sheeting compared with timber or timber and soldier beams should be explored. Because of the thinness of the sheeting it is often feasible to achieve reasonable compaction of backfill so that the steel sheeting may be withdrawn with about the same factor of safety against settlement of the surfaces adjacent to the trench as that for other types of sheeting left in place.

(ii) Trench Boxes

Several backfilling techniques are possible when a trench box is used. Granular material can be placed between the box and the trench wall immediately after placing the box and the box is advanced by lifting slightly before moving forward. Boxes can be made with a step at the rear which makes the trench wall accessible for compacting the embedment against the walls. Also, the box can be used where the sewer pipe is laid in a small sub-trench, and the box is utilized only in the trench above the top of the sewer pipe. Advancement of the box must be done carefully to avoid pulling pipe joints apart.

(iii) Pipe Bedding and Embedment

To assure that the sewer pipe is properly bedded or embedded it is suggested that compaction tests be made at selected or critical locations, or that the method of material placement be observed and correlated to known results.

Where compaction measurement or control is desired or required, the recommended references are:

* ASTM D2049, "Standard Method of Test for Relative Density of Cohesionless Soils"

* ASTM D698, "Standard Method of Test for Moisture Density Relations of Soils Using 2.5 kg Rammer and 204.8 mm Drop"

* ASTM D2167, "Standard Method of Test for Density of Soil in Place by the Rubber-Balloon Method"

*ASTM D1556, "Standard Method of Test for Density of Soil in Place by the Sand-Cone Method"

*ASTM D2922, "Standard Method of Test of Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)."

4. Field Conditions. The field engineer should be furnished with sufficient design date to enable him to evaluate unforeseen conditions intelligently. The field engineer should be instructed to confer with the design engineer if changes in design appear advisable.

5. Sheeting. Where sheeting is to be removed, pulling should be done in stages. The space formerly occupied by the sheeting must be backfilled completely, and field inspection should verify this.

(i) Effect of Trench Sheeting

Because of the various alternative methods employed in sheeting trenches, generalizations on the proper construction procedure to follow to ensure that the design load is not exceeded are risky and dangerous. Each method of sheeting and bracing should be studied separately. The effect of a particular system on the sewer pipe load, as well as the consequences of removing the sheeting or the bracing, must be estimated.

It is difficult to obtain satisfactory filling and compaction of the void left when wood sheeting is pulled. Wood sheeting driven alongside the sewer pipe should be cut off and left in place to an elevation of 450 mm above the top of the sewer pipe.

If granular materials are used for backfill it is possible to fill and compact the voids left by the wood sheeting if the material is placed in lifts and jetted as the sheeting is pulled. If cohesive materials are used for backfill, a void will be left on pulling the wood sheets and the full weight of the prism of earth contained between the sheeting will come to bear on the sewer pipe.

Skeleton sheeting or bracing should be cut off and left in place to an elevation of 450 mm over the top of the sewer pipe if removal of the trench support might cause a collapse of the trench wall and a widening of the trench at the top of the conduit. Entire skeleton sheeting systems should be left in place if removal would cause collapse of the trench before backfill can be placed.

Where steel soldier beams with horizontal lagging between the beam flanges are used for sheeting trenches, efforts to reclaim the steel beams before the trench is backfilled may damage pipe joints. It is recommended that use of this type of sheeting be allowed only on stipulation that the beams be pulled after backfilling and the lagging be left in place.

must be able to deflect without cracking liner failure, or other distress, and they should be designed accordingly with a reasonable factor of safety.

Certain types of plastic pipes are subject to strain deterioration. There is a limiting strain which , if exceeded over a period of time, will eventually result in cracking of the pipe wall, particularly if exposed to an acid environment. Manufacturers should be consulted on the value of this limiting strain.

(d) Recommendations for Field Procedures

The factor of safety against ultimate collapse of sewer pipe is about the same as that used in the design of most engineered structures of monolithic concrete. However, the design of sewer pipe is based on calculated loads, bedding factors, and experimental factors, which are less well-defined than the dead and live loads used in building design. It is therefore, important that the loads imposed on the sewer pipe be not greater than the design loads.

To obtain the objective of imposed loads being less than design loads, the following procedures are recommended:

1. Specifications. Construction specifications should set forth limits for the width of trench below the top of sewer pipe. The width limits should take into account; the minimum width required to lay and join sewer pipe and the maximum allowable width for each class of sewer pipe and bedding to be used. Where the depth is such that a positive projecting condition will be obtained, maximum width should be specified as unlimited unless the width must be controlled for some reason other than to meet structural requirements of the sewer pipe. Appropriate corrective measures should be specified in the event the maximum allowable width is exceeded. These measures may include provision for a higher class of bedding or concrete encasement. Maximum allowable construction live loads should be specified for various depths of cover if appropriate for the project.

2. Inspection. Construction should be observed by an experienced engineer or inspector who reports to a competent field engineer.

3. Testing. Sewer pipe testing should be under the supervision of a reliable testing laboratory, and close liaison should be maintained between the laboratory and the field engineer.

adequate factors of safety must be realized in installation and operation to provide reasonable assurance of acceptable longterm system performance.

The relationship between safety factors and design performance values as related to structural requirements is similar for rigid and flexible sewer pipe. However, an understanding of the differences between design requirements for rigid sewer pipe and the requirements for flexible sewer pipe system design is important.

Recommended field procedures and safety factors are discussed below.

(b) Rigid Sewer Pipe

Design performance limits for rigid sewer pipes generally are expressed in terms of strength under load. There are two alternative methods of determining the service strength of reinforced concrete sewer pipe - by "strength design" analysis, or by testing. The design method more accurately represents the relative performance of pipe in the field under the service load. However, testing is generally used for all types of precast or prefabricated rigid pipe.

Strengths of rigid sewer pipe usually are measured in terms of the ultimate three-edge bearing strength, and of ultimate and 0.3-mm (0.01-in.) crack, three-edge bearing strengths for reinforced concrete sewer pipe. A factor of safety of 1.0 should be applied to the 0.3 mm (0.01-in.) crack load for reinforced concrete sewer pipe and of 1.25 to 1.50 should be applied to the specified minimum ultimate three-edge bearing strength to determine the working strength for other rigid pipes. Common practice is to use a factor of safety of 1.25 for the ultimate load of reinforced concrete sewer pipe, and up to 1.50 for vitrified clay.

(c) Flexible Sewer Pipe

Design performance limits for flexible sewer pipes may be expressed in terms of stress or strain in the pipe wall, crushing or buckling in the pipe wall, or deflection. The most common limitation is deflection. The deflection limitation is established as a design performance limit to provide a factor of safety, against structural failure or any type of distress which might tend to limit the service life of the pipe. This design limit will vary with different pipe materials and the pipe manufacturing process. Deflection limits of up to 7.5% have been used for the design of certain types of flexible pipe. Pipes

generally are not related to the design of the sewer pipe. Under special embankment conditions or induced trench conditions, final backfill may play an important part in the sewer pipe design. However, for most trench installations, final backfill does not affect the pipe design.

The final backfill of trenches in traffic areas, such as under improved existing surfaces, is usually composed of material that is easily densified to minimize future settlement. In undeveloped areas, the final backfill often will consist of the excavated material placed with little compaction and left mounded over the trench to allow for future settlement. Studies have indicated that with some soils, this settlement may continue for over 10 years.

Trench backfilling should be done in such a way as to prevent dropping of material directly on top of a sewer pipe through any great vertical distance. When placing material with a bucket, the bucket should be lowered so that the shock of falling earth will not cause damage.

1.9.4 Design Safety Factor and Performance Limits

(a) General Concepts

In the design of sewer pipes, the selection of a factor of safety is an essential element of the structural design requirements. When the structural design requirements are defined for a given construction material, the most severe or maximum performance limits of that material as related to the proposed service or design application limit must be determined; the ratio of the two limits is the "factor of safety". Maximum performance limits and design values may be defined in terms of strength, stress, strain, or product deformation depending on the characteristics of the material under consideration. The selected factor of safety is applied to the maximum performance limit to calculate a lower value which is then used as a design or service performance value.

The selection of a desired factor of safety is essentially based on a risk/value assessment which must relate to the specific conditions anticipated in an application, the failure mode of the construction material, and the potential cost of system failure. Factors of safety compensate for unexpected construction deficiencies. They should not be relied on to compensate for poor construction practice or for inadequate inspection. Properly established design performance values, including

For flexible pipe, compaction of the haunching material is essential. For rigid pipe, compaction can ensure better distribution of the forces on the pipe. Material used for sewer pipe haunching should be shovel sliced or otherwise placed to provide uniform support for the pipe barrel and to fill completely all voids under the pipe.

Because of space limitations, haunching material is often compacted manually. Results should be checked to verify that the class of bedding or installation are achieved.

Material used for haunching may be crushed stone or sand, or a well graded granular material of intermediate size. If crushed stone is used, it should be subject to the same size limitation and caution regarding use against corrosion protection coatings. Sand should not be used if the pipe zone area is subject to a fluctuating groundwater table or where there is a possibility of the sand migrating into the pipe bedding or trench walls.

(e) Initial Backfill

Initial backfill is the material which covers the sewer pipe and extends from the haunching to some specific point [generally (150 to 300 mm)] above the top of the pipe, depending on the class of bedding. Its function is to anchor the sewer pipe, protect the pipe from damage by subsequent backfill, and ensure the uniform distribution of load over the top of the pipe.

The initial backfill is usually not mechanically tamped or compacted, since such work may damage the sewer pipe, particularly if done over the crown of the pipe. Therefore it should be a material which will develop a uniform and relatively high density with little compactive effort. Initial backfill should consist of suitable granular material, but not necessarily as select a material as that used for bedding and haunching. Clayey materials requiring mechanical compacation should not be used for initial backfill.

The fact that little compaction effort is used on the initial backfill should not lead to carelessness in choice or placement of material. In particular for large sewer pipes, care should be taken in placing both the initial backfill and final backfill over the drain to avoid damage to the sewer pipe.

(f) Final Backfill

The choice of material and placement methods for final backfill are related to the site of the sewer line; they

Table 9-2 Soil Classifications* (Cont.)

Soil Class (1)	Group Symbol (2)	Typical Names (3)	Comments (4)
III	GM	Silty gravels	
	GC	Clayey gravels	
	SM	Silty sands	
	SC	Clayey sands	
IV	MH,ML	Inorganic silts	Not recommended for bedding, haunching or initial backfill
	CH,CL	Inorganic clays	
V	OL,OH	Organic silts and clays	
	PT	Peat	

* For a more detailed description, see ASTM D2321 - "Recommended Practice for Underground Installation of Flexible Thermoplastic Sewer Pipe."
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In general, crushed stone or gravel meeting the requirements of ASTM Designation C33, Gradation 67 [19 to 4.8 mm] will provide the most satisfactory sewer pipe bedding.

In some locations the natural soils at the level of the bottom of the sewer pipe may be sands of suitable grain size and density to serve as both foundation and bedding for the pipe. In such situations, as determined by the design engineer, it may not be necessary to remove and replace these soils with the special bedding materials described above. If the natural soil is left in place, it should be properly shaped for the class of bedding required.

(d) Haunching

The soil placed at the sides of a pipe from the bedding up to the spring line is the haunching. The care with which this material is placed has a significant influence on the performance of the sewer pipe, particularly in the space just above the bedding. Poorly compacted material in this space will result in a concentration of reaction at the bottom of the pipe.

weight of the excavated soil. In this case, foundation pressures are not increased from the initial condition, and the designer should be concerned primarily with the presence of unsuitable soils, such as peat or other highly organic or compressible soils, and with maintaining a stable trench bottom.

If the full benefit of the bedding is to be achieved, the bottom of the trench or embankment must be stable.

(c) Bedding

The contact between a pipe and the foundation on which it rests is the sewer pipe bedding. The bedding has an important influence on the distribution of the reaction against the bottom of the sewer pipe, and therefore influences the supporting strength of the pipe as installed.

Some research has indicated that a well-graded crushed stone is a more suitable material for sewer pipe bedding than well-graded gravel. Both materials, however are better suited than a uniformly graded pea gravel.

Even though larger particle sizes give greater stability, the maximum size and shape of granular embedment should also be related to the pipe material and the recommendations of the manufacturer. For example, sharp angular embedment material over 12 to 20 mm should not be used against corrosion protection coatings. For small sewer pipes, the maximum size should be limited to about 10% of the pipe diameter.

Soil classifications under the Unified Soil Classification System, including manufactured materials, are grouped into five broad categories, according to their ability to develop an interacting sewer pipe-soil system. (Table 9-2). These soil classes are described in ASTM D 2321.

Table 9-2 Soil Classifications*

Soil Class (1)	Group Symbol (2)	Typical Names (3)	Comments (4)
I		Crushed Rock	Angular, 6-40 mm
II	GW	Well graded gravels	
	GP	Poorly graded gravels	40 mm maximum
	SW	Well graded sands	
	SP	Poorly graded sands	

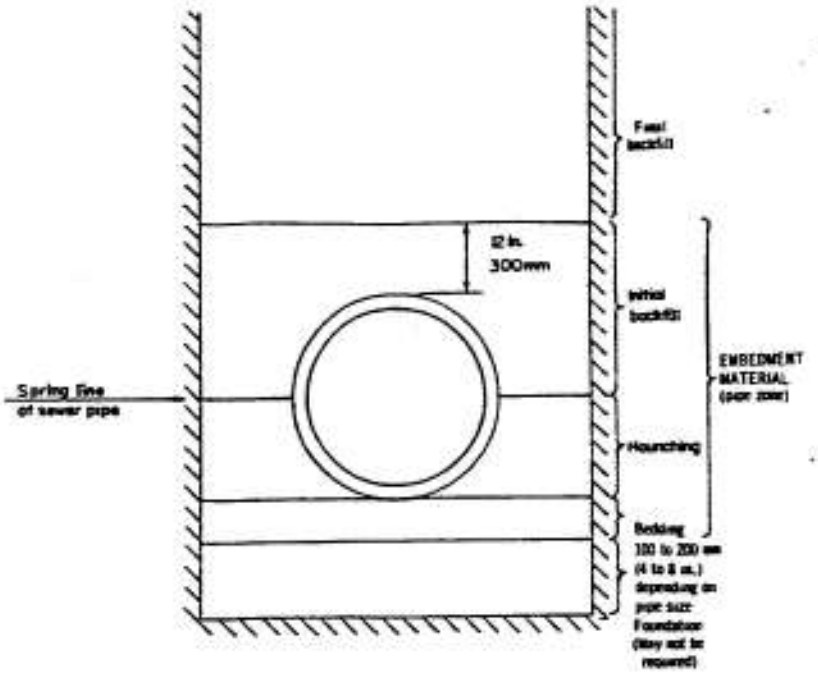


Fig.9-6. Trench cross section illustrating terminology. Copyright, American Society for Testing and Materials, 1916 Race St., Philadelphia, PA 19103, reprinted with permission.

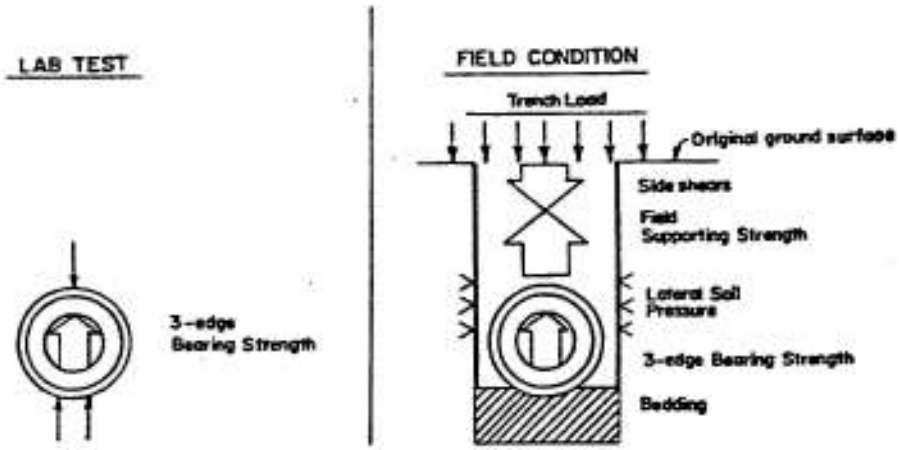


Fig.9-7. Both laboratory testing and field conditions should be used for rigid sewer pipe strength determination.

1.9.3 Pipe Bedding and Backfilling

(a) General Concepts

The ability of a sewer pipe to safely resist the calculated soil load depends not only on its inherent strength but also on the distribution of the bedding reaction and on the lateral pressure acting against the sides of the sewer pipe.

Construction of the sewer pipe-soil system focuses attention on the pipe zone which is made up of five specific areas: Foundation, bedding, haunching, initial backfill and final backfill (see Fig. 9-6 for definitions and limits of these five areas). However, all of these areas are not necessarily referred to in all pipe design standards. The discussion in this section is in general terms and is intended to describe the effects of the various areas on the pipe-soil system.

For all sewer pipe materials, the calculated vertical load is assumed to be uniformly distributed over the width of the pipe. This assumption was originated by Marston's work; the assumption is part of the bedding factors developed by Spangler and presented in this chapter. Many years of field experience indicate the assumption results in conservative designs.

The load carrying capacity of sewer pipes of all materials is influenced by the sewer pipe-soil system, although the importance of the specific areas may vary with different pipe materials. The information in this chapter includes descriptions of pipe beddings for the following:

* Rigid sewer pipe in trench,
* rigid sewer pipe in embankment,
* flexible non-metallic pipe,

Detailed information on pipe bedding classes is contained in the various ANSI and ASTM Specifications or industry literature for each material. The design engineer should consult the applicable specification or literature for information to be used in design.

(b) Foundation

The foundation provides the base for the sewer pipe-soil system. In trench conditions, the total weight of the pipe and soil backfill will normally be no more than the

soil-structure interaction. Analysis should consider both the geometry of the system and material properties of the sewer pipe and the surrounding soil mass.

A simpler method based on arch analysis, which considers only the geometry of the sewer pipe and section properties of the sewer pipe material can also be used for any specified loading condition.

In the method of soil-structure interaction analysis, loadings on the sewer pipe are automatically generated from the specified boundary conditions, the material properties, and the constitutive relationships of material behavior. Most solutions consider elastic behavior of the materials. Elastoplastic behavior and nonlinear analysis are also available.

Table 9-1 "At-Rest" Pressure Coefficients

Soil Type (1)	"At-Rest" Coefficient (2)
Granular Soils	0.5 to 0.67
Cohesive soils, medium to hard	0.67 to 0.88
Cohesive soils, soft	0.75 to 1.0

The arch analysis method requires specification of vertical and lateral loads. The vertical loads can be determined by the Marston method, as described in preceding sections, and distributed uniformly over the full width of the sewer pipe. Lateral loads depend on the soil type and geologic history of the soil deposit. Design parameters should be obtained from a soils consultant knowledgeable of the subsurface conditions in the area. For sewer pipe installed in tunnel or in a trench with properly compacted backfill, the recommended design lateral pressures are those corresponding to "at- rest" conditions. Where the backfill on the sides of the sewer may be loosely placed or insufficiently compacted, "active" pressure coefficients should be used to determine the lateral pressures. For preliminary analysis, the "at-rest" pressure coefficients in Table 9-1 are suggested. Since active and passive earth pressures are the result of lateral strain in the soil mass, the at-rest condition refers to the lateral pressures existing in a large soil mass not subject to horizontal forces or strains except those resulting from its own weight.

From Fig. 9-4, B_d/B_c = 2.39 (the ratio of the width of the trench to the width of the sewer at which loads are equal by both ditch-sewer theory and projecting-sewer theory); B_d = 2.33 x 2.39 = 5.57 5.33; H/B_d = 14/5.33 = 2.63; C_d (from Fig.9-3) = 1.85; and W_c = 1.85 x 120 x $(5.33)^2$ = 6,300 lb/ft (91,700 N/m.)

Example 9-3. Determine the load on the same sewer if (rough) sheeting is left in place.

B_d becomes 4 in. less - 5.00 ft; H/B_d = 14/5.00 = 2.8 C_d (from Fig. 9-3) = 1.92; and W_c = 1.92 x 120 x $(5.00)^2$ = 5,750 lb/ft (84,040 N/m).

Example 9-4. Determine the load on a 30 in. diameter flexible sewer pipe installed in a trench 4 ft 6 in. wide at a depth of 12 ft.

Assume the soil is clay weighing 120 lb/cu.ft. and that it will be well compacted at the sides of the sewer pipe. Then H = 12 ft; B_d = 4.5 ft; B_c = 2.5 ft; H/B_d = 2.67; C_d = 1.9; and W_c = 1.9 x 120 x 4.5 x 2.5 = 2.565 lb/ft (37,540 N/m).

For conservative design, the prism load should be determined. The prism load on flexible sewer pipe will be W = 2.5 x 12 x 120; = 3,600 lb/ft (52,460 N/m).

(d) Loads for Embankment Conditions.

A sewer pipe is described as a projecting sewer pipe when it is installed in a wide trench or in such a manner that the top of the sewer pipe is at or near the natural ground surface or the surface of thoroughly compacted soil and subsequently is covered with an embankment. If the top of the sewer pipe projects some distance above the natural ground surface or it it is installed in a wide trench, it is a positive projecting sewer pipe. There are, however, other methods of installing sewer pipe under embankments which have the favorable effect of minimizing the load on the sewer pipe, In these cases, the installation is classified as a negative projecting sewer pipe or an induced trench sewer pipe (Fig. 9-1).

(e) Alternate Design Method

For larger diameter sewers, such as greater than 1,200 mm in diameter, designs based on the Marston method may yield conservative results. In such cases a more precise analysis can be made utilizing the principles of

Steel sheeting to be removed should be pulled in increments as the trench is backfilled, and the soil should be compacted to prevent formation of voids. The portion of wood sheeting to be removed should be handled similarly.

Loads on sewer pipe in sheeted trenches should be calculated from a trench width measured to the outside of the sheeting if it is pulled or to the inside if it is left in place. Voids created by removal of the sheeting should be backfilled with a flowable material such as pea gravel.

Sewers which are to be constructed in sloping-sided trenches with the slopes extending to the invert, or to any plane above the invert but below the top of the sewer, should be designed for loads computed by using the actual width of the trench at the top of the sewer pipe, or by the projecting-sewer formula (covered in the next subsection), whichever gives the least load on the sewer pipe.

If for any reason the trench becomes wider than that specified and for which the sewer pipe was designed, the load on the sewer pipe should be checked and a stronger sewer pipe or higher class of bedding used, if necessary.

Sample Calculations:

Example 9-1. Determine the load on a 24-in. diameter rigid sewer pipe under 14 ft. of cover in trench conditions.

Assume that the sewer pipe wall thickness is 2 in.; B_c = 24+4 = 28 in. = 2.33 ft; B_d = 2.33 +2.00 = 4.33 ft; and w = 120 lb/cu.ft. for saturated top soil backfill. Then H/B_d = 14/4.33 = 3.24; C_d (from Fig.9-3) = 2.1; and W_c = 2.1 x 120 x $(4.33)^2$ = 4,720 lb/ft (68,880 N/m).

Example 9-2. Determine the load on the same size sewer laid on a concrete cradle and with trench sheeting to be removed.

Assume that the wall thickness is 2 in.; the cradle projection outside of the sewer pipe is 8 in. (4 in. on each side); and the maximum clearance between cradle and outside of sheeting is 14 in. Then B_d = 24 + (2 x 2 in.) + 8 (2 x 14) = 64 in. = 5.33 ft.

As this seems to be an extremely wide trench, a check should be made on the transition width of the trench; B_c = 2.33 ft. H = 14 ft; $r_{sd}p$ = 0.5; and H/B_c = 14/2.33 = 6.0.

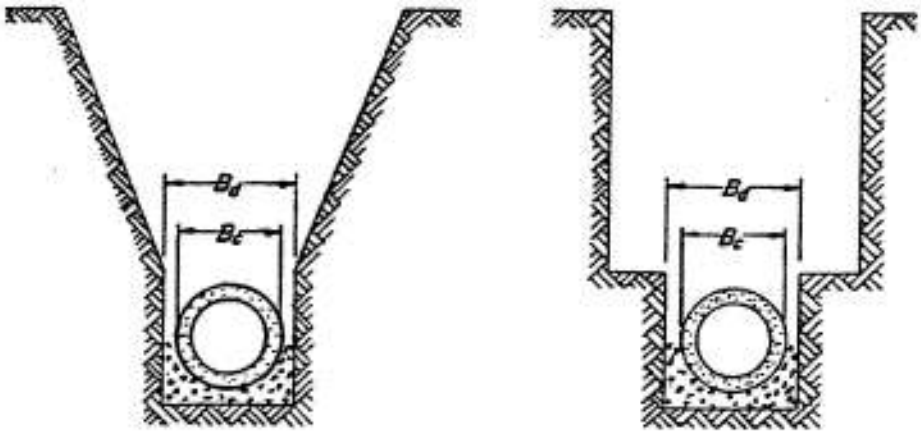


Fig.9-5. Examples of subtrench.

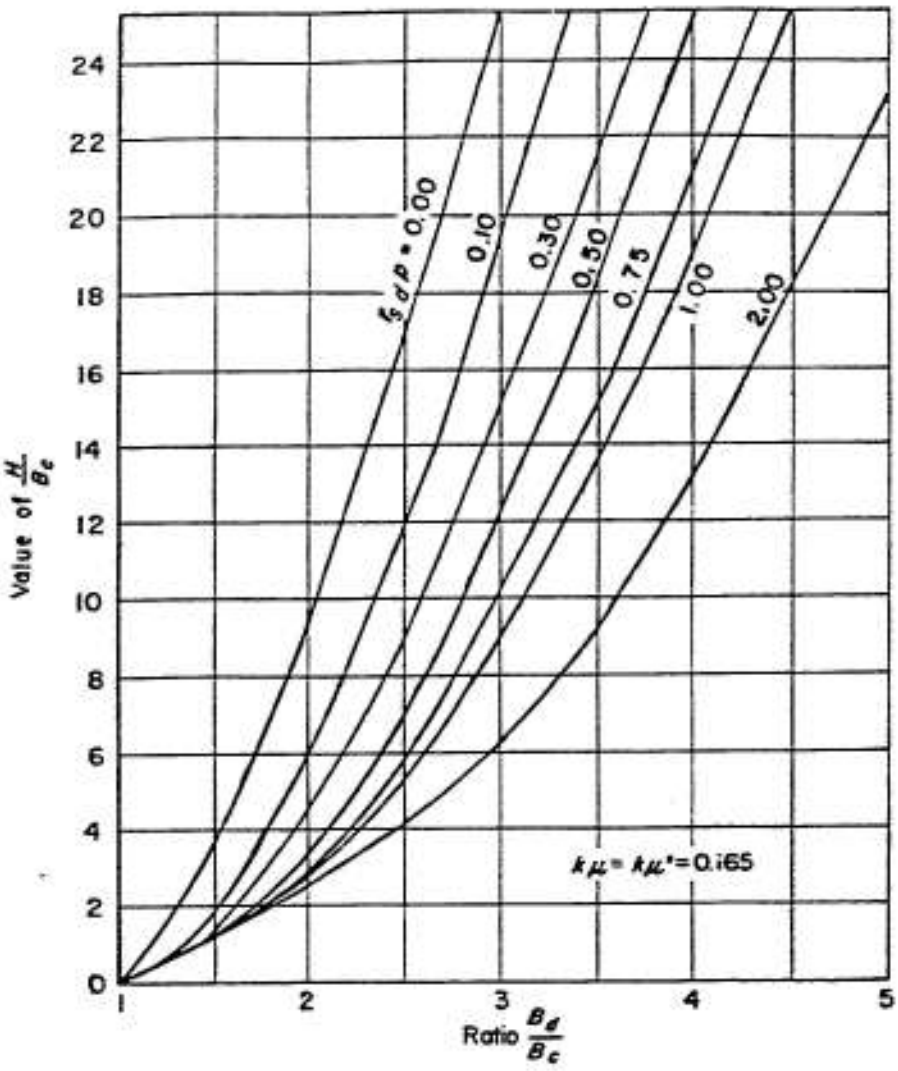


Fig.9-4. Values of B_d/B_c at which pipe in trench and projecting pipe load formulas give equal loads.

formula (discussed in the next section on "Embankment Conditions"). In other words, as the width of the trench increases (other factors remaining constant), the load on a rigid sewer pipe increased in accordance with the theory for a trench sewer pipe until it equals the load determined by the theory for a projecting sewer pipe. The width of trench at which this transition occurs may be determined from Fig. 9-4. The curves in Fig. 9-4 are calculated for sand and gravel, where $K\mu = 0.165$, but can be used for other types of soil since the change with varying values of $k\mu$ is small. In any event, the design engineer can check by calculating loads for both trench and embankment conditions. There is little research on the appropriate value of $r_{sd}p$ (the projection ratio times the settlement ratio) to use in the application of the transition width concept. In the absence of specific information, a value of +0.5 is suggested as a reasonable good working value. These quantities are defined in the subsection on "Loads for Embankment Conditions."

It is advisable, in the structural design of sewers, to evaluate the effect of the transition width on both the design criteria and the construction latitude. A contractor, for instance, may wish to place well points for drainage in the trench. If this requires a wider trench than usual, a stronger sewer pipe or higher class of bedding may be necessary.

It may be economical and proper to excavate the trench with sloping sides in undeveloped areas where no inconvenience to the public or danger to property, buildings, subsurface structure, or pavements, will result. A subtrench (Fig.9-5) may be used in such cases to minimize the load on the pipe. When sheeting of the subtrench at the pipe is necessary, it should extend about 450 mm above the top of the pipe.

When sheeting of the trench is necessary, it should be driven at least to the bottom of the pipe bedding or foundation material, if used. In general, in a constantly wet or dry area, sheeting and bracing should be left in place to prevent reduction in lateral support at the sides of the pipe because of voids formed by removal of the sheeting. Sheeting left in place should be cut off as far below the surface as practicable, but in no case less than 900 mm below final ground elevation.

When wood sheeting is to be removed, the sheeting should be cut off 450 mm above the top of the pipe and the sheeting alongside the pipe left in place.

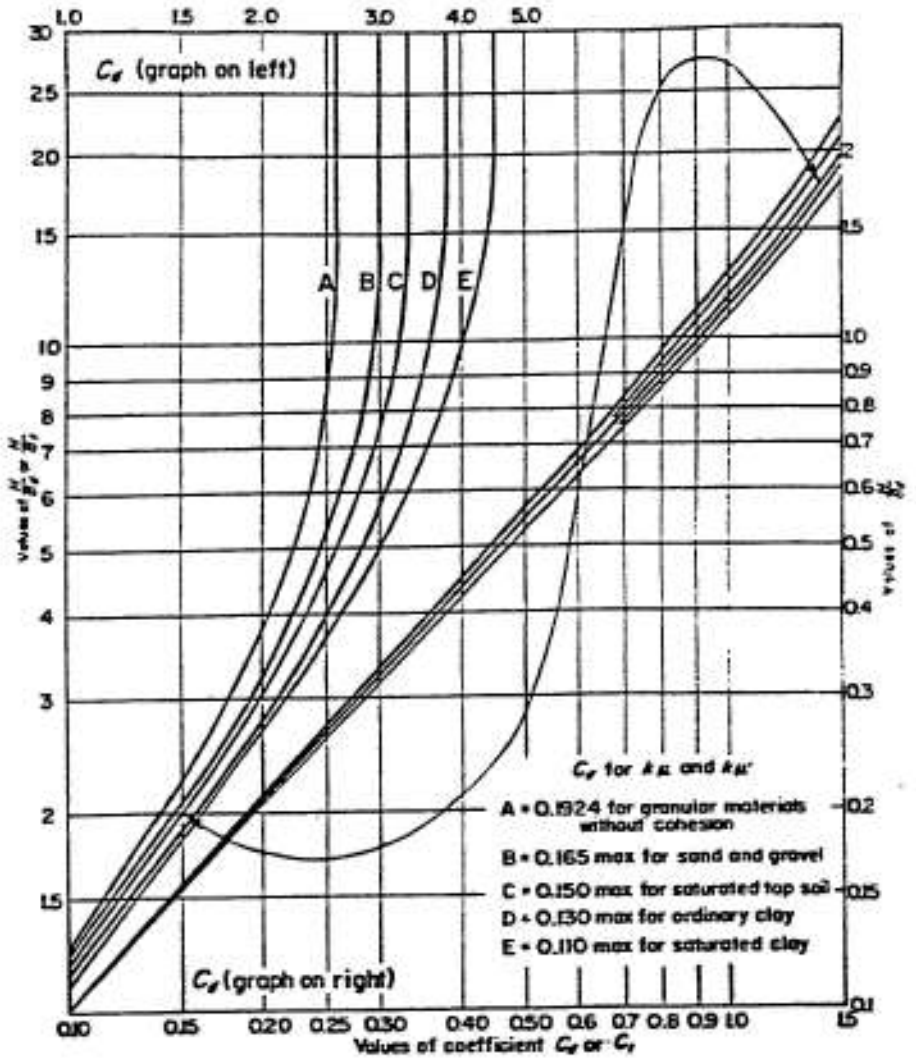


Fig.9-3. Computation diagram for soil loads on trench installations (sewer pipe completely buried in trenches).

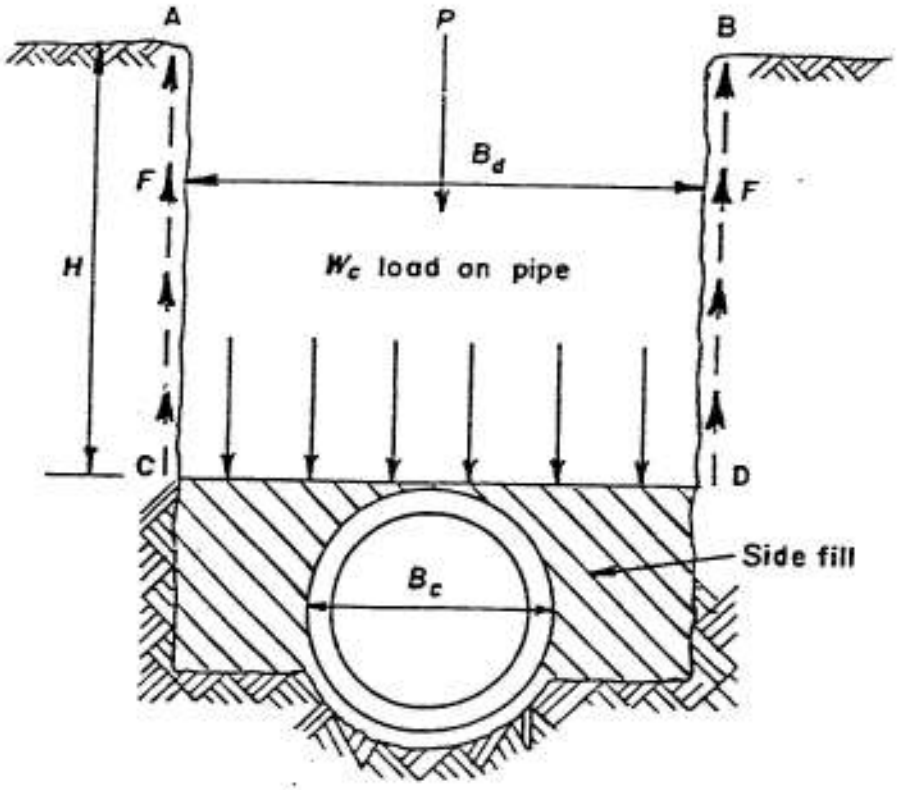


Fig.9-2. Load-producing forces; P = weight of backfill ABCD; F = upward shearing forces on AC and BD; and, W_c = P-2F.

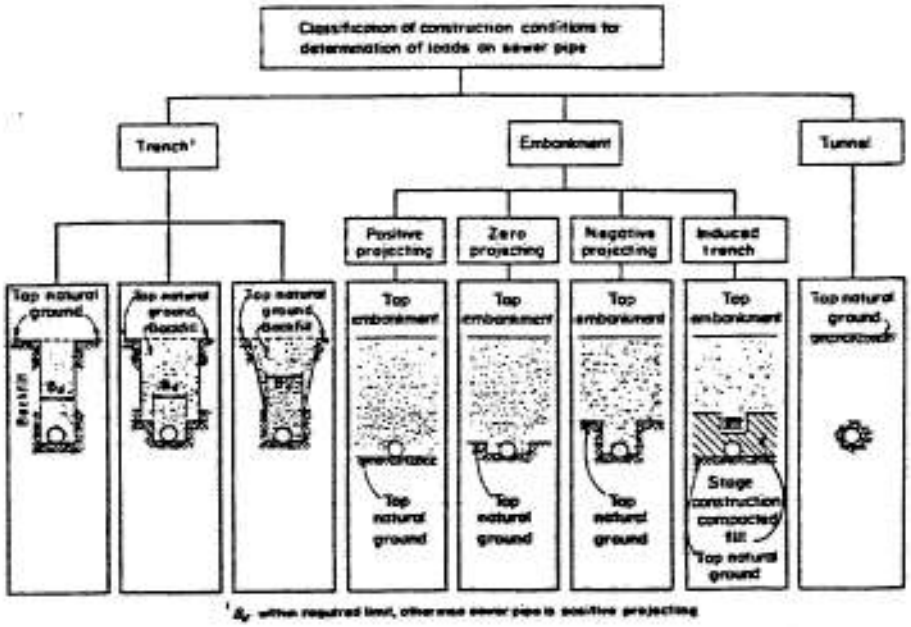


Fig.9-1. Classification of construction conditions.

fill to width of trench and of the friction coefficient between the backfill and the sides of the trench. The load coefficient, C_d, is computed as follows:

$$C_d = \frac{1 - e^{-2k\mu' \frac{H}{B_d}}}{2k\mu'} \qquad (9.3)$$

in which e is the base of natural logarithms and k is Rankine's ratio of lateral pressure to vertical pressure:

$$k = \frac{\mu^2 + 1 - \mu}{\mu^2 + 1 + \mu} = \frac{1 - \sin \phi}{1 + \sin \phi} \qquad (9.4)$$

The other terms are: $u = \tan \phi$ = the coefficient of internal friction of backfill material; $\mu' = \tan \phi$ = the coefficient of friction between backfill material and sides of trench (μ' may be equal to or less than u but never greater than μ); and H is the height of fill above top of pipe, in meters. The value of C_d for various ratios of H/B_d and various types of soil backfill may be obtained from Fig. 9-3.

The width of trench below the top of the sewer pipe is also important. It must not be permitted to exceed the safe limit for the strength of sewer pipe and class of bedding used. The minimum width must be consistent with the provision of sufficient working space at the sides of the sewer pipe to assemble joints properly, to insert and strip forms, and to compact backfill. The design engineer must allow reasonable tolerance in width for variations in field conditions and accepted construction practice.

The position of the lower wale usually will determine the proper width of trench from face-to-face of sheeting, where sheeting and bracing are required. A working-room allowance of 300 mm from each side of the sewer pipe or sewer pipe cradle to the face of the sheeting is a workable minimum for small and medium-sized sewer pipe for trenches up to about 4 m deep.

At any given depth and for any given sewer pipe size there is a certain limiting value to the width of trench beyond which no additional load is transmitted to the sewer pipe. This limiting value is called the "transition width". There are sufficient experimental data at hand to show that it is safe to calculate the imposed load by means of the trench-conduit formula (Eq.9.2) for all widths of trench less than that which gives a load equal to the load calculated by the projecting-conduit

(b) Types of Loading Conditions

Although the general form of Marston's equation includes all the factors necessary to analyze all types of installation conditions, it is convenient to classify these conditions, write a specialized form of the equation, and prepare separate graphs and tables of coefficients for each.

The accepted system of classification is shown diagrammaticaly in Fig. 9-1 and is described briefly below.

Trench conditions are defined as those in which the sewer pipe is installed in a relatively narrow trench cut in undisturbed ground and covered with soil backfill to the original ground surface.

Embankment conditions are defined as those in which the sewer pipe is covered above the original ground surface or when a trench in undisturbed soil is so wide that trench wall friction does not affect the load on the sewer pipe. The embankment classification is further subdivided into two major subclassifications - positive projecting and negative projecting. Sewer pipe is defined as positive projecting when the top of the sewer pipe is above the adjacent original ground surface. Negative projecting sewer pipe is that installed with the top of the sewer pipe below the adjacent original ground surface in a trench which is narrow with respect to the size of pipe and the depth of cover (Fig.9-1), and when the native material is of sufficient strength that the trench shape can be maintained dependably during placement of the embankment.

A special case, called the induced trench condition, may be employed to minimize the load on a conduit under an embankment of unusual height.

(c) Use of Marston's Formula

Marston's formula for loads on rigid sewer pipe in trench conditions is:

$$W_c = C_d w B_d^2 \qquad (9.2)$$

in which W_c is the load on the sewer pipe, in newtons per meter; w is the density of backfill soil, in kilograms per cubic meter; B_d is the width of trench at the top of the sewer pipe, in meters; C_d is a dimensionless load coefficient which is a function of the ratio of height to

His methods are based on both theory and experiment and have achieved acceptance as being useful and reliable.

More recent analysis and actual observation of field performance have shown that designs based on the Marston Theory yield satisfactory results especially for small diameter conduits in narrow trenches. For larger diameter conduits the results are conservative.

In general, the theory states that the load on a buried pipe is equal to the weight of the prism of soil directly over it, called the interior prism, plus or minus the frictional shearing forces transferred to that prism by the adjacent prisms of soil - the magnitude and direction of these frictional forces being a function of the relative settlement between the interior and adjacent soil prisms. The theory makes the following assumptions:

(i) The calculated load is the load which will develop when ultimate settlement has taken place.

(ii) The magnitude of the lateral pressures which induce the shearing forces between the interior and adjacent soil prisms is computed in accordance with Rankine's theory.

(iii) Cohesion is negligible except for tunnel conditions.

The general form of Marston's equation is:

$$W = CwB^2 \qquad (9.1)$$

in which W is vertical load per unit length acting on the sewer pipe because of gravity soil loads; w is the unit weight of soil; B is the trench width or sewer pipe width, depending on installation conditions; and C is a dimensionless coefficient that measures the effect of the following variables;

* The ratio of height of fill to width of trench or sewer pipe;

* the shearing forces between interior and adjacent soil prisms;

* the direction and amount of relative settlement between interior and adjacent soil prisms for embankment conditions.

1.9 STRUCTURAL DESIGN OF SEWERS

1.9.1 Introduction

The structural design of a sewer requires that the supporting strength of the installed sewer pipe, divided by a suitable factor of safety, must equal or exceed the loads imposed on it by the combined weight of soil and any superimposed loads.

This chapter presents generally accepted criteria and methods of determining combined loads and supporting strength of the sewer pipe, as well as procedures for combining these elements with the application of a factor of safety to produce a safe and economical design.

Methods are presented for estimating probable maximum loads caused by soil forces and for both static and moving superimposed loads. Where so noted, the methods apply to rigid and flexible conduits in the two most common conditions of installation: In a trench in natural ground and in an embankment.

The design of rigid and flexible pipes is treated separately. There are no specific design procedures given for flexible pipes of intermediate stiffness. For such cases, design procedures such as computer analysis based on soil-structure interaction or the designs for rigid or flexible pipes may be used (not interchangeably) for conservative results.

The supporting strength of a buried sewer pipe is a function of installation conditions as well as the strength of the sewer pipe itself. Structural analysis and design of the sewer line are problems of soil-structure interaction. This chapter presents procedures for determining the field or installed supporting strength of rigid sewer pipe based on its established relationship to the laboratory test strength. It also presents methods of predicting approximate field deflections for flexible pipe, based on empirical methods.

Since installation conditions have such an important effect on both load and supporting strength, a satisfactory sewer construction project requires accurate assumed design conditions from the job site. Therefore, this chapter also includes a section on recommendations for construction and field observations to achieve this goal.

1.9.2 Loads on Sewers Caused by Gravity Earth Forces

(a) General Method - Marston Theory

Marston developed methods for determining the vertical load on buried conduits caused by soil forces in all of the most commonly encountered construction conditions

proper selection of primer, mastic material, and good workmanship in application.

(viii) Sealing Band Joints

External Sealing bands of rubber made in conformance with ASTM C 877 are also used on noncircular concrete sewer pipe. These elastomeric bands are wrapped tightly around the exterior of the pipe at the joint and extend several centimeters on each side of the joint. Sealing against the concrete is achieved by a mastic applied to one side of the band.

1.8.5 SUMMARY:

With consideration of appropriate factors essential to the proper design of sewer systems, it is apparent that the broad variety of materials for sewer construction available is a source of significant benefit. Each sewer pipe product and each type of pipe joint offer distinct advantages which can prove to be specifically beneficial in given applications. Each sewer pipe product and each type of pipe joint also present limitations which must be understood and can generally be accommodated in proper system design for given applications.

(iv) Elastomeric Sealing Compound Pipe Joints

Elastomeric sealing compound may be used in jointing properly prepared concrete gravity sewer pipe. Pipe ends must be sandblasted and primed for elastomeric sealant application. The sealant, a thixotropic, two-compound elastomer, is mixed on the job site and applied with a caulking gun and spatula. The pipe joint, when assembled with proper materials and procedures, provides a positive seal against leakage in gravity sewer pipe.

(v) Solvent Cement Pipe Joints

Solvent cement pipe joints may be used in jointing thermoplastic pipe materials such as PVC pipe. This type of pipe joint involves bonding a sewer pipe spigot into a sewer pipe bell or coupler using a solvent cement. Solvent cement joint can provide a positive seal provided the proper cement is applied under proper ambient conditions with proper techniques. Reference should be made to ASTM D 402 for safe handling procedures. Required precautions should be taken to assure adequate trench ventilation and protection for workers installing the pipe. Solvent cement pipe joints may be desired in special situations and with some plastic fittings.

(vi) Heat Fusion Pipe Joints

Heat fusion pipe joints are commonly specified for PE sewer pipe. The general method of jointing PE sewer pipe involves butt fusion of the pipe lengths, end to end. After the ends of two lengths of PE pipe are trimmed and softened to a melted state with heated metal plates, the pipe ends are forced together to the point of butt fusion, providing a positive seal. The pipe joint does not require thrust restraint in pressure applications. Trained technicians with special apparatus are required to achieve reliable watertight pipe joints.

(vii) Mastic Pipe Joints

Mastic pipe joints are frequently used for special non-round shapes of concrete gravity sewer pipe which are not adaptable for use with gasket pipe joints. The mastic material is placed into the annular space to provide a positive seal. Application may be by trowelling, caulking, or by the use of preformed segments of mastic material in a manner similar to gaskets. Satisfactory performance of the pipe joints depends upon the

(i) Gasket Pipe Joints

Gasket joints effect a seal against leakage through compression of an elastomeric seal or ring. Gasket pipe joint design is generally divided into two types - push-on pipe joint and mechanical compression pipe joint.

(a) Push-on Pipe Joint

This type of pipe joint uses a continuous elastomeric ring gasket which is compressed into an annular space formed by the pipe, fitting or coupler socket and the spigot end of the pipe providing a positive seal when the pipe spigot is pushed into the socket. When using this type of pipe joint in pressure sanitary sewers, thrust restraint may be required to prevent joint separation under pressure. Push-on pipe joints (fittings, couplers, or integral bells) are available on nearly all pipe products mentioned.

(b) Mechanical Compression Pipe Joint

This type of pipe joint uses a continuous elastomeric ring gasket which provides a positive seal when the gasket is compressed my means of mechanical device. When using this type of pipe joint in pressure sewers, thrust restraint may be required to prevent joint separation under pressure. When incorporated into a coupler, this type of pipe joint may be used to joint two similarly sized plain spigot ends of any commonly used sewer pipe materials.

(ii) Bituminous Pipe Joints

This type of pipe joint involves use of hot-poured or cold packed bituminous material forced into a bell-and-spigot pipe joint to provide a seal. The use of this joint is discouraged in that reliable, watertight joints are not assured.

(iii) Cement Mortar Pipe Joint

This type of pipe joint involves use of a shrink-compensating cement mortar placed into a bell-and-spigot pipe joint to provide a seal. The use of this joint is discouraged in that reliable, watertight joints are not assured. Cement mortar joints are not flexible and may crack due to any pipe movement.

Potential disadvantages of PVC pipe include:

- subject to attack by certain organic chemicals,
- subject to excessive deflection when improperly bedded and haunched,
- limited range of sizes available,
- subject to surface changes effected by long-term ultra-violet exposure.

PVC pipe is specified by nominal diameter, dimension ratio, pipe stiffness and type of joint. PVC pipe should be manufactured in accordance with one or more of the following standard specifications:

- CSA Standard B 137.0 and B 137.3.
- ASTM D3034-73.

Additional information relative to the selection and design of PVC pipe may be obtained from the Uni-Bell Handbook of PVC Pipe - Design and Construction.

1.8.4 PIPE JOINTS:

a) General Information

The requirement for the control of ground water infiltration and wastewater exfiltration in sewer systems renders the specification of pipe joint design essential to proper sewer design. A substantial variety of pipe joints is available for the different pipe materials used in sewer construction. A common requirement which must be imposed on the design of all sewer systems, regardless of the type of sewer pipe specified, is the use of reliable, tight pipe joints. A good pipe joint must be watertight, root resistant, flexible and durable. In general, today, various forms of gasket (elastomeric seal) pipe joints are used in sewer construction. They generally can be assembled by unskilled labor in a broad range of weather conditions and environments with good assurance of a reliable, tight seal preventing leakage.

In common practice, water infiltration/exfiltration testing or air exfiltration testing is specified for typical nonpressure sewer system construction to demonstrate that infiltration/exfiltration is within acceptable limits.

1.8.4 b) Types of Pipe Joints

Commonly specified sewer pipe joints include:

Potential advantages of PE pipe include:

- long laying lengths (in some situations),
- light weight,
- high impact strength,
- ease in field cutting.

Potential disadvantages of PE pipe inlcude:

- relatively low tensile strength and pipe stiffness,
- limited range of sizes available,
- subject to environmental stress cracking,
- subject to excessive deflection when improperly bedded and haunched,
- subject to attack by certain organic chemicals,
- subject to surface change effected by long-term ultra-violet exposure,
- special tooling required for fusing joints.

PE pipe is specified by material designation nominal diameter (inside or outside) standard dimension ratios and type joint. PE pipe should be manufactured in accordance with one of the following specifications:

- CSA Standard B137.0 and B137.1

or

CGSB 41-GP-25.

1.8.3 c) Polyvinyl Chloride (PVC) Pipe

PVC pipe is used for both gravity and pressure sewers. Nonpressure PVC sewer pipe is available in nominal diameters from 100 through 675 mm. PVC pressure and non-pressure fittings are available. PVC pipe is generally available in lengths up to 6.1 m. Jointing is primarily accomplished with elastomeric seal gasket joints, although solvent cement joints for special applications are available.

PVC pipe is manufactured by extrusion of the plastic material.

Nonpressure PVC sanitary sewer pipe is provided in two dimension ratios (S.D.R.28 and S.D.R.35) which relate to two pipe stiffness values.

Potential advantages of PVC pipe include:

- light weight,
- long laying lengths (in some situations),
- high impact strength,
- ease in field cutting and tapping.

1.8.3 FLEXIBLE SEWER PIPE MATERIALS:

Sewer pipe materials in this classification derive load carrying capacity from the interaction of the flexible pipe and the embedment soils effected by the deflection of the pipe to the point of equilibrium under load. Commonly specified flexible sewer pipe materials are discussed below.

1.8.3 a) Corrugated Steel Pipe (CSP)

CSP is used for gravity sewers under extreme design conditions. CSP is available in nominal diameters from 150 mm to 2400 mm and a variety of pipe arch dimensions up to 1800 mm x 1100 mm. CSP is available in galvanized, asphalt coated, paved invert and asbestos bounded forms. Jointing is usually achieved with a bolted strap coupler. Neoprene gaskets are available to reduce exfiltration and infiltration.

Potential advantages of CSP include:

- high flexibility,
- range of gauges to suit loading,
- good for sewer on fill,
- good for extreme grades.

Potential disadvantages of CSP include:

- subject to corrosion with high water table,
- high roughness factor (n=0.024),
- bedding around pipe should be thoroughly compacted.

CSP should be manufactured in accordance with the standard specification:

- CSA Standard CAN 3-G401-M81 "Corrugated Steel Pipe Products".

1.8.3 b) Polyethylene (PE) Pipe

PE pipe is used for both gravity and pressure sewers. Nonpressure PE pipe, primarily used for sewer relining, is available in nominal diameters from 100 mm through 1200 mm. PE fittings are available. Jointing is primarily accomplished by butt-fusion or flanged adaptors.

PE pipe is manufactured by extrusion of PE plastic material. Nonpressure PE pipe is produced at this time in accordance with individual manufacturer's product standards.

Concrete pipe is normally specified by nominal diameter, class or D-load strength and type of joint. The product should be manufactured in accordance with one or more of the following standard specifications:

- CSA standards A257.1, A257.2, A257.3,
- Ministry of the Environment Standard Specification No. 11.

Additional information relative to the selection and design of concrete pipe may be obtained from the American Concrete Pipe Association Concrete Pipe Design Manual and Concrete Pipe Handbook.

1.8.2 c) Vitrified Clay Pipe (VCP)

VCP is used for gravity sewers. The product is manufactured from clay and shales. Clay pipe is vitrified at a temperature at which the clay mineral particles become fused. The product is available in diameters from 100 through 1050 mm. Clay fittings are available to meet most requirements with special fittings manufactured on request. A number of jointing methods are available.

VCP is manufactured in standard and extra-strength classifications, although in some areas the manufacture of standard-strength pipe is not common in sizes 300 mm and smaller. The strength of vitrified clay pipe varies with the diameter and strength classification. The pipe is manufactured in lengths up to 3 m.

Potential advantages of vitrified clay pipe include:

- high resistance to chemical corrosion,
- high resistance to abrasion,
- high range of fittings available.

Potential disadvantages of vitrified clay pipe include:

- limited range of sizes available,
- high weight,
- subject to shear and beam breakage when improperly bedded.

VCP is specified by nominal pipe diameter, strength, and type of joint. The product should be manufactured in accordance with one or more of the following standard specifications:

- CSA Specification A60.1 and A60.3.

- subject to shear and beam breakage when improperly bedded,
- low beam strength.

Asbestos-cement pipe is specified by pipe diameter and class or strength. The product should be manufactured in accordance with one or more of the following specifications:

- "Standard Specification for Asbestos-Cement Non-Pressure Pipe", ASTM C 428 and C500

- "Standard Specification for Asbestos-Cement Non-Pressure Small Diameter Sewer Pipe," ASTM C 644.

1.8.2 b) Concrete Pipe (CP)

Reinforced and nonreinforced concrete pipe are used for gravity sewers. Reinforced concrete pressure pipe and prestressed concrete pressure pipes are used for pressure as well as gravity sewers. Nonreinforced concrete pipe is available in nominal diameters from 150 mm through 450 mm. Pressure pipe is available in various diameters. Concrete fittings and appurtenances such as wyes, tees and manhole sections are generally available. A number of jointing methods are available depending on the tightness required and the operating pressure. Various linings and coatings are available.

A number of mechanical processes are used in the manufacture of concrete pipe. These processes use various techniques including centrifugation, vibration, packing and tamping for consolidating the concrete in forms. Gravity and pressure concrete pipe may be manufactured to any reasonable strength requirement by varying the wall thickness, concrete strength, quantity and configuration of reinforcing steel or prestressing elements (4).

Potential advantages of concrete pipe include:

- wide range of structural and pressure strengths,
- wide range of nominal diameters,
- wide range of laying lengths (generally 1.2 to 7.3 m).

Potential disadvantages of concrete pipe include:

- high weight,
- subject to corrosion where acids are present,
- subject to shear and beam breakage when improperly bedded.

been limited to the commonly accepted pipe materials currently available today for sewer applications. These products are listed below alphabetically within the two commonly accepted classifications of rigid pipe and flexible pipe:

1) Rigid Pipe

   a) Asbestos-cement pipe (ACP)
   b) Concrete pipe (CP)
   c) Vitrified clay pipe (VCP)

2) Flexible Pipe

   a) Corrugated steel pipe (CSP)
   b) Polyethylene (PE)
   c) Polyvinyl chloride (PVC)

1.8.2 Rigid Sewer Pipe Materials:

Sewer pipe materials in this classification derive a substantial part of their basic earth load carrying capacity from the structural strength inherent in the rigid pipe wall. Commonly specified rigid sewer pipe materials are discussed in the following subsections.

1.8.2 (a) Asbestos Cement Pipe (ACP)

Asbestos-cement pipe is used for both gravity and pressure sewers. The product, produced from asbestos fiber and cement, is available in nominal diameters from 100 through 900 mm and in some areas up to 1050 mm. A full range of fittings is manufactured that is compatible with the class of pipe being used. Jointing is accomplished by compressing elastomeric rings between pipe ends and sleeves or couplings.

ACP manufactured for gravity sewer applications, is available in five strength classifications. The class designation represents the minimum crushing strength of the pipe.

Potential advantages of asbestos-cement pipe include:

- long laying lengths (in some situations),
- wide range of strength classifications,
- wide range of fittings available.

Potential disadvantages of asbestos-cement pipe include:

- subject to corrosion where acids are present,

1.8 MATERIALS FOR SEWER CONSTRUCTION

1.8.1 Introduction:

Pipe for sewer construction generally is manufactured from various basic materials in accordance with nationally recognized product specifications. Each type of sewer pipe, its advantages and limitations, should be evaluated carefully in the selection of pipe materials for given applications.

Various factors are involved in the evaluation and selection of materials for sewer construction and are dependent on the anticipated conditions of service. Factors which may be involved and should be considered are:

1) Intended use - type of wastewater

2) Scour or abrasion conditions

3) Installation requirements - pipe characteristics and sensitivities

4) Corrosion conditions - chemical, biological

5) Flow requirements - pipe size, velocity, slope and friction coefficient

6) Infiltration/exfiltration requirements

7) Product characteristics - pipe size fitting and connection requirements, laying length

8) Cost effectiveness - materials, installation, maintenance, life expectancy.

9) Physical properties - crush strength for rigid pipe, pipe stiffness or stiffness factor for flexible pipe, soil conditions, pipe beam loading strength, hoop strength for force main pipe, pipe shear loading strength, pipe flexural strength

10) Handling requirements - weight, impact resistance.

No single pipe product will provide optimum capability in every characteristic for all sewer design conditions. Specific application requirements should be evaluated prior to selecting or specifying pipe materials.

With the advancement of technology, new pipe materials are periodically being offered for use in sewer construction. Discussion of pipe materials provided in this chapter has

21.9 CITY OF LONDON - ENGINEERING STANDARDS

HEAD LOSSES IN CURVED SEWERS AND MANHOLES WITH BENCHING TO CROWN

$H_L = K_L \frac{V^2}{2g}$

R = ℄ RADIUS OF BEND
D = DIAMETER OF LARGEST PIPE

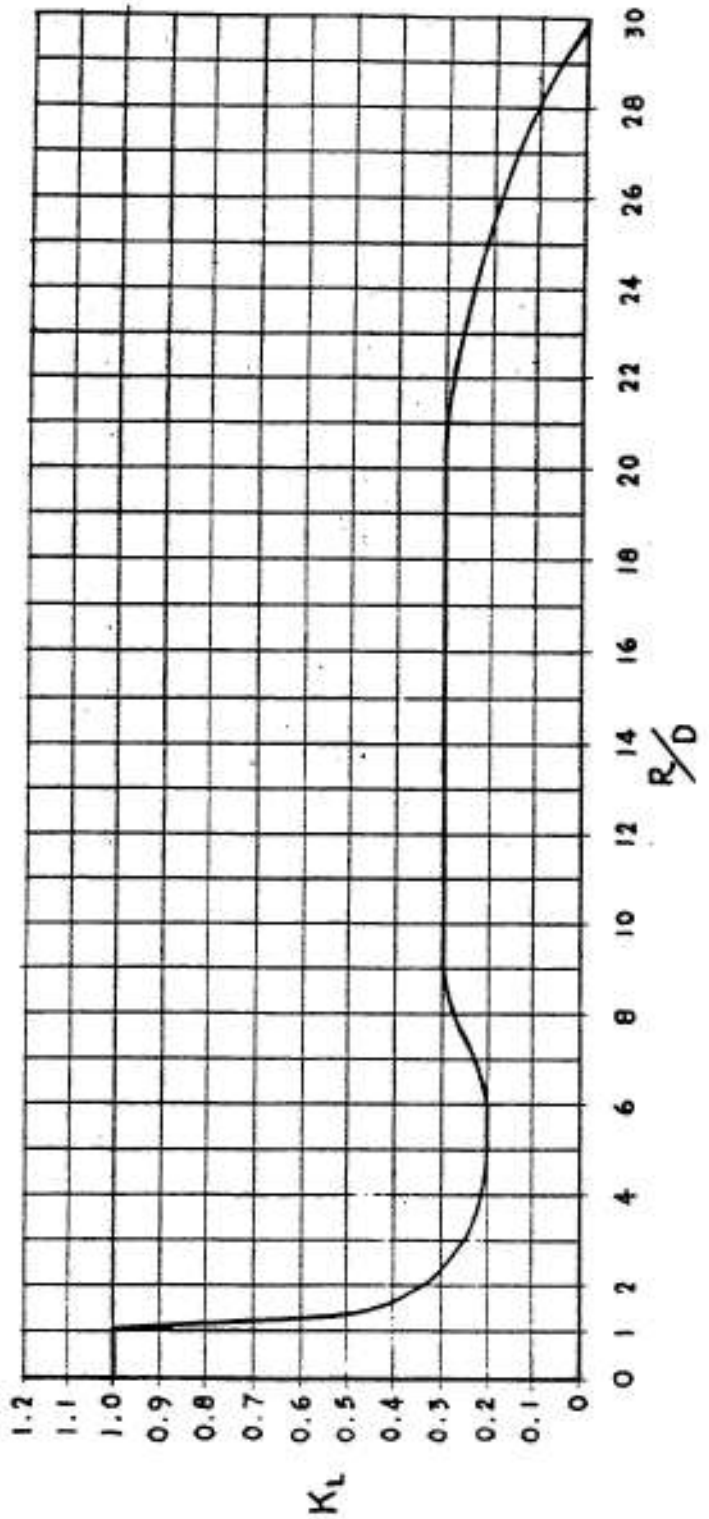
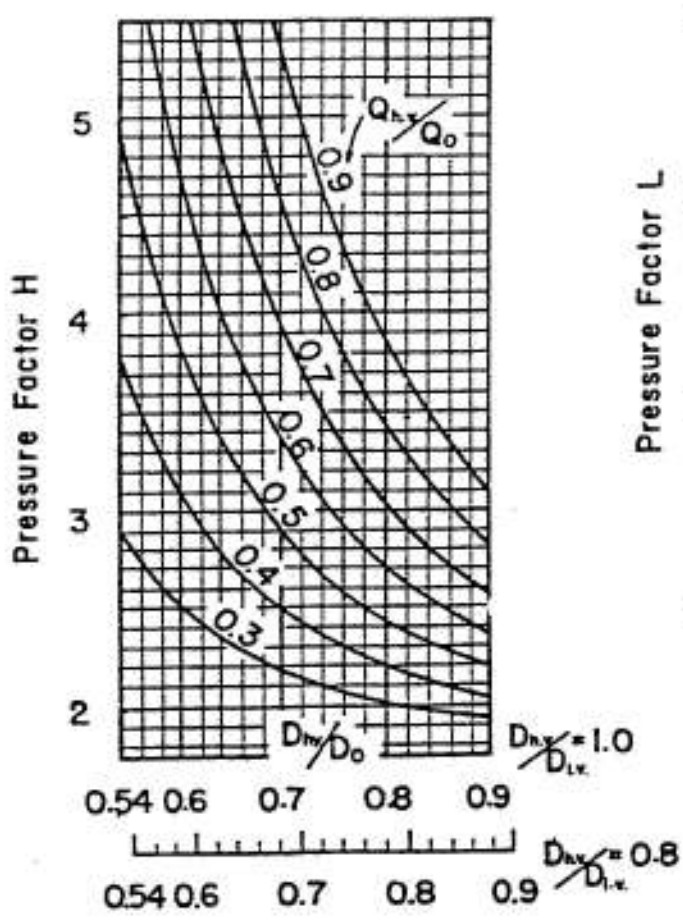
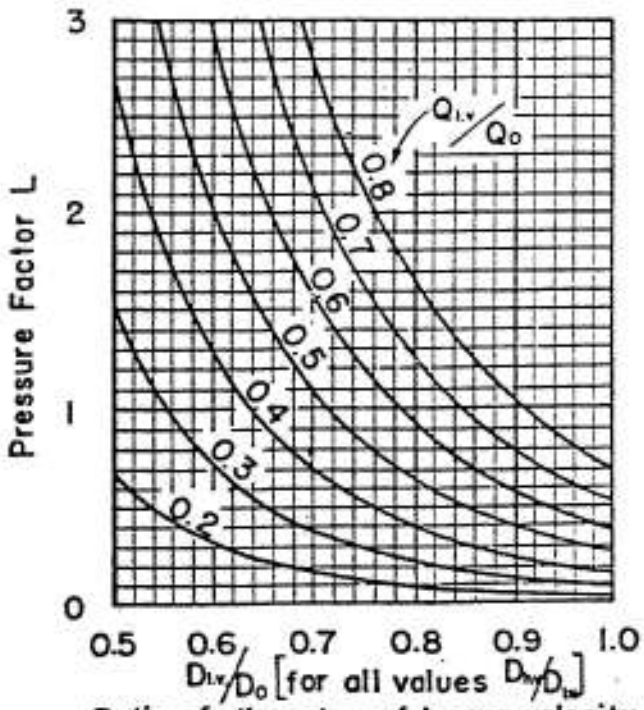


Figure 1.78



Ratio of diameter of higher velocity lateral to diameter of outfall



Ratio of diameter of lesser velocity lateral to diameter of outfall

$D_{h.v.}$ = diameter of lateral with higher-velocity flow.

$Q_{h.v.}$ = rate of flow in lateral with higher-velocity flow.

$D_{l.v.}$ = diameter of lateral with lower-velocity flow.

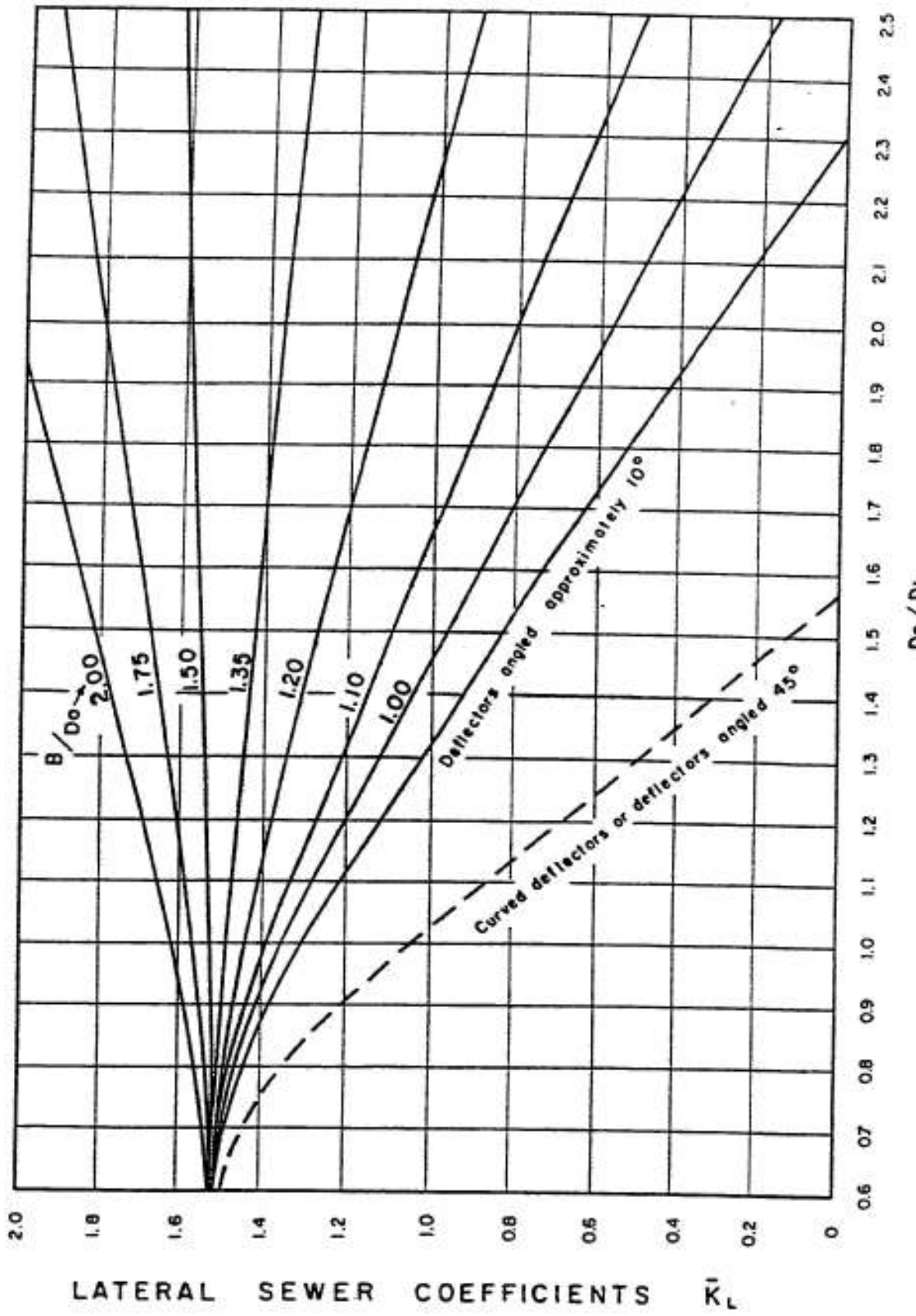
$Q_{l.v.}$ = rate of flow in lateral with lower-velocity flow.

PRESSURE FACTORS

FOR

UPSTREAM SEWERS - OPPOSED AND EACH AT 90° TO THE OUTLET

Figure 1.77.



LATERAL SEWER COEFFICIENTS FOR HEAD LOSSES IN MANHOLES

Figure 1.7.6

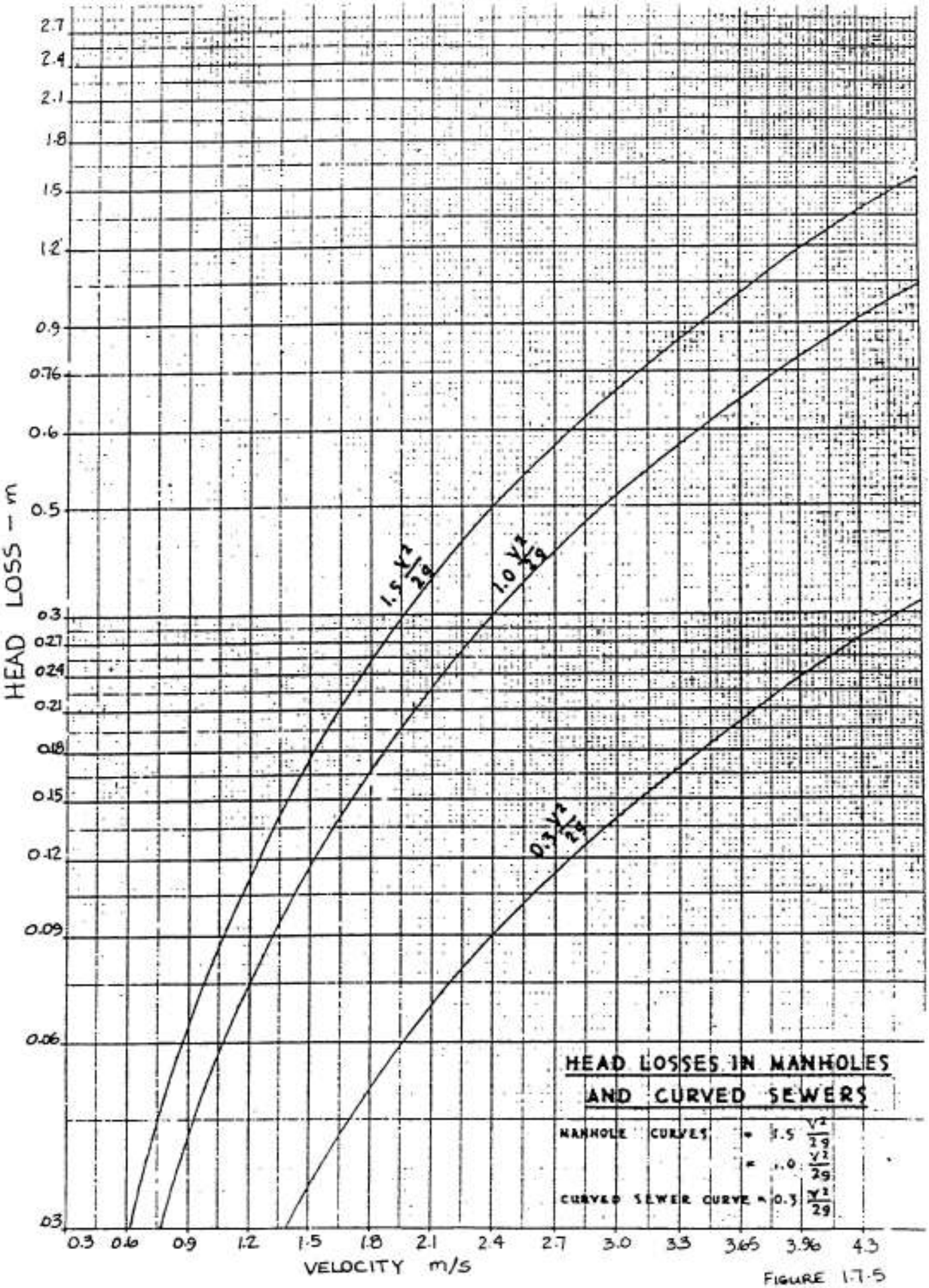
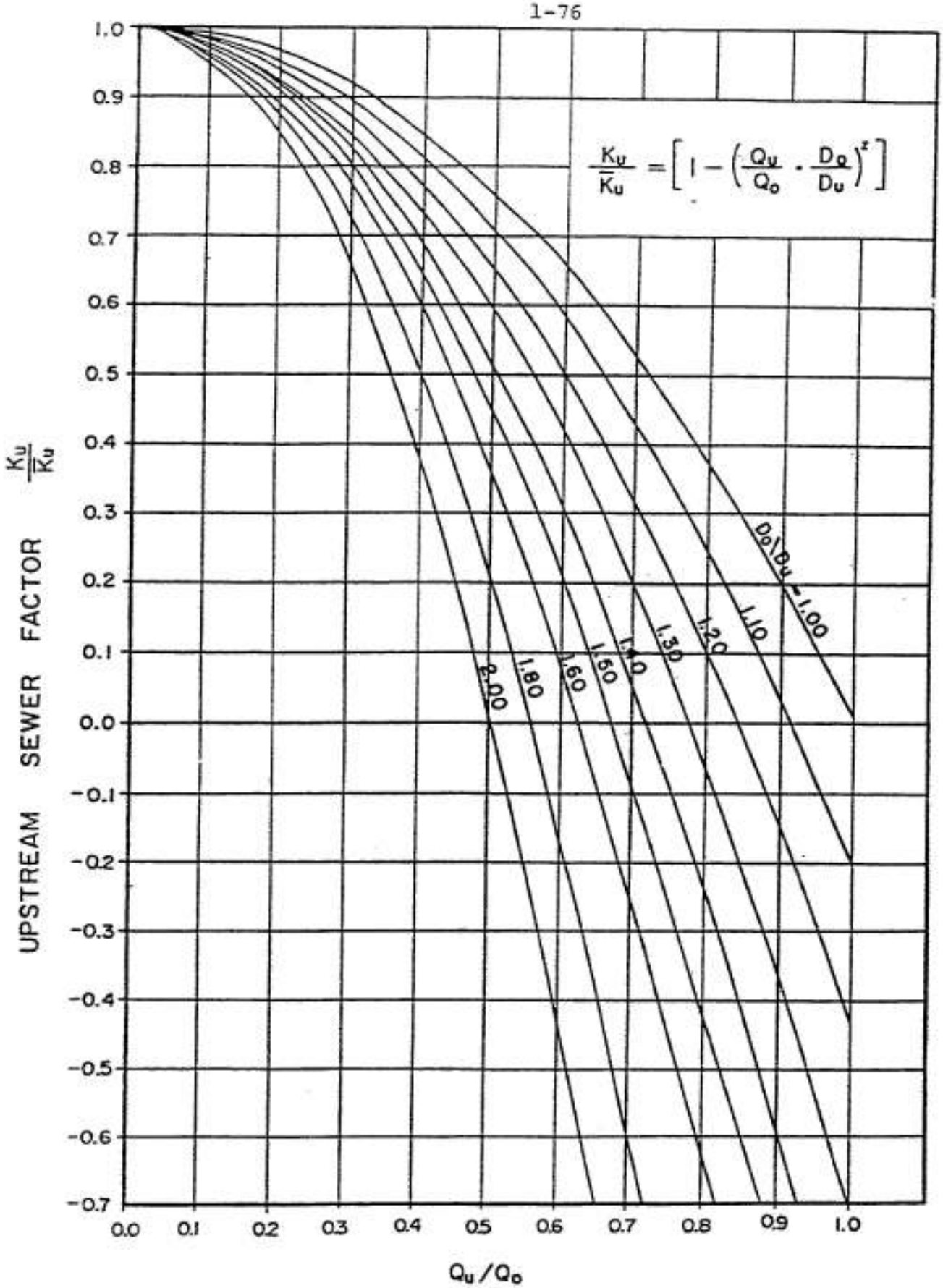

HEAD LOSS - m
2.7
2.4
2.1
1.8
1.5
1.2
0.9
0.76
0.6
0.5
0.3
0.27
0.24
0.21
0.18
0.15
0.12
0.09
0.06
0.03
$1.5\frac{V^2}{2g}$
$1.0\frac{V^2}{2g}$
$0.3\frac{V^2}{2g}$
0.3 0.6 0.9 1.2 1.5 1.8 2.1 2.4 2.7 3.0 3.3 3.66 3.96 4.3
VELOCITY m/s
HEAD LOSSES IN MANHOLES AND CURVED SEWERS
MANHOLE CURVES = 1.5 $\frac{V^2}{2g}$
= 1.0 $\frac{V^2}{2g}$
CURVED SEWER CURVE = 0.3 $\frac{V^2}{2g}$

FIGURE 1.7.5



UPSTREAM SEWER FACTORS
FOR HEAD LOSSES IN MANHOLES

FIGURE 1.7.4

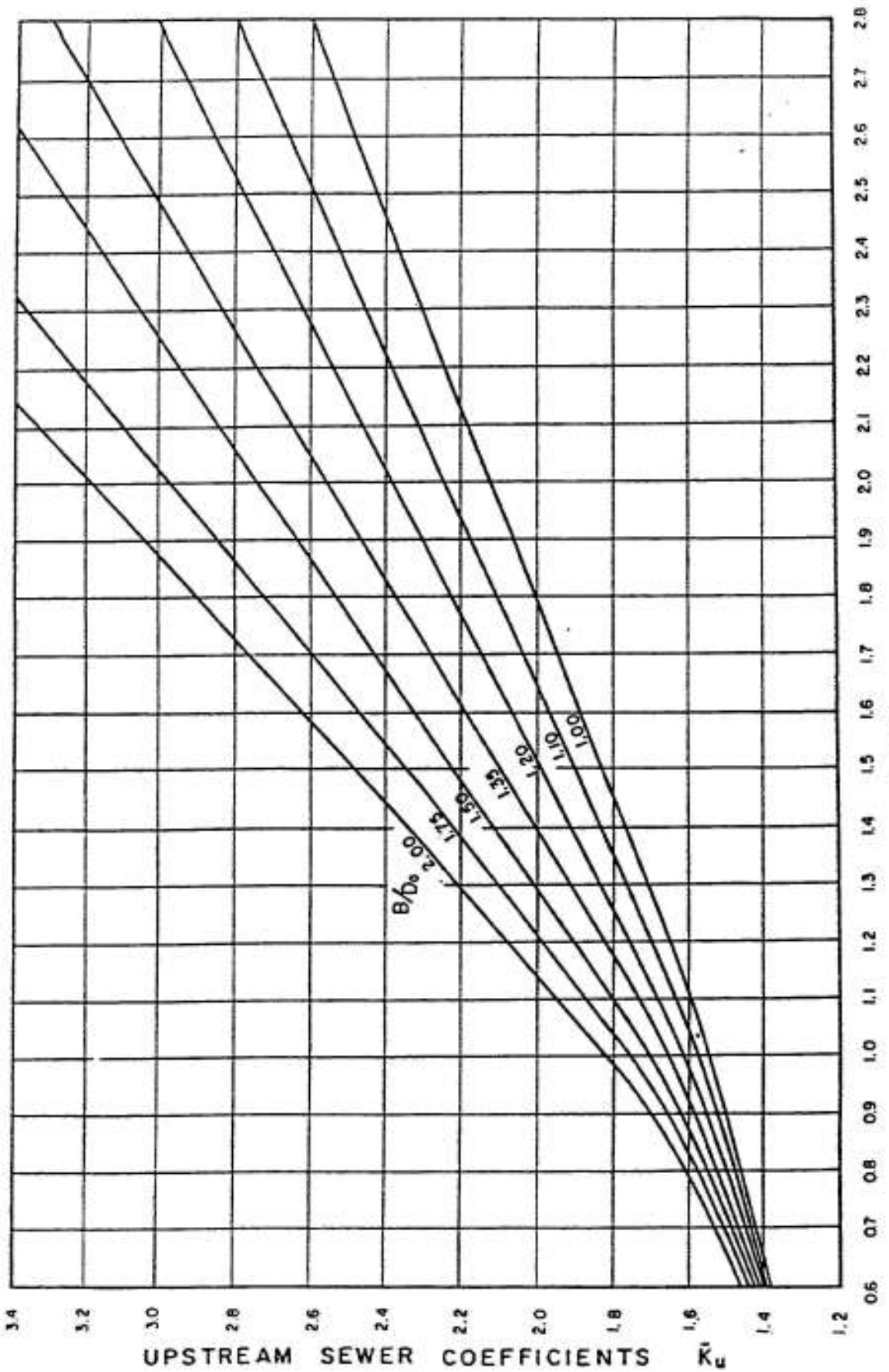

UPSTREAM SEWER COEFFICIENTS FOR HEAD LOSSES AT MANHOLES
UPSTREAM SEWER COEFFICIENTS K_u'
D_o / D_L
B/D_o
2.00
1.75
1.50
1.33
1.29
1.10
1.00
3.4
3.2
3.0
2.8
2.6
2.4
2.2
2.0
1.8
1.6
1.4
1.2
0.6
0.7
0.8
0.9
1.0
1.1
1.2
1.3
1.4
1.5
1.6
1.7
1.8
1.9
2.0
2.1
2.2
2.3
2.4
2.5
2.6
2.7
2.8

Figure 1.7.3

Values of K_L are obtained from the curve, "Head Losses in Curved Sewers".

The head loss required for curved sewers having a deflection other than 90° shall be determined by multiplying the head loss for a 90° bend by the "Head Loss Ratio" obtained from Figure 1.7.1

It can be seen, therefore, that the allowance for head loss in a curved sewer turning 90°, is less than one-third of that which would be required for the same turn in a standard manhole.

The head loss for radius pipe is allowed for by increasing the grade around the curve or by increasing the grade between the manholes on which the curve is located, by the amount of the head loss.

1.7.6 Special Junction Chambers

When special junction chambers are used and they are not sufficiently similar to one of the cases sited above, the head loss should be determined by the basic principles of conservation of momentum. An allowance is to be provided to compensate for the turbulence in the manhole.

K_L is obtained from Figure 1.7.6

$\left[1 - \left(\frac{Qu}{Qo} \cdot \frac{Do}{Du}\right)^{\frac{2Do}{D_L}}\right]$ Must be calculated

$\frac{V_o^2}{2g}$ is obtained from Figure 1.7.5

1.7.4.4 Upstream Sewers - Opposed and Each at 90° to the Outlet

$K_L = K_{lv} \cdot \frac{V_o^2}{2g}$ $H_L = K_{hv} \cdot \frac{V_o^2}{2g}$

K_{lv} - is the coefficient for the upstream sewer with the lower velocity

K_{hv} - is the coefficient for the upstream sewer with the higher velocity

Calculate $\frac{Q_{hv}}{Qo}$, $\frac{Q_{lv}}{Qo}$, $\frac{D_{hv}}{Do}$, $\frac{D_{lv}}{Do}$, $\frac{D_{hv}}{D_{lv}}$

Obtain H and L from Figure 1.7.7

$K_{lv} = (H - L) - 0.2$

$K_{hv} = 1.6$

1.7.5 Curved Sewers

Where curved sewers are used, there is an energy loss as the flow goes around the curve, which is in addition to the normal friction loss allowed for in straight sewers.

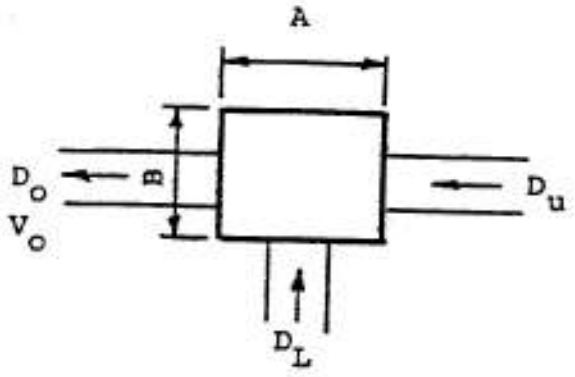
In addition to the grade required to overcome friction in a straight sewer, the minimum head loss to be allowed for a curved sewer with a 90° bend is obtained from the following formula:

$$H_L = K_L \frac{V_o^2}{2g}$$

1.7.4 Accurate Method of Head Loss Calculations

In this method, the head loss is calculated for each incoming sewer and is allowed for in the individual sewers as outlined in Section 1.7.2.2.

1.7.4.1 Manhole Abbreviations



1.7.4.2 Upstream In-line Sewer Head Loss

$$H_L = K_u \frac{V_o^2}{2g}$$

$$= \bar{K}_u \times \frac{K_u}{\bar{K}_u} \times \frac{V_o^2}{2g}$$

$\bar{K}_u$ is obtained from Figure 1.7.3

$\frac{K_u}{\bar{K}_u}$ is obtained from Figure 1.7.4

$\frac{V_o^2}{2g}$ is obtained from Figure 1.7.5

1.7.4.3 Upstream Lateral Sewer Head Loss

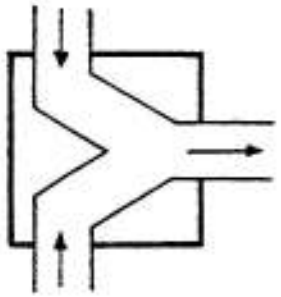
$$H_L = K_L \frac{V_o^2}{2g}$$

$$= \bar{K}_L \left[1 - \left(\frac{Qu}{Qo} \cdot \frac{Do}{Du} \right)^{\frac{2\,Do}{D_L}} \right] \frac{V_o^2}{2g}$$

1.7.3.2 Junctions

Tee - outlet at right angles to inlets and no deflector between inlets $K_L = 1.5$

- deflector between inlets for full height and width of incoming flows $K_L = 1.0$



Side and Cross Junctions - value of K_L is obtained from the following graph:

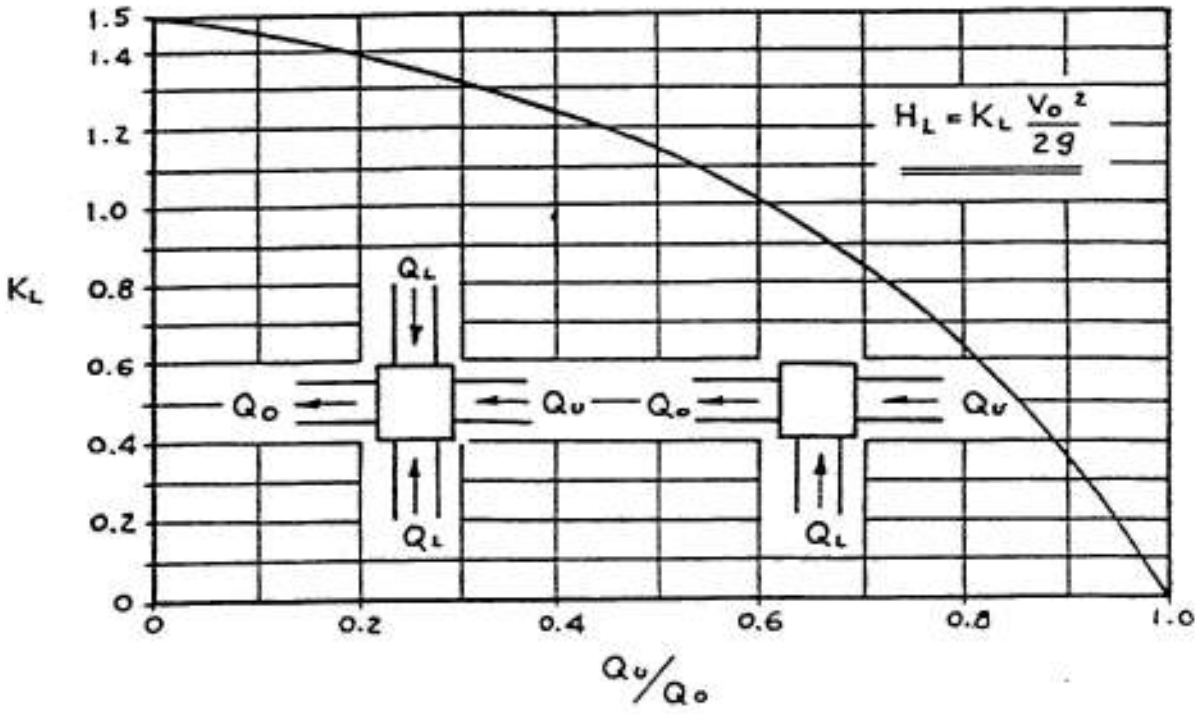


FIGURE 1.7.2.

1.7.3 Simplified Method of Head Loss Calculation

The head loss coefficient (K_L) for the particular bend or junction in the manhole is multiplied by the velocity head of the outlet sewer.

$$H_L = K_L \frac{Vo^2}{2g}$$

Head loss coeficients (K_L) to be used are:

1.7.3.1 Bends

90° - No benching or deflector, or where they are only up to the springline $K_L = 1.5$

90° - Benching or deflector to crown of sewers $K_L = 1.0$

Less than 90° - Multiply the head loss coefficient for a 90° bend by a head loss ratio factor from the following curve:

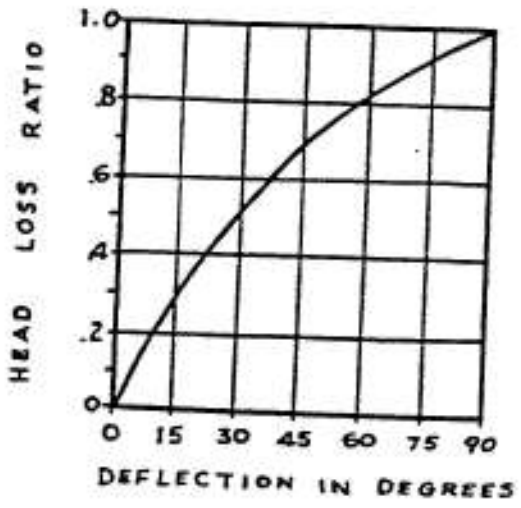


FIGURE 1.7.1.

The upstream sewers will be on a grade equal to the grade required to overcome the friction in the sewer plus the grade required to overcome the head loss in the downstream manhole.

An "accurate method" is also outlined in Section 1.7.4. This method is taken directly from the study with the addition of curves which aid in the solution of the formulas.

The head losses in radius pipe and in junction chambers are also described in Sections 1.7.5 and 1.7.6. For this purpose, a junction chamber is considered to be a manhole where two or more sewers enter the manhole with one or more entering at an angle other than 90° or 180° to the outlet sewer.

1.7.2 Addition of Energy to the System

1.7.2.1 Change in Pipe Size

In addition of energy to systems, the first point to keep in mind is that no energy is added when the crown of the inlet and outlet sewers are at the same elevation. Therefore, at changes in sewer size, the crown(s) of the incoming sewer(s) are to be at the same elevation where no energy is added or higher than the crown of the outgoing sewer where energy is to be added.

1.7.2.2 Addition of Energy

Energy is added to a system in two ways:

- Small Losses - where the head loss is 0.15m or less, the crown of the outlet pipe is dropped below the lowest incoming crown by the amount of the head loss.
- Larger Losses - where the head loss is greater than 0.15m, the crown of the outlet pipe is dropped below the lowest incoming crown by the amount of the head loss, and the upstream incoming crowns are dropped to the same elevation as the outlet crown. Sewers entering the manhole at 90° to the outlet need not be lowered.

1.7 HEAD LOSSES IN MANHOLES, CURVED SEWERS AND JUNCTION CHAMBERS

1.7.1 General

Where a manhole is placed on a sewer line, where two or more sewers meet in a manhole, where sewers meet in a specially designed junction chamber or where a sewer is placed on a curve there is a loss of energy greater than that of a straight length of sewer. These losses can be negligible as in the cases of small diameter sanitary sewers flowing partially full at minimum velocities or substantial as in the case of large diameter storm sewers turning 90° in a manhole at a high velocity.

The designer's responsibility is to ensure that he provides additional head energy in the sewer design to allow for the losses to be incurred. In cases where the head available is limited, he will have to adjust the design to provide for a system which is hydraulically smoother. This will require less head to be provided to overcome the energy losses.

The most complete study of this subject which has been undertaken to date has been done by Sangster, Wood, Smerdon and Bossy at the University of Missouri and published in their Bulletin No. 41 entitled "Pressure Changes at Storm Drain Junctions" and in the A.S.C.E. Journal of the Hydraulics Division entitled "Pressure Changes at Open Junctions in Conduits" - HY6 - #2057.

To facilitate the rapid determination of head losses in manholes, a number of typical manhole bends and junctions were studied. From this, a "simplified method" was established. This method has been found to be quite adequate for the vast majority of manholes and is outlined in Section 1.7.3.

eliminated.

Many designers, when designing deep sewers, will pre-install the connections to at least beyond the future curb line. This eliminates the need for re-excavating to the deep sewer when the connections are required, with its high cost and resultant traffic disruption, not to say anything about poor public relations.

1.6.3.3 Catch Basins

The locations and spacings of catch basins for roads is set out in the chapter on Transportation.

Most municipalities use standard pre-cast catch basins with standard frames and grates. Details of these can be seen in the "Municipal Services Design Standards."

At one time, there were probably as many types of storm water inlets (catch basins) and covers as there were designers. Each designer seemed to have his favourite designs, and, at least to him, valid reasons for his preference. The confusion that this caused to new designers, contractors and suppliers was quite inexcusable. The preparation of the "Municipal Services Design Standards" by MEA, as mentioned several times in this chapter, has done much to reduce the proliferation of choice, and to establish tried and true designs.

1.6.3.4 Other Appurtenances

Only the more commonly recognized sewer appurtenances have been listed above. Headwalls is included as a separate heading in this chapter, section 1.9. Others which bear mentioning here are as follows. The designer should be aware that they are not frequently used in design.

- Clean-outs
- Siphons
- Backwater Gates

is that where possible, the desirable spacing for manholes in a sanitary sewer system should be 100 m with 30 m being the maximum allowable. This same spacing is generally used for smaller storm sewers. However, for larger storm sewers, where men can actually enter the sewer, larger spacings are generally permitted.

1.6.3.2 Connections

Service connections may be known as building sewers, house connections, sewer laterals, etc., and are simply the connection between the street sewer and the property being served.

The designer of the sanitary and storm sewer should consider the design of the connections. In many municipalities, one department (Engineering, Works, or some other name) is responsible for the portion of the connections on the municipal road allowance while another department (known as Building or Plumbing, etc.) is responsible for the drains on private property. In some jurisdictions, the responsibility for both lies in one department.

The materials, joints and workmanship of sewer connections should be equal to those of the main sewer. Grades should be rigidly controlled at 2% or better; however, in extreme cases, 1% can be tolerated. Pipe alignment should be straight.

Whenever possible, connections should be installed by the use of factory made fittings at the time of construction of the main sewer. These connections should ideally terminate at the street line and be fitted with a water-tight plug.

The connections at the street line must be identified as to storm or sanitary so that when extended, the possibility of a crossed connection is virtually

Pre-cast manholes can be used for most sanitary sewers and for the smaller diameter storm sewers. However, at the junction of larger diameter storm sewers, it is usually necessary to have the flows merge without creating large energy losses. Therefore, the designer has to be aware of the energy losses which may occur at manholes and then design the manhole in a way that will not use more energy than can be reasonably provided for at the manhole. In cases where excessive energy losses will occur with the use of pre-cast manholes, special design manholes are required which permit the flows to combine in a more streamlined method. This will often involve the use of radius pipe to direct the flows downstream or deflectors or curved benching in manholes. The manhole is capped with a frame and cover. These are usually of cast iron, and, lately, many engineers are agreeing on the types of standard frames and covers. In sanitary sewer manholes, the cover contains at least two holes for lifting purposes. Many designers prefer more than two holes in order to provide adequate ventilation of the sanitary sewers.

The channelization in the bottom of the manhole should be U-shaped and extend up at least to the springline of the pipe. It is not necessary to construct the benching in sanitary manholes to the crown of the pipes as is desirable in storm sewer junction manholes. The benching should be nearly flat with just enough slope (1 : 8) to drain.

There are many different opinions on the spacing of manholes. Manholes are expensive to construct and they do interrupt the flow in the system. Their indiscriminate use should not be permitted. In a survey conducted in 1967, of municipalities in Ontario, the maximum allowable spacing ranged from 100 to 130 m for sanitary sewers. The conclusion that one can reach from this

smaller in size than storm sewers, and hydraulic considerations in sanitary sewer manholes, while still necessary, do not need to be so critical.

Manholes on sanitary and storm sewer systems must provide easy access for personnel to the sewer for examination and maintenance activities. They should be so designed and constructed to cause a minimum of flow interference, and, naturally, be structurally capable of remaining in place for many many years.

Manhole design is becoming quite standardized, and many sewer designers have at their disposal standard drawings of various types for a specific set of conditions. The MEA - "Municipal Services Design Standards" are an example of such standards. It is highly recommended that every design office should have at least one copy of these standards. The standards act as models during the design stage, and can be modified or amended to site requirements.

One thing that must be remembered in sanitary sewer manholes is that all small diameter sanitary sewers may require inspection by closed circuit television or photography, and for cleaning by dragging. It is necessary to shape the benching so that the equipment can be fed into the pipe and also removed either upstream or downstream. Additional steps or hooks should be placed immediately above the pipes to aid in the cleaning process.

The designer should become very familiar with the use and limitations of pre-cast manholes. Their use is becoming more favoured primarily due to the speed of installation. Flexibility in depth can be provided by the use of several courses of brick.

Where storm sewers are higher than the sanitary sewers, a check is made to see if there is adequate space for the sanitary service connection under the storm sewer. Figure 1.22 is a typical table used to check this distance. If there is inadequate space, the sanitary sewer is lowered as required. Where the obvert of the storm sewer is 2.5 m or lower below the road, the sanitary service connections can go over the storm sewer. In these cases, the sanitary sewer can be placed in a common trench with the storm sewer.

Where storm sewers pick up house weeping tile, the storm obvert is placed at about the same elevation as the sanitary sewers. The service connections go over the sewers.

As the sewers are designed, they are checked to ensure that they can pick up low basements, buildings set well back from the road, and low drainage spots. If transformer vaults are placed within the road allowance, the storm sewer may have to be lowered to provide a storm connection which will drain the vault.

Special care should be taken at intersections to ensure that the sewers do not conflict with other utilities or other sewers coming from side streets.

1.6.3 Appurtenances

Details of standard sanitary and storm sewer appurtenances can be found in the Municipal Engineers Association publication entitled "Municipal Services Design Standards" (4). This publication is highly recommended and may be obtained from the address listed in the Introduction of this Chapter.

1.6.3.1 Manholes

Manhole design for sanitary sewers is not as complex and demanding as for storm sewers. The sanitary sewers are

approving authority for authorization.

1.6.2 Minimum Pipe Cover

The depth at which sewers are installed is generally based on the frost penetration, and the depth of the basement being served.

On some streets, where small diameter storm sewers have been installed with only 2 or 3 feet of cover, it has been found that the frost action, after a number of years, has lifted the sewer up to the surface of the road. In similar cases, where the storm sewers were large, they were not affected as much by the frost action. To prevent the frost from acting on storm sewers, most engineers place the sewers at elevations such that the frost will not penetrate down below the springline. This usually results in the rule-of-thumb that the minimum depth of cover over small diameter storm sewers is 1.6 m and, over large diameter storm sewers, it is 1.25 m.

On fully serviced streets, the watermain is the shallowest liquid-carrying service. In Southern Ontario, the watermains and service connections are placed 1.6 to 2 m deep, and in Northern Ontario, 2 to 2.2 m deep, to avoid being frozen during severe winters.

If storm sewers do not pick up basement weeping tile, they can be placed just below the water service connections. If they pick up basement weeping tile, the storm sewer is placed at the same depth as the sanitary sewer. Sanitary sewers can generally be placed with their obvert 2.5 m below the road centre line. If the boulevard and front yard have a minimum grade of 2%, the sanitary house connection will usually be about .6 m below the house basement floor. This is adequate for the internal plumbing.

pose of the design is to accommodate a planned land use, and road alignments should not be adjusted to suit a sewer design matter.

1.6.1.2 Sanitary Sewer Layout

Sanitary sewer systems are generally laid out in the same manner as storm sewers. The first and second starting points mentioned under storm sewer layout are similar for sanitary sewers.

The storm and sanitary sewers generally parallel one another. This is desirable, as it reduces the conflicts between sewers and thereby helps prevent the sewers from getting too deep.

When it is advisable to construct the sanitary sewer at the same elevation as the storm sewer, then it usually is cheaper to install them in the same trench. This is very practical when rock is encountered during the excavation process.

1.6.1.3 Manhole Locations

Manholes are generally placed at all locations at which sewers change size, grade, depths and/or direction, at sewer junctions, and at locations which will ensure that the distance between manholes will not be too great for maintenance purposes.

Exceptions to this are in the larger diameter storm sewers, where radius pipe is permitted at changes in direction. Sometimes, in special cases, designers will put a vertical curve in large diameter storm sewers to change direction vertically in the same manner as radius pipe permits a horizontal change in direction.

Some municipalities now permit the installation of small diameter curved sanitary sewers. This helps to reduce the number of manholes required. Before a designer commences to design these, he should check with the

1.6 SEWER LAYOUT AND APPURTENANCES

1.6.1 Sewer Layout

1.6.1.1 Storm Sewer Layout

In general, when laying out a storm sewer system, the designer starts at the top end of the system being designed and works downstream from the lateral smaller sewers, to the sub-trunks, to the trunks and then to the outlets.

The starting points of storm sewer systems fall into three categories: The first points are where drainage from upstream areas within the watershed will in future enter the area now being designed. The second point is at the highest points in the road system. At these points, the storm sewers will flow downhill on one side of the high point and downhill in the opposite direction on the other side of the high point. The third points are at low locations within the area being sewered. These may be in parks, at the rear of lots on blocks or at other locations.

At the first and third locations, one generally has to start the storm sewer system at these points. At the second locations, it is recognized that the drainage system starts at this point. However, the storm sewer is started downstream at the first location needed for catch basin connections or connections from an abutting parcel of land.

From these locations, the storm sewers are designed flowing downstream. The sewers will connect with other sewers until they form sub-trunks and these will combine to form the major trunks which will eventually discharge into a creek, river or lake.

There are many factors that must be considered in the design of storm sewers. One of these is that the pur-

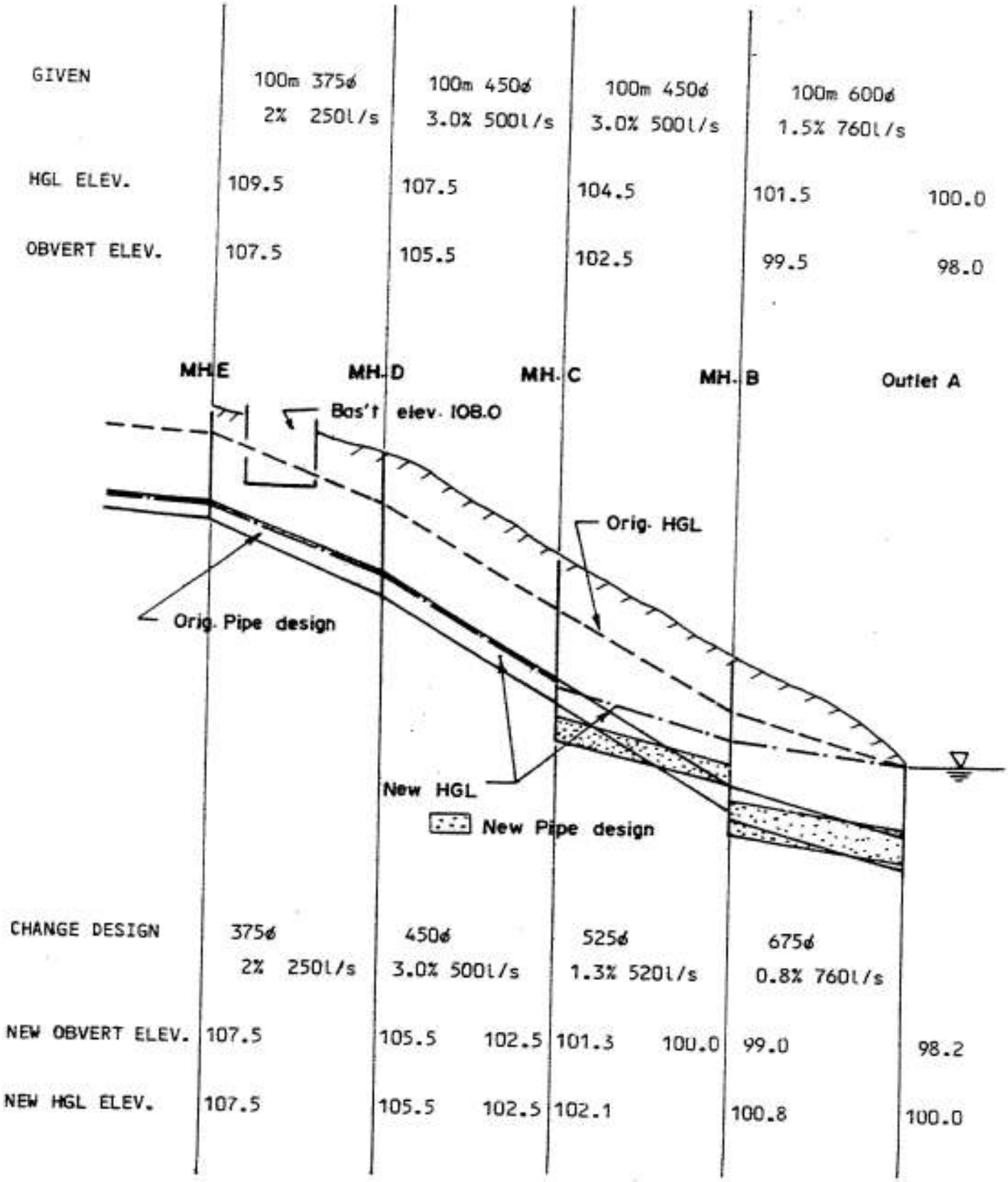

GIVEN
100m 375ø
2% 250l/s
100m 450ø
3.0% 500l/s
100m 450ø
3.0% 500l/s
100m 600ø
1.5% 760l/s
HGL ELEV.
109.5
107.5
104.5
101.5
100.0
OBVERT ELEV.
107.5
105.5
102.5
99.5
98.0
MH.E
MH.D
MH.C
MH.B
Outlet A
Bas't elev. 108.0
Orig. HGL
Orig. Pipe design
New HGL
New Pipe design
CHANGE DESIGN
375ø
2% 250l/s
450ø
3.0% 500l/s
525ø
1.3% 520l/s
675ø
0.8% 760l/s
NEW OBVERT ELEV.
107.5
105.5
102.5
101.3
100.0
99.0
98.2
NEW HGL ELEV.
107.5
105.5
102.5
102.1
100.8
100.0

FIG. 1.5.2.

of the receiving body of water at the storm sewer outlet. For example: the result of not taking into account a submerged outlet can be seen in Figure 1.52. The HGL as shown would cut the basement of the building between MH D & E though the pipe is sized correctly for the appropriate flow. To correct the HGL, the pipes could be redesigned as shown between outlet A and MH B & C. By increasing the diameter of the pipe and compensating by decreasing the slope of the pipe, the slope of the HGL is respectively decreased. In this example correcting these two lengths of sewer returns the HGL to the obvert of the pipe at MH C and the new HGL will follow the pipe from C to D to E.

designing sewer systems which are surcharged when carrying the design flows. Instead of placing the sewer obvert on or close to the piezometric graph line (surcharge line), it is placed lower. This procedure is not generally used. However, for the designer who wishes to consider it, an example can be found in the A.S.C.E. Manual Of Practice No. 37 (1) in Chapter 4.

1.5.5 Submerged Outlets

Some storm sewer outlets enter rivers or lakes and discharge below the surface of the water.

In these cases, the water level in the river or lake is to be used as the terminal piezometric elevation for the design of the storm sewer. Therefore, the available piezometric grade line (HGL) is taken from the water level at the outlet back along the storm sewer to the obvert of the sewer on the higher ground. Sewer sizes, grades, and manhole designs are selected which will result in their piezometric grade line, when carrying the design flows, following fairly close to the available piezometric grade line. Generally, the piezometric grade line of the sewer should meet the water surface elevation

the sewers, flows can be calculated. Also the head losses can be calculated for the manholes and junction chambers. If the manhole losses are small and do not affect the approximated surcharge line, the flows should be similar to those which actually occurred. If the head losses in the manholes are significant, the approximated surcharge line is to be re-estimated making allowance for the manhole losses. When a line is obtained which is close to the actual surcharge line, the flows as calculated should be close to those which occurred.

Care should be taken to look for anomalies. Where the recorded high water lines vary significantly from reasonably calculated lines there are frequently problems in the sewer system. Partial blockages in sewers, manholes with poor flow characteristics and locations with significant flows being added to the sewer will cause these discrepancies.

1.5.4.3 Sewer Systems Designed Based On Surcharged Sewers

The procedures vary slightly when

becomes higher than the permissible surcharging, comes back to the obvert of the sewer or comes to the top of the system.

If the surcharge line goes higher than the permissible limit, the increased flow has to be reduced and another trial made. If the surcharge line does not come up to any of the limits, additional trials can be made until the surcharge line meets, but does not exceed, one or more of the upper limits.. This will be the maximum capacity of the sewer.

1.5.4.2 Flows In A Sewer When Flooding Occurred

Establishing the flows in a sewer when flooding occurred is done in a manner similar to establishing the maximum capacity of a sewer. The known limits of flooding are plotted on the sewer profile. It is desirable to plot the high water levels in basements, manholes or other locations along the sewer. By striking grades through these known high water levels, the surcharge line can be approximated. Using these grades and the size of

cause problems. If it does, then the design surcharging has to be reduced to eliminate the problem on the lateral. The maximum capacity is determined by trial and error. The first trial is usually the total flow that it is desirable to have the system carry. Starting at the downstream end of the system, the first length of sewer having more flow than allowed for in the original design, is checked. If the grade at which the sewer would have to be laid, to carry the new flow was at, or less than, the grade of the sewer, no surcharging would occur.

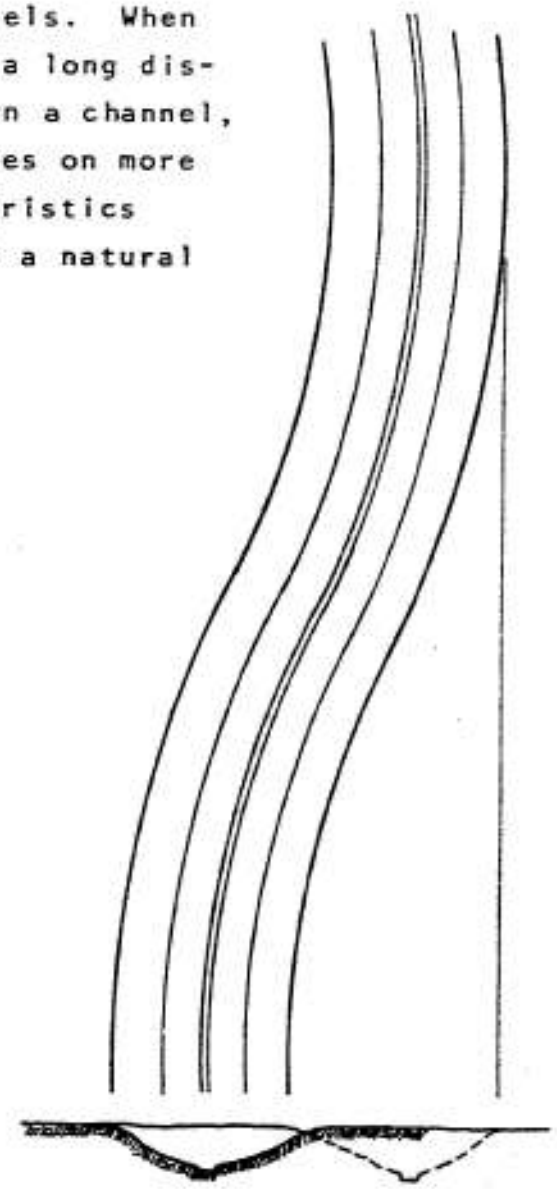
If the grade is greater than that of the sewer, surcharging will occur. To plot the surcharge line, it is normally adequate to start at the obvert of the sewer, at the downstream manhole, and plot the grade to the upstream manhole required to carry the new flow. This procedure is followed upstream. From the downstream surcharge line the head loss at the manhole, if any, is plotted, then the grade of the surcharge line is plotted for the next upstream sewer length. The surcharge line is plotted upstream until it

(a) Round Edges Off



Rounding the edges off has two advantages: they are less likely to be scalped when mowed and are much more pleasing aesthetically.

(b) Slow Longitudinal Curves - remove much of the canal look from channels. When one cannot see a long distance up or down a channel, the channel takes on more of the characteristics associated with a natural ravine.



(c) Trees and Bushes - also help soften the appearance of channels. These should be planted at random from the high water level up and along the berm adjacent to the channel. They should consist predominantly of varieties natural to the area to ensure a higher rate of survival. They should be planted in such a manner as to further break up the continuous view up and down the channel. They should be far enough apart to enable maintenance vehicles to travel along the berm (between the edge of channel and the limit of the right-of-way).

(d) Storm Sewer Outlets - should be kept to a minimum. Where provided, they should be as unobtrusive as possible. When placed under culverts, bridges, or at pedestrian foot bridges, they can be incorporated into the structures.

(e) Other Items - are frequently incorporated into the designs. These are combined pedestrian, bicycle, and cross-country skiing paths, exercise facilities, play equipment, park benches, and lighting.

1.12.1.7 Typical Channel Cross-sections

See Figure 1.70 for typical channel cross-sections. Actual cross-sections used vary widely due to the wide variety of conditions having to be met. However, the cross-sections, as shown on Figure 1.70, form the basis upon which most designs are established.

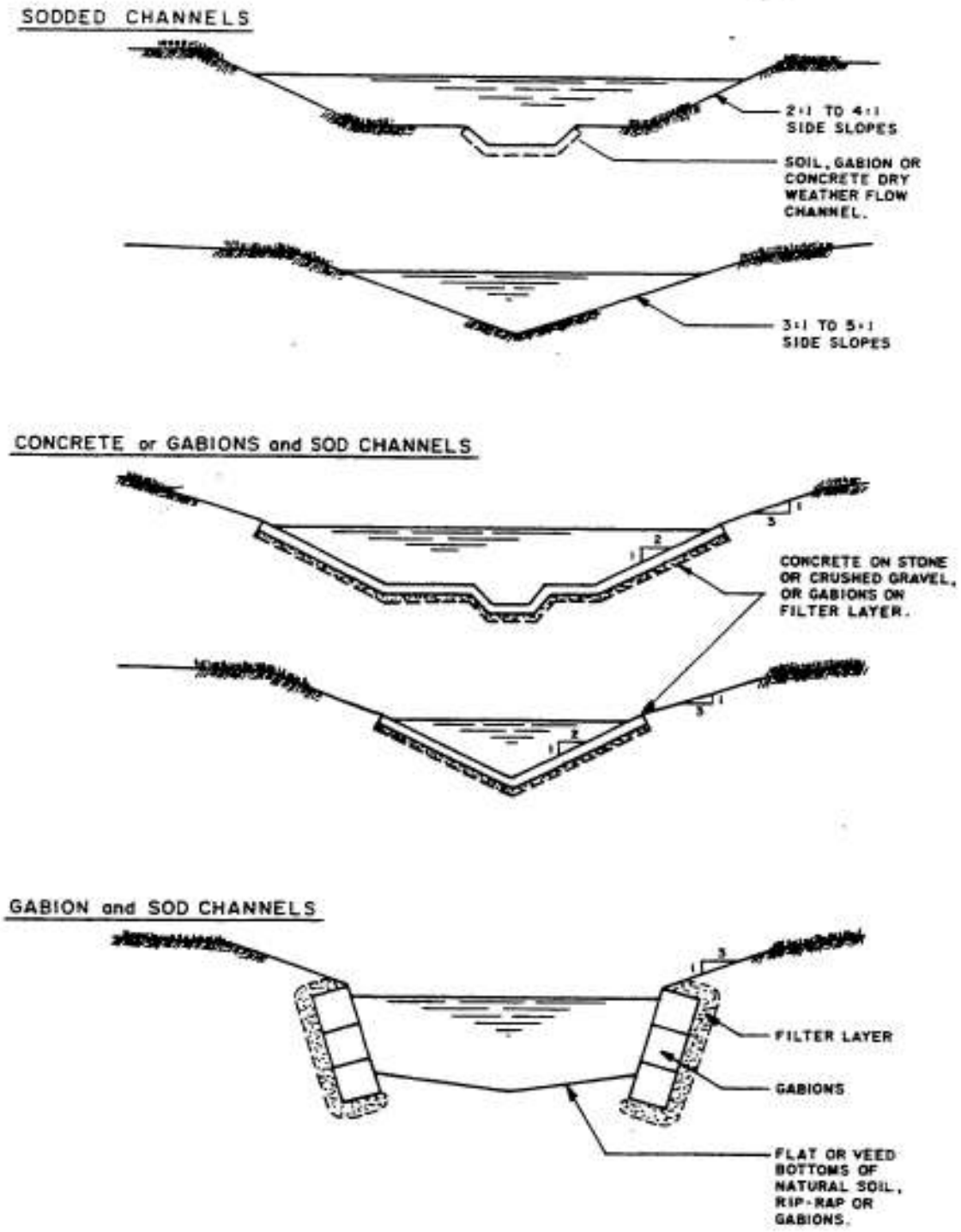

SODDED CHANNELS
2:1 TO 4:1 SIDE SLOPES
SOIL, GABION OR CONCRETE DRY WEATHER FLOW CHANNEL.
3:1 TO 5:1 SIDE SLOPES
CONCRETE or GABIONS and SOD CHANNELS
CONCRETE ON STONE OR CRUSHED GRAVEL, OR GABIONS ON FILTER LAYER.
GABION and SOD CHANNELS
FILTER LAYER
GABIONS
FLAT OR VEED BOTTOMS OF NATURAL SOIL, RIP-RAP OR GABIONS.

FIGURE 1-70

1.12.2 Dissipation of Excess Energy

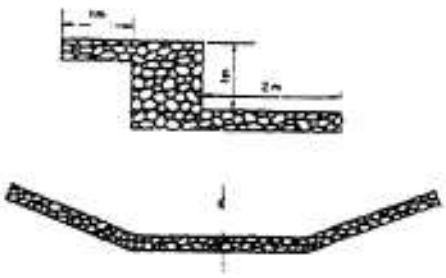
In earth and grass-lined channels, erosion is a major consideration. This is usually caused by the velocity of flow exceeding the erosion velocity of the soil in which the channel is constructed. To keep the velocity of the flow down below the erosion velocity of the soil, care has to be taken in the selection of the channel grade and the dissipation of excess energy where it occurs.

In normally rolling land, improved open channels have to be constructed at grades flatter than the natural creek grade it is replacing. The reason for this is that generally, the new open channel cross-section is a more efficient section than the natural creek, or the previously inadequate channel, and is hydraulically smoother than the one it is replacing. Therefore, to keep the velocities below the erosion velocity, it has to be constructed at a flatter grade.

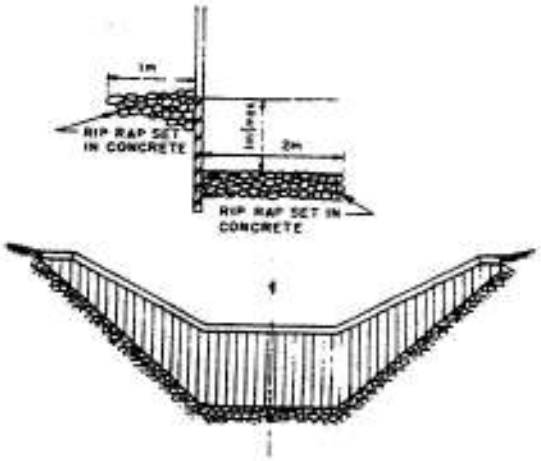
There are economical limits as to how deep a channel can be constructed and design limits on how shallow it can be. Therefore, there will be locations where there must be a drop in the invert of the channel. If the energy at these drops is not adequately removed, the additional energy will cause the velocity of flow to increase above the design velocity and thereby cause erosion.

In concrete, gabion, or riprap-lined channels, this increase in velocity is not normally a problem. The drops can be simple vertical drops as with a weir. However, checks should be made at the end of the lining to see if velocity will be down to an acceptable level.

TYPICAL DROP STRUCTURES



GABION DROP



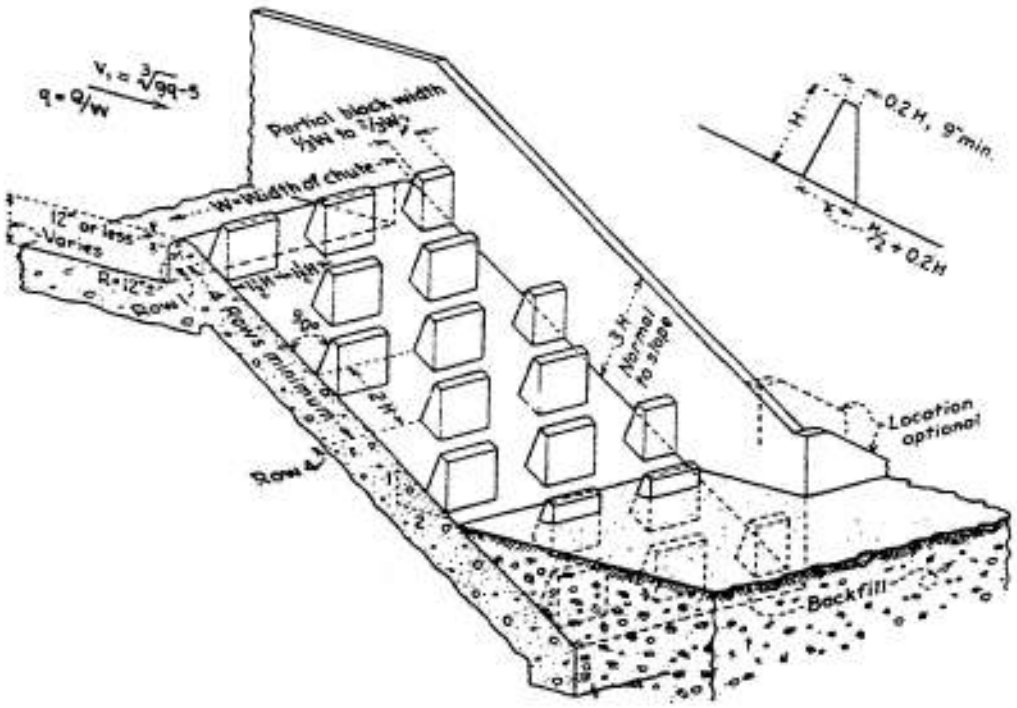
SHEET PILING

In earth and grass-lined channels which are temporary or will have major improvements made to them in the not too distant future, a value judgement is needed. It must be decided if an inexpensive drop along with minimum erosion protection with the expected erosion is acceptable. This is normally the case in farm drains and in small channels. The weirs are constructed of timber or gabions with crushed gravel behind them to act as a filter material. If this is not acceptable, a permanent type of energy dissipator is required, such as described in Reference (10).

There are two types of open channel energy dissipators generally used. One is the baffled chute type, which is shown on Figure 1.71. Another is the straight drop spillway stilling basin, and is shown in Figure 1.73.

1.12.2.1 Baffled Chute Energy Dissipator

This type of dissipator (10) has been used successfully for many years. It can be used for flows as high as 55 m ^{3}/s per metre of chute width, and drops may be as high as structurally feasible.

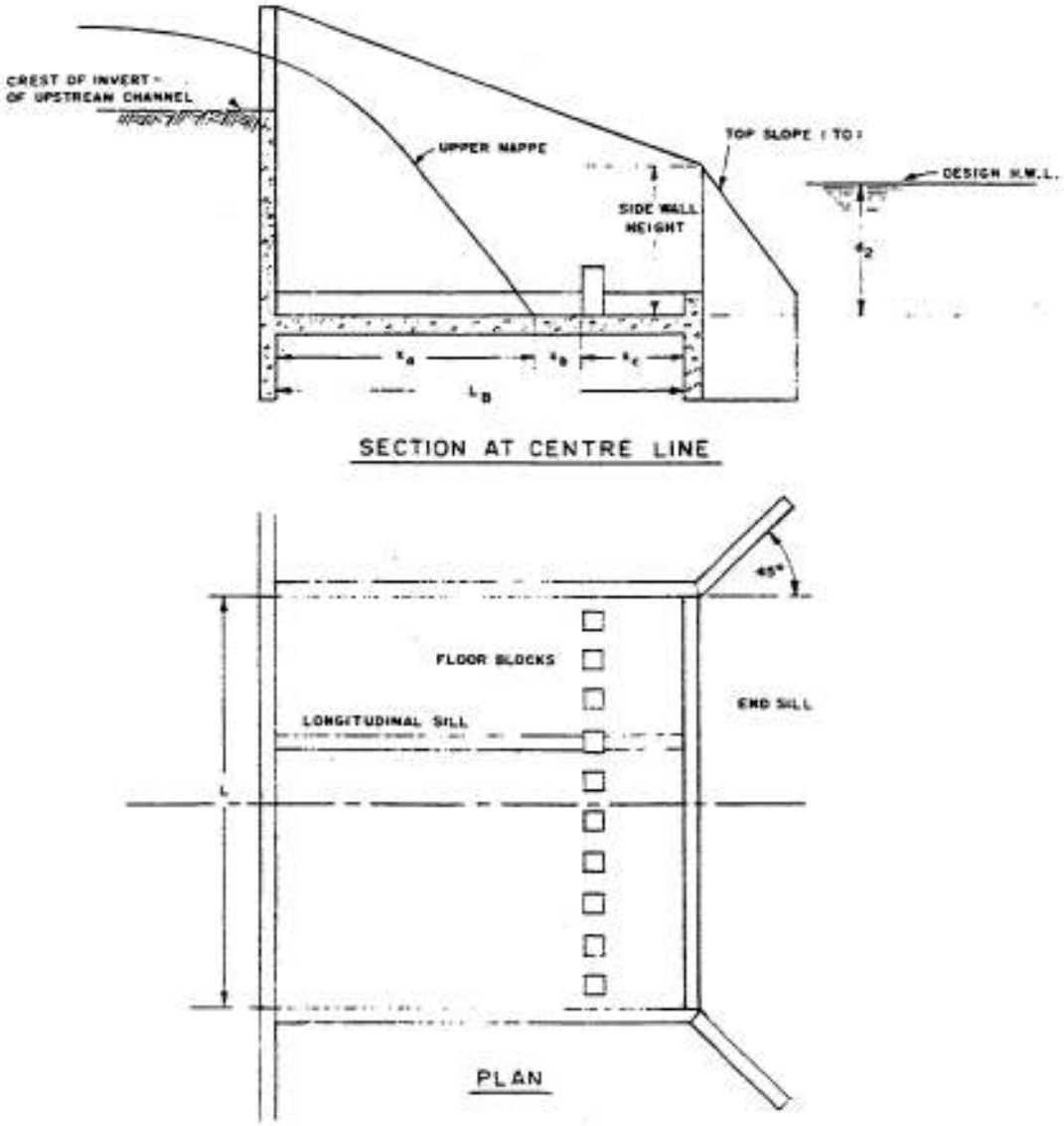


BASIC PROPORTION OF A BAFFLED CHUTE

FIGURE 1-71

1.12.2.2 Straight Drop Spillway Stilling Basin

Where the invert of the upstream channel is higher than the design high water level (H.W.L.) in the downstream channel, a straight drop spillway stilling basin (13) can be used to dissipate excess energy. See Figure 1.73. At locations where these higher drops occur, either this type or the baffled chute energy dissipator can be used. The selection will have to be based on



STRAIGHT DROP SPILLWAY STILLING BASIN

FIGURE 1·73

soil and flow conditions, construction costs, and other factors related to the particular application. The dry weather flow channel should pass through the crest and the end sill. Care should be taken to ensure the downstream dry weather flow channel is low enough to drain the stilling basin.

1.12.3 Design Nomographs and Charts

There are a number of design aids, such as nomographs and charts, which assist in selection of open channels. These are usually for the smaller channels, and can be found in various text books, some of which are listed in the bibliography.

1.13 INTRODUCTION TO STORM WATER MANAGEMENT

1.13.1 General

The design of a storm water system within a municipally controlled area is based on the Drainage Policies of the Municipality. Traditional policies prescribed that the storm runoff be collected in suitably designed systems and transported out of the boundaries of the development area as quickly as possible. The usual design technique employed was the Rational Method coupled with a storm frequency of either the 1 in 2 year or the 1 in 5 year storm. Using this technique, flows were calculated and a subsequent system of swales, street gutters, catch basins and storm sewers were developed for the area. This total flow was then dumped back into the natural drainage system located at the boundary of the development. Within the development area this man-made collection system works efficiently for the less intense design storms (i.e. 1:2 or 1:5). In general very little consideration was given to the problems of storms exceeding the design frequency where flooding and property damage may result.

The increased urban development creates large areas of land that are impervious. This includes streets, parking lots and roofed areas. This urbanization results in runoff rates and volumes that are much higher than existed in the natural drainage system prior to development. Another factor, not generally considered, was the effect of pollutant loading to the receiving waters. Pollutants accumulated on the land surface during dry weather conditions are washed readily from impervious surfaces into the storm sewers and then to the receiving waters. Among the products conveyed to the receiving waters are high levels of suspended solids, heavy metals, fecal materials of animal origin, phosphorus, street litter and debris. An inadequate assimilative capacity of the receiving waters would result in significant pollution downstream.

This traditional approach to stormwater drainage is characterized by an excessive concern for convenience on-site, and by a lack of consideration of environmental and hydrologic effects on-site and downstream. Continuation of the current methods for the design of storm drainage will lead to:

(a) Excessive capital funds being directed to drainage to increase convenience with limited assurances that property protection is provided against damage from severe storms.

(b) Excessive investment for improvements (i.e. channelization) to watercourses to carry stormwater from new development and to prevent large scale channel erosion.

(c) Increased downstream flooding.

(d) Possible liability claims against municipalities on the basis of negligence for ignoring modern drainage design concepts.

(e) Continued investment in combined sewer separation (without examining alternative approaches) often with little assurance that effective abatement of flooding or pollution will be achieved.

(f) Unnecessary soil loss in construction areas with resultant channel sedimentation and erosion.

(g) Loss of aesthetic features and recreational uses of many streams and watercourses.

(h) Continuing pollutant inputs following wet weather from storm sewers, combined sewer overflows and sewage treatment plant bypasses resulting in detrimental water quality impacts and high background levels of pollution.

(i) Increasing expenditures for controlling pollution at sewage treatment plants when in some cases, improvements in water quality might be achieved on a more cost-effective basis by dealing with diffuse sources carried in urban storm drainage.

From the foregoing it can be seen that there is a definite need for improved and more comprehensive stormwater drainage planning and design procedures.

1.13.2 Planning Procedures

The principle of natural drainage should be adopted wherever possible. This means that the natural predevelopment hydrologic conditons should be maintained, which includes groundwater recharge, maintenance of water tables, runoff volumes and rates, and the baseflow in dry periods.

The adverse effect of development on the hydrologic response can be remedied in some instances by the provision of storage facilities for excess drainage flows. Permanent or temporary ponds could reduce peak runoff, aid in groundwater recharge, provide a facility for the treatment of stormwater runoff and lessen downstream flooding, erosion and sedimentation. Where the storage of stormwater in ponds is practicable, the releases from the pond should simulate the predevelopment hydrograph for more frequent storms, or as required by downstream limitations.

The planning for drainage facilities should be coordinated with planning for open space, planning for sanitary sewage disposal, and planning for transportation. By coordinating these efforts, new opportunities may be identified which can assist in the prevention of drainage problems.

A master drainage plan should be set up for the municipality or specific development area. As the watershed acts as the natural boundary for water resources each master drainage plan should include the whole watershed area. In Ontario, Conservation authorities are organized on the basis of watershed boundaries for the purpose of flood control planning and operation of flood control systems. The master drainage plan must identify demands and problems within the watershed, established watershed management objectives and analyse alternative strategies to achieve those objectives.

1.13.3 Major and Minor Drainage System Concepts

The stormwater system is a single system having two purposes:

1. Control of stormwater runoff to prevent or minimize damage to property and physical injury and loss of life which may occur during or after a very infrequent or unusual storm; and
2. The control of stormwater to minimize inconvenience or disruption of activity as a result of runoff from frequently occurring but less intense storms.

This dual purpose is often characterized as the major and minor functions, and the drainage system itself is broken down into major system and minor system components.

The major system is the route followed by runoff waters when the capacity of minor system is exceeded. It generally consists of the roads and open channels. In certain cases where no surface route is available, storage elements or conduits may be required to maintain the major system.

The entire drainage system would be designed to provide a level of protection of property and life which fulfills the first or major function. The minor system would be designed for an appropriate level of convenience, (i.e. up to once in five year design frequency). It must be recognized that a total stormwater runoff system, subjected to an infrequent major storm, cannot be expected to prevent inconvenience and minor property damage. A design that would eliminate all such stress would be fundamentally unreasonable and almost certainly infeasible due to the high costs involved.

1.13.4 Technology Discussions

From the foregoing discussions it is apparent that the traditional approach should be updated. Today's Municipal engineers must be more demanding in their approach to storm drainage and understand the limitations

and strengths of the design techniques being used. A 1973 inquiry into urban drainage practice in Canada revealed that at that time all major muncipalities (with the exception of Toronto) employed the Rational Formula exclusively in urban drainage planning and design. The Rational Method provided a quick estimate of peak flow and was relatively well understood by all those involved in the drainage studies.

1.15.4.1 Rational Method: A Discussion

The Rational Method for peak runoff determination is usually expressed by the formula:

$Q = kCiA$ where;

Q = peak rate of runoff m^3/s

C = the runoff coefficient

i = the design rainfall intensity for a duration which equals the time of concentration (mm/hour) and A is the tributary area (hectares)

k = metrification constant to use the existing C values, with k = 0.0028

A review of Canadian practice indicated a wide discrepancy in the selection of parameters C, and the inlet time t_i used in conjunction with the Rational Method by different municipalities. Two questions arising on the use of the RM are whether the design gives conservative results and what is the maximum drainage area for which the RM can be applied.

The first item to be discussed is the runoff coefficient C. This coefficient lumps many parameters such as imperviousness ration, infiltration losses, depression storage, basin slope, ...etc. The simplest and most common way of selecting C is by relating it to land use

characteristics of the basin. Typical values can be seen in section 1.4.2. More recently, attempts have been made to determine the runoff coefficient in terms of imperviousness ratio which is a typical parameter in many of the newer sophisticated models. The simplest formula of this kind is a weighted relation of the type:

$$C = \frac{(Cperv \times Aper)}{Aper} + \frac{(Cimp \times Aimp)}{Aimp}$$

Where Cperv is the C value for the pervious area,
Cimp is the C value for the impervious area.

To improve the selection of C two directions were followed:

(a) expressing C in terms of rainfall intensity, and

(b) variable C in terms of time of concentration or time from start of rainfall.

To express C in terms of rainfall intensity a simple relation was developed which adjusts the runoff coefficient by multiplying the right hand side of the Rational Formula by a frequency factor "Cp" related to the storm frequency of occurence. The Formula therefore becomes:

$Q = kCiAC_f$

For more frequent storms different factors for C_f are used, for example, in Scarborough the integrated runoff coefficients used are:

for 1/25, $C_f = 0.7C_5 + 0.3$

for 1/100, $C_f = 0.5C_5 + 0.5$

Expressing C in terms of the time of concentration is related to the fact that at the beginning of a rainfall, abstractions (depression and infiltration) will be maximum, which means low runoff coefficient. As the rainfall continues, these abstractions will decrease with time,

resulting in an increase in the runoff coefficient. Graphs expressing C in terms of the time from beginning of rainfall are presented in articles such as ASCE and WPCF - Design and Construction of Sanitary and Storm Sewers, Manual No. 9.

The time of concentration (t_c) is a basic parameter which governs the selection of "i". Usually t_c is determined in terms of inlet time "t_i" plus travel time "t_t" which equals conduit or channel flow time. In most cases, a flat allowance is made for inlet time.

To determine the effectiveness of the Rational Method, it must be compared to accepted and proven techniques. A direct comparison between the Rational formula and the runoff hydrograph methods is not possible for various reasons. Some of these reasons are as follows:

(a) The Rational method does not take into account the time distribution of rainfall and assumes a uniform rainfall;

(b) The block rainfall data input into the Rational Method can be obtained from intensity-duration-frequency curves. Hydrograph methods can be used with hyetographs developed from the intensity-duration-frequency curves, however it is best to use storm profiles as close to real events as possible. Application of real storms with antecedent conditions may lead to peak flow-frequency relations. In the RM it is assumed that the return period of a storm is the same as that of the peak flow.

(c) For the hydraulic routing corresponding to a given storm event, the Rational Method substitutes consideration of different rainfalls of decreasing intensities. It does not give the designer the flow history of a runoff event. It gives only peak values and not hydrographs.
- The transformation of rainfall into overland runoff and channel routing are two distinct parts of the model analysis. This separation is not very obvious with the RM in which the two phases are lumped into a single process.

The IMPSWM PROJECT undertook a comparison under design conditions of the Ration Method and the Storm Water Management Model (SWMM). These results show that for a small area (approx. 20 acres) the Rational Method can be calibrated to give flows which are similar to SWMM peak flows, using a unique value of C in terms of the imperviousness and an inlet time related to the peak storm intensity. The peak intensity should be selected by analysis of real storms. In looking at the routing effects we find that the RM will produce larger errors as the drainage area increases. Also it seemed that the configuration of the sewer system has a bearing on the results, showing that a longer time of concentration creates a larger discrepancy.

Several alternatives for improving the RM results such that flows are in better agreement with SWMM are as follows:

1. <u>An increase in runoff coefficient "C"</u>: In fact many drainage manuals have recommended for residential areas C values which are slightly higher than those computed by the

weighting formula. This would lead to overestimation of flows for the upper small areas. However, the effect on the selection of pipe sizes may not be significant.

2. C variable with the time of concentration: It is possible to consider an increase of the C value in terms of the time of concentration.

1.13.5 Summary

The runoff coefficients adopted by many municipalities in conjunction with the RM is relatively high. The result is oversizing storm sewers designed by the Rational Method. On the other hand, for relatively large drainage areas, some increase of C may be required for sizing the pipes in the lower part of the drainage area. Relating the runoff coefficient "C" to the imperviousness (by the weighting formula) would result in C values lower than those currently used. Through systematic comparisons between the RM and SWMM flows under design conditions, it was found out that the RM flows (using "C" estimated in terms of imperviousness and adequate inlet time) may be in good agreement with SWMM flows for relatively small watersheds. However, for larger watersheds, the RM with the same parameters, tends to underestimate the flows as compared to SWMM flows, with the difference increasing with the area of watershed or the time of concentration. The "inlet time" should be selected interms of design storm "peakness". The latter should be determined by analysis of real storms. A logical approach to improve the performance of the RM is adopting a variable C which increases with the size of the drainage area (that is with the time of concentration). Still more testing should be done to safely select the parameters for the Rational Method but close comparisons are possible with small watersheds.

1.13.6 Models

For sophisticated drainage studies various models have been developed. In Canada the most common are STORM, SWMM, WREM, ILLUDAS and computerized unit hydrograph models such as HYMO. As model studies are still relatively new, there are improvements being made continually to get the "best" results. The best model for a municipality is the one that gives results as close as possible to the design events. Municipalities should limit the number of models used in routine studies based on such factors as: experience with the models and considerable testing,

- experience within the geographical area,
- priorities for the type of project,
- capabilities of staff reviewing modelling studies, etc.

For example, in Scarborough, they are advocating the use of two models, OTTSWMM and OTTHYMO. These are two models which give the Borough the best results.

In deciding to include model studies in a Municipality's Drainage Manual, the limitations, strengths and weaknesses of the models considered should be thoroughly investigated. Considerations could be as follows:

1. Multievent and single event model
2. Quantity and quality studies
3. Loading simulation for runoff
4. Imput conditions
5. The manual should include a good example
6. Should be a balance between practicality and accuracy. Sophistication does not always yield better results.
7. Should have a good routing routine capable of handling surcharges and various structures.

The following table developed by Paul Wisner, Ph.D., P. Eng., Philip Cheurg, B. Eng., P. Eng., Swapan Cupta, M. Eng., in a paper presented on Recent Model Comparisons shows the Canadian experience for several model studies.

CANADIAN EXPERIENCE WITH SEVERAL MODELS FOR VARIOUS STAGES OF SWM STUDIES

APPLICATION	RATIONAL METHOD	UNIT HYDROGRAPH	ILLUDAS	SWMM	WREM	STORM
Rural conditions	fair	good	fair	poor	poor	fair
Selection of critical events	N/A	N/A	N/A	N/A	N/A	very good
Preliminary Analysis of Urban conditions	good	very good	very good to excellent	very good	very good	good
Detailed analysis of Free Surface Peak Flows (pipes)	good	fair	good	very good	very good	poor
Detailed Analysis of Valley Storage	poor	good	fair	fair	fair	poor
Detailed Analysis of Surcharge	poor	poor	poor	poor	good	N/A
Quantity and Quality	——	——	——	good	good	very good

1.13.7 SWMM Model

The following is included as a cursory introduction to a specific modelling technique.

Reduction of runoff increases caused by urbanization has been adopted by many municipalities for the design of storm drainage systems. Peak flows are to be set to "the predevelopment level", or preferably to one determined by a site specific analysis.

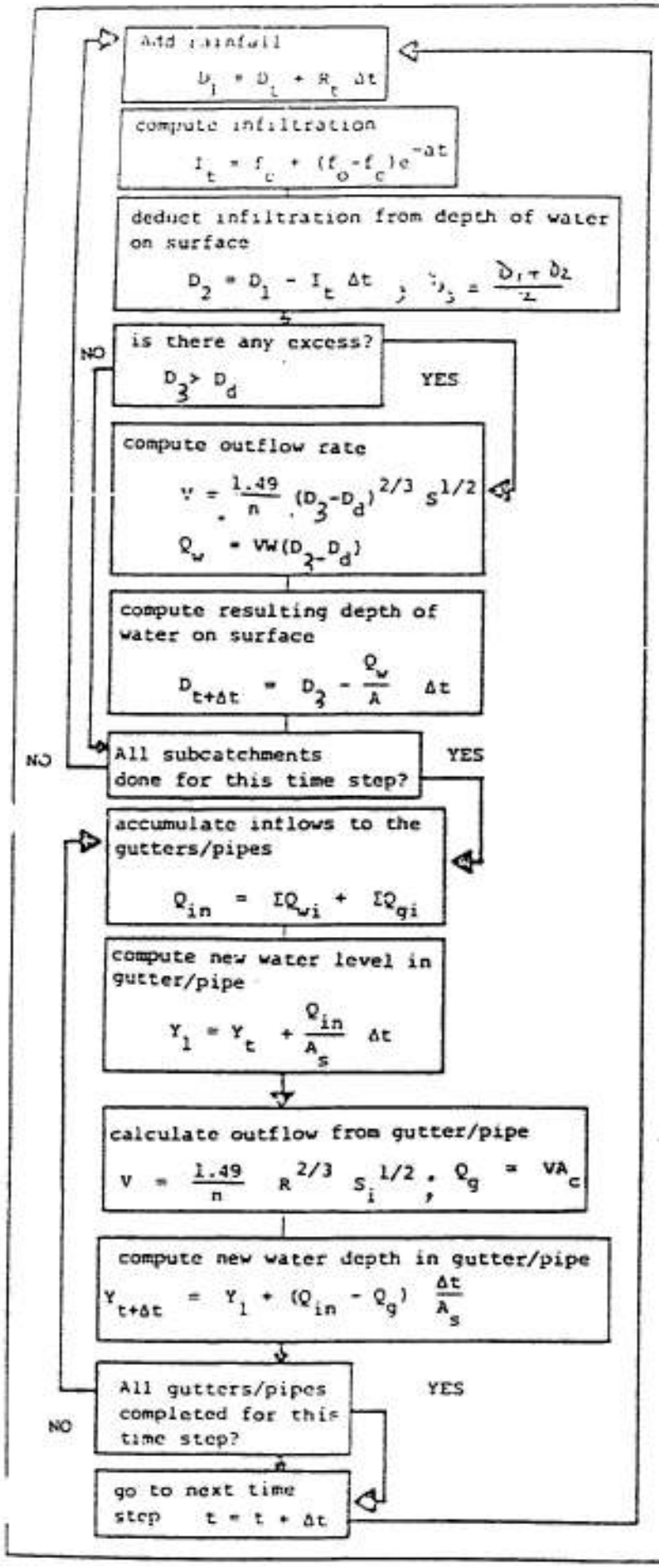
As a result of this approach: (i) the peak of the 1 in 2 or 1 in 5 year flow in the minor system may need to be reduced in many situations. The amount of excess water in the minor system, therefore, has to be stored temporarily, for example, by means of on-line underground storage; (ii) flows conveyed by the major system will also be directed by means of proper street grading to detention areas or parks where they are usually released back to the minor system at a very low rate.

The Storm Water Management Model is a single event computer model that simulates quantity and quality of urban storm runoff resulting from a real or design storm. The model performs first by converting rainfall into effective surface runoff hydrographs. These hydrographs are then hydraulically routed through the sewer system. Quality simulation by routing through storage or treatment facilities and simulations of receiving water body effects can also be done. The model consists of four main computational blocks: RUNOFF, TRANSPORT, STORAGE/TREATMENT, and receiving waters.

As no detail will be given for the model, the model algorithm is included.

1.13.8 Summary

The foregoing presents a general outline to storm water management. There are many publications available to give more details and the reader is suggested to obtain as much information as possible in order to make knowledgable decisions.



D_1 = water depth after rainfall
D_t = water depth on the surface at time t
R_t = rainfall intensity in time interval Δt
I_t = infiltration rate at time t
f_o, f_c, a = coefficients of Horton's equation
D_2 = water depth after infiltration
D_d = specified retention depth
V = velocity,
n = Manning's coefficient
S = ground slope
Q_w = total outflow rate
W = subcatchment width
A = surface area of the subcatchment
Q_{in} = inflow to a gutter
ΣQ_{wi} = outflow from tributary subcatchments
ΣQ_{gi} = flow rate of upstream gutters
Y_1 = new gutter water depth
Y_t = existing water depth in the gutter
A_s = mean water surface area between Y_1 and Y_t
R = hydraulic radius
S_i = the invert slope
Q_g = outflow from gutter
A_c = cross sectional area at Y_1

ALGORITHM OF THE SWMM MODEL

Fig. 1.15.1

1.14. CULVERT DESIGN

1.14.1 GENERAL

Culverts serve dual purposes, in that they function in both structural and hydraulic capacities. The structural function is served by allowing traffic to pass over the culvert, while the hydraulic function allows water to be conveyed through the culvert. As it has this dual function it is difficult to define what is a culvert and what is a bridge. In terms of span considerations, common MTC practice takes structures with individual clear spans of 6.0 m or less as culverts. Conversely, the Ontario Highway Bridge Design Code limits culverts to spans of less than 3.0 m. There are many crossings where either a bridge or a culvert will satisfy hydraulic and structural requirements. The MTC provides a list of considerations that should be taken into account to assist the designer in determining whether a culvert or bridge should be used, these are as follows:

1. The larger head generally allowed at closed invert culverts may permit use of a smaller effective waterway, thereby reducing the opening required.
2. The risk of a washout is often less for a closed invert culvert than for a bridge or culvert on spread fastings.
3. The risk of upstream flood damage is often higher at a closed invert culvert, since a larger head can be built up without causing a washout.
4. Well designed culverts are more traffic safe than bridges: firstly there is minimal danger to vehicles which run off the roadway, and secondly there is no reduction of shoulder or median width at culverts. On the other hand unprotected culvert ends can present a serious hazard to vehicles if left projecting from the fill.
5. Aesthetically, bridges are likely to be more acceptable than culverts in valleys or other locations where appearance is important.
6. Environmentally, bridges present less obstruction to fish and wildlife.
7. Culverts often present major interruptions to linear park systems in urban areas and can thereby create a hazard for pedestrians and cyclists who are forced to cross the roadway.
8. Bridge decks are prone to icing under certain weather conditions, creating a serious traffic hazard.
9. Maintenance of bridge decks is often expensive, and the avoidance of a bump at each end is difficult. On the other hand the road surface at culverts under very shallow fills may require frequent attention to counter the effects of differential heave and settlement.
10. Culverts can generally be designed and constructed in a shorter time.

To the foregoing, the designer should consider the economics of his choices. Although the cost of individual culverts is usually relatively small, the total annual cost of culvert construction constitutes a significant proportion of highway construction costs. Maintenance costs are also substantial. It can be seen that in some instances the choice between a bridge and culvert may be difficult, and should not rest entirely on cost considerations.

There are two other factors that should weigh heavily on the designers choice, and they are traffic safety and child safety. Where traffic barriers are not provided, culvert ends should be designed so that they will not present an obstruction to vehicles running off the roadway. This can be

achieved by covering exposed sides with fill, providing headwalls or wingwalls which will not present an obstruction, or mitering culvert ends parallel to the embankment slopes. High culverts in populated areas should be fenced whenever possible. Also for the hazard presented by culverts in private and side road entrances should be minimized by placing them as far as practicable from the roadway, and avoiding the use of headwalls.

With respect to child safety, streams and culverts will always attract children. Sudden deep drop-offs in the streambed should be avoided, particularly if submerged during normal summer flows. Where the invert must be placed unusually low to accomodate future deepening of the channel, and it is not feasible to fence off the culvert, consideration should be given to placing fill inside the culvert to reduce the depth of water. The fill should be protected from scouring by a temporary layer of riprap or gabions.

Should large verticle drops inside culverts be unavoidable, unauthorized access to the up stream end should be prevented by means of gratings. However, in view of their tendancy to accumulate debris, gratings, should be used only if no other alternative such as fencing is acceptable.

1.14.2 BACKGROUND INFORMATION REQUIRED

The following is a list of information required to adequately design culverts.

1.14.2.1 Field Investigations

(a) Site Survey - general
(b) Water shed data
(c) Site Specific Information for crossing
(d) Existing Structures
(e) Local Information
(f) Soils Information
(g) Environmental data - eg. fish migration data

After doing the field investigations the designer will have plan and profile information of the area including elevations of important buildings upstream to check on the backwater from the culvert. Using photo interpretation the drainage area can be ascertained including basin area, soil types, lake and swamp retention and slope of main channel. In looking at the site specific information available, the designer will have both a historical summary and the conditions at present. Included in this would be instances of ice or debris jamming, erosion problems, past or possible future channel alterations, streambed materials, a channel description for estimating roughness coefficients as well as photographs of the area which are valuable for design purposes and also helpful in the event of future claims. From interviews with local residents, knowledgeable of the area, will assist in the supplying of information not readily obtainable such as high water levels.

Having completed the field investigation the designer must next consider the culvert type and shape.

1.14.2.2 Culvert Data

In choosing the type of culvert the following should be considered:

(a) Open footing or closed invert
(b) Construction material
(c) Cross sectional shape
(d) Number of barrels

The first decision is whether to use an open footing culvert (usually having an invert of natural ground), or a closed invert type such as a pipe, pipe arch or box culvert whose invert forms an integral part of the structure. Open footed culverts should be used on bedrock or an equally scour resistant material, or if protected by a permanent floor slab with cutoffs. In most cases the size of the culvert required for safe performance would be larger for open footing than for the closed invert type.

The choice of material for culverts depends on the initial cost, service life, maintenance costs, hydraulic performance, ease of construction, salvageability, structural strength as well as environmental considerations. The optimum culvert will meet most of the considerations of the site as well as having the lowest annual maintenance cost.

The most suitable cross-sectional shape of a culvert depends largely on the height of fill and the depth of flow. The shapes include, circular pipes, arches, horizontal ellipses, rectangular shapes, and corrugated steel pipe arches.

1.14.2.3 Culvert Layout

The location, skew, profile, embedment and length of a culvert may have a significant effect on the construction and maintenance costs, hydraulics, channel stability, fish habitat, and safety of the culvert and road user.

(a) Location and Alignment

Wherever possible the culvert should be aligned with the natural stream channel. If in order to conform with the upstream and downstream channels it is necessary to break the culvert alignment then the culvert should be curved in plan or should have angular bends not exceeding 15° at intervals of 15m. Non-linear alignment could lead to debris blockage or increased head losses for culverts operating under outlet control.

Where flow from a roadside ditch on a steep grade is to be taken through a culvert under the highway, a skew angle of 40° is desirable to improve flow into the culvert and reduce the possibility that flow will over shoot the culvert entrance. The skew angle is the angle between the culvert centerline and a line normal to the highway.

(b) Culvert Profile

Considerations must be given to natural stream slope, barrel and outlet velocities, future deepening of drains, streambed degradation, differential settlement, undermining of culvert ends, deposition inside of culvert as well as environmental considerations, for the culvert profile and suitable embedment. It is common practice to place the floor of a closed invert culvert slightly below the natural streambed in order to increase the waterway area, permit future deepening of the channel, reduce

the possibility of undermining, accomodate a low road profile or provide for the passage of fish.

Experience has shown that deposits in properly designed culverts are normally cleaned out by moderate floods. The embedded cross sectional area (up to say 15% of the culvert height) should therefore be included in the hydraulic calculations as part of the effective area unless conditions are such that low-flow deposits are likely to remain permanently. This may occur if the culvert is located immediately down stream from an abrupt flattening of a steep channel, or if flood velocities through the culvert are unusually low.

1.14.2.4 Culvert Length

The length of a culvert should be sufficient to keep the fill from obstructing the waterway. Should an excessive length be used, for say future widening, consideration must be given to the possible uplift pressures on the excess length. Corrugated steel culverts are particularily succeptable to this uplift.

1.14.2.5 Road Profile and Relief Flow

Where it is possible to set road grades close to the culvert elevations, relief flow benefits can be realized. By allowing relief flow over the roadway during floods much larger than the design flood, the existing culverts could still be used. This allows water to flow over the road without increasing the backwater elevations.

Damage would be to the shoulder or roadway while leaving the culvert in tact. The principal disadvantage of relief flow is the increased risk to the road user, even though the inconvience of infrequent flooding of roads is generally accepted by the public. The road authority should ensure through warnings or road closures that the traffic hazards are minimized. Whenever possible, the lowest point of the road profile at a stream crossing shouldbe located at a distance from the culvert at least 3 to 4 times the culvert height, so that if the road washes out, the culvert will not be damaged.

1.14.2.6 Freeboard

The road grade should be set so that the freeboard from the normal design flood, measured from the upstream headwater elevation to the upstream edge of travelled way, is 1.0m for freeways, arterial roads and collector roads, and 0.3m for other roads.

1.14.2.7 Minimum Pipe Size

The minimum pipe sizes are necessary to facilitate maintenance. In special cases, such as a culvert under a very high embankment, the culvert would be oversized initially to allow a replacement culvert to be installed inside the old one.

1.14.2.8 Design Flood

Three classes of flood are applicable to culvert design: the normal design flood, the regulatory flood and the fish passage design flood. Estimation of normal and regulatory design flood discharges for small water sheds is dealt with in sections 1.4.1, 1.4.2, 1.4.3, 1.4.4, 1.4.5, and 1.4.6 of the Municipal Works Design Manual. It should be noted that design flood discharges may be modified at the designer's discretion to take account

of individual site conditions.

With respect to the Fish Passage Design Flood the designer should contact the Ministry of Natural Resources to check on their requirements.

1.14.2.9 High Water Level

A design high water level (HWL) is needed for establishing the allowable head water depth (AHW) or elevation (AHWEL), the tailwater depth (TW) or elevation (TWEL), minimum road grade, scour at open footing culverts and other related purposes.

The HWL used as the basis for backwater and scour computations is normally the natural unconstricted level corresponding to the design discharge under ice-free conditions. The HWL used for establishing the minimum road grade should include the backwater caused by the culvert.

The HWL can be found from the field investigation and transposed to the culvert site, allowing for the slope of the channel. Care must be taken in using the field data to ensure that the HWL corresponds to an ice free water level and not a water level caused by debris or ice blockage. The idea is to find a HWL as close to the design return period as possible, should no flow records be available. Should there be inadequate field data the HWL may be approximated by the Slope-Area Method. This method approximates the depth of water (d) at the culvert crossing. This method is reasonably accurate where the flow is confined to an artifical channel. For a natural channel with part of the flow on a flood plain inaccuracies may develop particularily in estimating the roughness coefficients and hydraulic gradients. It is important to check the calculated HWL with a known HWL and discharge.

The two basic equations used are:

(1) $V = \frac{1}{n} R^{2/3} S_o^{1/2}$ Mannings equation

(2) $Q = AV$

Where V = average velocity in cross section (m/s)
n = Manning roughness coefficient
R = hydraulic radius = $\frac{A}{P}$ (m)
A = cross sectional area of flow (m^2)
P = wetted perimeter of cross-section (m)
S_o = slope of water surface
Q = flow

Manning equation may be solved by the Nomograph of Fig 1.2.1 in this design manual. Through trial and error the appropriate depth d can be determined for the required flow in the channel.

An alternative would be to find the depth d through a stage-discharge curve. This is a plot of the height of water surface above streambed against discharge. Where records are available a stage-discharge table or curve should be obtainable from the local office of the Water Resources Branch of Environment Canada. The flood depth is read from the table or curve, and the HWL is obtained by adding the depth to the elevation corresponding to zero discharge.

1.14.2.10 Allowable Headwater (AHW)

Current practice is to not allow submergence of culvert inlets during design floods, i.e. to assume HW/D $\geq$ 1.0. The reasons for this are to reduce the possibility of ice or debris blockages, to facilitate removal of such blockages, and to place a reasonable limit on AHW. Selection of AHW or AHWEL should take into account the following considerations:

(a) Potential Flooding of Upstream Property - try to maintain a 1.0m minimum freeboard from the backwater curve to the property elevation.

(b) Downstream Considerations - care must be taken so that a large head of water would not wash out the embankment and cause serious damage to downstream properties.

(c) Road Grade - refer to Section 6 on freeboard.

(d) Seepage and Piping through fill - especially on narrow embankments (2 lane) of low head should be used.

(e) Overflow into Adjacent Watershed - this should be prevented entirely as it is illegal under common law.

1.14.3 CULVERT DESIGN

The design will be based upon a closed invert culvert having a uniform barrel section throughout. It is based upon Hydraulic Engineering Circular No. 5 published by the U.S. Federal Highway Administration.

1.14.3.1 Hydraulics

There are two major types of culvert flow: (1) Inlet control Fig 1.14.6
(2) Outlet control Fig 1.14.7

Under inlet control, the cross-sectional area of the culvert barrel, the inlet geometry and the amount of headwater or ponding at the entrance are of primary importance. Outlet control involves the additional consideration of the tailwater elevation in the outlet channel, and the slope, roughness and length of the culvert barrel; the inlet geometry is of less importance.

(a) Inlet Control

Inlet control means that the discharge capacity of a culvert is controlled at the culvert entrance by the depth of headwater and the entrance geometry, including the barrel shape, cross-sectional area and type of inlet edge. In inlet control the roughness and length of the culvert barrel and the outlet conditions are not factors in determining culvert capacity.

In all culvert design, headwater or depth of ponding at the entrance to a culvert is an important factor in culvert capacity. The headwater depth is the vertical distance from the culvert invert at the entrance to the energy line of the head water pool (depth + velocity head). The water surface and the energy line at the entrance are usually assumed to be coincident due to the low velocities in most entrance pools.

(b) Outlet Control

Culverts in outlet control can flow with the culvert barrel full or part full for all or part of the barrel length. The energy or head required to pass a given discharge through a culvert in

outlet control is given as:

$$H = H_v + H_e + H_f$$

H = total head (m)
H_v = velocity head (m)
H_f = friction loss (m)

The velocity head is given by: $H_v = \frac{V^2}{2g}$

where V = mean velocity in the culvert barrel (m/s)
The mean velocity is the discharge Q(m 3/s), divided by the cross sectional area A, in m^2, of the barrel.

The entrance loss H_e depends upon the geometry of the inlet edge. This loss is expressed as a coefficient K_e times the barrel velocity head, or;

$$H_e = K_e \frac{V^2}{2g}$$

Entrance loss coefficient K_e for outlet control are given in Fig. 1.14.21.
The friction loss H_f is the energy required to overcome the roughness of the culvert barrel;

$$H_f = \left[\frac{19.6n^2La}{R^{4/3}}\right] \frac{V^2}{2g}$$

where n = Manning roughness coefficient Fig 1.14.24
La = mean velocity in culvert barrel (m/s)
g = acceleration due to gravity (9.81 m/s^2)
R = hydraulic Radius = $\frac{A}{P}$(m)
A = cross sectional area of flow (m^2)
P = wetted perimeter (m)

Subsituting for $H = H_v + H_e + H_f$
and simplifying for <u>full flow</u>;

$$H = \left[1 + K_e + \frac{19.6n^2La}{R^{4/3}}\right] \frac{V^2}{2g}$$

Note the hydraulics in Fig 1.14.3 for culvert flowing full in outlet control.

The <u>energy line</u> represents the total energy at any point along the culvert barrel. The <u>hydraulic grade line</u> is defined by the

elevations to which water would rise in small vertical pipes attached to the culvert wall along its length. Both these lines are parallel over the length of the barrel except in the vicinity of the inlet where the flow contracts then expands. The difference between these two lines is the velocity head, $\frac{V^2}{2g}$.

The expression for determining the head H is derived by equating the total energy upstream from the culvert entrance to the energy just inside the culvert outlet, with consideration of all the major losses in energy. By referring to Fig 1.14.3 and using the culvert invert at the outlet as a datum, we get:

$$d_1 + \frac{V_1^2}{2g} + LaS - d_2 = H_v + H_e + H_f$$

Where d_1 and d_2 = depths of flow as shown in Fig

$\frac{V_1^2}{2g}$ = velocity head in entrance pool,

and LaS = length of culvert times barrel slope.

Then simplifying and substituting

$H = H_v + H_e + H_f$ we get:

$$H = d_1 + \frac{V_1^2}{2g} + LaS - d_2$$

As can be seen from Fig 1.14.3 that the head H is the difference between the elevations of the hydraulic grade line at the outlet and the energy line at the inlet. As with inlet control, the elevation of the energy line at the inlet can normally be assumed to coincide with the water surface of the head pond.

Nomographs are provided for solving of H with full flow. Each nomograph is drawn for a particular barrel shape and material and for the roughness coefficient noted on the respective charts.

(c) Head Water Depth

Head water depth HW can be determined from one equation for all outlet control conditions:

$$HW = H + h_O - LaS$$

Where H = head - determined from nomographs

h_O = greater of TW and $\frac{dc + D}{2}$

d_c = critical depth - see chart

D = culvert length (m) and S = slope of barrel

Determing h_o is done as follows:

i) Tailwater at or above top of opening - when the water surface in the outlet channel is at or above the top of the culvert outlet, h_o is equal to the tailwater depth. Tailwater depth TW is the depth from the culvert invert at the outlet to the water surface in the outlet channel.

ii) Tailwater below top of opening - If the tailwater elevation is below the top of the culvert opening at the outlet, h_o is more difficult to determine. In these cases, h_o is the greater of two values: (1) TW depth as defined above or (2) $\frac{dc + D}{2}$. This latter dimension is the height from the invert to the equivalent hydraulic grade line (located at half the height between critical depth and the culvert crown).

The critical depth is read from the chart (example Fig 1.14.18), and D is the culvert height. The value of d_c can never exceed D, making the upper limit of this fraction equal to D. Where TW is the greater of values (1) and (2) above, the critical depth is submerged sufficiently to make TW effective in increasing the head water.

(d) Outlet Velocity

The velocity at a culvert outlet is generally higher than the natural stream velocity. In erodible soils this may result in a scour hole immediately downstream. Calculation of the outlet velocity is necessary to assess whether it is acceptable, or whether erosion control measures or modifications to the culvert design are required.

i) Velocity for Inlet Control

Outlet velocities for culverts flowing in inlet control may be assumed equal to the mean velocity of flow in the culvert. This may be determined by trial and error based on the Manning equation.

ii) Velocity for Outlet Control

In outlet control, the outlet velocity is the discharge divided by the cross sectional area of the flow at the outlet. This flow area is measured up to critical depth or tailwater depth, which ever is higher, but cannot exceed the full area of the culvert.

(e) Non-Standard Roughness Coefficients

i) Roughness Coefficient other than Nomograph Value

If the culvert has a roughness coefficient different from that of the outlet control nomograph, as in the case of a timber culvert, the culvert length should be adjusted before entering the nomograph as follows:

$$L_1 = L\left(\frac{n_1}{n}\right)^2$$

Where L_1 = adjusted length of culvert

L = actual length of culvert with roughness coefficient n

n_1 = non standard coefficient

1.14.4 DESIGN PROCEDURE

The design procedure hinges on determination of the headwater depth HW for both inlet and outlet control for a selected trial size; the governing HW is then the larger of the two. If the governing HW exceeds or is considerably smaller than the allowable headwater AHW, a larger or smaller trial size is selected and the process repeated until HW is acceptable. The outlet control nomographs are acceptably accurate if the culvert is flowing at least three quarters full.

STEP 1 Prepare preliminary data

1. Field notes, existing culvert sizes, HWL etc. Fig 1.14.25 Plan, profile and sections of road and stream as required. Airphotos.
2. Select culvert types, shapes, materials, etc.
3. Design Discharge - calculate and check design flow.
4. Initial layout - decide on location, skew, embedment slope and culvert length.
5. Allowable headwater depth (AHW) - decide on AHW

STEP 2 Tailwater depth (TW)

1. Determine natural flood depth d corresponding to the design flow from field data where possible, otherwise by slope-area method.
2. Calculate TW = d + culvert embedment.

STEP 3 Select trial size of culvert

1. Enter onto Fig 1.14.15 with HW/D = 1 to find required dimensions.
2. If D is too large because of limitations on fill height or available sizes, try a flatter culvert shape (pipe arch, ellipse or box), or try multiple barrels.
3. If possible, check trial size with size of existing culvert.
4. Enter trial culvert size, type, inlet details, N, K_e and S on design form.

STEP 4 Inlet Control Calculations

This step determines HW and the size required for inlet control.
Fill in columns as follows:

Col. 1 Enter Q/N
Col. 2 Enter area A per barrel
Col. 3 Enter Q/NB for box culvert only
Col. 4 HW/D from charts
Col. 5 HW = D x col. 4

If HW is greater or significantly less than AHW, try other sizes until HW is acceptable for inlet control before going on to outlet control calculations.

STEP 5 Outlet Control Calculations

This step determines HW for outlet control, whether the flow is in inlet or outlet control, and if in outlet control, the size required. Note that the nomograph should not be used if HW/D<0.75 as inaccuracies arise.

1. Fill in columns as follows:
   Col. 6 Head H, obtained from Fig. 1.14.15
   Col. 7 Critical depth dc, from Fig. 1.14.18
   Col. 8 Calculate and enter $\frac{dc + D}{2}$
   Col. 9 TW if not already entered. (See step 2)
   Col. 10 h_O = the larger of $\frac{dc + D}{2}$ and TW
   Col. 11 LS = barrel length x barrel slope
   Col. 12 Headwater depth HW = cols. 6+10-11. If HW is negative enter 0.

2. Controlling HW: Larger of cols. 5 and 12 (circle on form)
   If outlet control governs and HW>AHW, repeat Step 5 with larger sizes until HW is equal to or slightly less than AHW.
   If HW is equal to or slightly less than AHW, size is acceptable. Go to next step.

STEP 6 Outlet Velocity V_O

Determine V_o for inlet or outlet control, whichever is governing (V_o will be evaluated in step 10)

(a) Inlet control
V_o for inlet control can be calculated on Fig. 1.14.31

(b) Outlet control
Determine cross-sectional area of culvert A_o up to d_c or TW whichever is higher. (Use Fig. 1.14.22 for pipe and pipe arch respectively.
If outlet is submerged, use full area).
Calculate outlet velocity $V_o = Q_D/A_o$. Enter in col. B.

STEP 7 Regulatory Hood
If there are buildings or developable land which are likely to be affected by the crossing during the regulatory flood go to the regulatory flood sub procedure in Subsection 7.1 of the MTC drainage manual.
If not go to the next step.

STEP 8 Fish Passage
If required go to subsection 7.3 of MTC drainage manual. If not go the next step.

STEP 9 Improved Inlet Option

Not included in this design as it deals with culverts with lengths greater than 50m operating under inlet control. Can be seen in Subsection 6.3 of MTC drainage manual.

STEP 10 End Treatment

Required if outlet velocity would cause scour or erosion downstream. Refer to subsection 9.4.1. and 9.4.2 of the MTC drainage manual for more details.

STEP 11 Final Culvert Selection

Select optimum design on basis of overall costs and benefits. Check that the culvert size is reasonably consistent with sizes of existing culverts known to be adequate.

EXAMPLE: Conventional Pipe Culvert Design

Given: $Q_{50} = 28.3\ m^3/s$
Channel is inerodible, with slope of bed S_o = o.oo5m/m, natural bed elevation at culvert outlet = 57.35. AHWEL = 61.0, AHW = 3.35m.
From field investigation, approx. 50 year flood depth is 1.9m above average streambed, and there is no flow through most of summer.
Culvert length L_a = 119m. Invert is to be embedded 0.3m below streambed. No restrictions on outlet velocity.

Required: Determine size of CSP and CSPA required with sharp-edged projecting inlet.

Solution:

Step 1 Preliminary data
see given data

Step 2 Tailwater depth
TW = 1.9 + 0.3 = 2.2m

Step 3 Trial size
From charts with HW/D=1, Q=28.3, N=1, required D=3.6m. For this size HW=3.6 which is larger than AHW, therefore try 2 barrels. With HW/D=1, N=2, Q/N=14.15, required D=2.75m. Nearest size of SPCSP= 2.74m (see Fig. 1.14.12). Use this for first trial of circular pipe. (Twin pipes).
Trial size of pipe arch is based on end area of final pipe size selected (see Fig. 1.14.8) i.e. 2 x 5.73m^2 = 11.46m^2.
Nearest SPCSPA size from Fig. 1.14.12 is a single 4.72m x 3.07m giving 11.38m^2.

Step 4 Inlet Control
See Fig. 1.14.8

Step 5 Outlet Control
For calculations see Fig. 1.14.8. The culvert is in outlet control since HW is larger than for inlet control.

Trial 1 (twin 2.74m SPCSP) is OK.
Trial 2 (twin 2.70m SPP) is also OK and would be more economical. Note effect of smaller reoughness on HW.
Trial 3 (twin 2.40m SPP) is too small.
Trial 4 (single 4.72 x 3.07m SPCSPA) is OK.

Step 6 Outlet velocity

Since the culvert is in outlet control, V_o is calculated as follows:

(a) Circular pipes (2CSP 2.70m dia.)

Pipe opening area A, from Fig. 1.14.8, Col. 2 = $5.73m^2$ each.
TW (>dc) = 2.20m
TW/D = d/D = 2.2/2.7 = 0.81
corresponding A_o/A = 0.88 (Fig. 1.14.22)
Therefore A_o = 0.88 x 5.73 = $5.04m^2$
$V_o = QD/A_o$ = 14.15/5.04 = 2.81 m/s

(b) Pipe Arch (1 SPCSPA 4.72 x 3.07m)

Pipe arch opening area A, from Fig. 1.14.8, Col. 2= $11.38m^2$
TW (dc) = 2.20m
TW/D = d/D = 2.20/3.07 = 0.72
corresponding A_o/A = 0.83 (from Fig. 1.14.22)
Therefore A_o = 0.83 x $11.38m^2$ = $9.44m^2$
$V_o = Q_D/A_o$ = 28.3/9.44 = 3.0m/s

Step 7 - 8 - 9 and 10 are not applicable in this example.

Step 11 Final Culvert selection

Required CSP = 2 barrel 2.7m diameter (projecting ends)

In the preceeding we have looked at a standard culvert selection. The designer should be aware that more efficient inlets are available as well as end treatments. The designer is referred to the MTC Drainage Manual - Volume 1 for more detailed information. The material in this manual should suffice to do any standard designs.

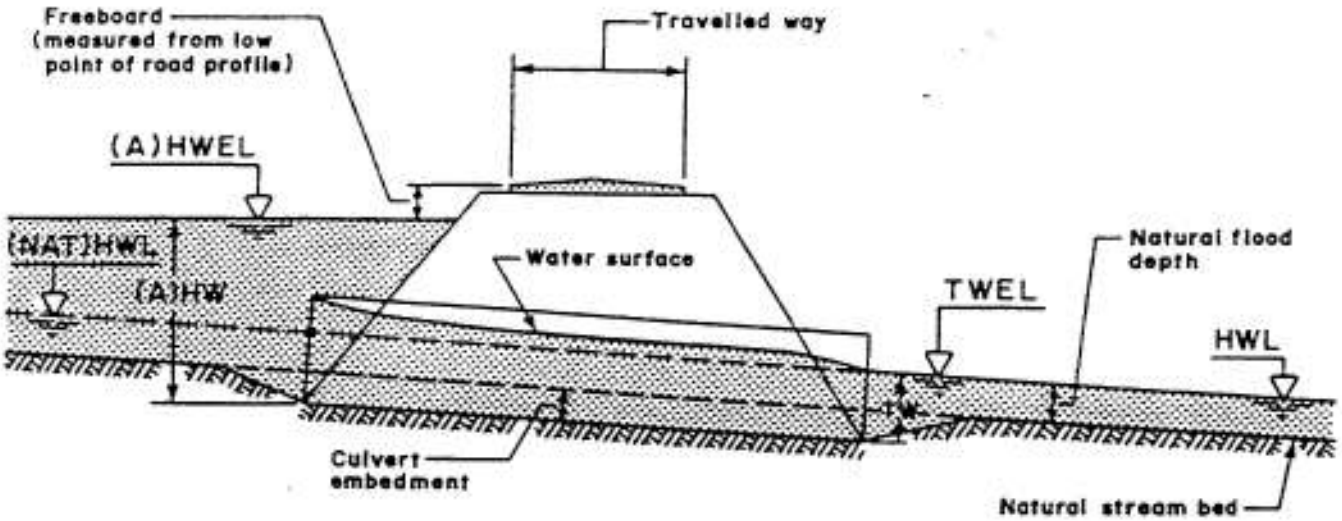


Fig.1.14.1 - Water depths and elevations at a culvert.

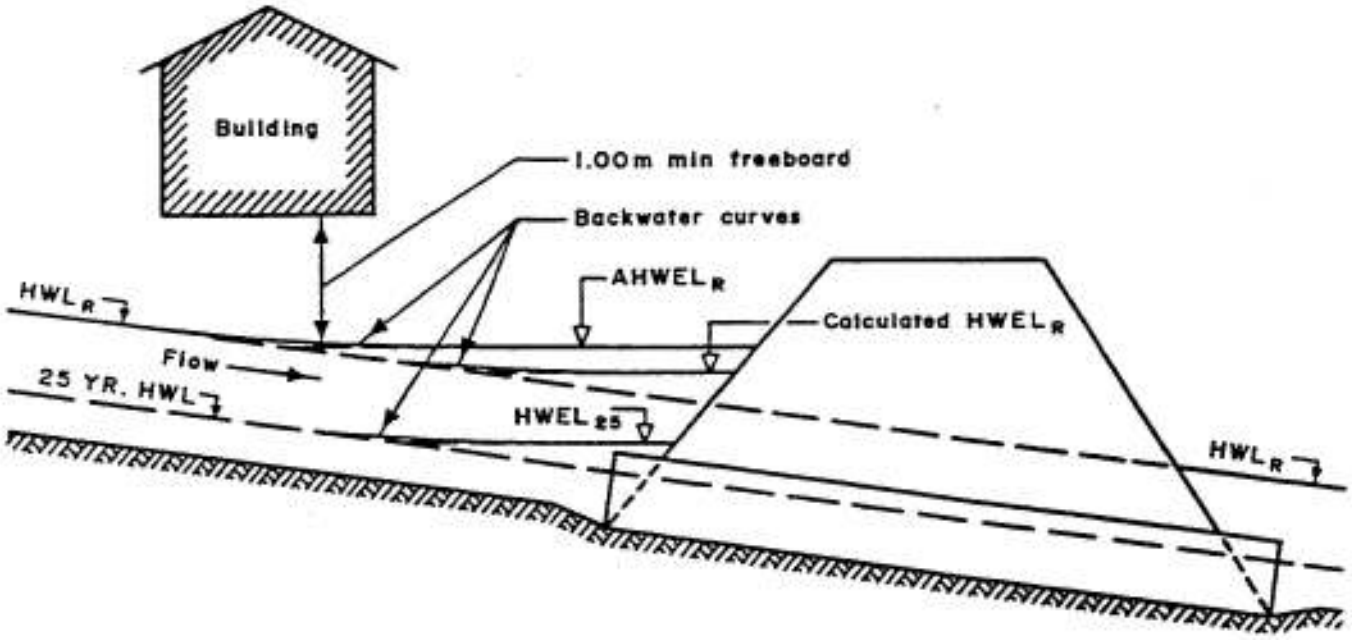


Fig. 1.14.2- Typical selection of allowable headwater elevation ($HWEL_R$) for regulatory flood.

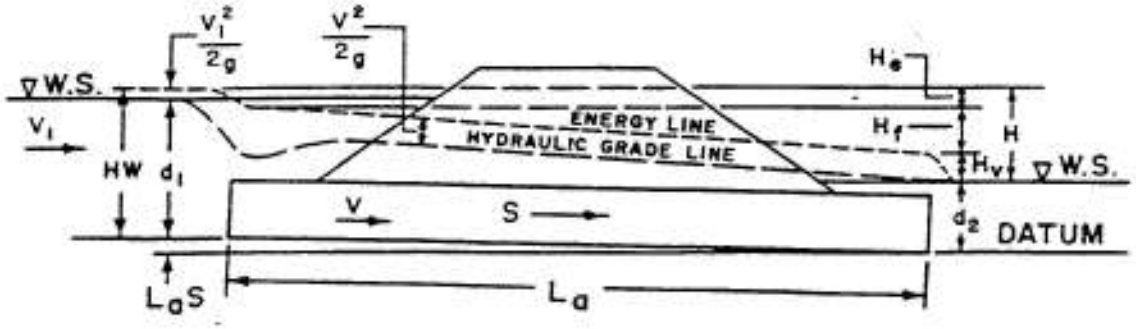


Fig. 1.14.3 - Hydraulics of culvert flowing full in outlet control (1).

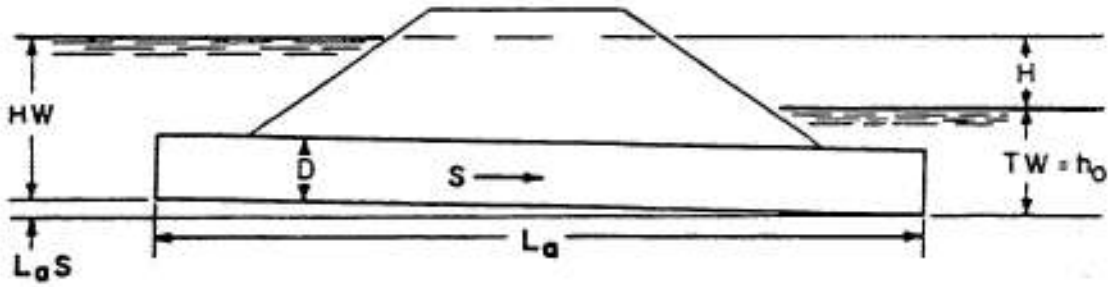


Fig. 1.14.4 - Determination of h_o for high tailwater (1).

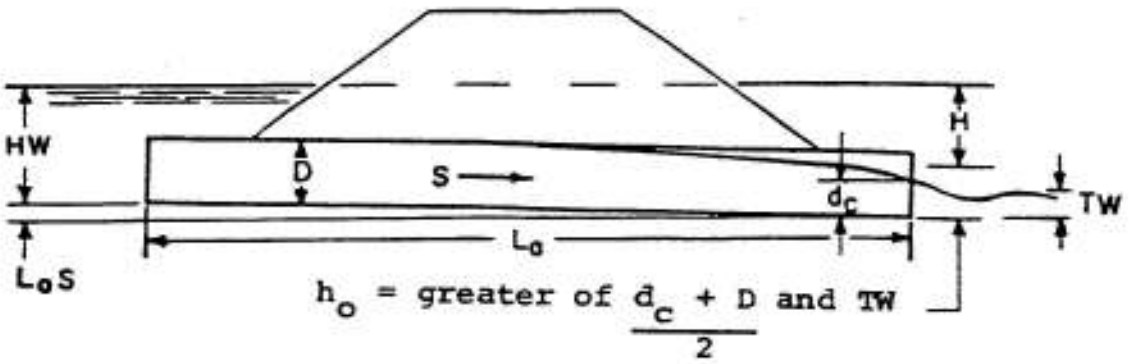
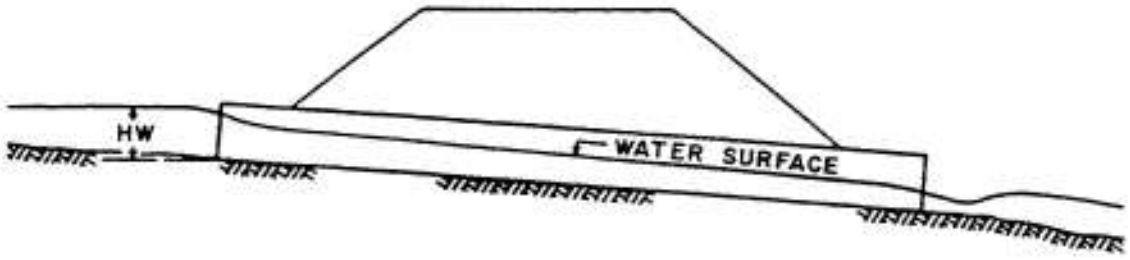
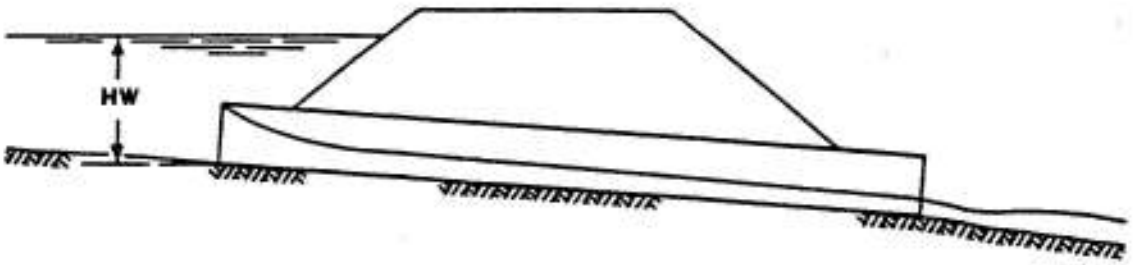


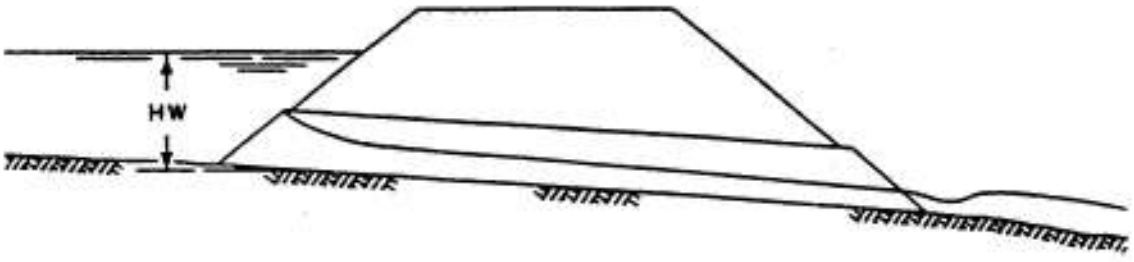
Fig. 1.14.5 - Determination of h_o for tailwater below top of opening (1).



a. Projecting end - unsubmerged inlet

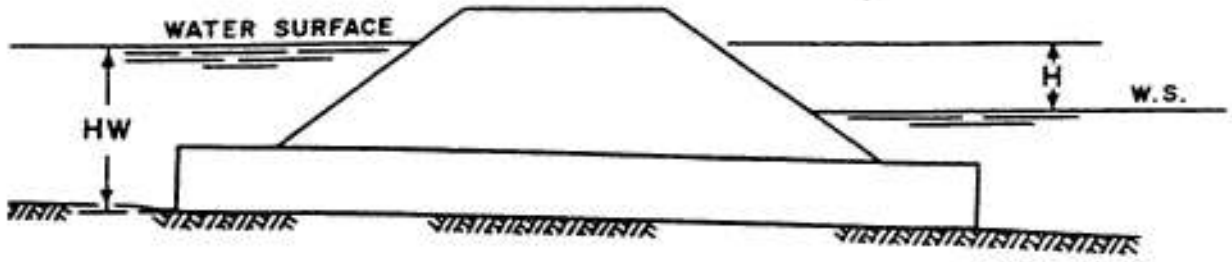


b. Projecting end - submerged inlet

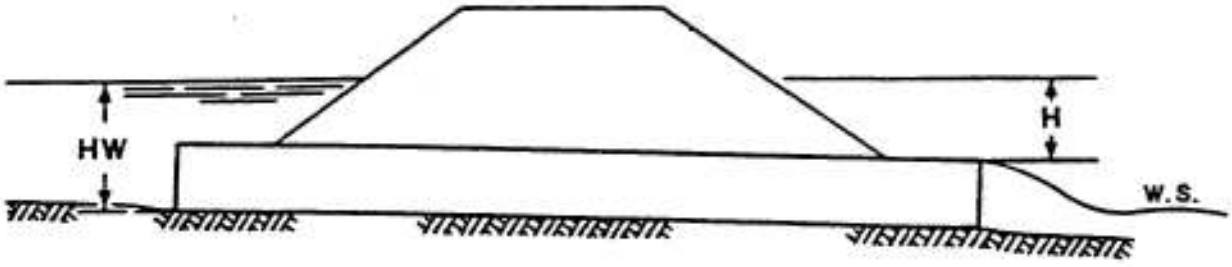


c. Mitered end - submerged inlet

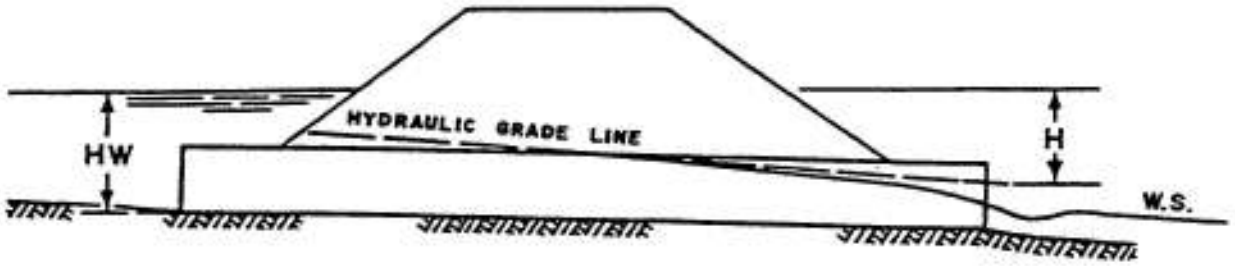
Fig. 1.14.6 - Inlet control (1).



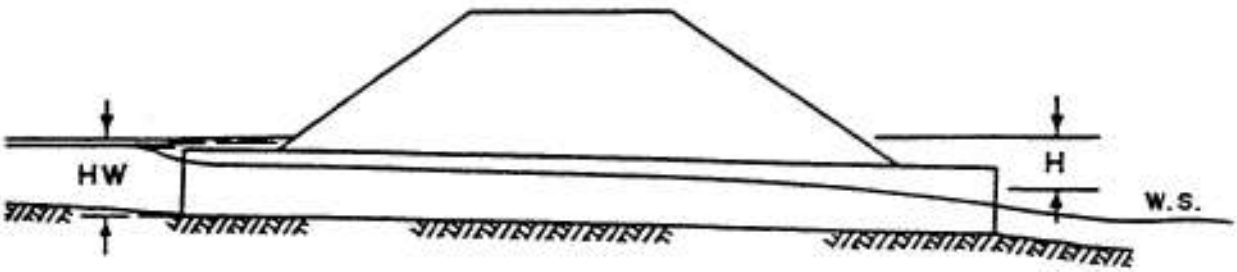
a. Culvert flowing full, submerged outlet.



b. Culvert flowing full, unsubmerged outlet.



c. Culvert flowing full for part of length.



d. Culvert not flowing full.

Fig. 1.14.7- Outlet control (1).

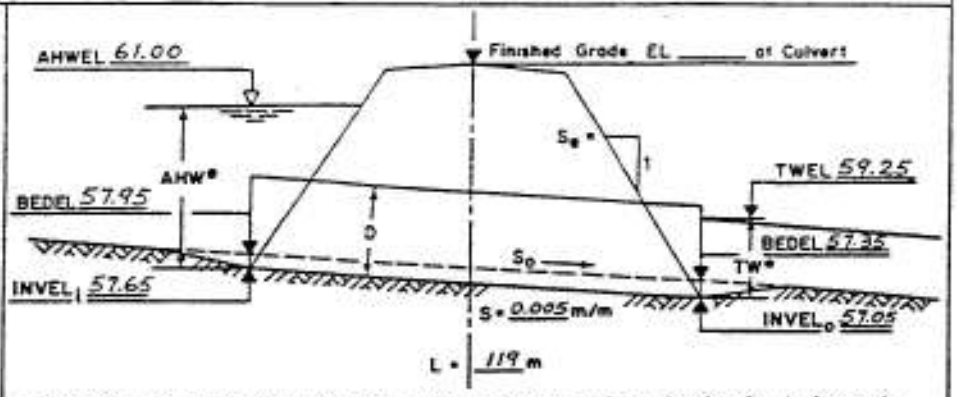
TABLE D4-3 - EXAMPLE OF CONVENTIONAL PIPE CULVERT DESIGN

M.I. No. _____ Culvert No. ___ Sta. _____ CONVENTIONAL CULVERT DESIGN Calc. By _____ Date _____ Check by _____ Date _____

Location _____

INITIAL DATA

Q_{50}	28.30	m³/s
TW*	2.20	m
AHW*	3.35	m
Nat. S_o	0.005	m/m
Nat. bed el. at outlet	57.35	m
Skew No.	90°	



* AHW, HW, & TW are measured from culvert invert unless depth of embedment is excessive.

	AREA PER BARREL	INLET CONTROL			OUTLET CONTROL							OUTLET VEL.
$\frac{Q}{n}$ (m³/s)	A (m²)	$\frac{Q}{NB}$ (m²/s/m)	$\frac{HW}{D}$ (m)	HW (m)	E (m)	d_c (m)	$\frac{d_c+D}{2}$ (m)	TW (m)	h_o	LS	HW (m)	V_o (m/s)
1	2	3	4	5	6	7	8	9	10	11	12	13
1st Trial, Descrip.: SPCSP, projecting, sharp edge			N 2		B —		D 2.74m		k_e 0.9			
14.15	5.91	—	1.02	2.79	1.67	1.68	2.21	2.20	2.21	0.60	3.28	—
Comments			(Chart D5-1G)		(Chart D5-2D)		(Chart D5-3B)		OK. but try CSP (lower roughness)			
2nd Trial, Descrip.: CSP, projecting, sharp edge			N 2		B —		D 2.70m		k_e 0.9			
14.15	5.73	—	1.04	2.81	1.40	1.67	2.19	2.20	2.20	0.60	3.00	2.8
Comments			(Chart D5-1G)		(Chart D5-2C)		(Chart D5-3B)		OK. Try smaller diam. CSP			
3rd Trial, Descrip.: CSP, projecting, sharp edge			N 2		B —		D 2.40m		k_e 0.9			
14.15	4.52	—	1.36	3.26	2.60	1.71	2.06	2.20	2.20	0.60	4.2	—
Comments			(Chart D5-1G)		(Chart D5-2C)		(Chart D5-3B)		Too small. Try single SPCSPA			
4th Trial, Descrip.: SPCSPA, projecting, sharp edge			N 1		B 4.72m		D 3.07m		k_e 0.9			
28.3	11.38	—	0.97	2.98	1.31	1.65	2.36	2.20	2.36	0.60	3.07	3.0
Comments			(Chart D5-1H)		(Chart D5-2E)		(Chart D5-3F)		OK.			
5th Trial, Descrip.: _____			N _____		B _____		D _____		k_e _____			
Comments												

Notes:
(3) Enter for box culvert only
(4) Charts D5-1A to J
(5) or (12) Circle controlling HW (i.e. the larger of cols. 5 & 12).
(6) Charts D5-2A to G
(7) Charts D5-3A to F (d_c & D)
(10) h_o = larger of cols. 8 & 9
(12) HW = H + h_o - LS

Fig. 1.14.8

CHART D1-1
CHART D1-2
CHART D2-1

CHART D1-1 - HAZARD PROTECTION DISTANCE WARRANTS (6)

Design Speed (km/h)	Unprotected clearance from edge of pavement (m)	
	Minimum for new highways, desirable for existing hwys.	Absolute minimum for existing highways
120 or more	10.0	5.0
110 to 119	9.0	4.5
100 to 109	7.0	4.0
90 to 99	6.0	3.0
80 to 89	5.0	2.5
70 to 79	4.0	2.0
60 to 69	3.0	1.5

CHART D1-2 - SUGGESTED CLEARANCES FOR PATHWAYS THROUGH CULVERTS

	Culvert span (m)	Min.horiz. clearance (m)	Min. vert. clearance (m)	Minimum freeboard from NWL (m)
Pedestrians	<3.0	1.0	2.2	0.3
	3.0 to 6.0	1.5	2.5	0.5
Cyclists (single lane)	3.0 to 6.0	1.5*	2.5	0.5

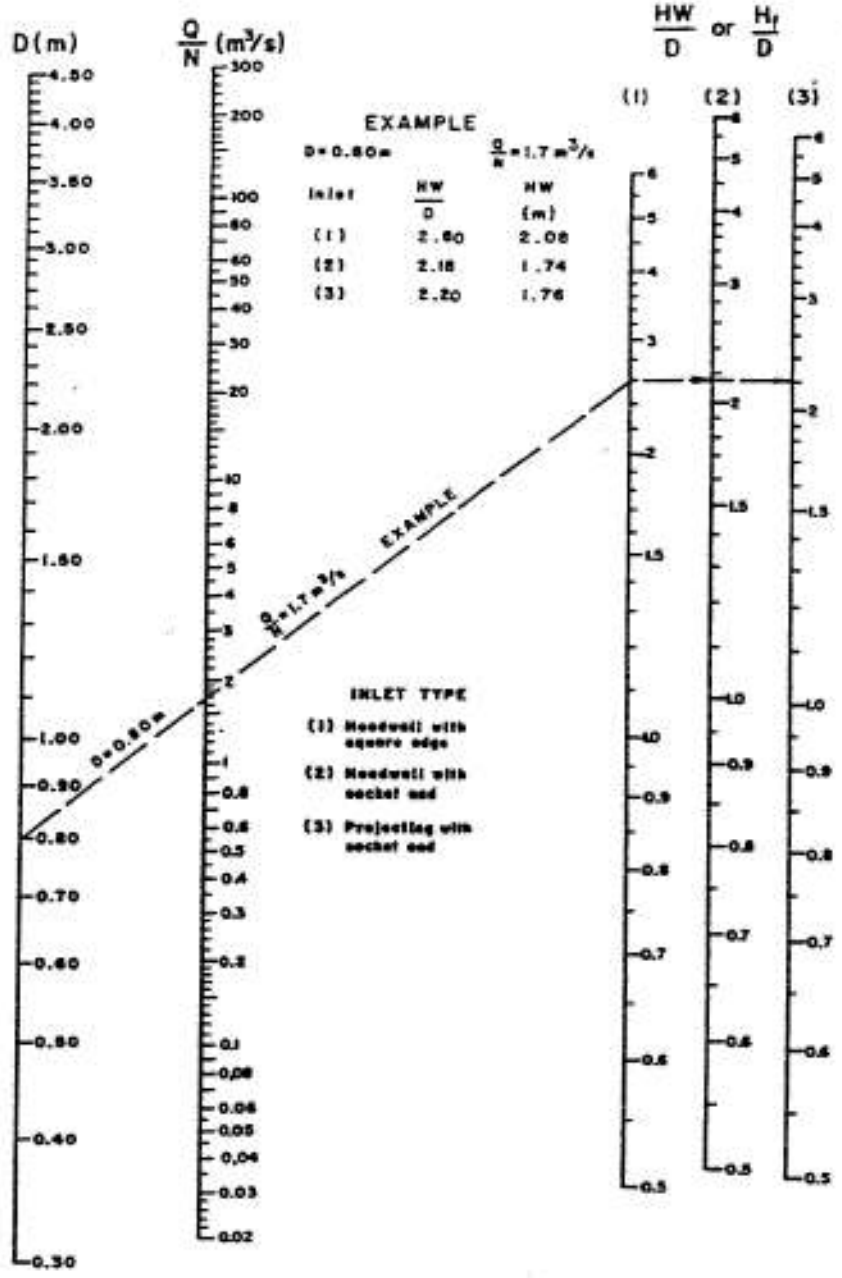
* assuming handrail on stream side.

CHART D2-1 - MINIMUM PIPE SIZES (44)

	Diameter (mm)
Provincial highways	800 or equivalent pipe arch
Other roads	600 or equivalent pipe arch
Private entrances	500 if more than 10 m length
Private entrances	400 if length 10 m or less

Fig. 1.14.9

CHART D5 - 1 E

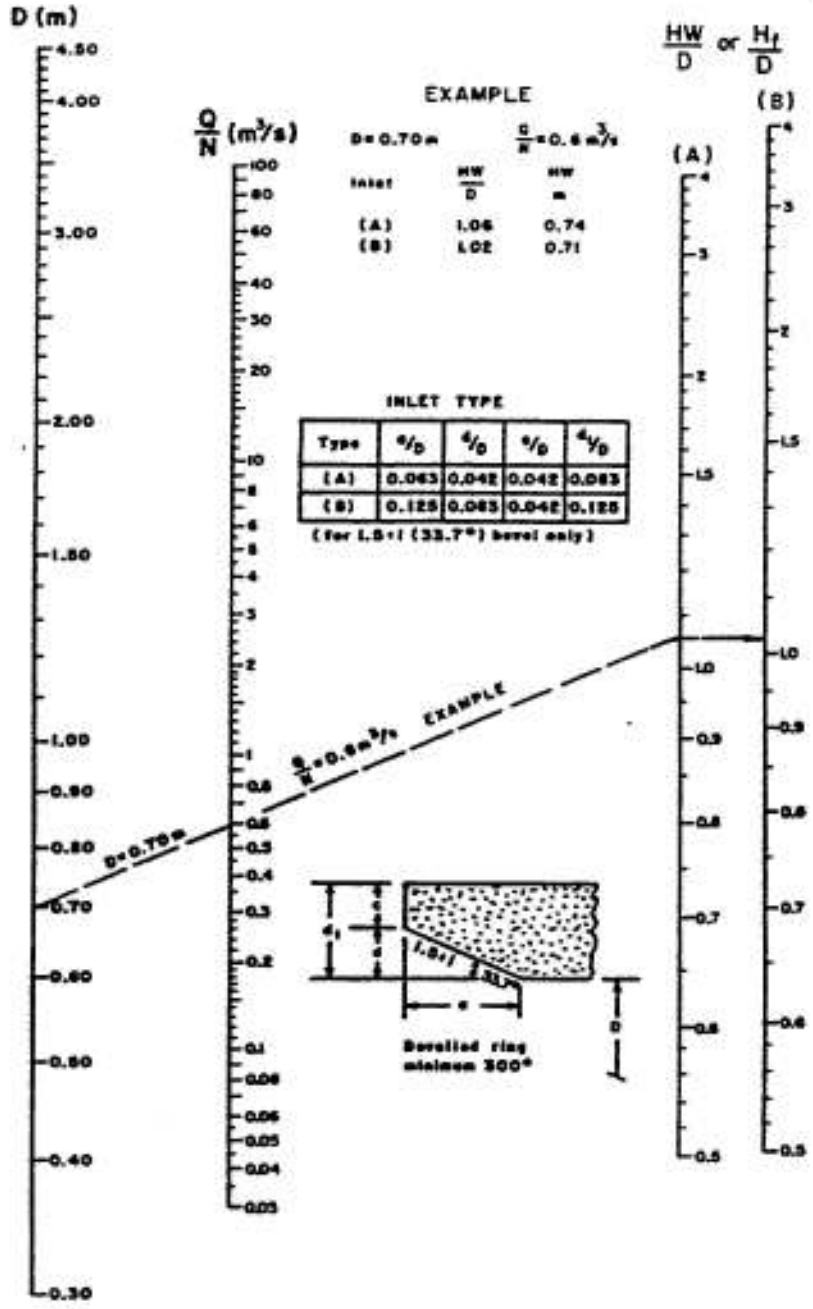




FHA 64 05
METRIC FHA 74 06 (81)

INLET CONTROL
CONCRETE PIPE CULVERT
Fig. 1.14.10

CHART D5 - 1F



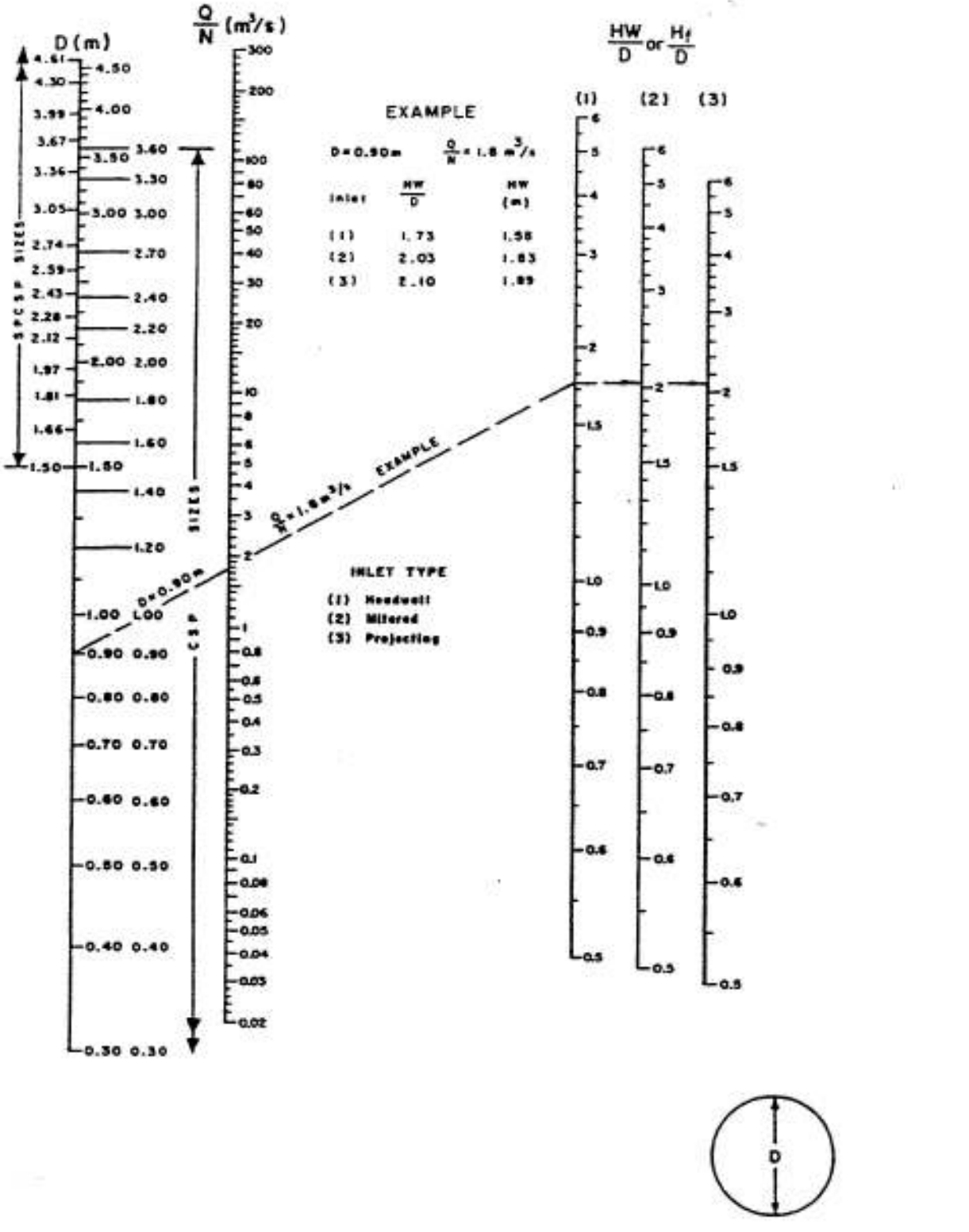


FHA 64 03
METRIC FHA 74 06(81)

INLET CONTROL
CIRCULAR PIPE CULVERT
WITH BEVELLED RING

Fig.1.14.11

CHART D5-1G

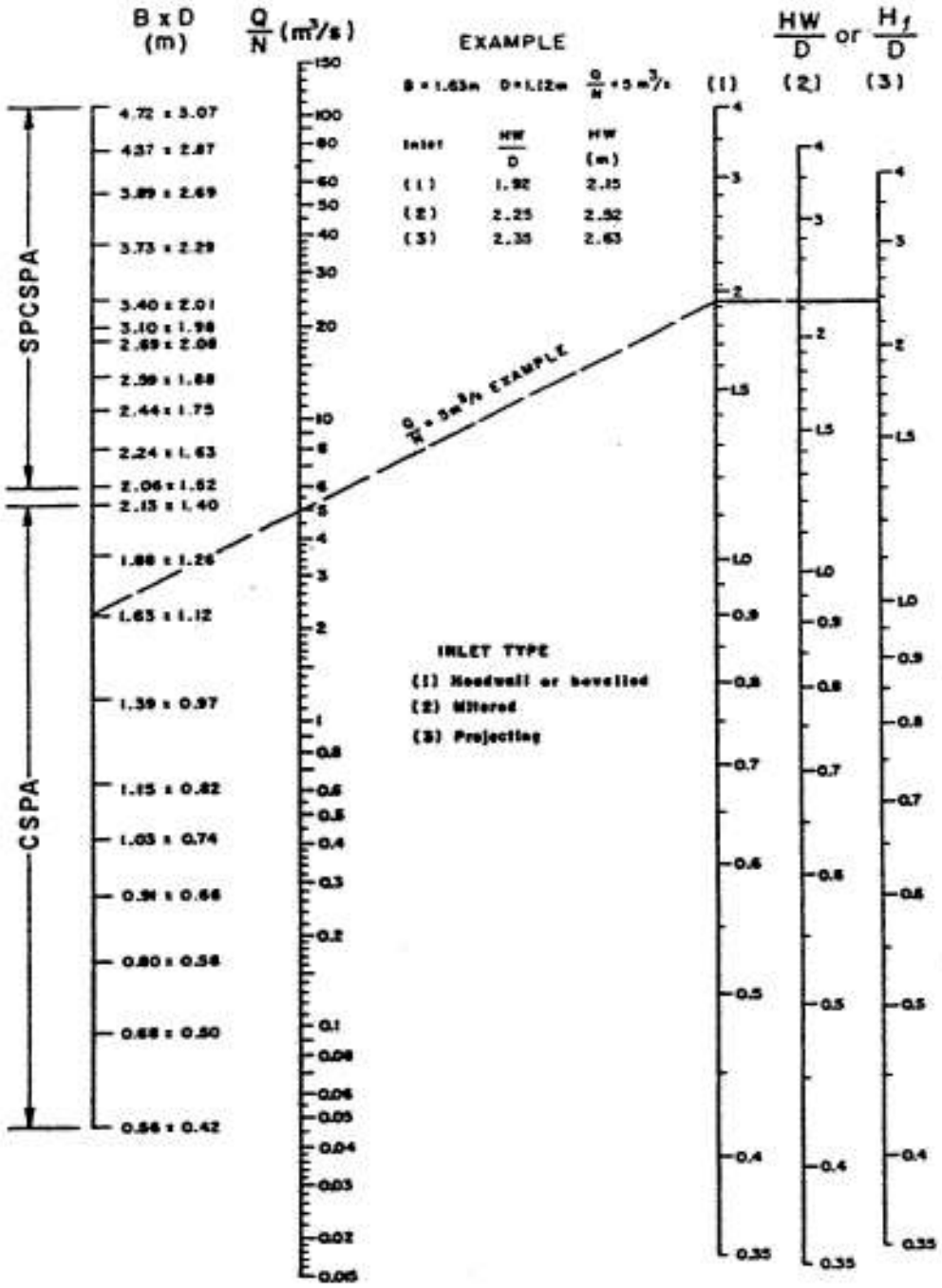


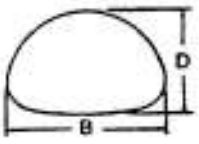
FHA 83 01
ETRIC FHA 74 06 (SI)

INLET CONTROL
CSP & SPCSP CULVERT

Fig.1.14.12

CHART D5 - IH



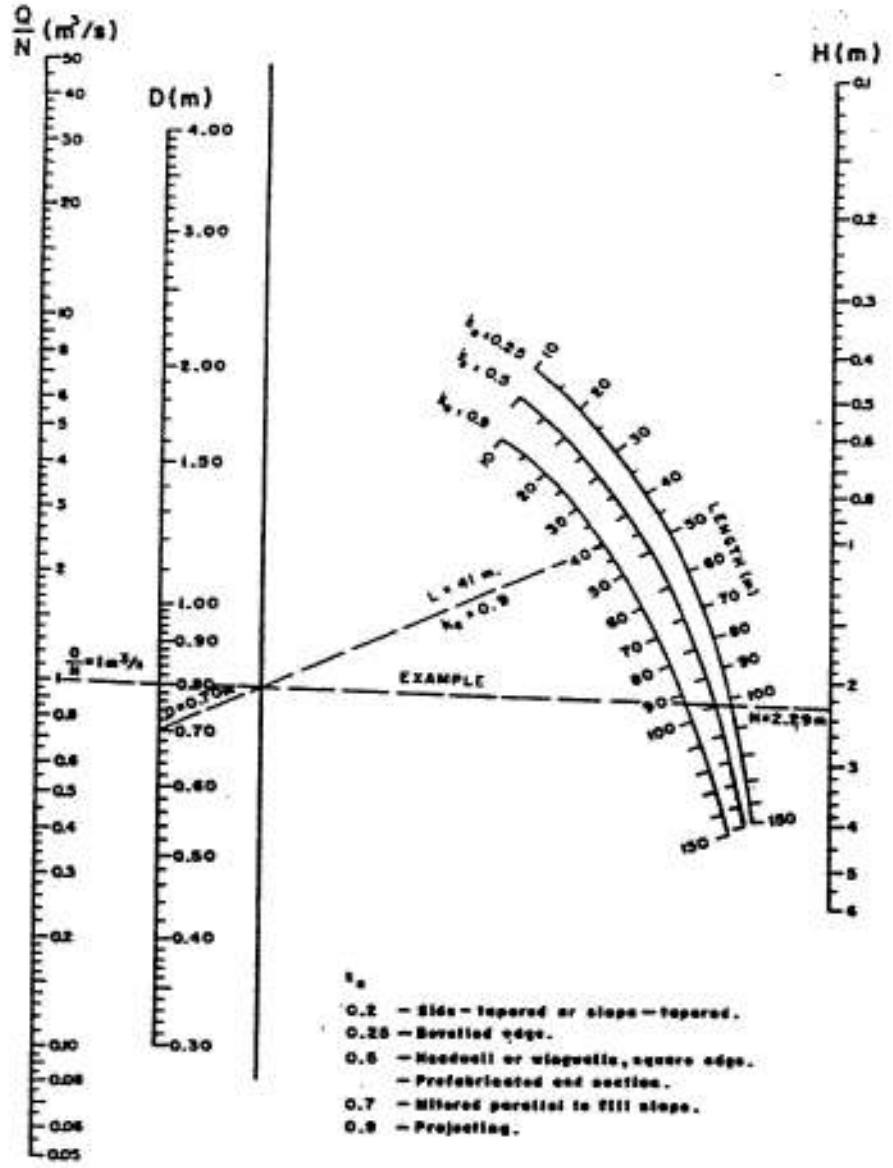


INLET CONTROL
CSPA & SPCSPA
CULVERT

Fig. 1.14.1

FHA 63 01
METRIC FHA 74 06 (61)

CHART D5-2C



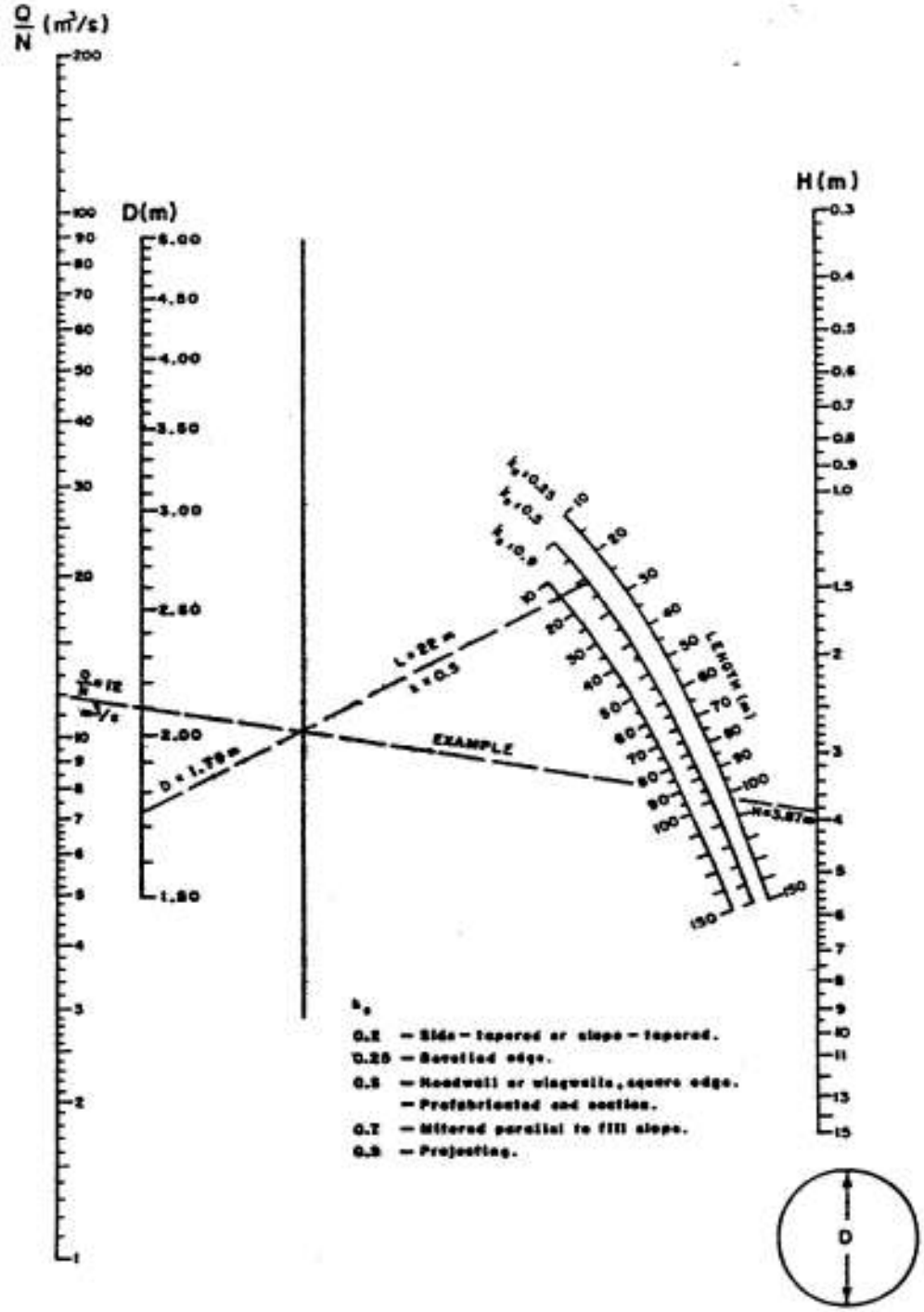


FHA 6301
METRIC FHA 74 08 (61)

OUTLET CONTROL
CORRUGATED STEEL PIPE
FLOWING FULL
n = 0.024

Fig. 1.14.14

CHART D5-2D

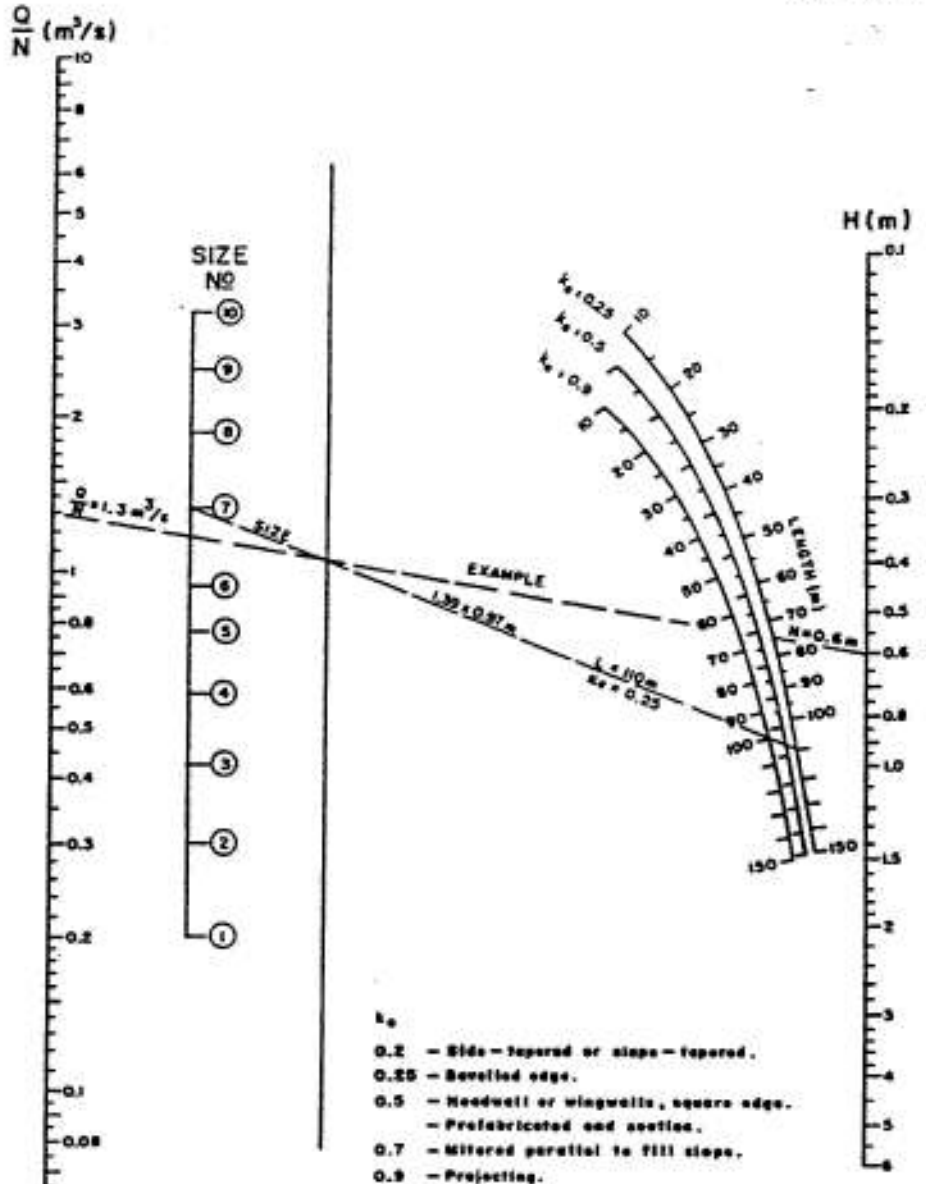


OUTLET CONTROL
STRUCTURAL PLATE
CORRUGATED STEEL PIPE
FLOWING FULL
n = 0.0328 TO 0.0302

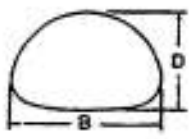
FHA 63 01
METRIC FHA 74 08 (SI)

Fig. 1.14.15

CHART D5-2E



CSPA SIZES	
Nº	B(m) x D(m)
1	0.56 x 0.42
2	0.68 x 0.50
3	0.80 x 0.58
4	0.91 x 0.66
5	1.03 x 0.74
6	1.15 x 0.82
7	1.39 x 0.97
8	1.63 x 1.12
9	1.88 x 1.26
10	2.13 x 1.40

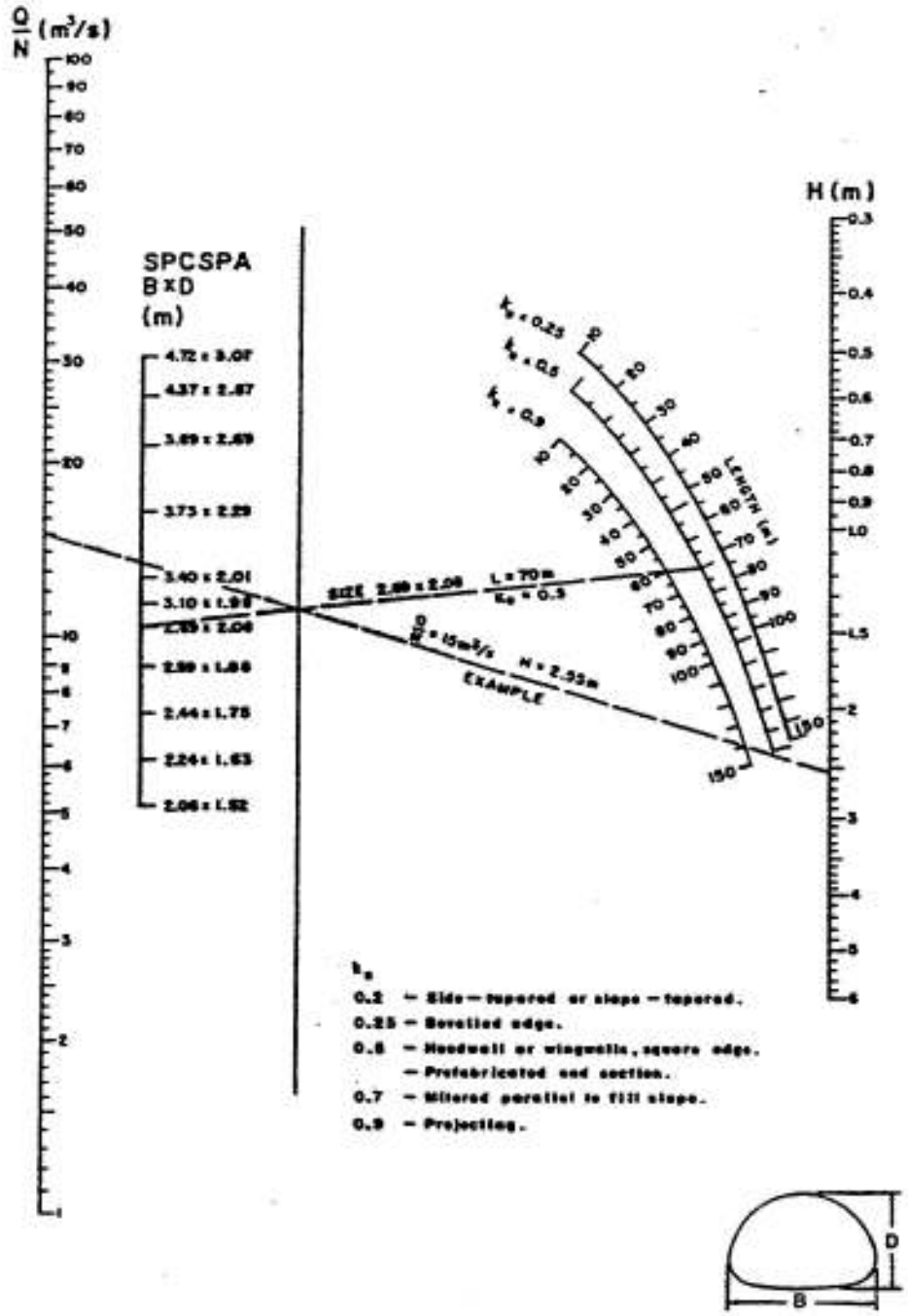


FHA 63 01
METRIC FHA 74 06 (51)

OUTLET CONTROL
CORRUGATED STEEL PIPE ARCH
FLOWING FULL
n=0.024

Fig. 1.14.16

CHART D5-2F

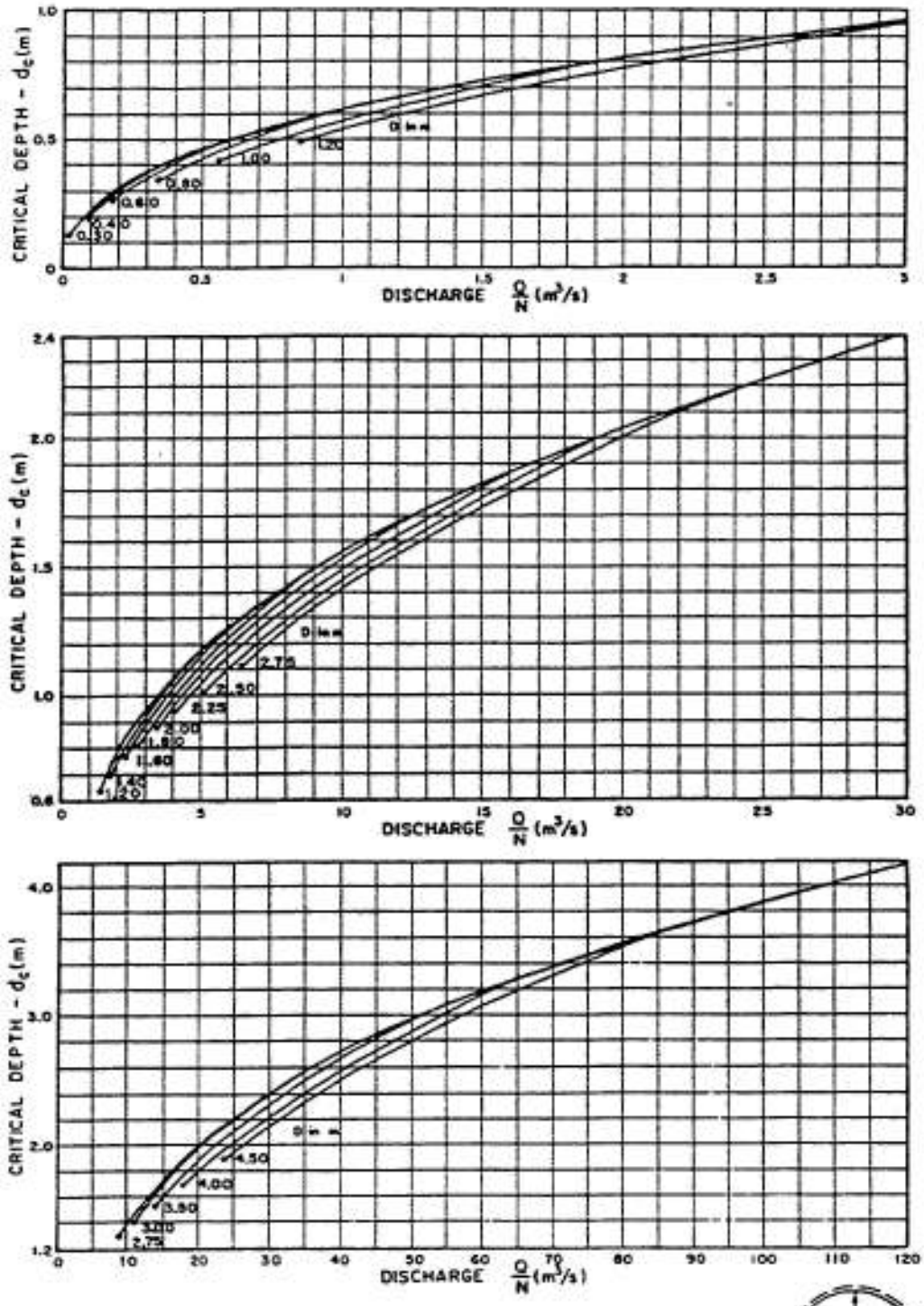


FNA 83 01
METRIC FNA 74 06 (81)

OUTLET CONTROL
STRUCTURAL PLATE
CORRUGATED STEEL PIPE ARCH
FLOWING FULL
n=0.0327 TO 0.0306

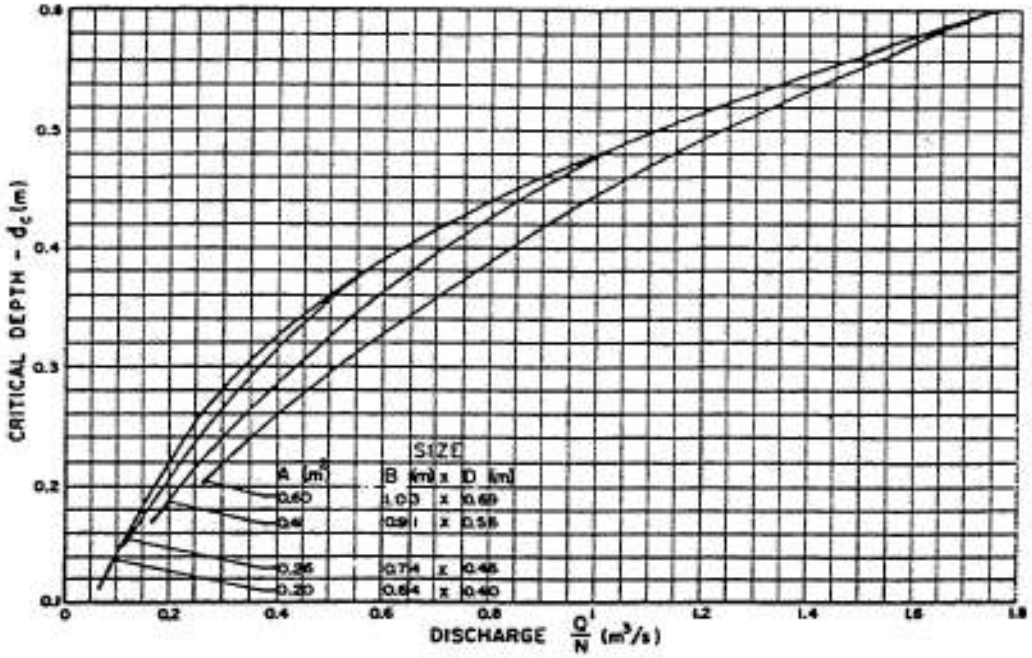
Fig. 1.14.17

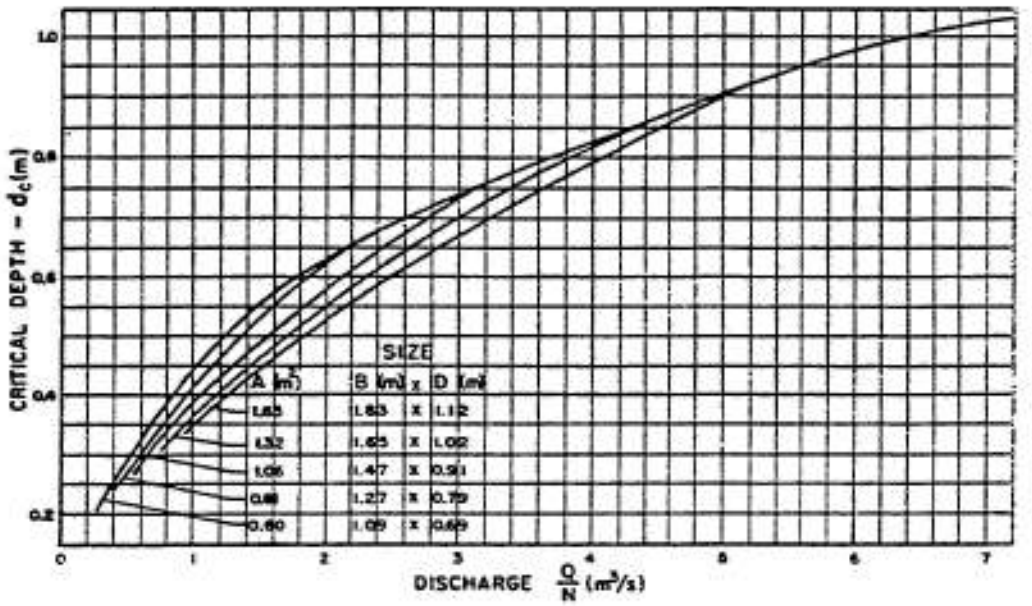
CHART D5-3B



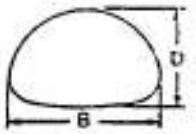
FHA 64 01
METRIC FHA 74 06 (51)

CRITICAL DEPTH
CIRCULAR PIPE
Fig. 1.14.18





($d_c \neq D$)



A = Cross-sectional area per barrel.

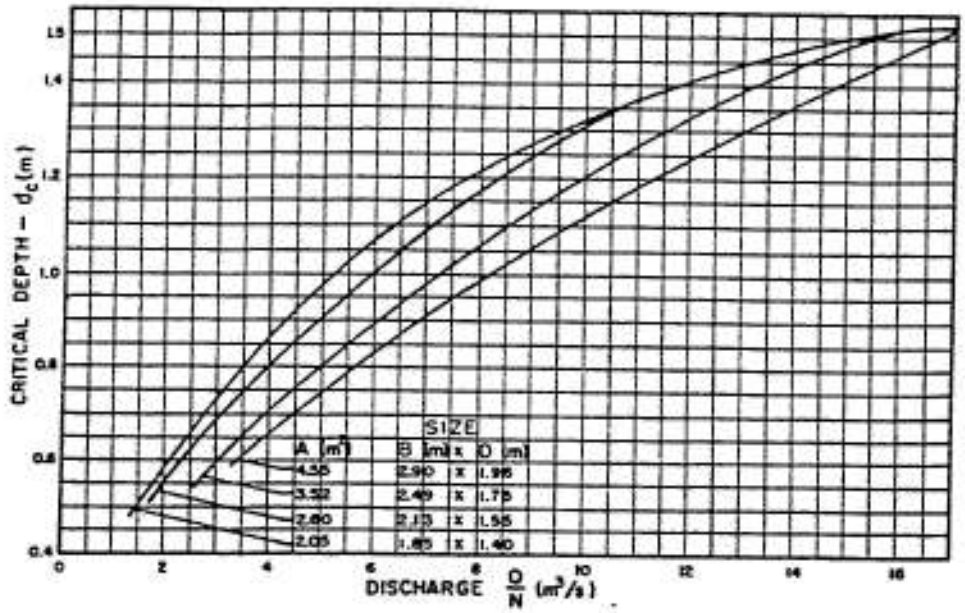
Other sizes to be interpolated

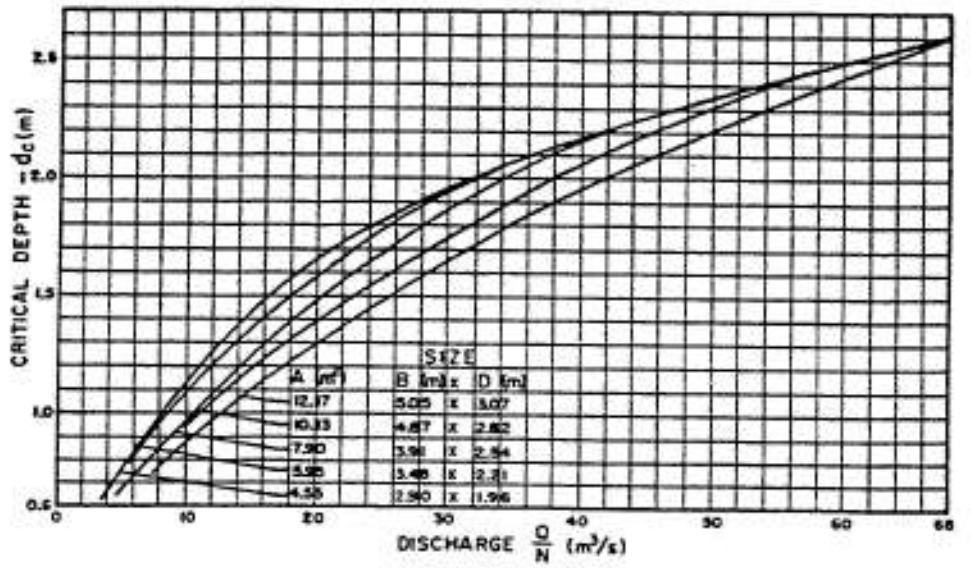
FHA 64 01
METRIC FHA 74 06 (51)

CRITICAL DEPTH
CORRUGATED STEEL PIPE ARCH

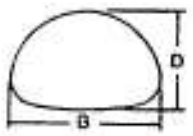
Fig. 1.14.19

CHART D5-3F





(d_c > D)



A = Cross-sectional area per barrel.

Other sizes to be interpolated

FHA 64 01
METRIC FHA 74 06 (SI)

CRITICAL DEPTH
STRUCTURAL PLATE
CORRUGATED STEEL PIPE ARCH

Fig. 1.14.20

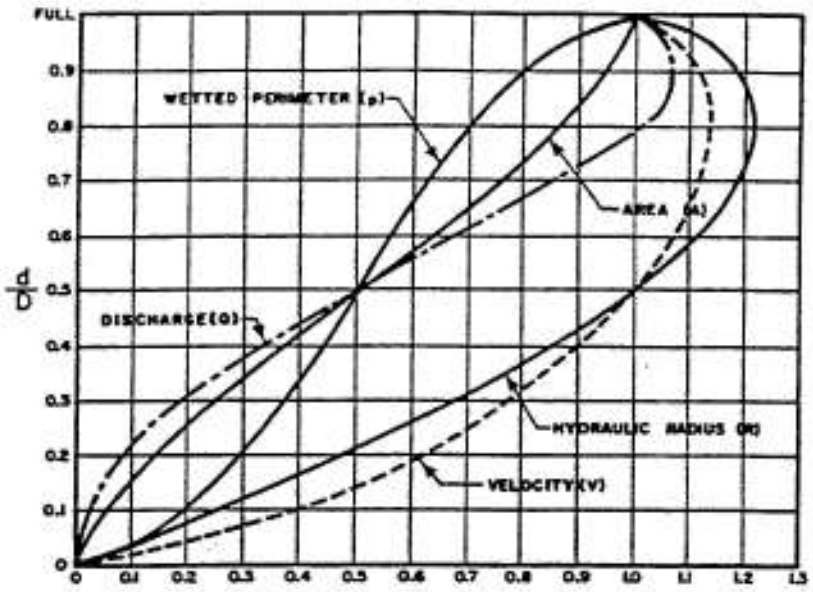
CHART D5-8. ENTRANCE LOSS COEFFICIENTS (2)

Coefficient k_e to apply to velocity head $V^2/2g$ for determination of head loss at entrance to a culvert operating under outlet control.

$$\text{Entrance head loss } H_e = k_e \frac{V^2}{2g}$$

Type of Barrel and Inlet	Coefficient k_e
Pipe, Concrete	
Projecting from fill, socket end	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe	0.2
Square-edge	0.5
Rounded (radius = 1/12D)	0.2
Mitered to conform to fill slope	0.7
End-Section conforming to fill slope (standard precast)	0.5
Bevelled edges, 33.7° or 45° bevels	0.2
Side-tapered or slope-tapered inlets	0.2
Pipe, or Pipe-Arch, Corrugated Steel	
Projecting from fill	0.9
Headwall or headwall and wingwalls, square edge	0.5
Mitered to conform to fill slope	0.7
End-Section conforming to fill slope (standard prefab)	0.5
Bevelled edges, 33.7° or 45° bevels	0.25
Side-tapered or slope-tapered inlets	0.2
Box, Reinforced Concrete	
Headwall	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension, or bevelled edges on 3 sides.	0.2
Wingwalls at 30° to 75° to barrel	
Square-edges at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or bevelled top edge.	0.2
Wingwalls at 10° to 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side-tapered or slope-tapered inlet	0.2
Projecting	
Square-edge	0.7
Bevelled edges, 33.7° or 45° bevels	0.2

CHART D5-9A
CHART D5-9B

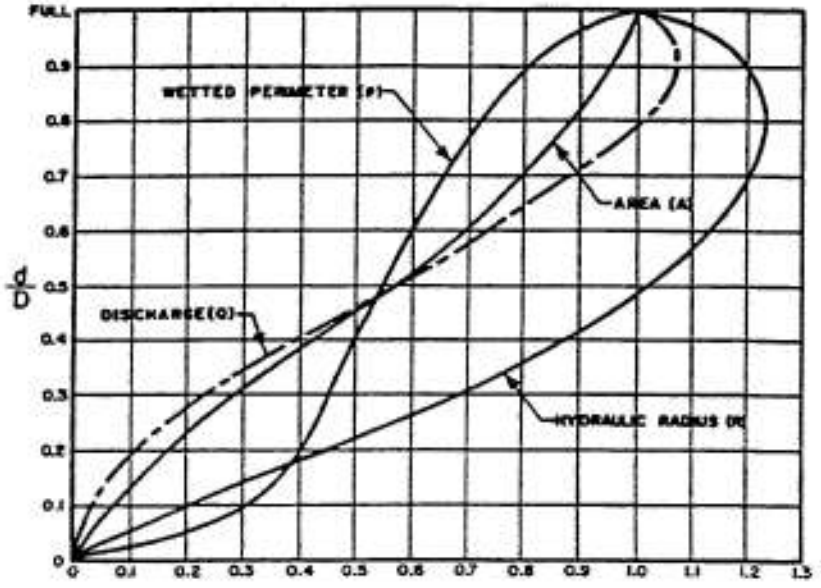


NOTE

OBTAIN FULL FLOW AREA AND PERIMETER OF CSP AND SPCSP FROM CHART D5-14B



Chart D5-9A - Hydraulic properties of pipes flowing partly full (45).



NOTE

OBTAIN FULL FLOW AREA AND PERIMETER FROM CHART D5-14C

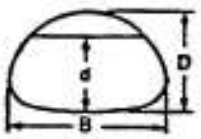


Chart D5-9B - Hydraulic properties of pipe arches flowing partly full (45).

Fig. 1.14.22

CHART D5 - II

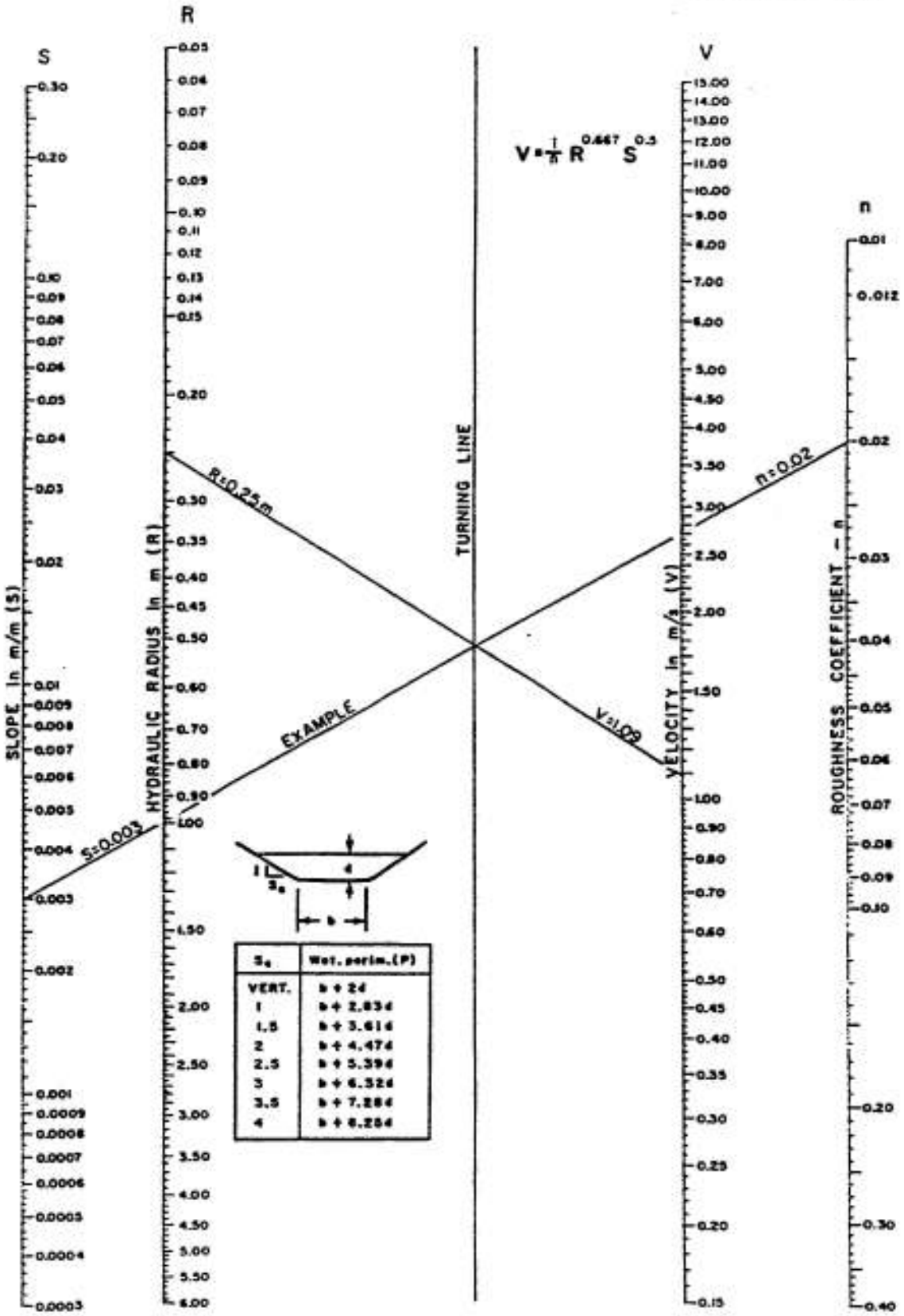


Chart D5-11 - Manning nomograph.(41) Fig. 1.14.23

CHART D5-12

Chart D5-12 - MANNING ROUGHNESS COEFFICIENTS FOR CULVERTS
(Based on references 1,47,48)
Increase values up to 15% for curved alignment.

		Manning 'n' range *
I	CLOSED CONDUITS	
	A. Concrete pipe and box	0.012
	B. Corrugated steel pipe or pipe arch	
	68 x 13 mm corrugation (riveted, annular)	
	Unpaved	0.024
	25% paved	0.021
	100% paved	0.012
	68 x 13 mm helical	
	Unpaved: 600 to 1525 mm Ø range:	0.016-0.024
	25% paved: " " " " " "	0.015-0.021
	100% paved: all sizes	0.012
	76 x 25 mm riveted (annular)	
	Unpaved	0.027
	25% paved	0.023
	100% paved	0.012
	76 x 25 mm helical	
	Unpaved: 900 to 1980 mm dia.:	0.021-0.027
	25% paved: " " " " "	0.019-0.023
	100% paved: all sizes	0.012
	152 x 51 mm corrugation (annular)	
	Unpaved 1550 - 4500 mm dia. or) 1900 to 5050 mm span)	0.033-0.030
	25% paved	0.026±
	C. Vitrified clay pipe	0.012-0.014
	D. Steel pipe	0.009-0.011
	E. Monolithic concrete:	
	1. Wood forms, rough	0.015-0.017
	2. Wood forms, smooth	0.012-0.014
	3. Steel forms	0.012-0.013
	F. Cemented rubble masonry walls:	
	1. Concrete floor and top	0.017-0.022
	2. Natural floor	0.019-0.025
	G. Laminated treated wood	0.015-0.017
	H. Smooth-wall plastic pipe	0.012

* Range is for good to fair construction unless otherwise stated.
For n values for channels and flood plains see Chapter C.

Fig. 1.14.24

CHART D5-13A
CHART D5-13B

CHART D5-13A - MAXIMUM PERMISSIBLE VELOCITIES IN ERODIBLE CHANNELS[1]

Material	Maximum permissible velocities for		
	Clear water	Water carrying fine silts	Water carrying sand and gravel
	m/s	m/s	m/s
Fine sand (noncolloidal)	0.46	0.76	0.46
Sandy loam (noncolloidal)	0.52	0.76	0.61
Silt loam (noncolloidal)	0.61	0.91	0.61
Ordinary firm loam	0.76	1.07	0.67
Volcanic ash	0.76	1.07	0.61
Fine gravel	0.76	1.52	1.13
Stiff clay (very colloidal)	1.13	1.52	0.91
Graded, loam to cobbles (noncolloidal)	1.13	1.52	1.52
Graded, silt to cobbles (colloidal)	1.22	1.68	1.52
Alluvial silts (noncolloidal)	0.61	1.07	0.61
Alluvial silts (colloidal)	1.13	1.52	0.91
Coarse gravel (noncolloidal)	1.22	1.83	1.98
Cobbles and Shingles	1.52	1.68	1.98
Shales and hard pans	1.83	1.83	1.52

[1]As recommended by Special Committee on Irrigation Research, American Society of Civil Engineers, 1926, for channels with straight alinement. For sinuous channels multiply allowable velocity by 0.95 for slightly sinuous, by 0.9 for moderately sinuous channels, and by 0.8 for highly sinuous channels.
Based on uniform flow in continuously wet channel (47).

CHART D5-13B - MAXIMUM PERMISSIBLE VELOCITIES IN GRASSED CHANNELS,[1,2] LINED WITH UNIFORM STANDS OF VARIOUS GRASS COVERS.

Cover	Slope range	Maximum permissible velocity on	
		Erosion-resistant soils	Easily eroded soils
	Percent	m/s	m/s
Bermudagrass	0-5	2.44	1.83
	5-10	2.13	1.52
	Over 10	1.83	1.22
Buffalograss, Kentucky bluegrass, Smooth brome, Blue grama	0-5	2.13	1.52
	5-10	1.83	1.22
	Over 10	1.52	0.91
Grass mixture	0-5[3]	1.52	1.22
	0-10[3]	1.22	0.91
Lespedeza sericea, Weeping lovegrass, Yellow bluestem, Kudzu, Alfalfa, Crabgrass	0-5[4]	1.07	0.76
Common lespedeza[5], Sudangrass[5]	0-5[4]	1.07	0.76

[1] From Handbook of Channel Design for Soil and Water Conservation.
[2] Use velocities over 1.5 m/s only where good covers and proper maintenance can be obtained.
[3] Do not use on slopes steeper than 10 percent.
[4] Use on slopes steeper than 5 percent is not recommended.
[5] Annuals, used on mild slopes or as temporary protection until permanent covers are established (47).

Fig. 1.14.25

Chart D5-14B - DIMENSIONS OF CORRUGATED STEEL PIPES

CSP				SPCSP			
D (m)	A (m^2)	P (m)	R (m)	D (m)	A (m^2)	P (m) *	R (m)
0.30	0.07	0.94	0.07	1.50	1.77	4.88	0.36
0.40	0.13	1.26	0.10	1.66	2.16	5.37	0.40
0.50	0.20	1.57	0.13	1.81	2.58	5.86	0.44
0.60	0.28	1.88	0.15	1.97	3.04	6.34	0.48
0.70	0.38	2.20	0.17	2.12	3.54	6.83	0.52
0.80	0.50	2.51	0.20	2.28	4.07	7.32	0.56
0.90	0.64	2.83	0.23	2.43	4.65	7.81	0.60
1.00	0.79	3.14	0.25	2.59	5.26	8.30	0.63
1.20	1.13	3.77	0.30	2.74	5.91	8.78	0.67
1.40	1.54	4.40	0.35	3.05	7.32	9.76	0.75
1.60	2.01	5.03	0.40	3.36	8.89	10.74	0.83
1.80	2.54	5.65	0.45	3.67	10.61	11.71	0.91
2.00	3.14	6.28	0.50	3.99	12.47	12.69	0.98
2.20	3.80	6.91	0.55	4.30	14.49	13.66	1.06
2.40	4.52	7.54	0.60	4.61	16.66	14.64	1.14
2.70	5.73	8.48	0.68	** 4.92	18.99	15.62	1.22
3.00	7.07	9.42	0.75	5.23	21.46	16.59	1.29
3.30	8.55	10.37	0.82	5.54	24.08	17.57	1.37
3.60	10.18	11.31	0.90	5.85	26.86	18.54	1.45
				6.16	29.79	19.52	1.53
				6.47	32.87	20.50	1.60
				6.78	36.10	21.47	1.68
				7.09	39.48	22.45	1.76
				7.40	43.01	23.42	1.84
				7.71	46.70	24.40	1.91
				8.02	50.53	25.38	1.99

Note: dimensions are shown in metres for direct use in calculations.

Source: MTC Standard DD-812-A dated 79 10 01 (columns D and A only).

* Based on manufacturers' periphery hole spaces.
** Limit of nomographs.



Fig. 1.14.26

Chart D5-14C - DIMENSIONS OF CORRUGATED STEEL PIPE ARCHES

CSPA					SPCSPA				
B (m)	D (m)	A (m^2)	p (m) *	R (m)	B (m)	D (m)	A (m^2)	p (m) +	R (m)
0.56	0.42	0.19	1.57	0.12	2.06	1.52	2.49	5.86	0.42
0.68	0.50	0.27	1.88	0.14	2.24	1.63	2.90	6.34	0.46
0.80	0.58	0.37	2.20	0.17	2.44	1.75	3.36	6.83	0.49
0.91	0.66	0.48	2.51	0.19	2.59	1.88	3.87	7.32	0.53
1.03	0.74	0.61	2.83	0.22	2.69	2.08	4.49	7.81	0.57
1.15	0.82	0.74	3.14	0.24	3.10	1.98	4.83	8.30	0.58
1.39	0.97	1.06	3.77	0.28	3.40	2.01	5.28	8.78	0.60
1.63	1.12	1.44	4.40	0.33	3.73	2.29	6.61	9.76	0.68
1.88	1.26	1.87	5.03	0.37	3.89	2.69	8.29	10.74	0.77
2.13	1.40	2.36	5.65	0.42	4.37	2.87	9.76	11.71	0.83
					4.72	3.07	11.38	12.69	0.90
				**	5.05	3.33	13.24	13.66	0.97
					5.49	3.53	15.10	14.64	1.03
					5.89	3.71	17.07	15.62	1.09
					6.25	3.91	19.18	16.59	1.16
					7.04	4.06	22.48	18.06	1.24
					7.62	4.24	25.27	19.28	1.31

Note: dimensions are shown in metres for direct use in calculations.

* Based on equivalent round diameter.

Source: MTC Standard DD-812B.

\+ Based on manufacturers' periphery hole spaces.

** Limit of nomographs.

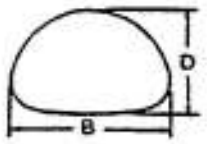


Fig. 1.14.27

CHART D8-3

CHART D8-3 - ALLOWABLE MEAN VELOCITIES FOR COHESIVE SOILS*

Depth of flow	Soil Erodibility**		
	High	Medium	Low
m	m/s	m/s	m/s
1.0	0.5	0.9	1.6
1.5	0.6	1.0	1.8
3.0	0.6	1.2	2.0
6.0	0.7	1.3	2.3
15.0	0.8	1.5	2.6

Above table is based on RTAC Guide to Bridge Hydraulics (49).

* Allowable velocities should be based on local experience whenever possible, taking into account saturation and weathering of the soil.

** It is not considered advisable to relate the tabulated values to soil property indices because of the strong effect of saturation and weathering on the scourability of cohesive soils. However the following tentative relationship to soil consistency is offered as a rough guide.

High erodibility . . . . very soft to soft clays
Moderate erodibility . . firm to stiff clays
Low erodibility . . . . stiff to hard clays; some glacial tills.

Soil consistency can be judged by the following field test applied with the soil at or near its natural water content (50).

Very soft - easily penetrated several centimetres by fist.
Soft - easily penetrated several centimetres by thumb.
Firm - moderate effort for thumb to penetrate several centimetres.
Stiff - readily indented by thumb, but penetrated only by great effort.
Very stiff - readily indented by thumbnail.
Hard - indented with difficulty by thumbnail.

Fig. 1.14.28

FORM D2 - 1

FORM D2-1 - FIELD INVESTIGATION SHEET FOR LARGE CULVERTS

REG./DIST._____ HWY NO. _____ WP NO. _____

STATION _____ INSP. BY. _____ DATE -_____

EXISTING CULVERTS (At site and upstream/downstream crossings)

Location			
Description			
Dimensions			
Year constr. (±)			
*Flood rise			
*Relief flow			
Nat. bed material			
Degradation			
Artif. deepening			
Other problems			
Est. adequacy			

PROPOSED CROSSING (Provide sketch)

Properties affected _____

Flood rise _____

Natural bed material _____

Bed degradation _____

Artificial deepening _____

Other problems _____

*Fisheries potential _____

Remarks _____

_____

*Obtain information from long-term residents(s)/patrolmen (10-15 yrs+).

Name & address of informant _____

_____

Period of knowledge_____Phone No._____ _____

Remarks_____

_____

_____

_____

Fig. 1.14.29

FORM D4-1

W.F. No. _____ Culvert No. ___ Sta. _____

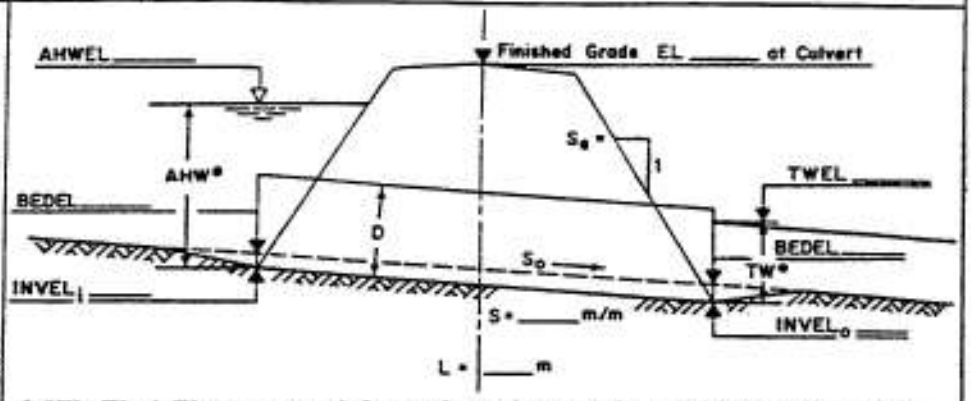
CONVENTIONAL CULVERT DESIGN

Calc. by _____ Date _____
Chkd.by _____ Date _____

Location _____

INITIAL DATA

Q	_____	m^3/s
TW*	_____	m
AHW*	_____	m
Nat. S_o	_____	m/m
Nat. bed el. at outlet	_____	m
No.	_____	



* AHW, HW, & TW are measured from culvert invert unless depth of embedment is excessive.

		INLET CONTROL			OUTLET CONTROL							OUTLET VEL.
$\frac{Q}{n}$ (m^3/s)	AREA PER BARREL A (m^2)	$\frac{Q}{nB}$ ($m^2/s/m$)	$\frac{HW}{D}$ (m)	HW (m)	H (m)	d_c (m)	$\frac{d_c+D}{2}$ (m)	TW (m)	h_o	LS	HW (m)	V_o (m/s)
1	2	3	4	5	6	7	8	9	10	11	12	13
1st Trial, Descrip.: _____ N _____ B _____ D _____ k_e _____												
Comments												
2nd Trial, Descrip.: _____ N _____ B _____ D _____ k_e _____												
Comments												
Trial, Descrip.: _____ N _____ B _____ D _____ k_e _____												
Comments												
4th Trial, Descrip.: _____ N _____ B _____ D _____ k_e _____												
Comments												
5th Trial, Descrip.: _____ N _____ B _____ D _____ k_e _____												
Comments												

Notes:
(3) Enter for box culvert only
(4) Charts D5-1A to J
(5) or (12) Circle controlling HW (i.e. the larger of cols. 5 & 12).
(6) Charts D5-2A to G
(7) Charts D5-3A to F (d_c & D)
(10) h_o = larger of cols. 8 & 9
(12) HW = H + h_o - LS

Fig. 1.14.30

FORM D4-2

FORM D4-2 - CULVERT VELOCITY CALCULATION FOR INLET CONTROL

Trial (m)	$\frac{d^*}{D}$	$\frac{A_o}{A}^*$	A_o (m^2)	$\frac{p_o}{p}^*$	p_o (m)	R_o (m)	V_o (m/s)	Q (m^3/s)	Remarks
1	2	3	4	5	6	7	8	9	10

* not required for rectangular culverts.

Col. 2 D = height of culvert.

3 From Chart D5-9A and B for circular and pipe arch culverts respectively.
A_o = partly-full area.
A = full area (Charts D5-14B or C for CS culverts).

4 A_o = col. 3 x A for non-rectangular culverts, B x d for rectangular.

5 From Chart D5-9A and B for circular or pipe arch culverts respectively.
p_o = partly-full wetted perimeter.
p = full wetted perimeter (Charts D5-14B or C for CS culverts).

6 p_o = col. 5 x p for non-rectangular, B + 2d for rectangular.

7 R_o = col. 4 ÷ col. 6.

8 $V_o = \frac{1}{n} R_o^{0.667} S^{0.5}$ (or use Chart D5-11).
Obtain n from Chart D5-12.

9 $Q = A_o V$ = col. 4 x col. 8.

Fig. 1.14.31

FORM D6-1A

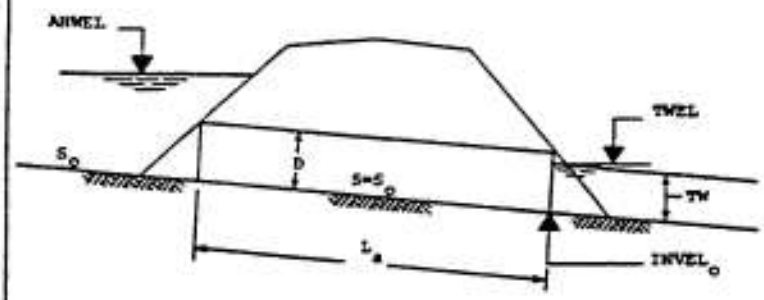
WP No. _____ Calc. by _____ Date _____

Culvert No. ___ Sta. _____ IMPROVED INLET DESIGN OUTLET CONTROL Chkd. by _____ Date _____

Location _____

INITIAL DATA

Q	_____	m^3/s
AHWEL	_____	m
S_o	_____	m/m
L_a	_____	m
$INVEL_o$	_____	m
TWEL	_____	m



L_a includes inlet but excludes wingwalls.

C	Q/N	A,E or BxD	E	d_c	$\frac{d_c + D}{2}$	TW	h_o	$INVEL_o$	$HWEL_o$	V_o	COMMENTS
1	2	3	4	5	6	7	8	9	10	11	
Trial No. ___ Descr. _____ k_e ___ N ___ B ___ D ___											
Trial No. ___ Descr. _____ k_e ___ N ___ B ___ D ___											
Trial No. ___ Descr. _____ k_e ___ N ___ B ___ D ___											

NOTES

(3) A per barrel for box sections, D for circular, BxD for other.

(4) From Charts D5-2A to G.

(5) From Charts D5-3A to F. ($d_c \div D$).

(7) TW = TWEL - $INVEL_o$.

(8) h_o = the larger of cols. 6 and 7.

(10) $HWEL_o$ = cols. 4 + 8 + 9.

(11) V_o = Q/Area defined by d_c or TW whichever is higher. If TW (or d_c) > D use full area. Defer calculation until control is known.

Fig. 1.14.32

1982 05 06

FORM D6-1B

WP No. _____

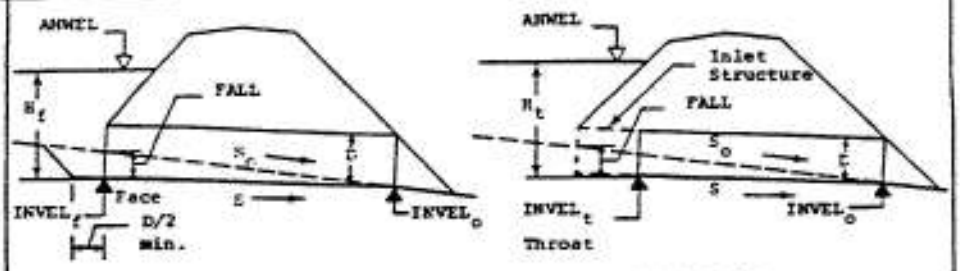
Culvert No. ___ Sta. _____

IMPROVED INLET DESIGN
INLET CONTROL

INITIAL DATA

Q	_____	m^3/s
AHWEL	_____	m
S_o	_____	m/m
L_a	_____	m

Nat. Stream Bed Elevation at Face _____ m



CONVENTIONAL INLET:
FACE CONTROL SECTION
(Upper Headings)

TAPERED INLET:
THROAT CONTROL SECTION
(Lower Headings)

DEFINITIONS OF INLET CONTROL SECTION

Q	Q/N or Q/NB $\frac{1.811C}{NBD^{1.5}}$	$\frac{H_f}{D}$ $\frac{H_t}{D}$	H_f H_t	$INVEL_f$ $INVEL_t$	Nat. Stream Bed El. at Face	FALL	$HWEL_f$ $HWEL_t$	S (m/m)	V_o	Note: Use upper headings for conventional inlet and lower headings for tapered inlet(throat control). COMMENTS
1	2	3	4	5	6	7	8	9	10	
Trial No. ___ Descr. _____ N_____ B_____ D_____										
Trial No. ___ Descr. _____ N_____ B_____ D_____										
Trial No. ___ Descr. _____ N_____ B_____ D_____										

NOTES

(2) Upper: (a) Use Q/N for non-rectangular sections.
(b) Use Q/NB for rectangular sections.
Lower: "B" = "D" for circular pipe.

(3) From Charts D5-1A to J (upper), D5-4A or B (lower).

(4) H_f or H_t = D x col.3.

(5) $INVEL_f$ = AHWEL - H_f. ($INVEL_f$ ≠ col.6).
$INVEL_t$ = AHWEL - H_t. ($INVEL_t$ ≠ nat. bed el. at throat). (See also Step 7(5)(c) re perform. curves).

(7) FALL = col.6 - col.5.

(8) $HWEL_{f(or\ t)}$ = col.4 + col.5.

(9) $S = S_o$ = FALL/L_a.

(10) V_o to be based on Manning formula applied to flow within the culvert. (See Step 5(4)).

Fig. 1.14.33

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2.

DEVELOPMENT POLICIES and PROCEDURES

CHAPTER 2 – DEVELOPMENT POLICIES & PROCEDURES

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CHAPTER 2 - DEVELOPMENT POLICIES & PROCEDURES

INTRODUCTION

This Chapter of the Municipal Works Design Manual deals with policies and procedures required to bring about new development or redevelopment within a municipality. It does not deal with technical details relative to sewers, roads, watermains, etc., which have been covered in other chapters of this manual.

Proper zoning or rezoning is a prerequisite to any development or redevelop- and this matter is usually dealt with by Municipal Council and staff.

Large scale development is carried out by means of a registered plan of subdivision and requires local, regional and/or provincial approvals. Small scale development or redevelopment can be done through Committee of Adjustment or Land Division Committees.

It is general policy that developers install or pay for all municipal services required within their development, as specified by the municipalities. e.g. paved roads, curbs, sidewalks, sewers, etc.. External services required in connection with the development of a parcel of land may be provided by the municipality and paid out of development levies, or financed jointly by the municipality and the developer or by a group of developers who would benefit, although policies and local circumstances vary considerably in this respect. Oversizing of municipal services required as a result of other developments, may be paid for by the municipality.

It should be realized that development and redevelopment procedures have a local character, and vary considerably from municipality to municipality and the following material should be considered in that context only.

2.1 OFFICIAL PLANS AND ZONING BY-LAWS

2.1.0 General

All Planning, Zoning and Development in the Province of Ontario is guided by The Planning Act, available from the Queen's Printer of Toronto.

The Act deals with the appointments of Planning Boards, Committees of Adjustment, Land Division Committees, the preparation of Official Plans, Official Plan Amendments, Subdivisions, Zoning and Building by-laws and appeal procedures to the Ontario Municipal Board.

Decisions made by local Councils affecting the use or development of land may be appealed to the Ontario Municipal Board, established by provincial legislature under the Ontario Municipal Board Act and then on to Cabinet of the Province.

This section will elaborate on Official Plans, Official Plan Amendments, Zoning by-laws and Zoning by-law Amendments.

2.1.1 Official Plans

An Official Plan of an area or Regional municipality is a document containing objectives and policies to guide the future growth and development of the municipality. An Official Plan may deal with a wide range of policies relating to such matters as land use, transportation, urban renewal, recreation, urban design, economics and servicing areas.

The intention of an Official Plan is to help deal with the physical development of a community while having regard to relevant social, economic and environmental matters. The implementation of an Official Plan is tempered by the ability of the community to pay for the municipal services required.

An Official Plan document consists of text and maps. An Official Plan may be prepared by a Planning Board but it must be adopted by the municipal council and be approved by the Ministry of Municipal Affairs and Housing or its delegated authority. Once approved, it becomes binding on public authorities. All public works proposed by municipal council, school boards, public utility commissions or other local authorities must conform to the Official Plan. No Council in the Planning Area may pass a by-law that conflicts with the Official Plan. In essence, the Official Plans can be regarded as fundamental planning legislature.

2.1.2 Official Plan Amendments

Official Plans should be reviewed periodically to ensure that the policies and the basis of the policies set out are still valid. Desired changes to the Official Plan can be made by an Amendment.

2.1.2 Official Plan Amendments Cont'd

Amendments can be initiated by citizens, agencies of council such as Planning Boards or Committees, or Municipal Councils directly. Amendments go from municipal council to the Minister or its delegated authority, either with a recommendation for the Amendment or a recommendation opposing the Amendment. The Minister can be asked by the parties effected to refer the matter to the Ontario Municipal Board.

2.1.3 Zoning By-laws

A Zoning By-law is a legal instrument for implementing the general policies of the Official Plan. Zoning By-laws deal specifically with the manner in which land is to be used. A Zoning By-law is considerably more detailed than the Official Plan. Generally, a Zoning by-law specifies the uses and standards for individual or tracts of land the Council considers to be desirable for the community. This will include types of buildings permitted, height of buildings, size of lots and a host of other specific items.

The Zoning by-law when prepared and passed by the Council and approved by the Ontario Municipal Board, or under Ontario Regulation 785, enables that municipality to guide development into specific patterns that are within the general policies of the Official Plan.

2.1.4 Zoning By-law Amendments

Any Amendment to a municipal zoning by-law is processed through Council and may be submitted to the Ontario Municipal Board for approval or processed under the provisions of Ontario Regulation 785.

A minor variance that does not alter the scope of the By-law and involves individual parcels of property can be dealt with by a Committee of Adjustment.

2.2 COMMITTEE OF ADJUSTMENT OR LAND DIVISION COMMITTEES

2.2.1 Introduction

Committees of Adjustments and more recently, Land Division Committees are established by local or regional governments to consider requests of property owners to sever a portion of their total holdings.

Before the formation of Regional Governments, Committees of Adjustment were empowered to perform both the duties of severing land into smaller parcels as well as giving consideration to minor variances to existing Zoning By-laws. In the case of minor variances, the Committee of Adjustment would consider such things as a 1 m side yard versus an existing Zoning By-law requirement of 1.2 m. The Committees of Adjustment have the power to give consideration to the above topics and, for all intents and purposes, have the same powers as the Minister of Housing or its delegated authority. All decisions of the Committees of Adjustment are subject to an appeal to the Ontario Municipal Board. The intent of the Committees of Adjustment is to provide local input into the aforementioned types of severances and/or minor variances.

In areas with Regional governments, the Regional government has established Land Division Committees to deal specifically with land severances.

Municipalities may include suggested or recommended conditions of approval for the Land Division Committtees' consideration including, cash payments. The thinking behind the suggestion of "imposts or cash payments" relates to the comparison of a severance being a form of subdivision of lands. In the development of lands of subdivision, municipalities usually suggest "imposts" of one type or another. In establishing suggested or recommended conditions of approval, an overriding policy should be that severances should not replace development of lands by plans of subdivision. In this regard, it is necessary, usually through official plans to determine guidelines respecting policies on the number of severances and the types of development acceptable for severances.

It must be understood that the conditions of approval suggested by a municipality (usually through municipal staff) are not mandatory to the Committee of Adjustment and/or Land Division Committee. If a committee considers the suggested conditions appropriate for a given application, the Committee can include the suggested conditions as a requirement of approval. If the conditions are not deemed desirable for a particular application, the Committee can disregard the suggestions of a municipality and either approve the application with no conditions, or reject the application.

In cases where the suggested conditions or recommendations are disregarded by the Committee of Adjustment or Land Division Committee, the municipality has the right to appeal the decision to the Ontario Municipal Board. In the same way, the owner of the lands can appeal any decisions including conditions of approval, to the Ontario Municipal Board.

7.2.7 Engineering and Planning Requirements

(A) General Service Charges:

Usually general service charges or specific levies are payable on the severed portion of the lands with a minimum charge being reviewed from time to time, as well as a maximum charge. In reviewing the maximum charge, costs of subdivision lots should be kept in mind as well as the minimum size of lot usually allowed. The general service charge would be applied to such items as oversizing of underground facilities, construction of roadways to service an area, the acquisition of park lands and development of parks, the installation of street trees, and a contribution towards treatment plants and other types of facilities. The argument for a general service charge relates to those services for which the remainder of the municipality is responsible. The argument extends to oversized services outside the general area of the individual application. In this way, consideration can be given to the total aspect of redevelopment, including a contribution towards any increased municipal expenditures caused by development throughout the municipality.

(B) Road Widenings:

Usually approval for consent to sever a parcel of land is conditional upon the granting by the owner to the municipality having jurisdiction over the abutting road, free of charge, all required road widenings, one foot reserves and corner roundings, along the severed parcel and sometimes along the remnant parcel as well. However, such requirements must be in accordance with the Official Plan or in accordance with an official policy adopted by the municipality with respect to road widenings.

(C) Sidewalks:

Sidewalks may be required as a condition of approval, along the severed portion, where feasible. Where actual construction is not feasible, monies should be obtained for the ultimate construction of sidewalks on streets where sidewalks probably will be constructed in a five to ten year period. An estimate of cost for the sidewalk should be included in the recommendation to the Committee of Adjustment or Land Division Committee so that a given amount of money can be included in the conditions of approval by the Committee of Adjustment or Land Division Committee. In this way, a fund can be established for the construction of the particular sidewalk when other sidewalks in the general area are considered.

In making this suggestion, sidewalks in the general area should be evident, thereby establishing the need for sidewalks across the severed portion. Whereas the monies obtained through the general service charge are usually used throughout the municipality, the intent of the sidewalk cash payment is for the construction of the sidewalk across the frontage of the severed portion itself. If the remainder of the street is to be done under the Local Improvement Act, the new owner of the severed portion of land would not contribute again.

2.2.2 Engineering and Planning Requirements Cont'd

(D) Easements:

Easements should be provided for any watercourses or other above or under ground services required in the area of the severance.

(E) Number of Severances:

A maximum number of severances may be permitted from any original township lot. In some municipalities, a potential maximum of two or three lots has been suggested. When severances in excess of two or three are proposed, a plan of subdivision should be considered.

(F) Size of Severances:

The size of the severances should generally conform to the zoning by-laws, including any new draft Zoning by-laws that might be under consideration. In this way, some uniformity of lot size can be established in a given area. In addition to the size of the severance, consideration should be given to a minimum frontage for the intended use. (Normally severances are for single-family use).

2.2.3 General Requirements

(A) All outstanding taxes should be paid on the severed and remnant parcels, prior to approval of severance.

(B) Any suggested policies with regard to imposts etc., should be applied with discretion when a severance is requested, such as:

(a) When conveying land to an adjacent owner. An example of this type of consideration would be the severance of a piece of property adjacent to another piece of land for the purpose of enlarging a property. In this case, no new lot is being created which will require servicing in the future. In this case, no imposts should be suggested as a consideration for approval of the application.

(b) When a parcel was previously held under separate ownership. With recent requirements of part lot control under the Planning Act, it is now necessary for severances to be required when lands were actually held under separate ownership in the past. In these instances, minimal conditions of approval should be suggested or recommended since no new lot has been created. It is merely a type of control vested in the municipality to insure proper planning of the area, and does not relate to services.

(c) When the land is located in a recently approved plan of subdivision. In these cases, the municipality usually obtains sufficient monies and/or services through the subdivision agreement.

..2.3 General Requirements (Cont'd

(B) (d) When an existing semi-detached or attached residential building is divided to permit separate ownership. In this case also, no new house is being created, and no additional strain will be placed on municipal services.

The conditions and policies described above are based on requirements as outlined in The Planning Act of Ontario. In addition, consideration must be given to Zoning by-laws and draft Zoning by-laws. An Official Plan of any municipality is of extreme importance when consideration is given to severances or extension of non-conforming uses. The Official Plan is a general guideline , and all suggestions or recommendations relating to severances particularly, should be related to the uses intended through the Official Plan.

2.3 REDEVELOPMENT AND SITE PLAN AGREEMENTS

Redevelopment is a common occurrence in most municipalities. It can be initiated by a private developer or the Federal, Provincial or Regional governments, through various programs, such as urban renewal or neighbourhood improvement plans. One of the main items to be considered in any "redevelopment" is proper control of the development so that it fits in properly with intended future uses and existing uses of adjacent lands.

In the case of "deregistration" of old plans of subdivision on which little development has taken place or resubdivision of blocks on a registered plan, it is the usual practice to require a new plan of subdivision to be placed on the entire land, including any existing buildings which might exist at the time of considering this type of redevelopment.

This procedure requires the agreement of existing owners of houses from the old plan of subdivision, and can be a lengthy procedure. In the long run, however, a new plan of subdivision through this procedure is sometimes advisable. With the high cost of land now prevailing, previous subdivisions with rather large sized lots are very uneconomical, and "deregistration" of plans of subdivision is one means of making better use of existing vacant lands. As indicated above, new plans of subdivision can then be created including subdivision agreements in which adequate services, etc. can be considered. The recent legislation now permits a number of "site plan" controls to be imposed by municipalities.

Some municipalities, in the past, had private bills passed allowing levies to be placed on such developments as apartment buildings, new commercial developments, etc., and to include certain items in a site plan agreement. With recent legislation, municipalities have permissive legislation to allow this type of control.

In most rezonings, or any other redevelopment, it is desirable that a "site plan" and site plan agreement become a condition of approval for rezoning. In this way, the municipality has control of the type of development which will occur on the rezoned section of land, and the municipality will also be able to acquire certain needed facilities through levies and/or dedications. Some municipalities have established a Site Plan Technical Committee to give consideration to the proposed site plans and site plan agreements relating to any rezoning. Technical input from the following points of view is desirable, planning, engineering, building, traffic and fire requirements. This committee acts as a liaison committee between the Council of a municipality and the developer.

The developer should meet with the committee on occasion, so that all points of view can be thoroughly discussed. Some of the items to be included on the site plan of any site plan agreement would include the following:

(1) Grading and Surface Runoff, Etc.

(a) Existing elevations should be shown on a 10-20 m grid and this information should include any drainage ditches with important elevations shown on the drainage ditch.

2.3 (1) Grading and Surface Runoff, Etc. (Cont'd)

(b) Proposed elevations should be shown on the plan in addition to existing elevations. Elevations should include those existing and proposed at corners of buildings and corners of lots, and any other elevations required to make sure that the general drainage pattern being proposed will function.

(c) Any catchbasin system required to accommodate the drainage together with proposed outlets to the municipal drainage systems must be shown on the plans.

(Note: The purpose of the above drainage scheme is to contain all water on the lot and to prevent surface water from running onto adjacent lands. In addition, the drainage scheme should accommodate any drainage which flows naturally to the lot.)

(2) All proposed fencing, landscaped or grassed areas should be shown on the plan including retaining walls where applicable.

(3) Roof Water

- All roof leads should be indicated on the plan showing the proposed direction of flow.

(4) Exterior Finish

- The type of material to be used on the building should be shown on all faces, although there is no enabling legislation for this.

(5) Parking Areas and Driveways

- The parking layout should be shown including the dimensions of of the proposed spaces, etc..

(6) Outside Storage of Garbage

- The type and location of this type of consideration should be shown on the plan.

(7) Location of Hydrants

- A means of access to all hydrants must be sufficient to permit proper fire fighting procedures. Consideration should also be given in the Site Plan Agreement to the necessity of all fire fighting procedures which might be pertinent to the particular application.

It should be noted that the conditions indicated above can usually be shown on the site plan. Reference should also be given to the above mentioned facilities in the Site Plan Agreement. The Site Plan Agreement should also include conditions regarding "perpetual" maintenance of such items as landscaping, shrubbery and so forth.

2.4 DEVELOPMENT BY REGISTERED PLAN

2.4.1 General

Except as noted in Section 2.2 under Land Division Committees, all subdividing of land for the purpose of creating residential, commercial, industrial or institutional parcels of land for building purposes, takes place through a plan of subdivision and a registered plan.

Prior to receiving approval from the local municipality, Region where applicable and the Ministry of Housing, for registration of his plan, the developer is required to prepare several plans and documents as described in the following Sections. Only after all governments and agencies give their approvals, can his plan be released for registration.

2.4.2 Draft Plan

The first plan of a specific subdivision is called the draft plan. This gives a relatively accurate display of proposed road patterns within the proposed subdivision as well as information on adjacent existing developments and land features. This plan must be prepared in accordance with Section 50, Sub-section 2, of The Planning Act of the Province of Ontario. Generally the plan should indicate the subdivision boundaries, the proposed lotting or block lay-outs, existing features such as contours, buildings, creeks and valleys. The plan is generally prepared by planners but may be produced by surveyors or engineers. It must contain a surveyor's certification with respect to the boundary survey.

The plan is submitted by the developer to the Provincial Ministry of Municipal Affairs and Housing or delegated authority where it is assigned a number, representing the Minister's file on that particular plan. This number always has a prefix "T". The Plan is circulated to all concerned, Federal, Provincial, Municipal and Regional authorities where applicable and all school boards, conservation authorities and utility agencies in the area.

All of these authorities submit comments to the approving body who will then prepare the draft plan conditions of approval, which must be satisfied prior to release of the plan for registration.

2.4.3 Draft stage including Consolidated Reports

Upon receipt of the draft plan, the local municipality will circulate the plan internally to the various departments for comments to establish the municipal requirements with respect to the subject subdivision. The coordination of this operation is generally carried out by the Planning Department if the municipality has such a department. Smaller municipalities may coordinate this operation through the Clerks Department.

2.4.3 Draft stage including Consolidated Reports Cont'd

The engineer responsible for subdivision development should review the plan at this stage in great detail. Normally a report of servicing requirements is prepared. The requirements as indicated by this report are then included in an overall report by the Planning Department, called the Consolidated Report, which includes the requirements of the other departments or agencies of the municipality and sometimes may also include requirements of external agencies.

The engineer's report should specify the general servicing requirements where these follow the standard requirements and specifications. More detail should be offered in situations peculiar to the plan of subdivision which may not be covered by standard requirements. Generally, the items requiring review are:

(i) sanitary and storm sewers - total areas to be drained within and outside the plan and location of existing trunk sewers and watercourses;

(ii) water supply - the area to be accommodated and location of existing feeders;

(iii) roads - road patterns as related to proposed use and anticipated traffic, geometric requirements for vertical and horizontal curves, major intersections, bus bays, road cross-sections including curbs and road surface, etc.;

(iv) sidewalks - type, width and where required;

(v) treatment of boulevards - sodding, tree planting, etc.;

(vi) walkways - to provide pedestrian accesses to off-street areas such as parks, schools.

The extent of detailed engineering required depends on the size of the project, but enough engineering should be carried out to make sure that the draft plan will not have to be changed after approval, and to enable the developer to estimate the construction cost of municipal services.

2.4.4 Draft Plan Approval

After completion of the Consolidated Report, the conditions are reviewed by the developer or his agents for comment and/or acceptance of the conditions stated. If the developer accepts these recommended conditions then the report and draft plan are submitted to the Municipal Council for approval and then to the approving authority. The approving authority in turn issues a set of requirements, which essentially describe all the conditions which must be met by the developer in order to obtain approval to proceed with the subdivision. They include the general requirements of the municipality's Consolidated Report as well as the requirements of Federal and Provincial agencies and other authorities.

2.4.5 Agreements

As soon as the engineering drawings have been completed by the developer's consulting engineers, they are submitted to the municipality for approval by the City Engineer, which usually requires several submissions until all of the requirements have been incorporated in the drawings. During the engineering processing, the consulting engineer and surveyor for the developer generally prepare the Schedules to the Engineering Agreement. The body of the Engineering Agreement itself is generally standard for each municipality and varies little between different subdivisions. The Schedules to the Engineering Agreement contain the specific requirements for that subdivision and generally the engineering drawings form a part of these Schedules.

At the same time the developer or the municipality prepares a Financial Agreement. Often this is done through the municipality's Clerk's Department, Legal Department or Finance Department. This agreement outlines the financial contributions the developer is required to pay to the municipality during the terms of the subdivision agreement: for example, payment of impost levies, special sewer area levies, contributions for construction of works or over-sizing of services to be carried out by the municipality or developer.

The Engineering and Financial Agreement generally stipulates, among other things, the duties of the developer's consulting engineers, identification of the Schedules attached, required securities and insurance policies, the responsibilities of the developer, the powers of the municipality, timing of works, conditions for issuance of building permits, final acceptance of works, etc..

The Schedules attached to the agreement normally contain the following: legal boundary description of the lands affected by the Agreement, either by reference plan or metes and bounds description; list of drawings forming a part of the agreement; list of lands and easements to be dedicated to the municipal authorities; special conditions with respect to application for building permits; a breakdown of the works in detail, their estimated unit costs and totals; miscellaneous schedules for municipal participation in service over-sizing, etc..

2.4.6 Administration Requirements

The developer should be required to post a Bond or Letter of Credit together with an Insurance Policy with the municipality at the time of execution of any development agreements. The security is usually for 100% of the estimated cost of the works and guarantees that the lots or blocks will be serviced after registration of the plan. The Insurance Policy naming the municipality as coinsured protects the municipality against claims for damages arising from operations carried out in connection with a particular development. The securities may be reduced in value as the work progresses, provided enough security is retained to provide for all oustanding work and possible maintenance work and to satisfy the requirements under the Construction Lien Act (min. 10% of the original amount).

2.4.6 Administrative Requirements Cont'd

A Statutory Declaration should be submitted by the developer stating that all completed works have been paid in full, prior to reduction in security. The developer is normally required to guarantee all works for a certain number of years after completion. (1-2 years for underground works, 2-3 years for above ground works).

After development agreements have been finalized and executed by the municipal officers and the developer has met all other conditions set out by the conditions of draft plan approval, the municipality will inform the approving authority that all municipal conditions have been met by the developer upon which, the approving authority may release the plan for registration as further described under "Final Plan for Registration" hereafter.

After registration of the plan, the installation of services may proceed.

In some municipalities special pre-servicing arrangements may be made whereby underground utilities may be installed prior to registration of the plan after finalization of the Engineering Agreements and drawings.

From this stage on, the municipal development engineer is charged with administration of the provisions of the Engineering Agreement. It is his responsibility to ensure that the work is carried out to meet the specifications and requirements of the municipality. He must also administer securities, insurance policies and issue approvals for works completed at various stages including the recommendation for final acceptance of the works by his Council. The Engineering Agreement shall also ensure that the developer rectifies any defaults which arise during established maintenance periods including drainage problems, particularly where these affect the public and residents.

Prior to final acceptance of the subdivision by the municipality, the developer should be required to submit the following documents:

1) Statutory Declaration that all accounts relative to the development have been paid.

2) A Statement from an Ontario Land Surveyor that all standard iron bars as shown on the registered plan have been found or replaced.

3) The original linens of the municipal services corrected to show "as constructed" dimensions and elevations.

2.4.7 Final Plan for Registration

As mentioned previously, after draft plan approval the developer's surveyor prepares an accurate plan of the subdivision lands showing all proposed lots, blocks, roads, walkways, road widenings, reserves and all existing survey information necessary to establish the plan, all in accordance with the requirements of the Land Titles Office and the aforementioned conditions of approval for the subdivision.

2.4.7 Final Plan for Registration Cont'd

The plan also shows certificates which are to be signed by the owner, surveyors, registrar and approving officer (Minister or Chairman of Region). If mortgages exist on the lands then the plan should also be signed by the mortgagees.

The owner's certificate generally identifies all of the lots, blocks, streets with their names and dedicates the streets and street widenings as public highways. This dedication eliminates the need of the municipality to pass by-laws opening the roads and widenings as public highways.

After the plans intended for registration are given final approval by the Minister or his delegated authority and after the developer has complied with all the requirements of the conditions of draft approval, the plans are sent to the Master of Land Titles.

The Master of Land Titles gives detailed scrutiny to the plans to ensure that the requirements of the Land Titles Act are met. The Land Titles Officer signs the final plans and sends them to the local Land Registrar.

At this stage the plan is put on title and given a registration number. From this point on all transfer of land within the plan are described based on the plan; for example, Lot 23, Registered Plan M-40.

Prior to January 1st, 1974, it was possible to register a plan of subdivision either in the Registry Office or Land Titles Office at the option of the developer. After that date, however, it has become mandatory to register all new subdivisions under Land Titles. The Land Titles registry system is an improvement over the old registry system as the need to search title beyond the current records is eliminated. The Land Titles Office maintain an up-dated status of each parcel. Consequently each individual purchaser of a lot receives better protection against liens or other charges outstanding against the subject lands with less time required for searching.

Upon registration of the plan, the roads are dedicated as public highways and come under the jurisdiction of the Provincial, Regional or local municipality and the ownership of lots and blocks may now be transferred.

2.4.8 Lot Grading

As pointed out in the introduction of the Chapter, technical details with respect to sewers, watermains and roads will not be indicated here. However, since lot grading is generally associated with new developments and can create many problems sometimes for years after completion of the subdivision, a short summary of typical lot grading requirements is included here.

A. Need for Lot Grading Plan

When a large parcel of land is to be divided into a number of lots, blocks and streets (usually by means of a plan of subdivision) careful consideration has to be given to the surface drainage on the newly created parcels.

2.4.8 Lot Grading Cont'd

A. On the original undeveloped parcel, surface water was drained by means of natural watercourses. These watercourses conveyed the water to road side ditches, drainage ditches, creeks and rivers.

When land is developed, roads and buildings are often placed across these natural or improved drainage systems. Therefore, it is necessary to establish a new system of drainage. This new system will re-establish a drainage pattern which ensures that the surface water will drain away effectively when the land is divided into the various parcels.

B. Basic Information

Lot grading plans usually have the following basic information on them:

a) Lot, block and street layout at a scale of 1:500;

b) All lot and block numbers and letters, street names and other identifying notations;

c) Contours showing the existing ground elevations, and trees;

d) Finished centre line of road elevations every 25 metres;

e) Typical lot cross-sections and plans showing the lot grading. These usually show the following typical patterns. (Refer to drawing on page 2-17).

C. Design Criteria

The following is design criteria used by many municipalities:

a) Grass surfaces shall have a minimum slope of 2% (in special cases 1%).

b) Grading around houses and buildings shall direct the water away from the structure.

c) Drainage flows which are adjacent to houses are to be in defined swales located as far from the house as possible.

d) The desirable swale depth is 225 mm with a minimum depth being 150 mm. Maximum swale depths will vary depending on location and safety consideration. (preferrably not more than 600 mm)

e) Swale grades shall be at least 2%.

f) The maximum runoff allowed in a swale between two houses is the runoff from 4 backyards.

g) The normal maximum runoff allowed in rear yard swales shall be the runoff from 8 backyards maximum depending on the lot size and grade.

2.4.8 Design Criteria Cont'd

C. h) When rear lot catch basins are installed to pick up surface drainage, some municipalities maintain them (in which case easements must be provided) and others do not. Where the municipality does not maintain the catchbasins, the owner of the lot, on which the catchbasin is located, is responsible for its maintenance. The catchbasin and drain should therefore be located on one lot and not on the lot line.

D. Final Lot Grading Plan

When the basic information is prepared and the design criteria is applied, the lot grading system can be established. To ensure that the lot grading, as designed, is carried out in the field, the following design information is shown on the lot grading plan.

a) The finished ground elevation at the four corners of each lot.

b) On page 2-17, typical lot grading patterns are shown. Each of these patterns is given a designation. These designations are shown on the plan in accordance with the type of grading proposed for the lot.

c) The elevation of the first floor of the house for each lot.

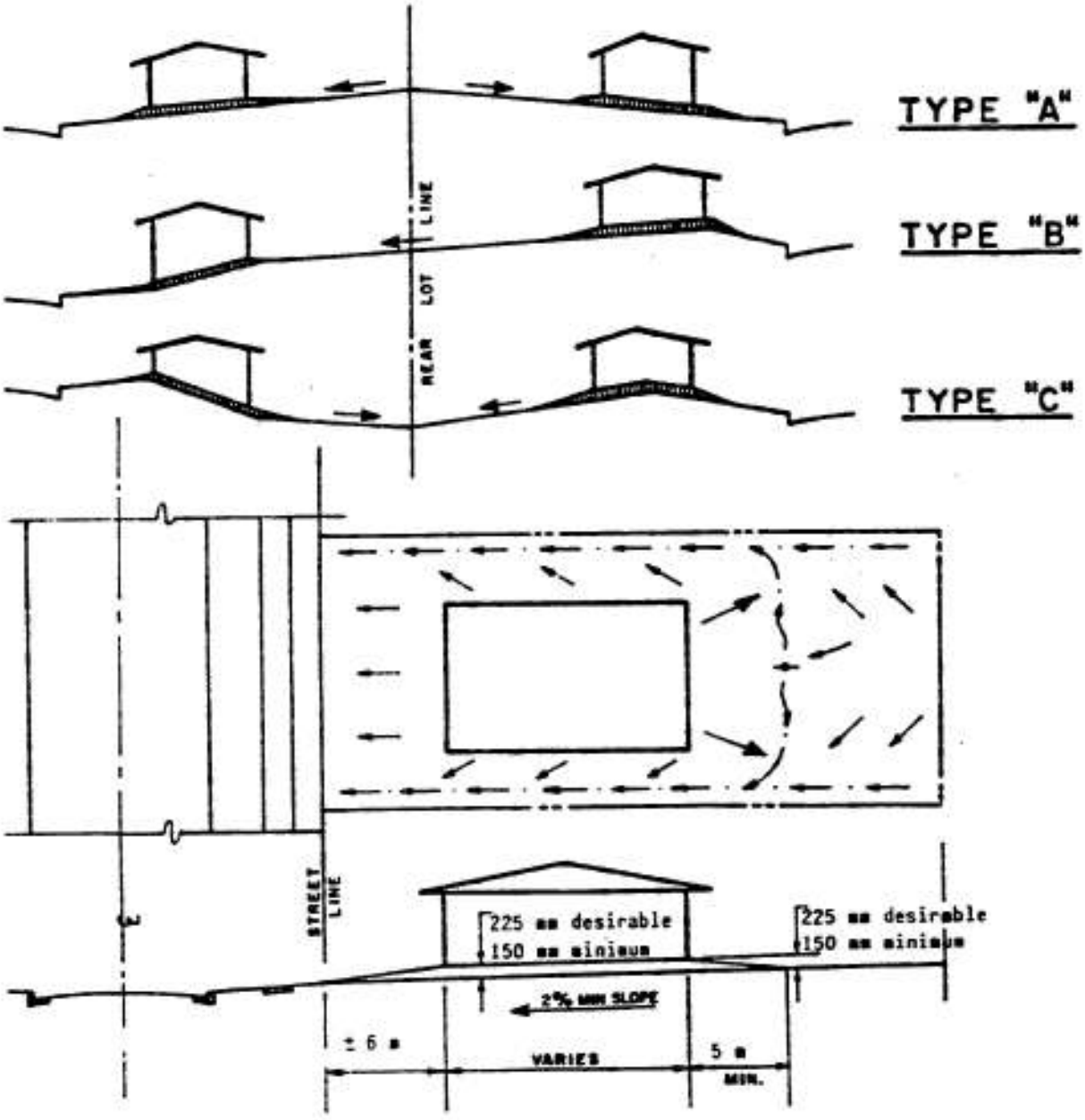
d) Arrows showing the basic drainage pattern for each lot.

e) Rear lot catch basins and their easements. Easements are to be shown only where the municipality maintains the catch basin and lead.

E. General

It should be realized that these grading plans establish a basic drainage pattern only and should not be interpreted as establishing the fine grading for each individual lot and house, since this will vary with the house type to be built on the individual lot. It is desirable to have the Engineers for the developers certify the plot plan (or site plan) for each house confirming that the proposed house design and grading can be accommodated in accordance with the approved grading plan. If not, the house design should be changed or the grading plan revised where possible prior to the issuance of the building permit.

Some municipalities are also requesting the Consulting Engineers for the developers to produce a certificate after completion of the lot grading, stating that the lot grading as completed is in overall conformity with the approved grading plan.



TYPICAL LOT
CROSS-SECTIONS AND PLAN

2.5 ZERO LOT LINE CONCEPT

2.5.1 General

Although zero lot line development is a form of development by registered plan, it is mentioned separately because it is a new concept for residential development. The terminology, "zero lot line housing" as a land use designation is a misnomer. It is not a type of land use such as single family housing, townhouses or apartments. The use of zero lot lines is a building siting technique closely interrelated with the architectural design of the house and adjoining houses. Utilized in conjuction with other closely associated concepts, densities approaching a probable maximum of 25 detached units per hectare can be achieved.

2.5.2 Existing Practices

Many of today's subdivisions are being built utilizing antiquated zoning by-laws and concepts of residential planning formulated to develop large lots for individual sale. Since the type of house to be constructed on each lot is usually unknown prior to registration, standard front, side and rear setbacks were instituted to exercise a degree of public control. This may result in inefficient land use, such as 1-2 m sideyards which may or may not be necessary for access or drainage, large ornamental front yards and backyards that rarely provide any outdoor privacy and seldom bear any relationship to the interior of the house. It is evident that a major drawback of conventional subdivisions is a lack of privacy. In essence antiquated planning concepts and zoning can hinder good land use planning and house production.

2.5.3 Alternative

One alternative is to integrate the overall design of a plan of subdivision and the proposed housing. Streets, land uses, facilities and structures are designed as a unit. Such a planning unit development (P.U.D.) permits the development of new neighbourhoods with a variety of siting treatments. Houses can be sited utilizing zero side yards and reduced or varied front yard setbacks. Privacy can be obtained by careful design of the house itself and in relation to adjoining houses.

The basic P.U.D. concepts relating to the development of the dwelling units can be summarized as follows:

1. Use of zero side yards on one side of the unit and reduced front yard setbacks on the other side of the unit and in the back yard.

2. Houses designed to provide maximum privacy within the home and with outdoor "privacy spaces" closely related to indoor living areas.

3. The walls of an adjacent dwelling on a zero lot line acts as a containing element for the patio area allowing for privacy and definition.

2.5.3 Alternative Cont'd

4. Overlooking windows are restricted to the dwelling utilizing the patio.

5. Fencing is an integral part of the design of the houses and in the creation of privacy areas.

6. Houses are sited in a manner to obtain maximum privacy.

2.5.4 Implementation

1. Plans are prepared and submitted to the Municipality for site plan approval.

2. Houses are located in accordance with the site plan.

3. Reference plans prepared by O.L.S. when basements established and lot lines (especially the "O" lot lines" are determined).

4. Houses are conveyed by way of registered reference plan.

It should be realized that the "Zero Lot Line" concept can be utilized for densities varying from 10 - 25 single family units per hectare. However, since it does enable the developer to achieve densities of up to 25 units per hectare, it is usually associated with higher densities and lower priced single family homes.

A team approach of planners, architects and engineers utilizing the zero lot line and the P.U.D. concept can produce quality single family homes at considerable cost savings to the home buyer.

2.6 POLICIES REGARDING MUNICIPAL SERVICES REQUIRED FOR NEW DEVELOPMENT

2.6.1 General

As mentioned in the introduction, it is generally the policy of municipal governments to require developers to install or pay for all municipal services required within the boundaries of their development and generally the municipality will provide the external services required such as sanitary sewers, trunk watermains, roads, etc.. Internal servicing standards vary from municipality to municipality. The Ontario Housing Advisory Committee has prepared "Recommended Guidelines for Residential Servicing in Ontario" available from the Ministry of Housing and Municipal Affairs which is an attempt to bring about greater uniformity in the different standards employed throughout Ontario. These guidelines can be used to great advantage when establishing new standards or revising old ones.

Where extensive development takes place within one municipality and especially where many different developers are involved, development can proceed more equitably if the municipality takes an active role in providing external trunk service. In order to finance the cost of this work, the municipality establishes an "impost charge" or "levy" expressedas a certain amount per person, hectare, lot or dwelling unit.

2.6.2 Levies

It should be realized that most municipalities will permit development or redevelopment only on the basis that it will not create an additional taxation burden on existing ratepayers in the municipality. In addition to the "hard" services required, such as trunk sewers, watermains, roads and treatment facilities, there are other financial obligations on the municipality resulting from development such as additional libraries, parks and recreational facilities, fire and police stations, etc.. To provide funds for all of these requirements, a municipality might adopt a levy scale, shown on Page 2-22, as an example only.

Levies for Sanitary Trunk Sewers and Trunk Watermains may be calculated as indicated below:

"A" Residential Zoning

Average cost/hectare	$3,000.00	
Based on 60 persons/hectare		$50.00/person
Single Family Semi-Detached Town Houses	4 persons/unit	$200.00/unit
Medium Density		
Low Rise – Deck Housing	3 persons/unit	$150.00/unit
High Rise	2 persons/unit	$100.00/unit

2.6.2 Levies Cont'd

"A" Trunk Watermains

Average cost/hectare	$4,500.00	
Based on 60 persons/hectare		$75.00/person
Single Family) Semi-Detached) Town Houses)	4 persons/unit	$300.00/unit
Medium Density		
Low Rise - Deck Housing	3 persons/unit	$225.00/unit
High Rise	2 persons/unit	$150.00/unit

"B" Industrial and Commercial Zoning

For trunk watermains - $5,000.00/hectare
For trunk sanitary sewers - $3,500.00/hectare

It should be realized that the levies included herein are an example only and since the needs and costs in different municipalities vary greatly, levies will vary greatly.

The Association of Municipalities in Ontario appointed a Committee on Cash Imposts to investigate the extent and nature of cash imposts levied by municipalities throughout Ontario and to develop guidelines for the use of municipalities in formulating a cash impost policy. The reader may wish to refer to this report for further details on Procedures, Implementation and Administrative Guidelines for establishing lot levies or impost charges.

EXAMPLE OF UNIT LEVIES

	Persons Per Unit	Library $50/pers.	Fire $30/pers.	Parks & Rec. Facilities $230/pers.	General Admin. & Service Complex $70/pers.	Capital Sewer Levy/Unit	Capital Water Levy/Unit	Total Unit Levy
High Rise (Non Family)	2.0	100.00	60.00	460.00	140.00	100.00	150.00	1,010.00
High Rise	2.5	125.00	75.00	575.00	175.00	100.00	150.00	1,200.00
Low Rise or Deck Housing	3.0	150.00	90.00	690.00	210.00	150.00	225.00	1,515.00
Town Houses or Condominiums	4.0	200.00	120.00	920.00	280.00	200.00	300.00	2,020.00
Semi-Detached	4.0	200.00	120.00	920.00	280.00	200.00	300.00	2,020.00
Single Family	4.0	200.00	120.00	920.00	280.00	200.00	300.00	2,020.00
Industrial & Commercial	-	-	-	-	-	3,500.00/ Hectare	5000.00/ Hectare	

It should be noted that this is an example only and that actual levies vary considerably from one municipality to another, depending on needs and requirements.

..7 CONDOMINIUMS

2.7.1 General

Condominium ownership is a form of ownership which has been permitted since the passing of The Condominium Act, 1967. Usually it means that parts of a project are owned individually (space within walls of a particular unit) and the remainder is owned commonly between all the owners of the individually owned units. (corridors, elevators, parking areas, lawns, etc.). The structure can be in the form of a highrise, rowhousing, cluster units or other arrangements. All of the common elements are owned and maintained by the Comdominium Corporation made up of the owners of individual units. This makes the Condominium Corporation responsible for maintenance of private sewers, watermains, roads, etc. located on their property and for services such as snow plowing and garbage pickup unless other arrangements have been made with the municipality. It is especially because of this overlap in maintenance and servicing requirements that many municipalities have become concerned about the way in which these condominiums are set up, constructed and maintained, and some municipalities require through a site plan agreement that services on private property such as sewers, watermains, and roads are built to acceptable standards to make sure that no serious problems will arise after these facilities are transferred from the builder to the Condominium Corporation. One municipality has prepared a complete guide with respect to all aspects of condominium development and the municipal servicing part of this guide is included in this section based on the following categories:

Category I - Single lots

Category II - Single lots & semis

Category III - Row Townhousing

Category IV - Medium Density (stack townhousing, cluster townhousing)

Category V - High Rise Apartments

2.7.2 Typical Servicing Requirements for Condominium Developments

The following specifications and standards which are subject to amendment from time to time as circumstances warrant, will form the basis of the review of condominium applications in accordance with the municipality's Official Plan policies and condominium housing.

1. Safety

   (a) Provisions for safety from fires and structural defects shall be in accordance with relevant statutes, Building Code, Fire regulations and any other relevant requirements of the municipality. Nowithstanding the requirements of the Building Code, the following additional specifications must be met:

?.7.2. Typical Servicing Requirements for Condominium Developments

1. Safety Cont'd

Within any building in Category V and any building two or more units in height in Category IV (i.e. stacked townhouses):-

i) A standpipe together with a hose cabinet shall be located in the corridor of each floor within 1 m of the exit door/stair door serving that corridor.

ii) Such standpipe and hose cabinet shall be located at not greater than 50 m intervals and further, that no entrance door to a unit be greater than 17 m from such standpipe and hose cabinet.

iii) The hose cabinet shall be equipped with approved hose racks and water stops and not more than 25 m of pre-connected 375 mm approved synthetic type fire hose.

iv) Within every dwelling unit, inside the door of that apartment unit there shall be installed an approved smoke detector unit linked to the superintendent's office and the Fire Department.

v) Continuous maintenance and upgrading of standpipes, firehose cabinets, exit lights, emergency lighting, fire pumps, alarms and/or detectors, sprinklers and hydrants.

b) Emergency Routes and Hydrants:

i) Secondary access routes, walkways and rear yards identified as emergency access routes shall be kept clear of any type of fencing or other obstacle. Such emergency access routes shall completely encircle each building so that the furthest portion of the building requires minimum length of hose.

ii) Fire hydrants shall be sited adjacent to roadways and located at no greater than 100 m intervals and no unit shall be located further than 160 m from the nearest water hydrant.

iii) Where units are constructed with separate exterior access, in order to assist emergency vehicles to find quickly the unit of distress, illuminated directional signs are mandatory.

iv) If the project has only one access from a public street, an emergency access must be provided in addition.

2. Roadway and Driveway Standards

a) Structural Design

i) Condominium roadway and driveways shall be designed to provide not greater than 1.5 mm Benkelman Beam deflection, or in any case shall not have less than 230 mm of Granular "B", 150 mm of Granular "A" and 75 mm of Asphalt HL3.

2.7.2 Typical Servicing Requirements for Condominium Development

2. Roadway and Driveway Standards Cont'd

(a) ii) Driveways serving individual units and garages to have not less than 150 mm Granular "A" and 50 mm of Asphalt.

iii) In any event, the type of subgrade soil shall be the governing factor in designing the pavement structure in each case.

(b) Widths

i) On a short crescent driveway or where a crossroad is located between two other driveways and where a one-way road pattern is permitted, the minimum pavement width shall be 6 metres. On such roads no parking shall be permitted.

ii) For the two-way roads where no street parking is permitted, the minimum pavement shall be 7 metres.

iii) On roads where two-way traffic is permitted and street parking is tolerated, the minimum pavement width shall be 8.5 metres.

iv) A one-way driveway providing convenience access to the front door of a condominium building in Categories IV and V shall be not less than 3.5 m in width.

(c) Alignment

i) Roadways serving condominium projects shall be designed to facilitate passage of emergency and other vehicles such as garbage trucks and snow removal equipment. Curb returns having a 7.5 m radius and inside bends having a 15 m radius are required for most vehicle turning movements. At dead-ends, the cul-de-sac pavement radius should be at least 10 m and no cul-de-sac shall be greater than 50 m in length.

ii) In Categories I, II and III, the driveway design shall provide direct access for each unit to the common driveway.

(d) Set-back Requirements

i) An entrance and/or exit to underground garage ramp or an above ground ramp shall be set back such that the start of the ramp shall be not less than 7 m from any intersecting driveway, "required" or "optional" sidewalk or street line; the intervening length of driveway shall be level and tangent.

ii) An individual unit garage in Categories I, II and III shall be set-back not less than 7 m from any condominium driveway, "required" or "optional" sidewalk or street line.

2.7.2 Typical Servicing Requirements for Condominium Development

2. Roadway and Driveway Standards Cont'd

(d) iii) Except for driveways serving individual units and garages, no roadway or driveway shall be closer than 1.2 m to the foundation or wall of any portion of a building.

(e) Curbs

All common drives and roadways shall have permanent type poured concrete barrier curbs as per municipal engineering specifications.

3. Parking Standards

(a) Category I

- a minimum of one space per unit enclosed in an individual unit garage, plus a minimum of one space per unit located between the garage and the condominium roadway. Visitor parking shall be accommodated on the common driveway at the required widths or in common parking lots at a rate of one space per four units.

(b) Category II

- a minimum of two spaces per unit one of which may be enclosed in an individual unit garage. Both spaces to be located within the land area assigned to that unit. Visitor parking shall be accommodated on the common driveway at the required widths or in common parking lots at a rate of one space per four units.

(c) Category III

- a minimum of one space per unit enclosed in an individual unit garage plus a minimum of one space located between the garage and the condominium driveway plus a minimum of one space for every four units located in a parking lot separate from condominium driveways to be reserved for visitor use only, and so designated by clearly visible signs. The parking lots to be distributed throughout the project for convenience to users.

(d) Category IV

- a minimum of 1.85 spaces per dwelling unit. Of this required parking supply, a portion being no less than 0.25 spaces per dwelling unit shall be reserved for visitors only. The total parking distributed throughout the project for convenience to users.

7.7.2 Typical Servicing Requirements for Condominium Development

3. Parking Standards Cont'd

(e) Category V

- a minimum of 1.75 spaces per dwelling unit. Of this required parking supply, a portion being no less than 0.25 spaces per dwelling unit shall be reserved for visistors use only, shall be so designated by clearly visible signs and shall be located convenient to an "intercom" equipped entrance to the building either in a grade level parking lot or in an open portion of the underground garage.

(f) Categories IV & V

- of the required parking supply, a portion being not more than 0.2 spaces per dwelling unit may be provided in the form of tandem parking spaces (one without independent freedom of access) provided that the two tandem spaces are assigned to the dwelling unit occupant only.

(g) Categories II & V

- Recreational Vehicle Parking Area to be provided for the parking of recreational vehicles on the basis of 5 m^2 for each dwelling unit, such area to be separate from any other parking area. Recreational vehicles are defined as trailers, boats, campers, motorized homes, snowmobiles, motercycles and any other recreational vehicle. The internal marking of spaces may vary from different size vehicles and provide for sufficient manoeuvering room, especially long vehicles.

(h) Underground Parking Garages

i) All walls and ceiling of an underground parking garage shall be painted with white luminous paint.

ii) All doors to an underground parking garage shall be fitted with a dead lock.

iii) Underground garages shall be designed with minimum ramp widths of 7 m for two-way traffic or 5 m for one-way traffic. The ramp widths for above grade parking structures shall be the same as for underground garages.

iv) Underground garage roofs shall be structurally adequate to bear the weight of a 12 tonne fire apparatus.

v) The roof of the underground garage shall be structurally adequate to bear the weight of any required landscaping equipment including soil and grass.

7.7.2 Typical Servicing Requirements for Condominium Developments

4. Sidewalk and Walkway Standards

(a) Standards - The required width for walkways is 1.5 m. Pedestrian walkways to be constructed with 125 mm thick poured concrete, except across driveways where the minimum thickness shall be 175 mm to accommodate heavier traffic loadings. Design and construction as per Municipal Engineering specifications. Where walkways are required for individual units from the unit itself to the common walkway or common driveway or public sidewalk, they shall be constructed to the same specifications as above.

(b) Condominium plans shall be examined to determine the nature and direction of pedestrian travel. Sidewalks shall be required on at least one side of major condominium driveways in Categories I, II and III and where required, to provide for pedestrian access to street sidewalks, transit, services, schools and other facilities off-site and to recreational, social and other facilities on-site.

(c) Location in Relation to Driveways

i) In Categories I, II and III, a minimum 1.5 m boulevard will be required between sidewalks and major condominium driveways for snow storage purposes.

ii) In Categories IV and V, sidewalks should wherever possible, not be located adjacent to major driveways, but where located adjacent there shall be a minimum 1.5 m boulevard.

5. Lighting

Street lighting shall be provided as follows:

(a) No light standard shall be higher than 6 m.

(b) A light standard shall be placed at each roadway intersection and/or intersection of sidewalks.

(c) Light standards shall not be more than 40 m apart.

(d) A minimum candle power of 175 watt/light standard.

(e) Light standards shall not be located in the middle of the unit frontage.

6. Utility Standards

(a) Watermains, storm and sanitary sewers shall be constructed to the same standards as required in any residential subdivision development, or the Plumbing Code whichever is the more stringent.

2.7.2 Typical Servicing Requirements for Condominium Developments

6. Utility Standards Cont'd

(b) In condominium developments of Categories I, II and III the required hydro service easements shall be located immediately adjacent to the road and a minimum setback for such easement should be approximately 6 m from each housing unit.

(c) In all buildings in Category IV where separate service and meter connections are not required and all buildings in Category V, a transformer room or rooms shall be provided within the buildings.

7. Recreational and Social Facilities

(a) A properly designed and fenced children's play area or areas shall be provided in Categories II - V. Further, the size of such play areas to be based on 5 m^2 per unit in Categories II - IV and 1.5 m^2 per unit in Category V, but in any case no play area to be less than 100 m^2. Children's play areas are to be a minimum of 25 m from any unit or alternatively provide ample screening to minimize annoyance to adjacent units. Such play areas to be provided with children's play equipment to the approval of the Parks and Recreation Director.

(b) The minimum requirement for recreational and social facilities shall be as follows in Categories II to V:

i) Swimming or Wading Pools

1.2 m^2 of water surface per unit. If the requirement based upon the number of units is less than 70 m^2, then a wading pool shall be acceptable. If the requirement is more than 125 m^2 a swimming pool and wading pool may be acceptable. If the requirement is more than 240 m^2, an enclosed indoor swimming pool with a minimum of 0.6 m^2 of water surface per unit is required. The depths of swimming or wading pool facilities, deck space, and ancilliary facilities and equipment to be subject to the approval of the City Parks and Recreation Director.

ii) Meeting/Pre-School/Social Facilities

- Multi-purpose and/or specific usage facilities shall be provided based upon 0.7 m^2 per unit. Such facilities will provide for toilet and storage facilities in relation to the size of the facility and the interior decoration and furnishing shall be subject to City approval.

2.7.2 Typical Servicing Requirements for Condominium Developments

7. Recreational and Social Facilities Cont'd

(b) iii) Maintenance and Storage Facilities

For use by the Condominium Corporation to provide for the storage of maintenance equipment. Such facilities may form a part of other structures but shall be a minimum of 20 m^2 of covered and secure indoor space.

iv) Hard Surface Areas

Multi-purpose or specific usage hard surface areas shall be provided based upon 2.0 m^2 per unit. In small projects the space would appropriately allow for ball hockey, outdoor basketball, volleyball, etc., tricycle areas and winter skating areas. Larger projects should also consider tennis facilities as appropriate to developments.

(c) Specialized Facilities

Category V Condominium projects may also be required to provide specialized facilities which may include; game and hobby rooms, indoor play areas, fitness facilities such as gymnasium, squash, saunas, sundecks, lounges and limited commercial facilities.

The criteria for the foregoing will respect such consideration as:

- number and type of units
- location of project in relation to municipal or commercial facilities
- adjacent property development

In any event Category V developments, because of the high density factor, should attempt to provide maximum opportunity for social interaction through the provision of recreational and social openness through the provision of a high degree of landscpaing and grade variations.

8. Landscaped Open Space

The provision of landscaped open space within each development shall increase in direct relation to the density of a development. In all cases, the preparation of site plans are to respect trees and natural features in order to enhance the development and preserve vegetation which is irreplaceable.

Each condominium development will be, of necessity, judged on its own merits, but as a general guideline, landscaped open space which includes all areas excepting residential building sites, roads, driveways, ramps and parking areas, shall be provided at no less than:

Categories I - IV - 50%
Category V - 60%

7.7.2 Typical Servicing Requirements for Condominium Developments

8. Landscaped Open Space Cont'd

In calculating this percentage, credit may be given for indoor and outdoor recreational and social facilities.

The quality and quantity of landscaping is considered an essential ingredient in the development of a condominium project. Standards respecting landscaping will take into consideration such matters as:

- topographical features
- existing trees and vegetation
- adjacent land use
- environmental hazards - i.e. arterial roads, railways, industry, etc.

As a minimum general guideline, the following criteria will apply:

- all landscaped open space to be fully sodded with #1 nursey sod after soil has been prepared in accordance with good horticultural practice.
- an average of three trees per unit on the overall site.
- Decidous trees to be a minimum of 75 mm caliper with larger trees planted at selected areas.
- Coniferous trees to be a minimum of 2 m tall from finished grade with larger trees planted at selected areas.
- ornamental trees and shrubbery to be provided to achieve aesthetic improvement to the unit or building and/or to provide screening where required.
- where sites are flat, contours are to be created through mounds and berms as a method to provide a variety of gradients, and where required, to provide screening through elevations on the site.
- fencing will form a part of the landscape plan and shall be required to provide privacy screening, boundary delineation (where required), safety barriers and screening from environmental hazards. The type and specifications of fencing shall be in accordance with the requirements of the City for each specific site.

2.7.2 Typical Servicing Requirements for Condominium Developments

9. Shared Facilities

In the event the municipality considers it feasible to allow recreational and social facilities to serve more than one Condominium Corporation as an area under joint ownership, the aforementioned standards may be increased or decreased at the discretion of the Parks and Recreation Director in order to achieve a viable and acceptable shared facility plan.

10. Location of Recreational and Social Facilities

It shall be the policy of the municipality to require the location of recreational and social facilities to be adjacent to municipality owned open space and/or parkland where applicable and in other instances on the periphery of a plan in order to minimize the noise factor eminating from active facilities.



3.
PUMPING STATIONS and FORCEMAINS

CHAPTER 3

PUMPING STATIONS AND FORCEMAINS

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CHAPTER 3

PUMPING STATIONS AND FORCEMAINS

3.1 Introduction

The need for sanitary and storm sewage pumping stations, to one degree or another, is prevalent in most municipalities. The design of the pumping system will be governed primarily by the development of the municipality with respect to the existing topography.

Sanitary sewage pumping systems range in size from the small single pump private home unit to the pumping station serving small or medium size areas up to the main installation pumping all of the Municipality's sewage to a sewage treatment plant.

Storm sewage pumping systems generally have high volume pumping capability and usually do not involve forcemains. These systems are usually located adjacent to a receiving stream or trunk gravity sewer conduit.

A cost benefit study should be undertaken to determine the merits of pumping vs gravity sewer services. Pumping installations are relatively expensive and ongoing expenditures for operation, maintenance and replacement can be excessive in light of the increasing cost of electrical energy and labour.

The usual applications for sewage pumping stations are as follows:

a) The pumping of sewage from a low area over a height of land to existing gravity mains.

b) Raising of sewage from gravity mains whose outlets are below the receiving waters.

c) Raising the sewage to provide a head for gravity flow through sewage treatment plants.

Many of the comments in this discussion are applicable to all sizes of pumping facilities but are directed primarily towards small and medium sized stations encountered most often by the Municipal Engineer.

3.2 Site Selection

The site for a sewage pumping station is deserving of adequate consideration and in the design of a subdivision, the location should be considered in the land use planning phase.

Matters that should be reviewed are as follows:

a) Accessibility

The site should be relatively accessible from a municipal street and sized to provide for the parking of vehicles used for inspecting and maintaining the station. Access should be maintained to permit removal and replacement of major items of equipment. Access should provide for standby electrical generating systems or pump or even tank trucks in emergency conditions. Access should provide for the storage or removal of snow in the winter season and size should be based on expected precipitation.

b) Drainage

The site chosen should be properly graded to ensure that surface drainage does not become a problem under all reasonable circumstances, having regard for the equipment located underground. Where possible, stations should not be located under travelled roadways.

c) Power

The voltage and type of electrical power available should be ascertained from the local power authority. The frequency and duration of power outages and availability of alternate supply, such as rerouting through an optional grid, should also be determined.

d) Soil Conditions

Adequate soil investigations should be carried out prior to the design of the installation, particularly with respect to the water table. Since sewage pumping stations are normally constructed in low areas, a high water table may lead to flooding of the excavation and associated costly construction methods. Provisions should be made in the contract to eliminate the

possibility of high extra charges. The presence of rock should also be determined as rock excavation can also result in a major increase in construction costs.

e) Appearance

Many pumping stations are constructed in residential areas, immediately adjacent to dwellings. Many modern pumping stations are now below ground level, yet there still must be access driveways, entrance chimneys and, in some cases, electrical control panels above ground.

Some station designs incorporate a small building above the surface to house the pumps and control equipment. In any of these areas, provisions should be made to adequately landscape the property and to ensure that the installation does not detract from the surrounding neighbourhood. The building portion that is above ground should be designed with compatability with other buildings in the area in mind.

3.3 Types of Pumping Stations

There are primarily two types of pumping stations commonly referred to as the dry well station and the wet well station.

The basic difference is that the wet well station has the pumps and motors in the same chamber into which the sewage flows. In some installations the motor is set considerably above the pump and drives the pump by a long shaft. A more common procedure, particularly in small stations, is to use a submersible pump wherein both the pump and the motor, as one unit, can be immersed in the liquid. The pump can be removed by lifting it vertically on guide bars clear of the station. When the pump is replaced, by lowering it along the guides, it automatically seals the pump outlet to the discharge pipes.

In the dry well type of station the sewage is collected in a separate well or pit from that in which the pumps, motors and controls are located and the two portions of the station are physically isolated so that the only connection between them is through the inlet pipes connecting the wet well to the pumps. In this type of facility the dry well and the wet well may not necessarily be immediately adjacent but could be separated by several feet, depending upon the type of construction used.

Either of these stations may be constructed wholly on the site or may be purchased partially as a factory fabricated unit. One common method of construction of the dry well type of station is to construct the concrete wet well portion on the site, together with the concrete foundation slab and to purchase the prefabricated pumping chamber and simply install it on the prepoured foundation to be secured with the anchor bolts previously installed. This type of unit would include the pumps, motors, valves, all internal piping, electrical control panel and all accessories including an entrance tube from the pumping chamber to grade level. This unit is normally manufactured from steel and is constructed to meet the required elevations of the pumping facility.

Variations of factory units are available almost to any degree of fabrication that may be required by the engineer.

On Figures 1, 2 and 3, three alternative pumping station types have been illustrated as follows:

1. Figure 1 - Dry Well/Wet Well Station (cast in place.) This type is usually for medium to high flow conditions.

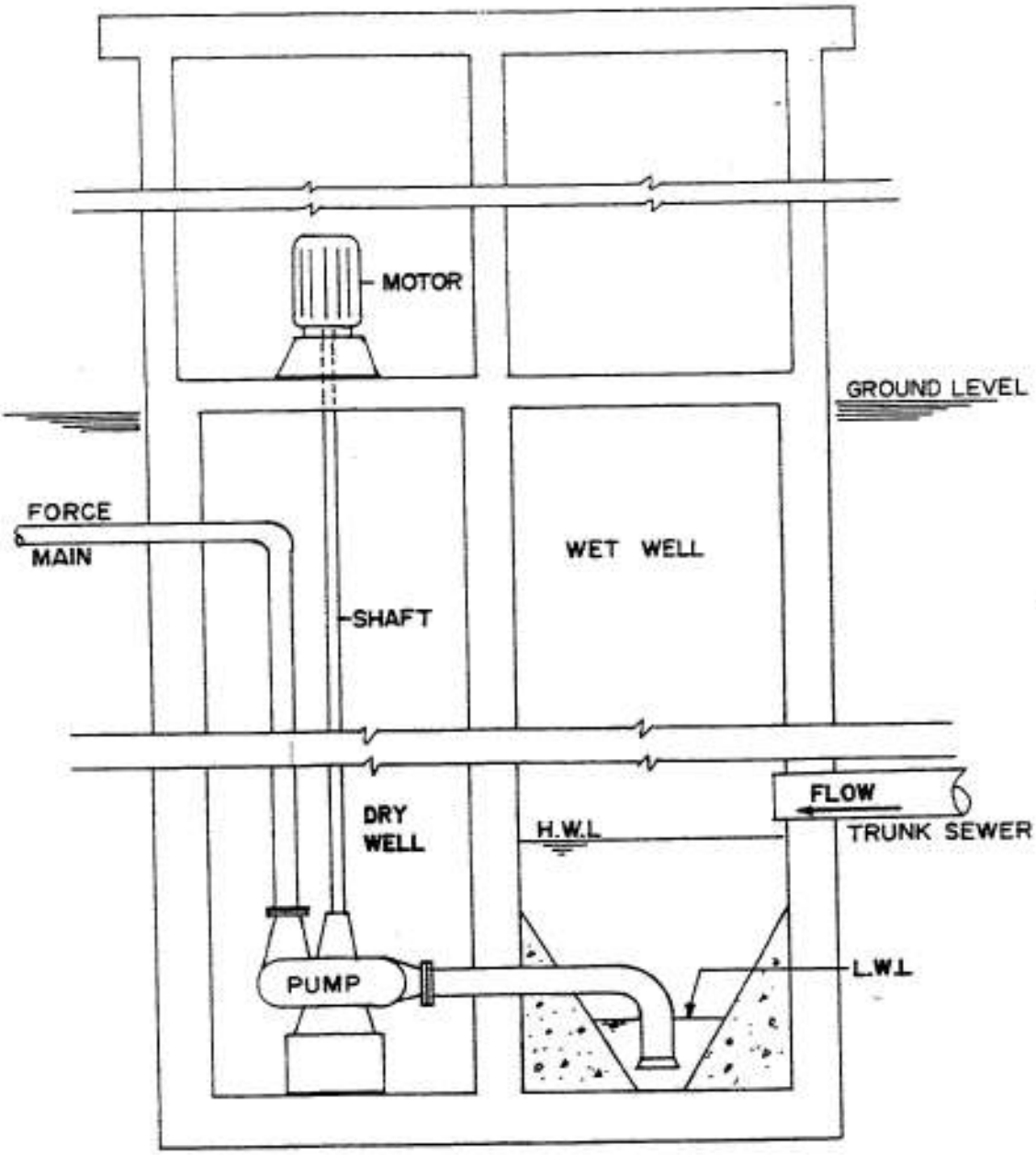
2. Figure 2 - Wet Well type using pre-cast concrete rings for the well and submersible pumps. This type is usually for low to medium flow conditions.

3. Figure 3 - Wet Well/Dry Well using pre-cast concrete wet well rings and a prefabricated pumping package unit. This type is usually for low to medium range flow conditions.

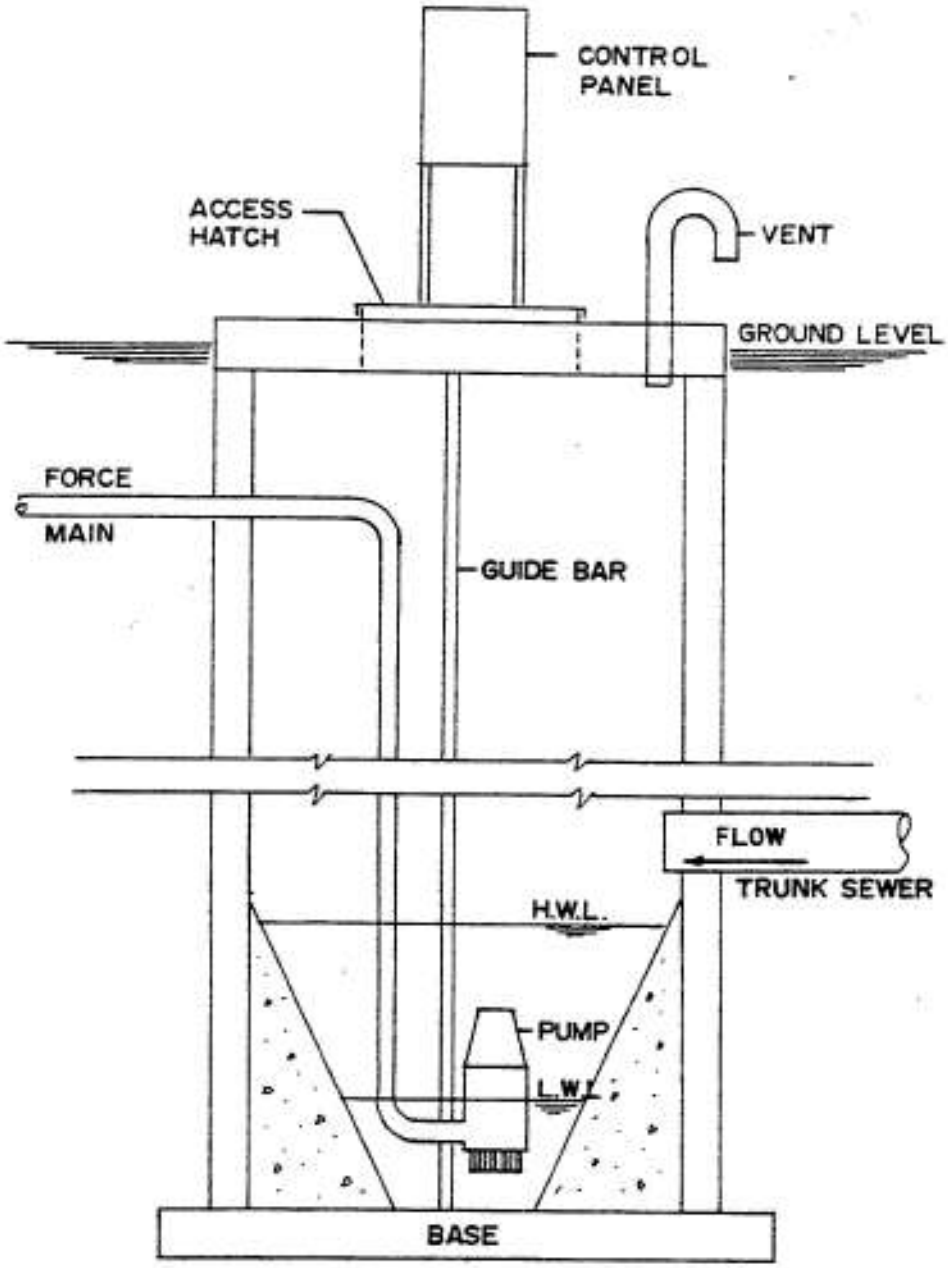
3.4 Station Design

a) Design Flow

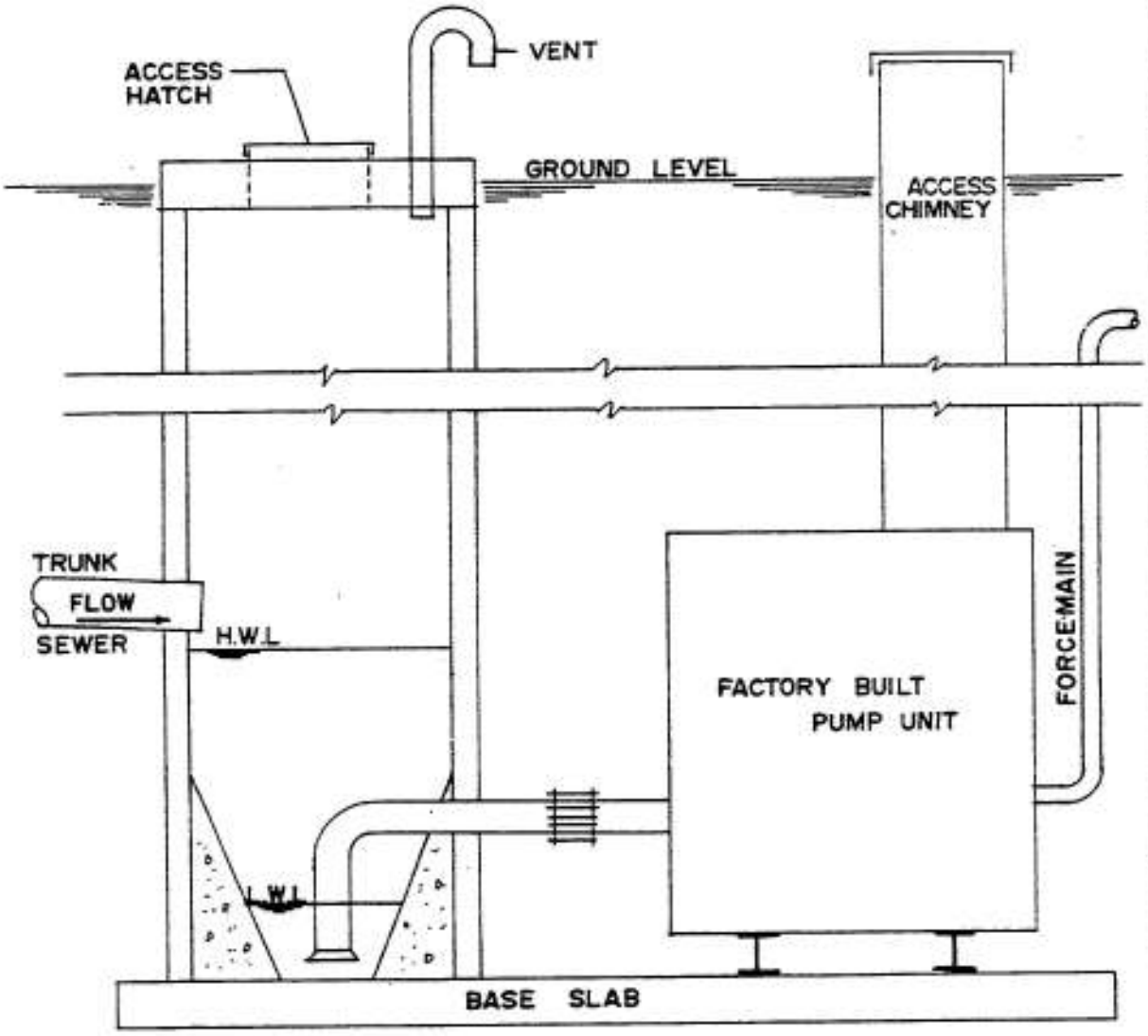
Major design features of any pumping station are of course the size of the wet well based upon design criteria and the capacity of the pumps(s) required to pump through the losses involved. These flow calculations are made in accordance with the design standards in effect in the municipality within the sewer area to be serviced and this information would be available from the designed gravity sewer system in the drainage area. Consideration should of course be given not only to the residential population density but to the possibility of industrial or



WET WELL/DRY WELL STATION - CAST IN PLACE | FIGURE 1



PRECAST WET WELL WITH SUBMERSIBLE PUMPS | FIGURE 2



PRECAST WET WELL & FACTORY BUILT PUMP UNIT | FIGURE 3

commercial development that might be anticipated in the area. Reference to the municipality's Official Plan should be made.

The Ministry of the Environment of the Province of Ontario in its "Guidelines for the Design of Water Storage Facilities, Water Distribution Systems, Sanitary Sewer Systems and Storm Sewers" - May 1979 sets out various requirements which should be taken into account early in the design stage.

Peak domestic sewage flows are to be calculated by the following equation:

$$Q(d) = \frac{PqM}{86.4} + IA$$

Where Q(d) = Peak domestic sewage flow in L/S (litres/second) (including extraneous flows)

P = Design population in thousands

q = Average daily per capita domestic flow in Lpcd (Litres per capita per day)

M = Peaking factor as derived from

i) Harmon Formula

$$M = 1 + \frac{14}{4 + P^{0.5}}$$

ii) Babbit Formula

$$M = \frac{5}{P^{0.2}}$$

or as determined from flow studies for similar developments in the same municipality.

The minimum permissible peaking factor shall be 2.0

I = Unit of peak extraneous flow in L/S per hectare

A = Tributary area in gross hectares.

Example

Population = 287 persons
Using the Harmon Formula

$$M = 1 + \frac{14}{4+.287^{0.5}} = 1 + \frac{14}{4+.536} = 4.09$$

M = peak factor, not greater than 4.0

q = 100 gpcd = 454Lpcd (Litres per capita per day)

$$\text{Peak Flow} = \frac{.287 \times 454 \times 4}{86.4} = 6.03 \text{ L/S}$$

Based on 60 persons per hectare

A = 287/60 = 4.78 hectares

I = .15 L/S . ha, based on the municipality's standard allowance for infiltration

Total Peak Flow = 4.78 x .15 + 6.03 = 6.75 L/S (Litres per seond)

b) Design of Wet Well

The design of the wet well into which the sewage from the gravity mains is based on a number of criteria which should keep in mind the economics of construction in that the size should be adequate for the purpose without overdesigning. The criteria set forth in the W.P.C.F. Manual of Practise No. 9 is as follows: "The desired results can be obtained with a minimum of objections, except for large capacity stations, if the size of the wet well is such that with any combination of inflow and pumping, the cycle of operation for each pump will not be less than 5 minutes and the maximum retention time in the wet well will not average more than 30 minutes."

For our example, the following conditions apply:

i) Pump Capacity = 10.43 L/S

ii) Peak Sewage Flow = 6.75 L/S

iii) Average Sewage Flow = 6.75/4.00=1.69 L/S

For 30 minute average retention, the capacity of the wet well would be 30 x 60 x 1.69 = 3,042 litres or 3.04 cubic metres.

The operating cycle of the pump would be 3042/(10.43 x 60) = 4.9 minutes, plus about an additional minute to pump the continuous flow.

Therefore the criteria of a minimum of 5 minutes of pump "on" time would be satisfied.

During peak flow conditions, the wet well would be refilled in 3006/(6.75 x 60) = 7.4 minutes which represents a reasonable balance of pump "on" and pump "off" time.

Consideration should be given to the amount of storage capacity in the station should pumping failure occur. For the design example providing say 4 hours of storage would increase the wet well by a factor of 8. Often, there is storage available in the gravity main system and the degree to which this storage is available should be determined before providing it in the wet well.

If the standby equipment to be used for operating the station during emergency conditions, such as complete failure or power outages, is of a mobile type that is used at more than one station, the retention period should of course be adequate to allow for transportation time.

c) Design of Pumps

To determine the proper pump(s) to be used in the station, the following information is required:

1. Pump Manufacturer's Data

Consideration should be given to selecting non-clogging sewage pumps. The pump information includes a performance curve, which relates the total head (metres) against capacity (litres per second) together with Motor H.P. and electrical power requirements. It is accepted practice to size the two pumps in a station to be capable of each pumping the maximum flow to the station.

2. Force Main Pipe Manufacturers Data

Pipe manufacturers provide friction loss data on various sizes of pipe in the form of graphs or nomographic charts. From these charts, the head loss for various rates of flow can be determined.

3. Pipe Fitting Losses

The supplier of various fittings used in the internal piping in the station provide equivalent "pipe lengths" for various fittings to permit the calculation of total station losses. These would consist of the equivalent length of pipe for such items as gate valves, check valves, tees, bends and other piping items between the discharge side of the pump and the force main. In some cases, particularly in larger systems, inlet losses may also be significant and contribute to losses. In smaller stations they are, however usually not considered.

4. Design Example

In the following example, the design is based on a new pumping station required to pump a maximum flow of Q = 10.43 L/S through 527.3 M of 100 mm P.V.C. forcemain connected to 80.8 M of existing cast iron pipe force main, thence to discharge. The wet well lower water level after all pumps are stopped is 91.75 and the overflow level is 95.4. Discharge elevation is 98.6 M. Overflow level is either the actual overflow pipe invert or the level in the station at which basement or other flooding will occur.

Calculation of Total Dynamic Head - Q = 10.43 L/S

Static Lift from 91.75 to 98.6 = 6.85 M

Station Loss: The equivalent length of fittings is 24.7 metres and the head loss for CIP @ 10.43 L/S is 3.12 M/100 M

Loss = 3.12x24.7/100 = 0.77 M

100 mm P.V.C. Loss

Length of P.V.C. is 527.3 M
and loss is 1.31 M/100 M
Loss = 527.3x1.31/100 = 6.91 M
Velocity = 1.233 M/S
(4.04 f.p.s.)

100 mm C.I.P. Loss

Length of C.I.P. = 80.4 M
and loss is 3.12 M/100 M
Loss = 80.4x3.12/100 = 2.51 M
Velocity = 1.235 M/S

Total Discharge
Head required =17.04 M

Pump manufacturers performance curves are used to select pump model and power requirements for a flow of 10.43 L/S and a total discharge head of 17.04 metres.

It should be noted that the head losses are determined by use of Hazen Williams Tables which values relate to the following conditions:

a) C = 120 and low water level in the wet well

b) C = 130 and median water level over the normal operating range in the wet well

c) C = 140 and overflow level in the wet well

It should be noted that the 'C' value or friction loss coefficient is related to the size, material and age of the forcemain conduit. These factors are discussed in the Hazen Williams Tables.

The point on the graph represented by these requirements should be at or near the pump curve but not above it. This point is referred to as the true operating point of the pump.

System Curves

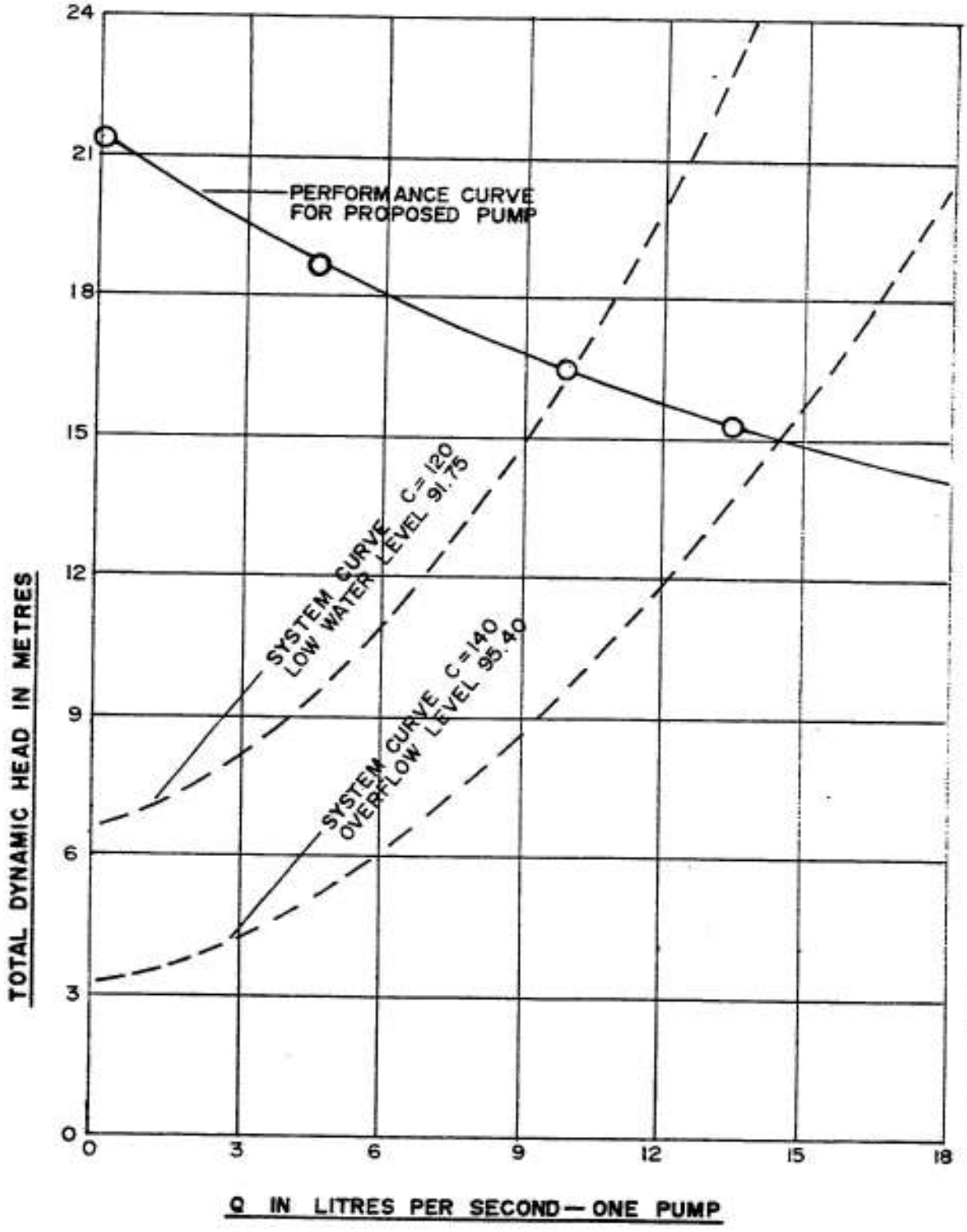
The Ministry of the Environment of the Province of Ontario application for pumping station approval requires the submission of pump selection data, including the manufacturers' pump performance curve and three system curves, two of which are shown on Figure 4. The first system curve is calculated for the low water level condition (Elev. 91.75) using a value of C = 120 and the second for overflow condition (Elev. 95.4) using C = 140.

The system curve calculation for low water level, various pump discharge rates and C=120 is shown below. The various head losses are obtained from manufacturer's nomographic charts.

System Curve C = 120 - Low Water Level 91.75

Q	0 L/S	3.16 L/S	6.32 L/S	9.48 L/S	12.64 L/S	15.80 L/S
Static	6.88	6.88	6.88	6.88	6.88	6.88
Station	-	0.10	0.30	0.64	1.10	1.77
100 mm P.V.C.	-	0.76	2.71	5.79	9.81	15.94
100 mm C.I.P.	-	0.27	1.01	2.10	3.57	5.82
T.D.H.	6.88	8.01	10.90	15.41	21.36	30.45

The system curve calculated for overflow condition will be relatively parallel to the first curve, but will be lower and to the right, due to the decrease in static head resulting from the higher water level in the station. In this curve the static head will be 98.60 - 95.40 or 3.20 metres. The slope of the curve will vary to some degree due to the value of C = 140 being used. It will be noted from the second curve that the TDH is less, and the value of Q is greater.



PUMP PERFORMANCE CURVE | FIGURE 4

Although not normally shown on manufacturers' data, as the flow (Q) increases more power is required and the rated motor horsepower can be exceeded under some conditions. It is general trade practice that if the two system curves are within the limits of the performance curve supplied, the motor will be adequate for the anticipated capacities. Confirmation should be obtained from the pump supplier on this requirement.

It is good practice to select a pump whose true operating point is near the middle of the curve to provide adequate allowance for conditions resulting from abnormally high or low flows.

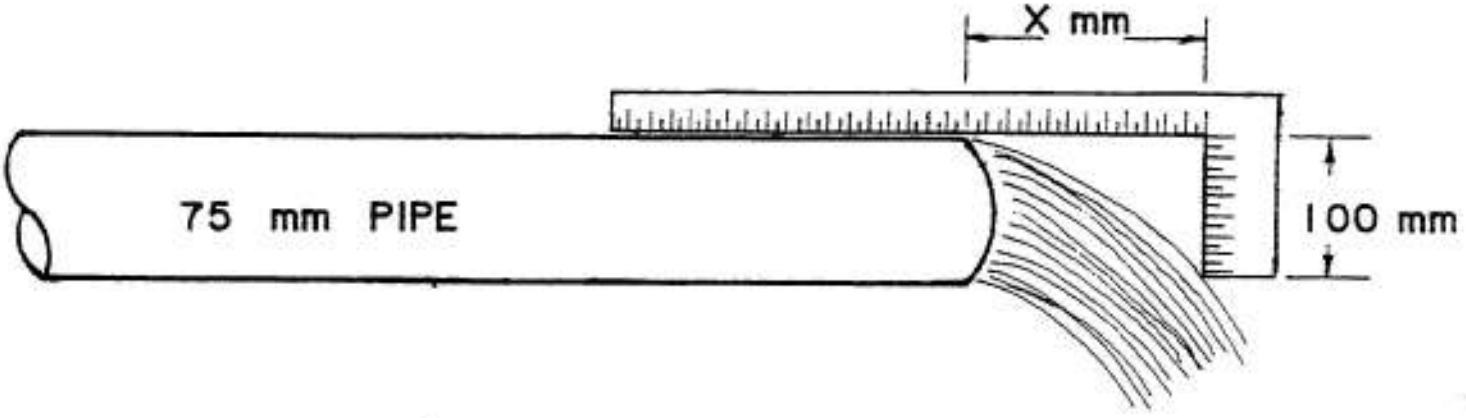
DETERMINING THE APPROXIMATE CAPACITY OF A PUMP.

To estimate the pumping capacity of any given unit, construct an "L" shaped measuring device similar to that shown in the accompanying sketch. The shorter side should be 100 mm long. The longer side may be any convenient length marked off in mm. With the water flowing from the horizontal open discharge, place the long side of the "L" along the top of the discharge pipe allowing the shorter side to hang downward as shown in the drawing. Slide the "L along the pipe until the 100 mm length barely touches the flow of water. Note the distance "X" traveled by the flow of water before it drops 100 mm. Presume, for example, that the distance is 375 mm and the inside diameter of the pipe is 75 mm. Consulting the table as shown, find 375 mm in the column at extreme left labeled "Horizontal Dist. "X" in mm". Then move horozontally to the right of the column showing the pipe diameter being used (75 mm). The discharge rate is found to be 13.9 L/S.

DISCHARGE RATE (Litres per second)

Nominal Pipe Diameter (mm)

Horizontal Distance "X" (mm)	75	100	125	150	200	250	300
100	3.7	6.3					
125	4.6	7.9	12.3				
150	5.5	9.5	14.8	21.6			
175	6.4	11.0	17.3	25.3	43.9		
200	7.4	12.6	19.7	28.8	50.3	80.2	
225	8.3	14.2	22.2	32.3	56.8	90.1	125.7
250	9.2	15.7	24.7	36.0	62.8	100.7	140.0
275	10.1	17.3	27.3	39.7	69.3	110.5	166.5
300	11.1	18.9	29.5	43.1	75.7	121.1	174.1
325	12.0	20.4	32.2	46.9	81.8	131.0	181.7
350	12.9	22.1	34.5	50.7	87.8	140.8	196.1
375	13.9	23.6	37.1	53.7	94.6	151.4	210.4
400	14.8	25.3	39.4	57.5	100.7	160.5	224.1
425	15.7	26.9	41.6	61.3	106.7	171.1	237.7
450	16.7	28.4	44.7	65.1	113.6	180.9	252.1
475	17.6	29.9	46.9	68.9	119.6	190.8	265.0
500	18.5	31.4	49.2	71.9	125.6	201.4	280.1
525	19.4	32.9	51.9	75.7	132.5	212.0	
550		34.8	54.5	79.5	140.9	221.0	
575			56.8	83.3	144.6	231.6	
600				86.3	151.4	242.0	



d) Forcemain

The sewage forcemain is constructed much in the same manner as a watermain, with the pipe being placed below the level at which it could be affected by frost. The size of the forcemain should be calculated to provide for a minimum velocity of 0.8 metres per second. The maximum velocity as indicated by the Ministry of the Environment of Ontario is 4.0 metres per second. The reduced cost of pumping due to a larger diameter forcemain should be weighed against the increased cost of a larger diameter forcemain.

The velocity in the preceeding example for 10.43 L/S in a 100 mm diameter forcemain is 1.233 M/S or 4.04 f.p.s. This would be in tne lower range of the 0.80 to 4.0 M/S guidelines. Using of a 150 mm diameter forcemain would result in a velocity of 0.50 M/S which is less than the recommended minimum. The use of a 100 mm diameter forcemain in this case would be the optimum selection.

In general, the cost/benefit analysis associated with the sizing of the forcemain would compare the increased construction cost of a larger diameter forcemain against the reduced costs of possibly smaller pumps and in all cases of the reduced power requirements associated with less total discharge head. In the case of power costs the savings over say 20 years is related to a present value which takes into account increased charges for electrical power and interest charges over the 20 year period.

The forcemain should discharge into a manhole at the point of the highest elevation of the forcemain and be drained then by gravity flow. Care should be taken to ensure that no point of the forcemain between the pump and the point of discharge into free air is above the discharge invert elevation, to prevent the formation of air locks.

Should the elimination of high points in the forcemain not be possible, air relief valves should be installed. These are located in a chamber and automatically discharge trapped air.

Between the pump discharge piping and the forcemain a check valve should be installed to prevent back flow when the pump(s) are not operating. A gate valve is also required to isolate the check valve for repairs. This point will be covered in more detail in the details of station design. A capped connection should be provided to permit pumping of sewage by portable pump from the wet well into the forcemain in times of station failure, by bypassing the permanent pumps.

e) Provision for Overflow

Present regulations do not permit overflow from sewage pumping stations into receiving waters and consequently care must be taken to prevent excessive damage to equipment in the station and to provide facilities for removing the sewage in cases of emergency. In particular, if overflow does occur, the design of the station should preclude sewage from entering the dry area where the equipment is located, in the extreme event that the wet well was completely flooded.

There are normally two primary reasons for station failure, being firstly the result of power supply being unavailable, either from the electrical distribution system or as a result of breakdown within the station, or secondly as a result of failure of the pumps or other equipment. Generally the second problem, that is pump failure is not too serious as present day stations should have at least two pumps installed so that one may continue to operate in such circumstances. Each pump should be capable of pumping the maximum flow to the station. The lack of power is, however, a problem that can only be overcome by supplying an additional source of energy or alternate methods of operating the pumps. This can be accomplished by either a built-in diesel or gasoline operated electric generator or a built-in diesel or gasoline engine operating the pumps by direct drive. In addition, the station can be designed so that a portable generator can be connected to the system to provide power for pumping the well at reasonable intervals. In this method a single portable generator can adequately serve three or four stations in the municipality depending upon the distance that they are apart.

Facilities can be installed so that a portable pump on the surface can be used to empty the wet well, thence into a connection to the forcemain or alternately by a length of hose to the nearest gravity manhole. In most cases, however, the distance between the pumping station and the nearest manhole may be such that the hose method is not practical, particularly in temperatures below freezing or where it will have an adverse affect on vehicular traffic. As a last resort arrangements can be made to empty the well into tank trucks to be transported to an adjacent satisfactory point of disposal into the gravity system.

f) Design Details - Dry Well Stations

The following design details should be given adequate consideration by the engineer:

1. Wet Well

- The floor of the wet well should be sloped to the pump suction inlets to ensure that a buildup of grit and solids does not take place.

- The access to the wet well should be properly designed to "encourage proper and regular inspection and maintenance by the municipal staff". If the wet well is difficult to enter and proper provision for inspection landings and lighting are not provided, there will be a tendency to overlook this necessary feature of ongoing maintenance.

  The wet well is a "confined space" according to the Occupational Health and Safety Act.

- As noted previously, the design of the station should be such that there is a physical barrier between the wet well and the dry well or pump chamber so that there is no possibility of overflow into the operating area when an emergency gravity overflow takes place or is imminent.

- Ensure that a highwater alarm is installed in the wet well and that it is connected to provide a remote indication of the condition. This feature of design will be further discussed under the section entitled "Controls and Alarms" but the alarm switch should be operated from an independent power supply and be connected to the alarm board by a telephone line or other independent system.

- On larger stations the wet well should be divided and valved so that half of the well can be drained for inspection or maintenance while the other half remains in operation to permit the plant to function normally.

- In larger stations, consideration should be given to the provision of washroom and/or wash-up facilities where possible.

- Wet wells are classified as Class 1, Group D, Division 11, Hazardous Location and the requirements of the Ontario Electrical Safety Code, Section 18, must be satisfied for all electrical installations in wet wells, or the equipment must be CSA approved for use in sewage wet wells.

2. Pump Chamber

The design of the pump chamber or dry well portion of a dry pit station should include consideration of the following items:

- The dry well floor should be gently sloped to a gutter along one or more walls and sloped to a sump, to ensure that the floor is kept relatively dry. Water in the sump is removed by a sump pump discharging to the wet well and the discharge pipe should be equipped with at least two check valves mounted in series to prevent any wet well back flooding from discharging through the discharge pipe into the dry well in the event that one check valve should stick in the open position.

- In addition, the sump pump discharge pipe should enter the wet well at an elevation at or above the wet well maximum overflow level or floodlevel to prevent the possibility of backflooding.

- Where a standby engine or engine generator set is installed, adequate provision must be made for the discharge of exhaust fumes to the atmosphere if this equipment is mounted below grade to a depth that could be hazardous from carbon monoxide accumulation. The ventilating system should be equipped with an air flow sensing switch that will shut off the engine in the event that the ventilation system is not functioning.

- A "stop-lockout switch" should be provided at each pump to prevent pump operation during maintenance. Most electrical codes require this type of switch to be installed within the sight of the electric motor. It is also advisable to consider installing a second switch adjacent to the pump itself, particularly when the pump might be somewhat remote from the motor. In the case of a pump being driven through universal shafting from a motor on the floor above, the switch permits access to the pump universal shafting by maintenance staff with the assurance that the shaft will not be placed in rotation and that the pump and impeller can be opened and inspected under the same conditions. This switch eliminates the necessity of going to the upper floor to turn off the pump switch or neglecting to do so and undertaking work without that precaution.

- Dehumidification of the station dry well has become an important consideration in pumping station design. It is necessary to control the humidity level to reduce the rate of deterioration of equipment, piping and paint. This is one of the reasons we have said that the station floor should be sloped to a sump and that all runoff or spillage be properly controlled and conducted to the sump pump.

- During the time of station design, the possibility of future maintenance, both major and minor, must be considered and costs to be borne in the future can be considerably reduced by providing proper access of facilities in the initial construction of the structure. In particular, openings of sufficient size should be provided in upper floors to permit pumps or motors or other major items of equipment to be hoisted out of the station for transportation. In a larger station, a small travelling hoist beam could be installed to permit the equipment to be raised and moved out of the building for loading on a truck to be transported to a shop for repairs. Shop maintenance is usually less expensive than field maintenance in this type of work.

3. Pump Discharge

Check valves should be equipped with outside levers to indicate the valve operation and to permit the application of springs or weights to accelerate the valve closure. In some applications, it is advisable to accelerate the check valve closure to control the effects of waterhammer when the pump stops.

- Pressure gauges should be provided on each pump discharge flange to indicate pump pressure. These gauges should be equippped with a shutoff cock and a diaphragm seal to isolate the gauge liquid components from the sewage. Condensation of bourdon tube gauges will quickly accumulate sediment preventing their proper operation unless the seal prevents the entry of sewage into the gauge. The sewage side of the diaphragm can be easily cleaned periodically but a gauge cock should be fitted between the diaphragm and the pump so that the gauge can be left off except when being read. These gauges are useful as a means of indicating a change in pump discharge characteristics that might occur, possibly due to impeller wear.

- In a dry pit station when the pumps are driven through universal shafting between the motor and the pump - i.e., with the motor at grade level and the pump some distance below in the well, - and the length

of the shaft or pit depth requires an intermediate steadying bearing, access must be provided for lubrication of the bearing and the universal joint. This lubrication will normally be about once every six months. Extended grease lines from the floor above or below to the bearing should be avoided as grease tends to harden in these lines and not flow as required. In any event, it is not possible to have such a grease line to the rotating universal joint.

3.5 Emergency Operation and Facilities

Since sewage pumping stations cannot be allowed to overflow in the event of operational failure, the situation becomes critical when the facility is not operating due to pump and/or power failure. As noted previously, the supply authority can quite often supply power to the area from an adjacent grid if the normal grid service is disrupted. Provision should, however, be made to supply power under emergency conditions from either a gasoline or diesel powered generator constructed as a part of the pumping station or facilities should be available to connect a portable electrical power source from a mobile generator. In addition, during a period of power failure, pumps may be run directly from a gasoline or diesel engine.

As an alternative to these procedures, provisions can also be made for temporary pumping from the wet well to the gravity sewer, either through the force main or by a flexible hose, depending on the type of construction used in the station. Depending on the storage capacity of the gravity mains leading to the station plus the wet well, periodic pumping into a tank truck for removal may be adequate, depending on flows, in emergencies.

3.6 Monitoring and Alarms

Since most sewage pumping stations are unmanned and maintenance staff is only present during routine inspection, it is advisable to install monitoring and alarm systems that will provide necessary information to Operations staff located at some other site, possibly the Sewage Treatment Plant. Almost any type of information that may be desired can be transmitted from the pumping station to the supervisory centre and a considerable number of functions can be transmitted over one telephone or communications line. Such

information as No. 1 Pump running, No. 2 pump running or Diesel Generator running is usually desirable and this information can be recorded on a circular chart, using either the same chart for all three pieces of equipment or one chart for each unit. If running time recorders are on each pump at the station, that information, together with the chart recording, can permit the calculation of the "on" time of each unit.

An indication of highwater in either the wet well or dry well is usually also a desired addition to monitor as well as the failure of electrical power supply to the station. One type of monitoring equipment can provide sixteen different status readouts on each printed circuit board that is fed by one communications line from the station to a scanner that constantly looks at each signal received in succession. If a warning is received, the coloured light designating the problem remains on so that the operator may be aware of the problem. In addition to those matters monitored as noted above, a communications failure is also included so that a breakdown in the communications line from the plant may be recorded. It may not be necessary to have some of these functions recorded individually and the signal may be combined to indicate that there is a problem in one of four or five areas as may be decided. For example, one indicator light could show that an overload had occurred on No. 1 or No. 2 pump and that the circuit breaker on either of these pumps had been tripped and since it is usually not important as to which failure has occurred, one indicator light could be used to show that one of the four conditions existed.

If the maintenance system in the municipality is such that 24 hour watch is not provided by the municipal staff and/or weekends or holidays are not staffed, this system can be equipped with an alarm system that will provide an indication at a remote location such as a police station so that the operating staff may be alerted. Here again the alarm usually indicated that there is a problem and any one of a number of functions in any pumping station may have caused the alarm to be activated. Since the maintenance staff must be called out in any event, it is not necessary to provide a complicated system to determine the nature and location of the failure, at that time.

Bibliography

The designer should refer to the Ministry of the Environment of the Province of Ontario's "Guidelines for the Design of Water Storage Facilities, Water Distribution Systems, Sanitary Sewage Systems and Storm Sewers" - May 1979, for information on Ministry requirements for pumping station design.

Additional information may be obtained from the "WPCF Manual of Practice No. 9 - Design and Construction of Sanitary and Storm Sewers" - Fourth Printing 1976, prepared by the Water Pollution Control Federation and the American Society of Civil Engineers.

A number of specifications have been prepared by the Ministry of the Environment to assist designers of sewage pumping stations. The following is a list of those specifications:

a) "Standard Specification for Factory-Built Underground Sewage Pumping Stations" MOE Spec. No. 1

b) "Standard Specification for Diesel Generator Sets" MOE Spec. No. 2.

c) "Standard Specification for Submersible Sewage Pumps, Auxiliary Equipment and Controls" MOE Spec. No. 3.

d) "Standard Specification for Dry Pit, Non-Clog Vertical Sewage Pumps" MOE Spec. No. 4.

e) "Standard Specification for Magnetic Flow Meters for Water and Sewage Works" MOE Spec. No. 9.

Reference to the Water Pollution Control Federation Manual of Practice No. FD-4, Facilities Development entitled "Design of Wastewater and Stormwater Pumping Stations" - 1982, could also be made.



4.

WATER DISTRIBUTION SYSTEM DESIGN

CHAPTER NO. 4 - WATER DISTRIBUTION SYSTEM DESIGN

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CHAPTER NO. 4 - WATER DISTRIBUTION SYSTEM DESIGN

4.1 GENERAL

Society is totally dependent upon the availability and accessability of good quality water, consistent with guidelines set forth by Environment Canada and the Ontario Ministry of the Environment.

Once a source, lake, river, etc., has been determined, along with an appropriate treatment facility, the question of transporting the treated water to the potential user becomes the task of the Engineer. The facility commonly used for the purpose is referred to as a "water distribution system". The principal components of such a system are generally, trunk mains, distribution mains, distribution reservoirs, booster stations, valves, hydrants, main line meters, service connections, customer meters and finally a suitable outlet. Distribution may be by gravity, by pumps or a combination thereof.

4.2 ESTIMATING WATER QUANTITY REQUIREMENTS

It is taken for granted that a community requires a sufficient supply of water for present and future needs. There are very few communities of more than 1,000 population that do not have a municipal water supply system. Design considerations may differ

somewhat, depending on whether the supply and system is entirely new or an extension to an established system. An entirely new community can be designed as a completely integrated system using the latest developments and equipment. Suburban development sometimes would fall under this category as do communities which are developed in connection with new industrial locations. Old communities which must extend or enlarge a water supply system are faced with the problem of integrating new works with existing facilities and must design accordingly.

In planning any water supply system, new or enlarged, a basic unit of capacity must be used in the design criteria. The unit most commonly used is expressed as the annual average litres per capita per day (lpcd).

The LPCD unit varies widely from one community to another, depending on many factors such as type of community, use of water for domestic, commercial, and industrial establishments, leakage, etc., and the value varies from year to year depending on the degree of conservations and changes in industrial/residential ratios. Most important in any consideration of design criteria is the period for which the design must be developed. The size and design period depends on two factors: 1) probable life of the system being planned, and 2) method by which construction is to be financed. If the proposed addition has a long life expectancy, and will benefit future generations, it should be planned and financed over an extended period so that present generations will not be saddled with a major portion of the cost when the benefit extends to future generating. If the expansion project is of a relatively short life,

it should be planned and financed accordingly, but it is unwise and uneconomical to plan and build for a very short period. If the project is to be financed by a debenture issue to be retired out of earnings, the life expectancy of the work should be no less than the debenture life. If the project is financed out of surpluses, its life should be equal at least to the period required to rebuild the surplus out of future earnings.

Having determined the period for which the expansion or construction is to serve, the next step is to estimate the capacity required at the end of that period. This estimate will depend on two factors: a) population at that future time, and

b) per capita consumption at that time. Prediction of the population to be served at the future date for which plans are made is generally made on the basis of past performance, modified by such factors as may be foreseeable. Many municipalities now have an official plan, and it may include the forecast of population at certain horizon years. If the official plan is reasonably current, and the population projections still seem to be valid, this probably is the best document on which to base future water needs. Whatever the population estimate, it is probably good practice to plan future facilities for a population greater than the predicted value or to make provision for future expansion in case population growth and water demand become appreciably greater than predicted.

Some of the general consumption factors that have to be taken into consideration are: a) size of city both present and future b) make-up of the community - is it residential, residential and commercial, commercial and industrial? c) kind of industries in

the community; d) climate - the greater the average temperature range, the greater the use of water. When the summer months are hot and dry, water consumption will increase appreciably. Water consumption per capita is greater in areas of low rainfall and in periods of drought; e) cost of water -- when water rates are increased, there is generally a decrease in consumption, and the greater the increase in rates, the greater the percentage decreases in per capita water consumption; f) miscellaneous factors - quality of water, sewers, average water temperature, attitude of water utility toward lawn sprinkling, etc. When the quality of water is improved, per capita consumption increases. Previously unsewered residential areas show increased water usage after the installation of a sewer system.

Some of the specific factors which affect consumption are:

a) service pressure - the higher the average pressure, the greater the per capita consumption;

b) percentage of total services which are metered;

c) loss and waste tolerated by the water utility management;

d) public use of water for public buildings, sprinkling, street flushing, fire protection.

4.2.1 Quantity Consumption Data

Many designers prefer to base future needs upon past demands in the various consumer classifications rather than on total population estimates. Water systems should have accurate, continuous and up-to-date records of the number of customers served in the various service categories. Proper records of the amount of water

served these customers over the years, modified by proper analysis of changing conditions form a more accurate and logical basis upon which to determine future demands than perhaps basing estimates, population and per capita uses.

Because of the fact that estimate of population and rate of growth are extremely difficult to ascertain,and the existence of several methods, judgement must be exercised by the Engineer regarding the best applicable method. A thorough knowledge of the service area, its environs, its trade territory, the rate of growth of its industries, the type of development taking place, will all enter into estimation of an area's future population.

While there are several methods of determining growth, for the purposes of this manual, it will suffice to simply list the same, with a brief outline of one:

Arithmetic method - This method assumes a constant rate of growth. To test this hypothesis, the formula $dp/dt = K$ is utilized, where dp/dt is the rate of change of population with time, and K is a constant. The value of K is established graphically or from data of actual population in successive censuses as $K = P/t$. Future population is then estimated from $Pt = Po + Kt$ where Pt is future population,Po present population and t is projected time.

Other methods which can be obtained from textbooks are:

Uniform Percentage Method

Curvilivear Method

Logistic Method

Declining Growth Method

Ratio Method.

4.3 PRESSURE REQUIREMENTS

4.3.1 Maximum and Minimum Operating Pressures

It is a generally accepted practice to design water supply and distribution systems such that the normal operating pressure ranges between 350 kPa and 550 kPa (50 to 80 psi) under a condition of maximum daily flow.

The maximum pressures in the distribution system should, in general, not exceed 700 kPa (100 psi) to avoid damage to household hot water tanks, rapid wear of tap washers, unnecessary energy consumption, etc. Where localized areas must have pressures above this level, the homes affected should be provided with individual pressure-reducing valves on their services.

The distribution system should be sized so that under maximum hourly demand, the pressures do not fall below 275 kPa (40 psi). Under conditions of simultaneous maximum day and fire flow demands, the pressure should not drop below 140 kPa (20 psi).

4.3.2. Transient Pressures

The distribution piping system must be designed to withstand the maximum operating pressure plus the transient pressures to which it will be subjected. Transient pressures are caused by rapid valve operation, pump start-up and shut-down, power failure, etc. Wherever possible, the design of pumping systems should be such that surges caused by pumping station operations are minimized.

As a minimum allowance in the distribution system, it is recommended that the pipe strength be such that it can withstand the maximum operating pressure plus the pressure surge that would be created by instantaneous stoppage of a water column moving at 0.6 m/s (2 fps). The pressure created by such an event will vary depending upon the diameter, wall thickness and material of pipe used in the distribution system. When calculating transient pressures for flexible pipe materials, the designer is cautioned that celerity values given in texts, manufacturer's catalogues and other sources of information are usually for the unrestrained condition. When buried, such pipe materials will exhibit higher effective celerity values and correspondingly higher transient pressures. Celerity values utilized in transient analyses should be increased for such pipe materials for buried conditions and a rule of thumb is to use twice the value calculated from the celerity equation. The Ministry is currently researching this matter and further guidance will be provided when available.

4.4 CLASSES OF WATERMAINS

Watermains are normally considered to fall into three classifications:

1) Suply mains (also called a transmission or a trunk main) to carry water from source or pumping station to sections of the city.

2) arterial mains or secondary feeds which interconnect the supply mains with the distribution grid.

3) Minor distributors make up the grid of pipes between the secondary system and customer services, fire hydrants, etc.

If a supply main is being designed, the water pressure available to it to create flow would probably be either the minimum water level that could be expected in a reservoir from which it may be connected, or, alternately, the discharge pressure from a pump at the rated capacity of the pump. If you are designing an arterial main, the pressure available would be dependent upon the location of the connection to the trunk transmission main. The operating records of most existing water systems would provide the information as to to what pressure is available during various flow conditions. If the pipe being designed is a minor distributor, once again the pressure available to create flow should be available from distribution system records at the point on the arterial main at which the minor distributor will be connected.

4.5 PIPE SIZE REQUIREMENTS

a)Sizes of pipes in the distribution system are influenced by the total water demand for fire flow requirements on a maximum demand day. This capacity is usually taken as the fire flow demand plus the average flow for other purposes on the day of maximum demand.

b) Minimum pipe sizes may be indicated largely by recommendation of the I.A.O. Actual pipe sizes selected will depend on the determination of the required carrying capacity and pressure required for each area to be served.

c) In practice, it is customary to select the next larger nominal pipe size above the theoretical size indicated from the determinations of the required flows in the pipe systems.

4.5.1 Flow in Pipes

For practical purposes, the flow carrying capacity of a pipeline depends upon the pressure available (or that can be made available) to create flow, the inside diameter of the pipe, the condition of the interior of the pipe (relative roughness) and the length of pipe through which the water must be delivered. The relationship between these variables is expressed by what is called the Hazen-Williams formula, which has become more generally used than any other in the water supply field.

The formula is: $v - k(C)\ r0.63\ s0.54$

where v is velocity in pipe in distance per second, r is the hydraulic radius of pipe in the distance unit common to velocity, s is the hydraulic gradient, C is a constant depending upon the relative roughness of the pipe, and k is an experimental co-efficient and unit conversion equal to 0.849 (m/s.m) or 1.318 (ft./s.ft.) for example. The exponents of the formula were selected from experiments with pipes and channels ordinarily used in waterworks practice.

Below is a table containing different values of C:-

Hazen-Williams co-efficients for various pipe materials

Description of the Pipe	Values of C
Extremely smooth and straight	140
Cast Iron	

New	130
5 years old	120
10 years old	110
20 years old	90 - 100
30 years old	75 - 90
Concrete or Cement-lined	120 - 140
Welded Steel	130
Riveted Steel	110
Plastic	150
Asbestos Cement	120 - 140

Some of the formulae are difficult to use, however, tables, charts, and nomographs are available which may simplify the procedure. Most hydraulic textbooks, and many pipe supplier product manuals, provide charts that can be used for design purposes.

The level of fire protection to be provided by a municipally-owned water supply system is the decision of the municipality. In general, the minimum fire flow, as recognized by various fire underwriting groups is 30 L/s (400 gpm) and to receive credit, the water system must be simultaneously capable of satisfying the maximum day demand.

To estimate the fire flow requirements for a particular structure or area of a municipality, the designer should refer to a guide such as, "Water Supply for Public Fire Protect - A Guide to Recommended Practice," prepared by Public Fire Protection Survey Services, Insurers' Advisory Organization, 180 Dundas STreet West, Toronto, Ontario.

4.6 FIRE PROTECTION REQUIREMENTS

The level of fire protection to be provided by a municipally-owned water supply system is the decision of the municipality. In general, the minimum fire flow, as recognized by various fire underwriting groups is 30 L/s (400 gpm) and to receive credit, the water system must be simultaneously capable of satisfying the maximum day demand.

4.7 HYDRANT REQUIREMENTS

Being a factor in the level of fire protection to be provided, the provision, spacing and location of fire hydrants is largely the decision of the municipality. The designer should, therefore, acquaint himself with the requirements of the municipality involved.

The required hydrant spacing decreases as the fire flow requirement increases. Hydrants must, therefore, be placed much closer together in high risk areas, such as downtown core areas, than in residential areas with detached homes.

In residential areas, the line spacing for hydrants is normally recommended to be 120-150 m (400-500 ft.). For a more detailed discussion of hydrant spacing requirements reference should be made to "Water Supply for Public Fire Protection - A guide to Recommended Practice".

Fire hydrants should only be installed on watermains capable of supplying fire flow requirements. The hydrant leads should be 150 mm (6 in.) diameter pipe.

To allow for hydrant maintenance and repair with a minimum of disruption, it is preferred that all hydrant leads be valved to allow for ease of maintenance.

4.8 HYDRAULIC DESIGN

4.8.1 Design Water Demands

Water supply systems should be designed to satisfy the greater of either of the following demands:

a) maximum day plus fire flow

b) peak rate (maximum hourly demand).

The maximum day demand is the average usage rate on the maximum day. The fire flow demand will vary with the size of the municipality (chance of multiple fires at any time) and the nature of development (type of construction materials, building height and area, and density of development). The peak rate demand is the short-term demand placed upon the system by usages other than fire fighting. The peak rate demand is usually taken as the average water usage over the maximum hour. Wherever possible, peaking factors based on usage records for the water supply system should be used, but when such records do not exist or are unreliable, factors such as those given in Appendix A may be used.

4.8.1a Domestic Water Demands

Domestic water demands will vary greatly from municipality to municipality. Depending upon such factors as presence of service metering, lawn watering practices, water quality, leakage, etc., average daily per capita consumption rates can vary from less than 180 litres (40 gallons) to more than 450 litres (100 gallons).

For design purposes, existing reliable records for average, maximum day and peak rates should be used wherever possible. Otherwise, average daily per capita water demands of 280 to 450 litres (60 to 100 gallons) are recommended. Minimum rate, maximum day and peak rate factors for large systems should e in accordance with the values shown in Appendix A.

For small residential developments of less than 500 persons, a peak domestic demand of 0.076 to 0.23 L/x (1 to 3 gpm) per dwelling unit should be used for design. Demands as high as 0.45 L/s (6 gpm) per lot have been experienced in estate-type subdivisions during lawn watering periods.

4.8.1b Commercial and Institutional Water Demands

The water demands for commercial and institutional establishments vary greatly with the type of water using facilities present in the development, the population using the facilities, the presence of metering, etc.

In general, the method to use to estimate water consumption for large commercial areas is to calculate a population equivalent for the area covered by the development, then calculate the water demands on the same basis as discussed in the previous section. A

population equivalent of 90 persons per hectare (36 ppa) is often used.

For individual commercial and institutional uses, the following unit demands are commonly used. Where a range is stated, the lower figure is a minimum requirement.

	Water Usage (Ave. Daily)
Shopping Centres	2500-5000 L/1000 m^3.day
	(based on total floor area)
	(50 - 100 gpd/1000 ft^3)
Hospitals	900-1800 L/bed. day
	(200-400 gpd/bed)
Schools	70-140 L/student. day
	(15 to 30 gpd/student)
Campgrounds	225-570 L/campsite. day
	(50to 125 gpd/campsite)

When using the above unit demands, maximum day and peak rate factors must be developed. For establishments in operation for only a portion of the day, such as schools, shopping plazas, etc., the water usage should also be factored accordingly. For instance, with schools operating for 8 hours per day, the water usage rate will be at an average rate of say 70 L/student. day x 24/8 or 210 L/student. day (45 gpd/student) over the 8 hour period of operation. The water usage will drop to residual usage rates during the reminder of the day. Schools generally do not exhibit large maximum day to average day ratios and a factor of 1.5 will generally cover this variation. For estimation of peak demand rates, an assessment of the

water using fixtures is generally necessary and a fixture-unit approach is often used.

The peak water usage rates in campgrounds will vary with the type of facilities provided (showers, flush toilets, clothes washers, etc.) and the ratio of these facilities to the number of campsites. A peak rate factor of 4 will generally be adequate, however, and this factor should be applied to the average expected water usage at full occupancy of the campsite.

4.8.1c Industrial Water Demands

Industrial water demands are normally expressed in terms of water requirements per gross hectare of industrial development. These demands will vary greatly with the type of industry, but common allowances for industrial areas range from 35 m^3/gross hectare. day (3000 gpad) for light industry to 55 m^3/gross hectare. day (5000 gpad) for heavy industry. These are average daily demands. Peak usage rates will generally be 2 to 4 times the average usage rate.

4.8.1.d Fire Demands

To estimate the fire flow requirements for a particular structure or area of a municipality, the designer should refer to a guide such as, "Water Supply for Public Fire Protection - A Guide to Recommended Practice), prepared by Public Fire Protection Survey Services, Insurers' Advisory Organization, 180 Dundas Street West, Toronto, Ontario.

4.9 FRICTION FACTORS

It is recommended that the following "C" valves be used for the design of water distribution systems, regardless of material:

Diameter	C-Factor
150 mm (6 in.)	100
200 & 250 mm (8 & 10 in.)	110
300 to 600 mm (12 to 24 in.)	120
Over 600 mm (over 24 in.)	130

The above C-factors are intended to represent those which would be experienced in the long term. In calculating maximum velocities for transient pressure estimations, or for checking pump motor sizes for run-out conditions, it is recommended that a C-factor of 140 be used to allow for new pipe conditions.

In evaluating existing systems for expansion, the C-factors should be determined by actual field tests, wherever possible.

4.10 DESIGN CALCULATIONS

Design calculations in support of the sizing of minor extensions to water distribution systems will often not be required where minimum pipe size requirements have dictated the sizing. Where new systems, or major extensions are proposed, design calculations demonstrating the adequacy of the system should accompany the application.

When an existing distribution system which has, or is suspected to have pressure problems is proposed to be extended or modified, design calculations will be required to demonstrate the acceptability of the proposed works.

If a computer analysis has been carried out on the proposed system, the results of that analysis should be noted on a general plan of the system together with an indication of the conditions imposed in the program.

4.11 SYSTEM LAYOUT REQUIREMENTS

4.11.1 Grid Design

Wherever possible, water distribution system layouts should be designed to eliminate dead-end sections. Where dead-end mains cannot be avoided, they should be provided with a fire hydrant, blow-off or other acceptable measures taken to prevent problems associated with stagnation.

4.11.2 Valve Placement

In grid patterns, intersecting watermains should be equipped with shut-off valves, as follows, to minimize disruption during repairs:

	Number of Valves
'T' intersection	at least 2
Cross intersection	at least 3

Air release valves should be placed at all significant high points of the system. Where the needs for an automatic air release valve is uncertain, a manual air release valve or hydrant can be initially installed and later replaced with an automatic valve, if significant air accumulations are found. With large diameter mains, drain valves positioned at low points may also be required to permit main repairs. Small diameter watermains can generally be drained through hydrants by using compressed air and/or pumping.

4.12 DISTRIBUTION SYSTEM STORAGE REQUIREMENTS

The primary purposes and advantages of distribution reservoirs are:

a) to equalize peak hourly rates to average daily flows.

b) to provide and maintain desired pressures in remote areas.

c) to provide emergency storage for fire protection purposes.

Secondary benefits of distribution reservoirs are:

a) to equalize pumping heads which permits selection of pumps of highest efficiency.

b) to permit economies of construction by making it possible to reduce size of pumps and mains necessary to furnish a district, since peak rates on the pumps are diminished by the reservoirs.

c) to permit economies of operation by reduced shirt operations at pumping plants, and by being able to pump during off-peak periods when power rates may be lower.

d) to reduce pressure fluctuations and possibly avoid interruptions in service during main repairs.

Distribution reservoirs are normally classed as being ground or elevated storage. The former being at or below ground level while elevated storage systems may consist of either a standpipe or an overhead tank. Distribution reservoirs are most advantageously located at or near the centre of consumption, and incertain instances near areas where improvements in pressure are needed.

No attempt has been made to determine the amount of storage that might be desirable within any particular distribution system. Suffice it to say that distribution system storage can be a viable and economical alternative to increasing feedermain capacity, duplicating a feedermain for reliability purposes, eliminating the necessity for additional pumps, or additional water treatment plant capacity, by levelling out peak rates.

4.13 DEPTH OF COVER REQUIREMENTS

With the exception of those watermains which will be taken out of service and drained in winter, the minimum depth of cover for watermains and service connections should not be less than the depth of frost penetration.

As a general guide, the depths of frost penetration within the road allowance for areas of Ontario are as follows:

	Frost Penetration
Southwestern Region	1.5 to 1.8m (5 - 6 ft.)
West Central Region	1.7 to 2.0m (5½-6½ ft.)
Central Region	1.7 to 2.0m (5½-6½ ft.)
Southeastern Region	1.7 to 2.1m (5½-7 ft.)

Northeastern Region	1.8 to 2.4m (6 - 8 ft.)
Northwestern Region	2.1 to 2.6m (7 -8½ ft.)

For any particular location, the depth of frost penetration may differ from the above ranges and reference should either be made to local records, data available from the Atmospheric Environment Branch of Environment Canada, or the Ministry of Transportation and Communications publication "Frost Depth Penetrations/Ontario".

Large diameter watermains, over 300 m (12 in.) without service connections may be installed at such an elevation that the frost-free depth corresponds with the spring line of the pipe rather than the crown.

If for economic or practical reasons it is impossible to install watermains in the natural frost-free zone, consideration may be given to reduced cover installations provided the design method ensures that the watermain will not freeze or be damaged by heaving or increased trench loads caused by frost penetration. Recent studies have shown that the penetration of frost into the ground causes increases in the earth loads roughly doubled despite the fact that the frost penetration did not reach the tops of the pipes. The earth loadings prior to frost penetration were approximately equivalent to calculated prism loads. In these experiments, it is interesting to note that the loading increased to double the normal earth load as the frost penetration increased, but the closest that the frost layer came to the pipes was 0.22 m (0.75 ft.). It is, therefore, possible that higher loadings would occur if the frost penetrated closer to the pipe.

These increased external loads caused by frost may create beam breaks in the pipe when bedding is non-uniform. This points to the need for proper attention to the installation of the pipe bedding. It also suggests that great care must be taken in the selection of pipe materials, pipe classes and bedding types. Although various insulation and heating techniques are available, the designer is cautioned to consider the long-term effects and economics of such an approach. Reference should be made to Ministry of the Environment research papers "Design of Buried Pipelines in Cold Climates" and "Temperature Monitoring of an Insulated Watermain".

4.14 CROSS-CONNECTION CONTROL AND SPECIAL SEPARATION OF WATER AND SEWAGE WORKS

Precautions must be taken in the design of water distribution and plumbing systems to preclude the entrance of contaminating materials into the water supply.

Such contaminants can enter water supply systems from various sources such as cooling water systems, pump seal systems, industrial process piping, groundwater, etc. To control contamination from other non-potable piped systems, cross-connection control measures are necessary. To prevent or minimize the entrance of dangerous contaminants into the water supply from groundwater sources, certain precautions must be taken with the relative location of sewage and water systems.

For more information on cross-connection control equipment, reference should be made to CSA Standard B64.0, "Definitions, General

Requirements, and Test Methods for Vacuum Breakers and Backflow Preventers".

4.15 PIPE DESIGN

4.15.1 Alternate Pipe Materials

Acceptable alternate materials for watermains include ductile-iron (cement-lined), asbestos cement, reinforced concrete, polyethylene, polyvinyl chloride, cement lined-cement coated steel, and fibreglass reinforced plastic. Pipe selected shall have been manufactured in conformity with the latest standards issued by the American Water Works Association, the Canadian Standards Association, or the Canadian Government Specification Board. In the absence of such standards, pipe manufactured in accordance with certain commercial standards, if acceptable to the Ministry of the Environment, may be selected.

In selecting a pipe material, the designer should consider the following factors:

a) Costs (capital, operating, maintenance and other costs);

b) Trench foundation conditions;

c) Location;

d) Soil corrosivity;

e) Potable water corrosivity;

f) Available labour skills.

4.15.2 Pipe Strength

Section 4.3.1 and 4.3.2 discussed the operating and transient pressures which are experienced in distribution systems. Buried watermains are also subjected to external loads imposed by the trench backfill, frost loading, and superimposed loads (either static or dynamic, or both). The watermain pipe selected for a particular application must be able to withstand with an adequate margin of safety all the combinations of loading conditions to which it is likely to be exposed.

Pipe strength designations and the methods for selecting the required pipe strength vary with the types of materials used for watermains. For a thorough understanding of this subject, it will be necessary for the designer to evaluate the pipe supplier information and consult such references as the pertinent ANSI, AWWA, and CSA Standards and Design Manuals. A general discussion of the design approach used with some of the pipe materials is outlined in the following subsections.

4.15.2a Ductile-Iron Pipe

Ductile-iron pipe to be used for watermains should be manufactured in accordance with ANSI A21.51 (AWWA C151) with the cement lining applied in accordance with ANSI A21.4 (AWWA C104). For pipe manufactured to these standards, the design method and design stresses used to determine the thickness of pipe are covered in Standard ANSI A21.50 (AWWA C150).

The wall thickness requirement is determined by separately considering trench load-laying condition and internal pressure. To

the greater of the two thickness requirements is then added allowances for corrosion and casting tolerances. The thickness class (Class 50, 51, 52, etc.) is then selected. It should be noted that the design tables assume a transient pressure of 700 kPa (100 psi) and the design pressure is the sum of working pressure and transient pressure times a safety factor of two. If transient pressures greater than 700 kPa (100 psi) are anticipated the thickness requirement for internal pressure must be calculated by the equations provided.

4.15.2b Asbestos-Cement Pipe

Asbestos-cement pipe to be used for watermains should be manufactured in accordance with AWWA C400. The pipe design procedures should be in accordance with AWWA C401 (formerly AWWA H2). It should also be noted that installation procedures for asbestos-cement pipe are outlined in AWWA C603.

The pipe class selection curves contained in AWWA C401 are based on the combined-loading theory developed by W.J. Schlick. Tests of asbestos-cement pipe under various combinations of internal pressure and external load indicate that the point of pipe fracture follows a parabolic curve. With increasing external loading, asbestos-cement pipe will fail at progressively lower internal pressures. The class rating of the pipe (100, 150, 200, or 300) corresponds to the suggested maximum internal operating pressure in psi which should be used for design when the pipe is laid at 1.5 m (5 ft.) depth of cover, with a trench width equal to I.D. plus 0.6 m (2 ft.), a

bedding condition of Class C and a soil density of 1920 kg/m^3 (120 lb./ft.^{3}). With higher external loadings caused by different trench conditions than above, the design operating pressure of the pipe will decrease. The curves provided in AWWA C401 will allow the designer to select the proper pipe class for a wide range of trench and operation conditions.

The design curves incorporate safety factors of 4.0 for operating pressures and 2.5 for external loads. Ordinary transient (surge) pressures are allowed for in the safety factor of 4.0. Exceptionally high surge pressures should be calculated and allowed for separately.

For municipal water supply systems, a minimum of Class 150 asbestos-cement pipe is recommended by the Ministry of the Environment. For non-municipal type systems where operating pressures in the order of 700 kPa (100 psi) are not anticipated, Class 100 pipe may be used if operating and/or transient pressures and external loading conditions permit.

4.15.2c Reinforced Concrete Pipe

Reinforced concrete watermains should be manufactured in accordance with the appropriate AWWA specification for the type of pipe required, as follows:

- REINFORCED CONCRETE PRESSURE PIPE

  STEEL CYLINDER TYPE - AWWA C300

- PRE-STRESSED CONCRETE PRESSURE PIPE

  STEEL CYLINDER TYPE - AWWA C301

- REINFORCED CONCRETE PRESSURE PIPE
  NON-CYLINDER TYPE - AWWA C302
- PRETENSIONED CONCRETE PRESSURE PIPE
  STEEL CYLINDER TYPE - AWWA C303

With concrete pipe, the user does not have available the convenient selection curves in the AWWA specifications as are provided for ductile iron or asbestos cement pipe. The design methods are, however, outlined and they should be followed closely. To avoid errors in pipe selection, the designer should provide the pipe manufacturer with the information suggested in the appropriate AWWA specffication. The manufacturers' catalogues should also be consulted since they generally outline the class pressures routinely manufactured and contain design curves for pipe selection.

4.15.2d Steel Pipe

Steel watermains should be manufactured in accordance with the latest version of the appropriate AWWA specifications. The AWWA specifications pertaining to steel pipe are:

- STEEL WATER PIPE 150 mm (6 in.)
  AND LARGER (formerly C201 and C202) - AWWA C200
- COAL-TAR PROTECTIVE COATING AND
  LININGS FOR STEEL WATER PIPELINES -
  ENAMEL AND TAPE - HOT-APPLIED - AWWA C203
- CHLORINATED RUBBER-ALKYD PAINT SYSTEM
  FOR THE EXTERIOR OF ABOVE

GROUND STEEL WATER PIPING	- AWWA C204
- CEMENT-MORTAR PROTECTIVE LINING AND COATING FOR STEEL WATER PIPE - 100 mm (4 in.) AND LARGER - SHOP APPLIED	- AWWA C205
- FIELD WELDING OF STEEL WATER PIPE	- AWWA C206
- STEEL PIPE FLANGES	- AWWA C207
- DIMENSIONS FOR STEEL WATER PIPE FITTINGS	- AWWA C208
- COLD-APPLIED TAPE COATINGS FOR SPECIAL SECTIONS, CONNECTIONS, AND FITTINGS FOR STEEL WATER PIPELINES	- AWWA C209
- CEMENT-MORTAR LINING OF WATER PIPE LINES - 100 mm (4 in.) AND LARGER IN PLACE	- AWWA C602

For a detailed description of the design approach to be used for steel watermains, reference should be made to AWWA Manual M11 "Steel Pipe Design and Installations".

4.15.2e Thermoplastic Pipe

The two most commonly used thermoplastics for watermains are polyvinyl chloride (PVC) and polyethylene (PE). Both materials are acceptable for use in the Province of Ontario.

A. Polyvinyl Chloride (PVC)

PVC pipe should be manufactured in accordance with the latest editions of either CSA B137.3 or AWWA C900. With pipe manufactured to the former specification, the maximum allowable internal pressure in psi to which the pipe can be subjected (operating pressure + surge pressure) is designated by the Series number (Series 80, 100, 125, 160, and 200). With the latter specification, a Class rating system is used and the class number (100, 150 and 200) corresponds to the maximum allowable working pressure (operating pressure) in psi to which the pipe can be subjected. The class ratings provide for a pressure rise above the maximum working pressure caused by a surge that does not exceed that caused by an instantaneous velocity change of 0.6 m/s (2 fps). Both specifications rate the pipe at a working temperature of 23 degrees C, or lower; if pipe is to be used at higher working temperatures, the effective pressure rating of the pipe must be reduced. For normal watermain trench conditions, the pipe strength requirements for PVC will be dictated by the expected internal pressures. External loadings will not decrease the ability of the pipe to withstand internal pressures until a deflection of 10 percent of the diameter is reached. To avoid this weakening of the pipe, a maximum deflection limit of 5 percent is suggested.

For municipal water supply systems, a minimum of Series 160, or Class 100 PVC pipe (DR26 or DR25), is recommended by the Ministry of the Environment. For non-municipal type systems, Series 80, 100, or 125 PVC pipe may be used if operating and/or transient pressures and external loading conditions permit.

B1 Polyethylene (PE)

Polyethylene pipe to be used for watermains in the Province of Ontario should be manufactured in accordance with either CSA B137.0 and B137.1 or *CGSB 41-GP-25. The CSA Standards cover pipe up to and including 150 mm (6 in.) diameter and the CGSB Standard covers nominal sizes from 90 mm (3.5 in.) to 1600 mm (48 in.).

With PE pipe, the Series number (45, 60, 80, 100, 125 and 160) represents the maximum allowable working pressure (operating pressure) in psi to which the pipe should be subjected, at a "working temperature" of 23 degrees C (73.4 degrees F), with an allowance for surge pressure of 3 times the series number. Transient (surge) pressures normally experienced with water distribution systems can be permitted to raise the total internal pressures above the pipe series rating provided that the normal operating pressure is below the series rating and the duration of the excessive pressure is short and followed by a recovery period of equivalent, or longer, duration than the excessive pressure.

* As with PVC, the effective pressure rating of PE pipe must be reduced if it is to be used at working temperatures above 23 degrees C. For instance, at temperatures between 30 degrees and 38 degrees C the pipe should not be operating in excess of 80 percent of its series rating.

PE pipe strength requirements for watermains will be dictated normally by the internal pressures. External loadings may become a factor if watermains are to be watered for long time periods, or where excessive live and/or trench loads are to be experienced. Manufacturer's design curves will generally provide sufficient information to allow a designer to select the required

* Canadian Government Specification Board

pipe series. It is recommended that long term deflections not be in excess of 5 percent.

For municipal water supply systems, a minimum of Series 125 PE pipe is recommended by the Ministry of the Environment. For non-municipal type systems, Series 60, 80 or 100 may be used if internal pressure and external loading conditions permit.

4.16 PIPE BEDDING AND BACKFILLING

The bedding and backfilling requirements for watermains vary greatly depending upon the type and class of watermain material used. Reference should be made to the previous sections on alternate pipe materials and the specifications quoted therein for specific bedding and backfilling requirements.

In general, however, certain precautions should always be taken in pipe bedding. The trench bottom should be properly prepared to accept the watermain. It must be accurately graded so that the pipe may be evenly supported along its length. Special care should be taken at the joints to ensure that the weight of the pipe is not being supported by the joints alone. The trench bottom should be cleaned and free of stones and hard lumps and if this cannot be economically achieved, the trench should be over-excavated and backfilled with granular material. If bedrock is encountered, over excavation and granular cushioning should be carried out.

4.17 WATERMAIN APPURTENANCES

4.17.1 Hydrants

All hydrants used on distribution systems in the Province of Ontario should be of the dry-barrel type to prevent freezing. They should be manufactured in accordance with AWWA C502-73. In areas where the water table will rise above the hydrant drain ports, the drain ports should be plugged and the barrels kept pumped dry to prevent freezing damage to the barrel and water system contamination. Where hydrant drains are not plugged, they shall drain to the ground surface, if topography permits, or to dry wells provided exclusively for that purpose. Under no circumstances should such drains be connected to storm or sanitary sewers.

All fire hydrants should be provided with adequate blocking to prevent movement caused by thrust.

For other hydrant requirements refer to Section 4.7 and consult with the municipal fire department.

4.17.2 VALVES AND VALVE CHAMBERS

For the guidelines covering the placement of valves refer to Section 4.11.2. Also the Municipal Water Department should be consulted regarding their preferences with respect to valve locations at intersections; line valve spacing; types of valves permitted; direction of rotation of wrench nut to open; maximum size of valve permitted in valve box; etc. Wherever possible, valves to be used in water distribution systems should be manufactured in accordance with

recognized standards, such as those prepared by AWWA. The following AWWA standards cover valves used in water distribution systems:

- GATE VALVES - 75 mm (3 in.) through 1200 mm (48 in.) - AWWA C500
- RUBBER-SEATED BUTTERFLY VALVES - AWWA C504
- BALL VALVES-SHARE OR TRUNION-MOUNTED - 150 (6 in.) through 1200 mm (48 in.) FOR WATER PRESSURES UP TO 2100 kPa (300 psi) - AWWA C507

Valves 300 mm (12 in.) in diameter, or less, may have access provided to the operating nut via a valve box and stem assembly, but it is the recommendation of the Ministry of the Environment that all line valves larger than 300 mm (12 in.) in diameter be placed in valve chambers. Similarly, all air release valves and drain valves should also be located in chambers. To minimize the number of chambers required, consideration should be given to locating combinations of such appurtenances in a single chamber. These chambers and/or the discharge lines from air release, drain or blow-off valves should not be connected directly to any storm or sanitary sewer. Instead, they should drain to the surface of the ground where they are not subject to flooding by surface water; to absorption pits underground; or, to a sump within the chamber, if the ground level is above the chamber floor.

4.17.3 WATER SERVICES

Water services must be at least 20 mm (3/4 in.) inside diameter and under the provisions of the Plumbing Code (Regulation 647) the following materials are acceptable. Municipal works Departments should be consulted regarding local preferences.

a) Brass

b) Cast Iron

c) Copper

d) Ductile Iron

e) Open Hearth Iron

f) Steel

g) Wrought Iron

h) Polyethylene (rated working pressure 150 psi, or more)

i) Polyvinyl Chloride (rated working pressure 150 psi, or more)

j) Asbestos Cement

k) Polybutylene (CSA BI 37.7 and/or AWWA C902) for pipe up to 76 mm (3 in.)

Reference should be made to the Plumbing Code for further discussion of the acceptable service pipe materials and the conditions under which they may be used.

In selecting the diameter of a service connection, the designer should consider such factors as the following:

a) Peak water consumption of building services;

b) Total length of service which will be required to reach building;

c) Elevation of building;

d) Available head in watermain;

loss of head resulting from considerations a), b), c), fittings and meter; and required head at point of water usage.

Studies have shown that for residential water services, the peak water demands can be as high as 1.1 L/s (15 gpm). Past practice has been to design for peak flow rates of 0.4 to 0.5 L/s (5 to 7 gpm). Head losses become excessive with 20 mm (3/4 in.) services for water flows much higher than 0.4 L/s (5 gpm). The use of 25 mm (1 in.) residential water services should, therefore, be considered, especially in the case of large homes, large lot sizes, etc.

All water services should be equipped with a corporation stop and a curb stop. The curb stop should be provided with a curb box.

4.18 THRUST BLOCKS

Adequate restraint must be provided in water distribution systems to prevent pipe movement and subsequent joint failure. In the case of non-restraining mechanical and/or slip-on joints, this restraint should be provided by adequately sized thrust blocks positioned at all plugs, caps, tees, reducers, wyes, hydrants and bends deflecting 22½ degrees, or more. Depending upon internal pressures, pipe sizes and soil conditions, bends of lesser deflection may also required thrust blocking. An alternative approach that can be used to prevent joint failure is to either use pipe and joining methods capable of resisting the forces involved (such as welded steel pipe, or polyethylene pipe with thermal butt-fusion joints) or use joint restraining methods, such as suitable metal tie rods, clamps or harnesses. In designing thrust blocks and other restraint systems, the designer should remember that transient pressures should

be added to the normal operating pressures when calculating the thrust forces (if velocity of flow is very high, dynamic thrust should also be calculated); adequate corrosion protection must be provided for external clamps and tie rods; the safety bearing values of soils should be reduced substantially from textbook figures if shallow trenches are used. For further discussion of thrust blocking and joint restraint design, reference should be made to the pipe manufacturer's catalogue, and other sources such as textbooks, watermain design manuals, etc.

When installing Ductile Iron service connections with a tee connection at the main, the control valve should be properly blocked, and at least one length of pipe should be installed after the valve to keep it from moving when the excavation for the extension of the service pipe to the building is made.

4.19 WATER METERS

A water meter is the cash register of the waterworks system. While not all cities measure the water supplied to each consumer, the practice of metering is well-established.

The common types of meters now used in consumers' services are the mutating or wobbling disc, oscillating piston, and turbine types. The first named is the most common type used: it comprises a disc made of hard rubber which oscillates in guides, each oscillation indicating a filling and emptying of a fixed amount of water in the disc chamber. A Turbine meter measures the velocity of water passing the turbine and thus, by a series of gears, a reading is obtained of

the quantity of water passing through. A compound meter is two meters built into one. One of these meters is generally a large capacity turbine meter to register large flows; the second is a small by-pass disc meter to measure the small flows. Built into the compound meter is an automatic valve arrangement which directs small flows to the by-pass meter.

There is much new technology now available for meter installations where the meter can be read by various types of remote read-out arrangements. Experiments were carried out in the Borough of North York, Ontario, where telemetering is done by a computer link up to basement meter installations via the Bell Telephone lines. It is not the intention in this brief chapter to detail any of the technology of these systems, since this particular aspect of metering is under constant experimentation and change.

Meter manufacturers should be contacted for the details on the various commercial systems now on the market.

APPENDIX A

Peaking Factors for Municipal Water

Supply Systems

POPULATION RANGE	MINIMUM RATE FACTOR2 (Min. Hr.)	MINIMUM DAY FACTOR3	PEAK RATE FACTOR4 (Max. Hr.)
Up to 50	0.40	3.00	4.50
501 - 1000	0.40	2.75	4.13
1001 - 2000	0.45	2.50	3.75
2001 - 3000	0.45	2.25	3.38
3001 - 10000	0.50	2.00	3.00
10001 - 25000	0.60	1.90	2.85
25001 - 50000	0.65	1.80	2.70
50001 - 75000	0.65	1.75	2.62
75001 - 150000	0.70	1.65	2.48
Greater than 150000	0.80	1.50	2.25

Example: Assume a) Projected 20 year population of Town is 3,600 persons

b) Average per capita water consumption is 0.3 m^3/cap.d

Calculate a) Average day

b) Minimum rate

c) Maximum day

d) Peak rate

Solution a) Average day = 3600 x 0.3 = 1080 m^3/d

b) Minimum rate = 1080 x 0.5 = 540 m^3/d

c) Maximum day = 1080 x 2.00 = 2160 m^3/d

d) Peak rate = 1080 x 3.00 = 3240 m^3/d

Notes:

1. These peaking factors are suitable for use with an entire municipal system with its variety of water uses (residential, industrial and commercial). For portions of a municipal system such as an industrial subdivision, they may over-estimate actual peaks and under--estimate minimum rates, whereas, with a purely residential area they may under-estimate the actual peaks and over-estimate the minimum rates. For sub-areas of a municipality refer to Section 4.8.1 for a discussion of peaking factors for residential, industrial and commercial areas.

2. Minimum hourly consumptions may be estimated by multiplying the average day usage by the minimum rate factor. By definition, the minimum hour is the minimum usage rate on the maximum day.

3. Maximum day usage may be estimated by multiplying the average day usage by the maximum day factor.

4. Peak rate water usage may be estimated by multiplying the average day usage by the peak rate factor.

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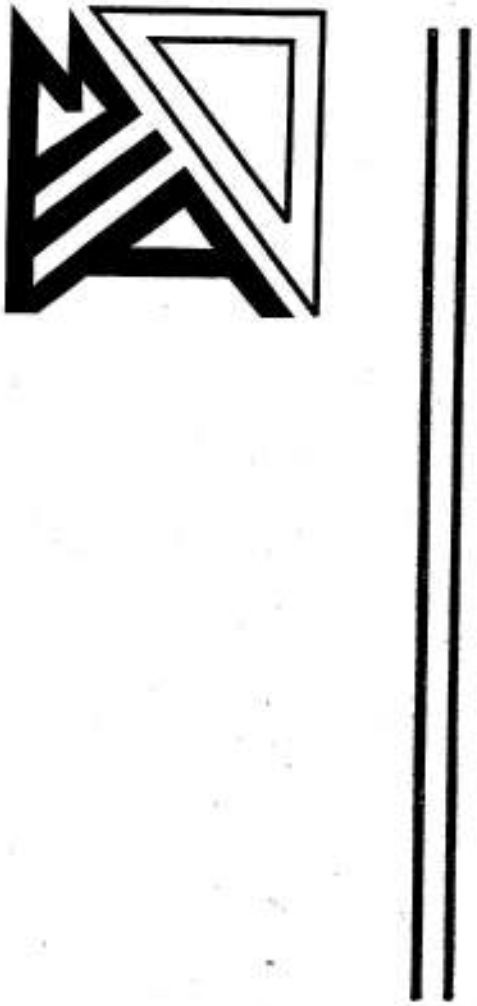

5.

TRANSPORTATION

CHAPTER NO. 5 - TRANSPORTATION

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- 2.4 Local Roads and Streets

3 Primary and Secondary Groups

4 Characteristics of Road Classes

5 Design Classification

5.1.2 TYPICAL FACILITIES PROVIDED

1 Introduction

2 Examples - Typical Facilities (Dwg)

3 Outline of Various Facilities (Dwg)

4 Local Residential Street (Dwgs)

5.1 ROAD ALLOWANCE CONSIDERATIONS

5.1.1 ROAD CLASSIFICATION

1.0 Introduction

A road and street system is composed of various road types. Each performs a particular service in facilitating vehicular travel between points and in providing access to property. Some roads are obviously more important, while others play a very minor role in the overall framework of a municipality. Knowing a road's ultimate use will allow that particular road to be designed and built to serve its purpose with a minimum of disruption and waste.

Through planning, present and future road needs are determined. Because road and street building is an engineering process, engineering knowledge and judgement should play a major role in preparing transportation and/or land use plans. When development patterns are known well in advance, dislocation of people and business is reduced as are right-of-way and other costs.

Some of the traffic factors that can be anticipated based on a good plan are:

- traffic volume;
- vehicular type - truck, bus, car;
- trip type - commuter, local, through, shopping, sight-seeing, recreational;
- public transportation needs;
- future traffic protection.

Thus, by assessing the planned use of a road and classifying roads according to requirements, standard design elements can be developed to meet the needs.

2.0 General Description of the Functional Classes

Eight major functional classifications are used:

Rural	Urban
1. Freeway	1. Freeway
2. Arterial	2. Arterial
3. Collector	3. Collector
4. Local	4. Local

(Rural and urban refer to the predominant characteristics of adjacent land use and not to jurisdictional boundaries.)

2.1 Freeways

Freeways are intended to accommodate heavy volumes of traffic at high speeds under free-flowing conditions. They are intended to serve long trip traffic.

2.0 (continued)

	Rural	Urban
Traffic volume:	8,000 plus	20,000 plus
Traffic flow:	Limited access roads with opposing traffic lanes separated, with grade separated interchanges and cross-overs, and restrictions on parking, pedestrians and cycling.	
	Rural	Urban
Running speeds:	80 to 100 km/h	70 to 100 km/h
Vehicle types:	All types with 20 to 30% trucks.	All types with 20% trucks and express buses only.
	Rural	Urban
Connections:	Other freeways, arterials and collectors only.	Other freeways and arterials, usually with some connection to collectors.
Percent of road system:	Up to 5%	Up to 10%

2.2 Arterial Roads

Arterials are major routes in the network. They connect industrial and commercial centres and concentrations of residential development. Where freeway service is not warranted arterials are the highest type of road.

	Rural	Urban
Traffic volume:	1,000 to 12,000	5,000 to 30,000
Traffic flow:	Usually uninterrupted flow. Intersections should be controlled (signals or signs).	Intersections Should be signalized and if close together, synchronized. Parking/loading should be restricted. Public transportation loading zones & turning lanes should be provided.
Running speeds:	60 to 90 km/h	50 to 70 km/h

2.2 (continued)

	Rural	Urban
Vehicle types:	All types with up to 20% trucks	All types with up to 20% trucks. Buses may be express or local.
Connections:	All other classes of rural roads.	Freeways, other arterials, collector and in some cases local, industrial or commercial.
Percentage of road system:	5 to 10%	Up to 30%

2.3 Collector Roads

Collectors collect traffic from local roads and distribute it to other local roads, arterials and sometimes freeways. They also service land and full access to adjacent land is usually permitted.

	Rural	Urban
Traffic volume:	200 to 5,000	1,000 - 12,000
Traffic flow:	Traffic flow is interrupted by stop conditions at intersections, and turning movements at land access points.	
	Rural	Urban
Running speeds:	60 to 80 km/h	30 to 50 km/h
Vehicle types:	All types, but with up to 30% farm related trucking.	All types, but trucks are usually service types.
Connections:	Connect with all other rural classifications.	Connect to all but freeways and are rarely found in central business districts.
Percentage of road system:	10 to 20%	Up to 30%

2.4 Local Roads and Streets

Local roads and streets provide land access. Trips on local roads have their origin or destination within the length of the road.

2.4 (continued)

	Rural	Urban
Traffic volume:	N/A	N/A
Traffic flow:	Interrupted flow.	
Running speeds:	50 to 80 km/h	20 to 50 km/h
Vehicle types:	Passenger vehicles and some light or service trucking.	
Connections:	Arterials, collectors and locals.	Collectors and local.
Percentage of road system:	65 to 85%	70% approximately.

3.0 Primary and Secondary Groups

Some jurisdictions may wish to further divide the major classifications into primary or secondary.

Examples: (a) Secondary local - a short crescent or cul-de-sac.

(b) Primary collector - a collector serving a predominantly industrial area where extra width and strength will be needed to handle trucking.

4.0 Characteristics of Road Classes

(A) Urban Road Classes

	Urban Locals	Urban Collectors	Urban Arterials	Urban Freeways
traffic service	traffic movement secondary consideration	traffic movement and land access of equal importance	traffic movement primary consideration	optimum mobility
land service	land access primary consideration		land access secondary consideration	no access
range of traffic volume ADT	not applicable	1000-12000	5000-30000	more than 20000
characteristics of traffic flow	interrupted flow	interrupted flow	uninterrupted flow except at signal & cross walks	free flow
average running speed off-peak conditions	20-50 km/h	30-50 km/h	50-70 km/h	70-100 km/h
vehicle type	passenger & service vehicles	all types	all types up to 20% trucks	all types up to 20% trucks
percentage of total length	70 approx.	up to 30	up to 30	up to 10
percentage of vehicle-kilometres	2 - 10	20 - 30	25 - 65	0 - 50
connects to	collectors locals	arterials collectors locals	freeways arterials collectors	freeways arterials

4.0 (continued)

(B) Rural Road Classes

	Rural Locals	Rural Collectors	Rural Arterials	Rural Freeways
traffic service	traffic movement secondary consideration	traffic movement and land access of equal importance	traffic movement primary consideration	optimum mobility
land service	land access primary consideration		land access secondary consideration	no access
range of traffic volume ADT	not applicable	200-5000	1000-12000	more than 8000
characteristics of traffic flow	interrupted flow	interrupted flow	uninterrupted flow except at signals	free flow
average running speed off-peak conditions	50-80 km/h	60-80 km/h	60-100 km/h	80-110 km/h
vehicle type	predominantly passenger cars & light to medium trucks & occasional heavy trucks	all types up to 30% trucks in the 3 t to 5 t class	all types up to 20% trucks	all types heavy trucks average 20% - 30%
percentage of total length	65 - 85	10 - 20	5 - 10	up to 5
percentage of vehicle-kilometres	2 - 15	15 - 25	40 - 70	0 - 40
connects to	arterials collectors locals	all classes	all classes	freeways arterials collectors

5.0 Design Classification

In order to select roadway design elements for each of the basic classifications, the designer must determine the physical limitation of the route and know the anticipated traffic.

Physical limitations govern - grades
- design speeds
- maximum width

Traffic governs - pavement width
- shoulder width
- number of lanes

Combining physical limitations with traffic and considering financial limitations, the road or street can then be designed to a desirable, acceptable or a minimum set of standards.

Road Design Classifications

location	class	type	design	code number
rural	local	undivided	60	RLU 60
			70	RLU 70
			80	RLU 80
			90	RLU 90
			100	RLU 100
rural	collector	undivided	60	RCU 60
			70	RCU 70
			80	RCU 80
			90	RCU 90
			100	RCU 100
rural	collector	divided	70	RCD 70
			80	RCD 80
			90	RCD 90
			100	RCD 100
rural	arterial	undivided	80	RAU 80
			90	RAU 90
			100	RAU 100
			110	RAU 110
			120	RAU 120
rural	arterial	divided	80	RAD 80
			90	RAD 90
			100	RAD 100
			110	RAD 110
			120	RAD 120
rural	freeway	divided	100	RFD 100
			110	RFD 110
			120	RFD 120
			130	RFD 130
			140	RFD 140

5.0 (continued)

Road Design Classifications

location	class	type	design	code number
urban	local	undivided	40	ULU 40
			50	ULU 50
urban	collector	undivided	50	UCU 50
			60	UCU 60
			70	UCU 70
			80	UCU 80
urban	collector	divided	50	UCD 50
			60	UCD 60
			70	UCD 70
			80	UCD 80
urban	arterial	undivided	50	UAU 50
			60	UAU 60
			70	UAU 70
			80	UAU 80
urban	arterial	divided	60	UAD 60
			70	UAD 70
			80	UAD 80
			90	UAD 90
			100	UAD 100
urban	freeway	divided	80	UFD 80
			90	UFD 90
			100	UFD 100
			110	UFD 110
			120	UFD 120

5.1.2 TYPICAL FACILITIES PROVIDED

1.0 Introduction

Although the roadway is historically and even now often considered as being the prime reason for having a road allowance, there are a number of other necessary facilities and utilities that have rights to and must be provided for along the same right-of-way.

These may include sidewalks, bicycle paths, telephone or telegraph lines, electrical power lines, gas mains, water mains, sanitary sewers and storm sewers or municipal drains.

It has been found from experience that both the sanitary and storm sewers will normally require little maintenance. These have traditionally been placed under the road surface, since the need to block the roadway to provide entry to these sewers will be rare. Also since only a single sanitary and storm sewer main is used to provide service to each side of the street, the connections are approximately equal distance.

The other underground utilities do not require the same depth of installation as the sewers, and are often subject to more frequent maintenance, adjustment, or supplementary installations. These facilities have therefore been more usually placed clear of the road surface. Although a single watermain is used to provide service to both sides of the street, it is often the case that the underground installations for telephone lines, electrical power lines and gas mains are provided on both sides of the street, thus avoiding even the service connections from crossing under the road.

Where sidewalks are required, it is advisable to have these well clear of the road as well as clear of other underground utilities if possible. A minimum boulevard width of 3.0 m between the sidewalk and the road will provide room for the winter snow storage, protect pedestrians from vehicle splash and provide room for some of the underground utilities.

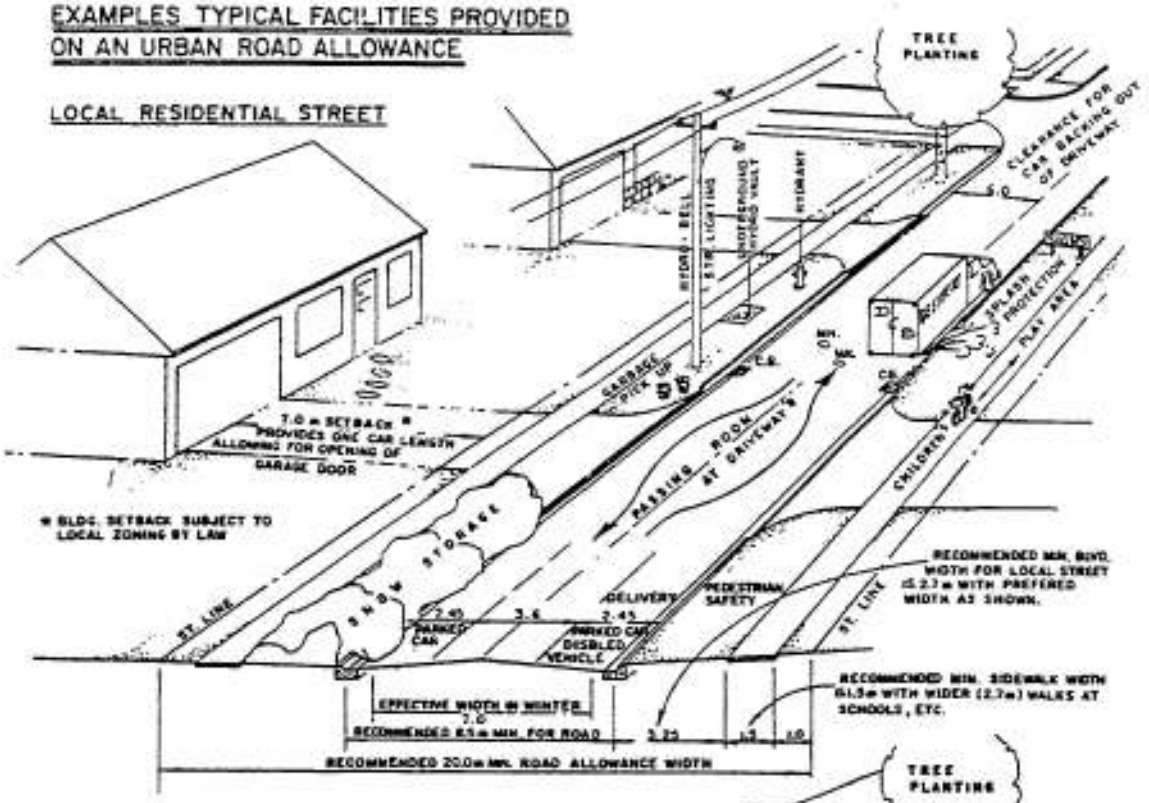
It should be noted that often large main trunk facilities often follow the same route as the collector and arterial streets, increasing the need for wider boulevard areas and overall road allowance widths. It is also more important on these major roads that the various utilities be located clear of the road area, ensuring that vehicle movement will be affected as little as possible by any needed maintenance operations on the utilities.

Municipal drains (providing agricultural and sometimes urban drainage) may be located within a road allowance. It should be the aim of a road authority to have as many of the open municipal drains as possible located off the road allowance in order to eliminate the pressures to expend road maintenance funds to clean them.

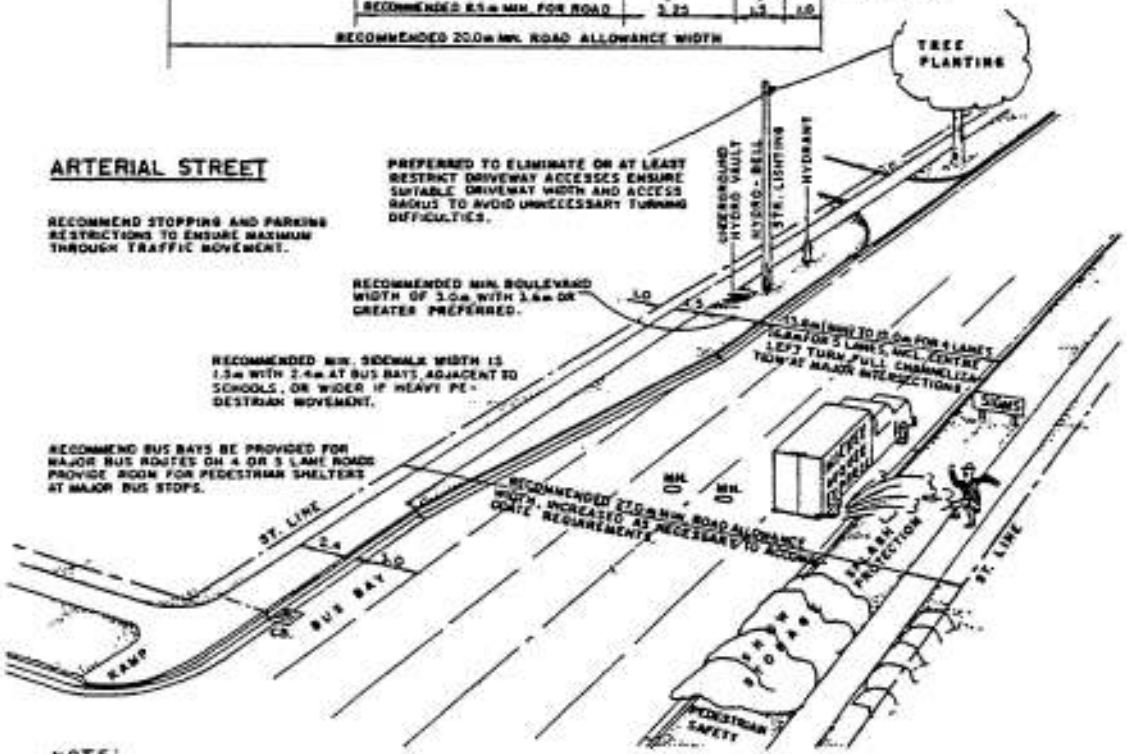
2.0

EXAMPLES TYPICAL FACILITIES PROVIDED ON AN URBAN ROAD ALLOWANCE

LOCAL RESIDENTIAL STREET



ARTERIAL STREET



NOTE:
SEE PAGE 3 FOR A FURTHER OUTLINE OF THE VARIOUS FACILITIES WHICH MAY BE PROVIDED FOR WITHIN THE ROAD ALLOWANCE, WITH PARTICULAR REFERENCE TO UNDERGROUND UTILITIES.

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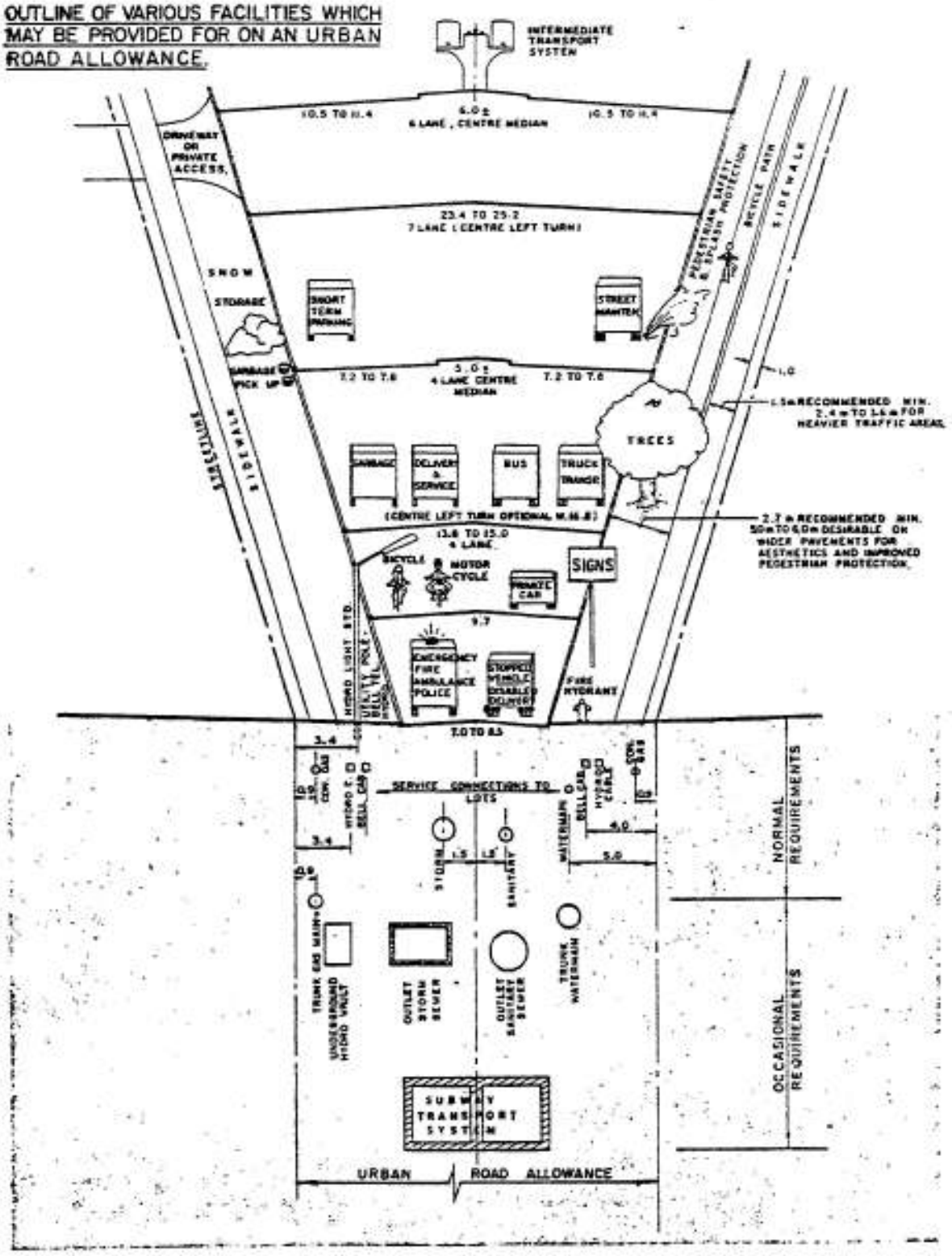
3.0

OUTLINE OF VARIOUS FACILITIES WHICH MAY BE PROVIDED FOR ON AN URBAN ROAD ALLOWANCE.



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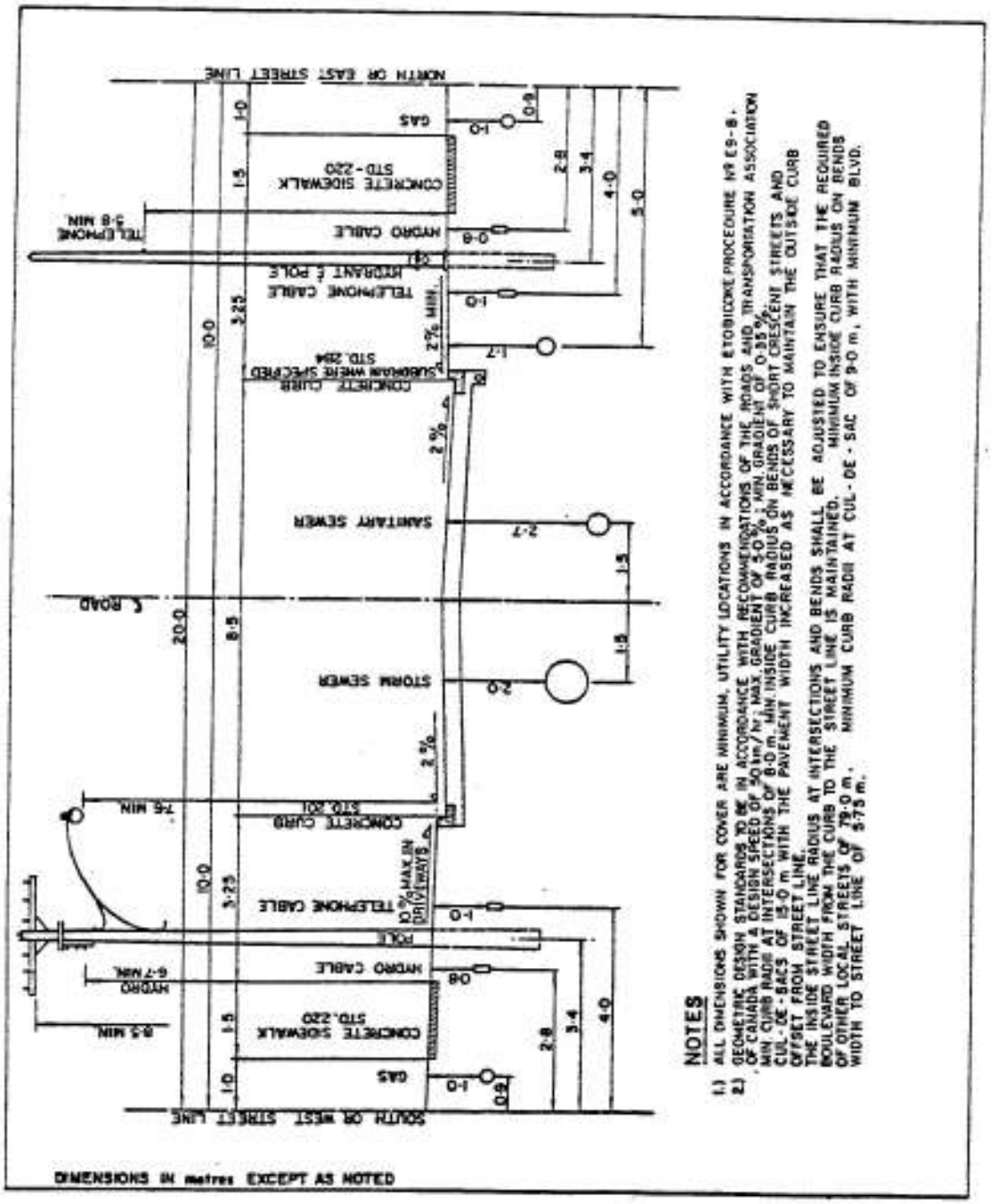
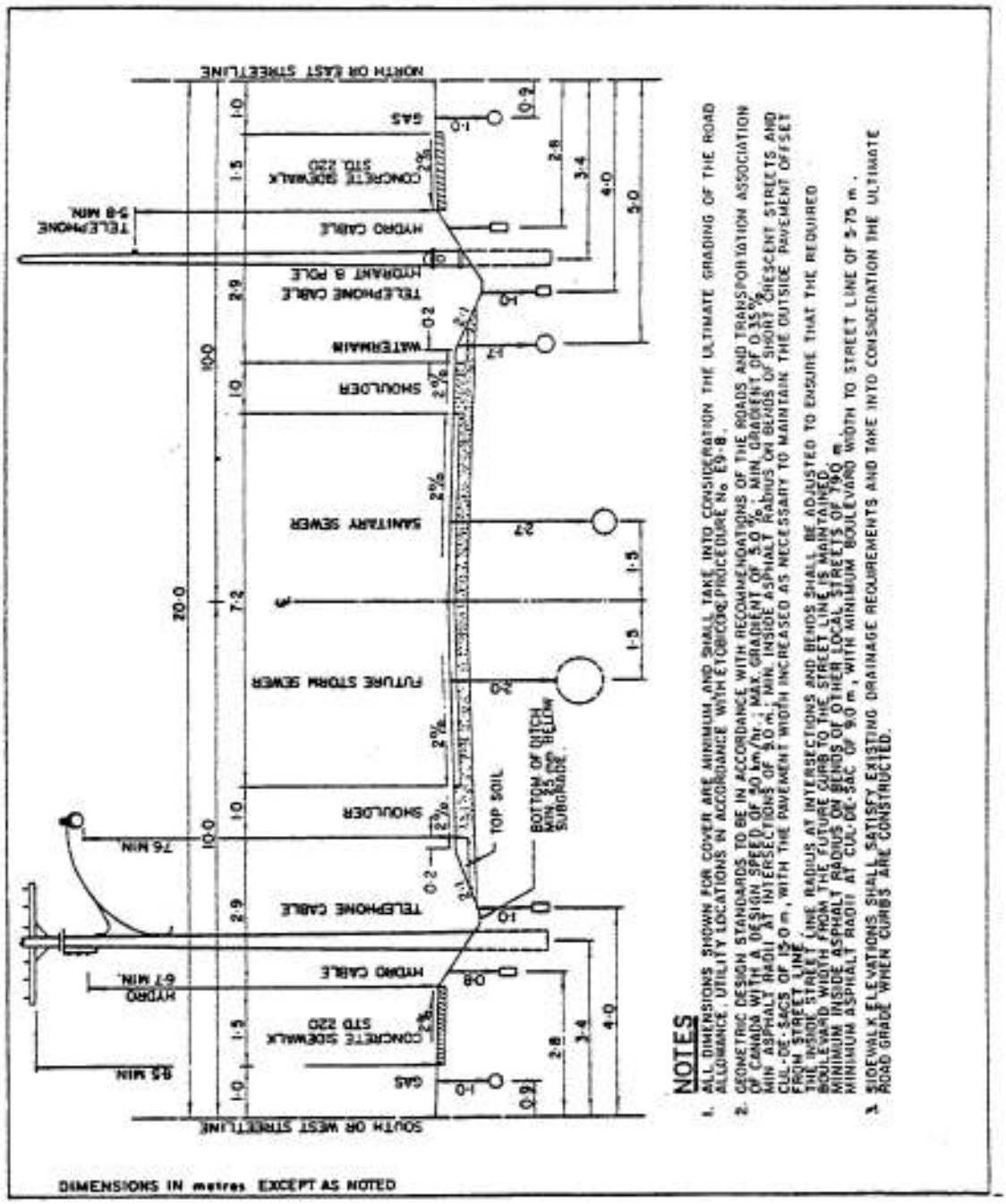
4.0 Local Residential Street



NOTES

1) ALL DIMENSIONS SHOWN FOR COVER ARE MINIMUM. UTILITY LOCATIONS IN ACCORDANCE WITH ETOBICOKE PROCEDURE № E9-8.
2) GEOMETRIC DESIGN STANDARDS TO BE IN ACCORDANCE WITH RECOMMENDATIONS OF THE ROADS AND TRANSPORTATION ASSOCIATION OF CANADA WITH A DESIGN SPEED OF 50 km/hr; MAX. GRADIENT OF 5-0%; MIN. GRADIENT OF 0-35%.
MIN. CURB RADII AT INTERSECTIONS OF 8-0 m. MIN. INSIDE CURB RADIUS ON BENDS OF SHORT CRESCENT STREETS AND CUL-DE-SACS OF 15-0 m WITH THE PAVEMENT WIDTH INCREASED AS NECESSARY TO MAINTAIN THE OUTSIDE CURB OFFSET FROM STREET LINE.
THE INSIDE STREET LINE RADIUS AT INTERSECTIONS AND BENDS SHALL BE ADJUSTED TO ENSURE THAT THE REQUIRED BOULEVARD WIDTH FROM THE CURB TO THE STREET LINE IS MAINTAINED. MINIMUM INSIDE CURB RADIUS ON BENDS OF OTHER LOCAL STREETS OF 7-9 m. MINIMUM CURB RADII AT CUL-DE-SAC OF 9-0 m, WITH MINIMUM BLVD. WIDTH TO STREET LINE OF 5-75 m.

4.0 Local Residential Street



NOTES

1. ALL DIMENSIONS SHOWN FOR COVER ARE MINIMUM AND SHALL TAKE INTO CONSIDERATION THE ULTIMATE GRADING OF THE ROAD ALLOWANCE; UTILITY LOCATIONS IN ACCORDANCE WITH ETOBICOKE PROCEDURE No. E9-8.
2. GEOMETRIC DESIGN STANDARDS TO BE IN ACCORDANCE WITH RECOMMENDATIONS OF THE ROADS AND TRANSPORTATION ASSOCIATION OF CANADA WITH A DESIGN SPEED OF 40 km/hr.; MAX. GRADIENT OF 5.0 %; MIN. GRADIENT OF 0.35 %; MIN ASPHALT RADII AT INTERSECTIONS OF 10 m; MIN. INSIDE ASPHALT Radius ON BENDS OF SHORT CRESCENT STREETS AND CUL-DE-SACS OF 13.0 m, WITH THE PAVEMENT WIDTH INCREASED AS NECESSARY TO MAINTAIN THE OUTSIDE PAVEMENT OFFSET FROM STREET LINE.
3. THE INSIDE STREET LINE RADIUS AT INTERSECTIONS AND BENDS SHALL BE ADJUSTED TO ENSURE THAT THE REQUIRED BOULEVARD WIDTH FROM THE FUTURE CURB TO THE STREET LINE IS MAINTAINED.
4. MINIMUM INSIDE ASPHALT RADIUS ON BENDS OF OTHER LOCAL STREETS OF 7.90 m
5. MINIMUM ASPHALT RADII AT CUL-DE-SAC OF 9.0 m, WITH MINIMUM BOULEVARD WIDTH TO STREET LINE OF 5.75 m.
6. SIDEWALK ELEVATIONS SHALL SATISFY EXISTING DRAINAGE REQUIREMENTS AND TAKE INTO CONSIDERATION THE ULTIMATE ROAD GRADE WHEN CURBS ARE CONSTRUCTED.

5.2 VEHICLE FACILITIES

5.2.1 ROAD GEOMETRICS INDEX

1 Introduction

2 Preliminary Design Review
- 2.1 Functional Road Classification
- 2.2 Design Criteria and Standards
- 2.3 Environmental Assessment of Project
- 2.4 Review of Proposed and Future Utility Requirements
- 2.5 Review of Proposed and Future Development Proposals
- 2.6 Review of Road Allowance Requirements
- 2.7 Review of Existing Utilities

3 On Site Information Requirements
- 3.1 A Recent Survey
- 3.2 Cross-sections and Elevations
- 3.3 Identification of Utilities
- 3.4 Soils Information
- 3.5 On Site Information Requirements (Dwg.)

4 Horizontal Alignment
- 4.1 Maximum Rate of Curvature
- 4.2 Maximum Superelevation
- 4.3 Side Friction Factor
- 4.4 Horizontal Sight Distance on Curves
- 4.5 Type of Curves
- 4.6 Curves on Major Roads and Streets.
- 4.7 Curves on Local Roads and Streets.
- 4.8 Horizontal Alignment Examples

5 Vertical Alignment
- 5.1 Gradient
- 5.2 Gradient at Intersections
- 5.3 Vertical Curves

6		Road Cross-section
	6.1	Urban
	6.2	Rural
7		Intersections
	7.1	Continuous Two-way Left Turn Lane
	7.2	Major Intersections
	7.3	Angle of Intersection
	7.4	Sight Distance at Intersections
	7.5	Intersection Curb Radii
	7.6	Cul-de-Sacs
8		Driveways
9		Vertical Design Analysis
	9.1	Urban
	9.2	Rural
10		Considerations in Urban Road Design
11		Preparation of Drawings

5.2 VEHICLE FACILITIES

5.2.1 Road Geometrics

1.0 Introduction

For design considerations and details, reference should be made to the following publications and design aids in particular:

"Geometric Design Standards for Canadian Roads and Streets"; Roads and Transportation Association of Canada, 1765 St. Laurent Blvd., Ottawa, Canada. K1G 3V4

"Geometric Design Standards for Ontario Highways and Streets"; Ministry of Transportation and Communications, Downsview, Ontario.

"Construction Manual"; Ministry of Transportation and Communications, Ontario.

"Turning Vehicle Templates"; Roads and Transportation Association of Canada.

"Vertical Curve Tables"; Ministry of Transportation and Communications, Ontario.

2.0 Preliminary Design Review

All proposed road projects should be assessed as to the scope of the work to be carried out. The following outline is provided as a guide as to some of the considerations that should be evaluated before the final design criteria are selected.

2.1 Functional Road Classification

As discussed in Section 5.1, this may involve an assessment in co-operation with planners. It is a policy decision. This would be of particular importance when work is planned on a road to assist the movement of existing or anticipated increased traffic volumes. Most local streets are relatively easy to establish. Similarly arterials are normally easily identified since they often follow traditional traffic routes (concession roads, etc.). Collector roads often pose particular problems to identify. Often it involves the upgrading of what the residents have considered a local street. Care must be exercised when reclassifying a road or street, and it may be wise to seek public participation in the process.

2.2 Design Criteria and Standards

Review and establish, in accordance with the functional road classification, the design classification and thus the number of lanes, width of lanes, width of pavement, curbs or shoulders and ditches, parking provisions, intersection channelization requirements, public transit requirements, pedestrian requirements. This assessment may include existing and projected traffic volumes; existing and projected pedestrian volumes; traffic type, trucks, buses, passenger vehicles and physical restrictions.

Collector streets are often the most difficult to establish the proposed design criteria to be used, since they range from a 2 lane primarily passenger vehicle usage (usually a 9.5 m minimum width) to a 4 lane primarily truck usage (usually 15.0 m).

The M.T.C. Geometric Design Standards illustrate the normal lane widths, etc.

GEOMETRIC DESIGN STANDARDS

FOR UNDIVIDED URBAN ROADS IN ONTARIO

DESIGN YR. TRAFFIC		DESIGN SPEED (km/h)	No. of Lanes	LANE WIDTH (m)	PARKING LANE (m)	CURB CLEARANCE (m)	MINIMUM C CURB to CURB DISTANCE (m)
A.A.D.T.	D.H.V.						
6000 and up	600 and up	80	4	3.5-3.75 a		0.25	14.5
		60-70 a	4	3.5		0.25	14.5
6000 to 3000	600 to 300	60-70 a	4 b	3.5		0.25	14.5
		80	2	3.5-3.75 a	2.5-3.0 a	0.25	10
		60-70 a	2	3.5	2.5-3.0 a	0.25	10
3000 to 2000	300 to 200	80	2	3.5	2.5-3.0 a	0.25	10
		60-70 a	2	3.25	2.5-3.0 a	0.25	9.5
		50	2	3.0	2.5-3.0 a	0.25	9
2000 to 1000	200 to 100	60-70 a	2	3.25	2.5-3.0 a	0.25	9.5
		50	2	3.0	2.5-3.0 a	0.25	9
less than 1000	less than 100	40-50 a	2	2.75-3.0 a	2.5-3.0 a	0.25	8.5

2.2 (continued)

a) Upper value is desirable, lower value is acceptable.

b) Four lanes are appropriate in the upper limit of this traffic range where there is a measurable capacity deficiency with only two lanes.

c) Lower value is desirable, upper value is acceptable.

- Horizontal and vertical geometric design standards shall conform to Alignment Standards.

- Auxiliary lane widths shall conform to criteria for Rural Highways.

Boulevards

Boulevards are measured from the edge of pavement to the edge of sidewalk. Where boulevards are provided the desirable minimum boulevard widths are 3.0 m on arterial roads and 2.0 m on collector and local roads. The absolute minimum boulevard width should be 1.5 m.

Sidewalks

The desirable minimum sidewalk width is 1.5 m. Under restricted conditions an absolute minimum of 1.25 m may be used. Sidewalks of 2.0, 2.5 or 3.0 m may be considered in areas of high pedestrian activity.

2.2 (continued)

GEOMETRIC DESIGN STANDARDS

FOR 2 LANE RURAL ONTARIO ROADS

Design Year		Design Speed km/h	CURVES			MAX. GRADE(%)	WIDTH (m)	
			Min. Hor. R(m)	Min. Vert. K(m)				
A.A.D.T.	D.H.V.			Crest	Sag		Lane	Shoulder
4000 & UP	600 & UP	120	650	120	60	6-7	3.75	3.00
		110	525	90	50	6-7	3.75	2.50 a
		100	420	70	45	6-8	3.75	2.50 a
		90	340	50	40	6-8	3.50 a	2.50
		80	250	35	30	6-8	3.50	2.50
4000 to 3000	600 to 450	110	525	90	50	6-7	3.75	2.50 a
		100	420	70	45	6-8	3.50 a	2.50
		90	340	50	40	6-8	3.50 a	2.50
		80	250	35	30	6-8	3.50	2.50
		70	190	25	25	6-12	3.25	2.00
3000 to 2000	450 to 300	110	525	90	50	6-7	3.75	2.50
		100	420	70	45	6-8	3.50 b	2.50
		90	340	50	40	6-8	3.50	2.00 b
		80	250	35	30	6-8	3.25	2.00
		70	190	25	25	6-12	3.25	2.00
2000 to 1000	300 to 150	110	525	90	50	6-7	3.50 c	2.50
		100	420	70	45	6-8	3.50	2.00 c
		90	340	50	40	6-8	3.25	2.00
		80	250	35	30	6-8	3.25	2.00 *
		70	190	25	25	6-12	3.00	1.00 *
		60	130	15	18	6-12	3.00	1.00
1000 to 400	150 to 60	100	420	70	45	6-8	3.50	1.00
		90	340	50	40	6-8	3.25	1.00
		80	250	35	30	6-8	3.25	1.00
		70	190	25	25	6-12	3.00	1.00
		60	130	15	18	6-12	3.00	1.00
Less than 400	Less than 60	80	250	35	30	8	3.00	1.00 d
		70	190	25	25	12	3.00	1.00 d
		60	130	15	18	12	3.00	1.00 d
		50	90	8	12	12	2.75	1.00 d

The minimum roadway cross-section for all paved two-lane King's Highways and Secondary Highways will be: 3.25 m lane width, 1.0 m shoulder width.

2.2 (continued)

NOTES TO
GEOMETRIC DESIGN STANDARDS
FOR RURAL ONTARIO ROADS

- Use D. H. V. for design

- * The 1.0 m shoulder width is considered the minimum desirable width for pavement support. This may be reduced to 0.5 m when paved.

- * The 2.0 m shoulder width is considered the minimum acceptable for a disabled vehicle.

- These cross-sections are desirable standards. An acceptable standard formed by a variation of (±) one increment may be considered, with the appropriate approval.

- a) if number of trucks greater than 10% increase by one increment.
  b) if number of trucks greater than 15% increase by one increment.
  c) if number of trucks greater than 25% increase by one increment.
  Lane width increments - 0.25 m.
  Shoulder width increments - 0.50 m.
  d) 0.5 m shoulders will be permitted where there is no foreseeable possibility of the road being paved within a 20 year period. (Note: 1.0 m shoulder must be used where guide rail is installed.)

- When curb and gutter is used in conjunction with a paved shoulder the gutter pan forms part of the paved shoulder.

- Lane widths for Multi-lane highways shall be 3.75 m in all cases except:

i) 4 lane divided or undivided with design speed less than 100 km/h where the lane width shall be 3.5 m. In rural fringe areas where urban development is likely within the life of the highway, rural cross-section of 1.0 m flush median, 2 x 3.75 m driving lanes and 0.5 m of full depth construction partial width paved shoulders should be considered where 4 lanes are required. (This 17 m cross-section provides for a 3.0 m continuous left turn lane with 3.5 m driving lanes.)

2.2 (continued)

ii) 6 or more lanes divided, all design speeds, the median lane shall be 3.5 m.

- The median shoulder on 4 lane divided highways shall be 1.0 m and paved.

- The median shoulder on divided highways with 6 or more lanes shall be 2.5 m except where 6 m medians or less are used.

Auxiliary Lanes

- Left turn lanes with no median strip are to be 0.25 m less than the width of the adjacent through lane, minimum 3.25 m.

- Left turn lanes with median strip (painted, rippled or raised) shall be 3.0 m exclusive of gutter or curb offset. The curb face on all raised medians adjacent to left turn lanes must be offset.

- Continuous left turn lanes shall be 4.0 m desirable, 3.0 minimum. The minimum standard should normally only be utilized where operating speeds are less than 60 km/h except where a future eventuality is being protected.

- Right turn lanes and acceleration lanes are to be 0.25 m less than the width of the adjacent through lane, minimum 3.25 m.

- Truck climbing lanes and passing lanes are to be 0.25 m less than the width of the adjacent through lane minimum 3.25 m. The adjacent shoulder should desirably be the same as that on the approach roadway but may be reduced to a minimum of 1.0 m where significant economies can be demonstrated.

Ramps

- Single lane interchange ramp widths shall be 4.75 m.

- Multi-lane interchange ramps shall have lane widths of 3.75 m except where pavement widening for curves is required.

- The right-hand shoulder on single and multi-lane interchange ramps shall be 2.5 m.

2.2 (continued)

- Left-hand shoulder widths on interchange ramps shall be:

  i) Zero on single lane ramps, and effective shoulder will be formed by insetting the pavement edge line (see DD-213, 214 A and B).

  ii) Zero on multi-lane ramps with curb and gutter (see DD-215 B).

  iii) 0.5 m on multi-lane ramps with no curb and gutter (see DD-215 A).

- Right-hand shoulder widths adjacent to speed change lanes shall be 1.0 m except for channelized intersections where the lesser of the shoulder widths on the two approach roadways shall be continued throughout the speed change lane and channelized ramp, or in the case of interchanges where the ramp shoulder width shall apply.

Rounding

- Shoulder widths do not include rounding. Rounding, when applied, shall be:

  i) 0.5 m when design speed less than 100 km/h
  ii) 1.0 m when design speed greater than 100 km/h

- Rounding on the left-hand side of single lane interchange ramps having no shoulders shall be 1.0 m.

Medians

- Flush medians - 1.0 m.
- Raised division islands usually associated with intersections - 2.0 m.
- Raised medians intended to accommodate left turn storage lanes within the median - 5.0 m.

The minimum basic median cross-section for use on freeways in urban areas shall consist of 6.0 m median utilizing a concrete median barrier. Bridge piers shall be accommodated in accordance with DD-918-C.

The standard median width for rural freeways will be 20 m except as noted. This 20 m median provides for an additional lane of 3.5 m in each direction with a resulting 13 m depressed median. The subsequent addition of 3.5 m lanes in each direction would result in the standard 6 m urban cross-section. (See Figure 1).

2.2 (continued)

Exceptions to the standard value of 20 m should be justified but may include:

i) Conditions where construction staging may include at-grade intersections.

ii) Locations where terrain or property costs are such that independent alignments are appropriate.

Under these conditions the basic median widths should be based on increments of 5 m.

Median widths designed for multiple use, including transit systems, should be designed on a project specific basis in conjunction with the Highway Design Office.

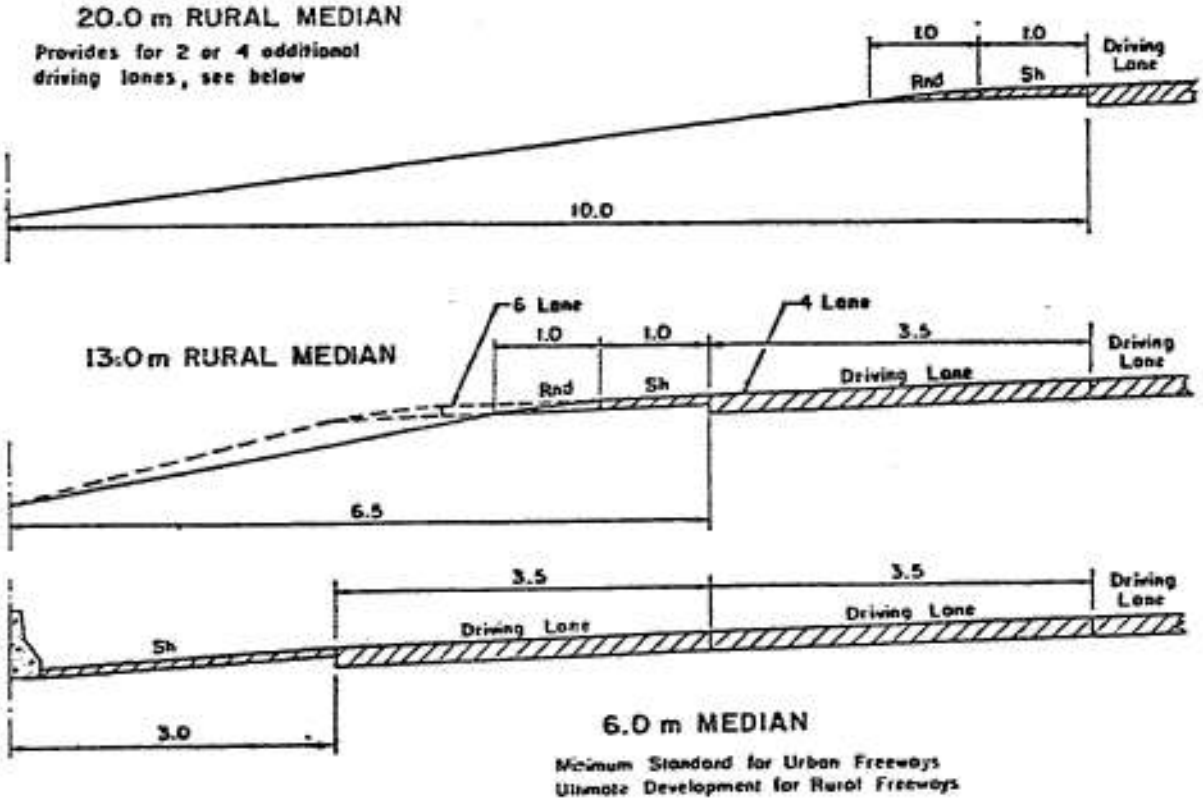


Figure 1

2.2 (continued)

MUNICIPAL ROAD AND BRIDGE DESIGN CRITERIA

MUNICIPALITY OF _____

(A) EXISTING DATA SUMMARY

Road ☐ or Bridge ☐ Urban ☐ Semi-urban ☐ or Rural ☐

Inventory Section No. _____, A.A.D.T. _____, Year _____, Capacity _____

Road: Name and/or Number _____ Length _____

From _____ to _____

Bridge: MTC Src No. _____ Inventory Structure No. _____ Local Bridge Name _____

Location _____ Type of Crossing _____

Safe Load Limit _____ Roadway Width _____

(B) PROPOSED DESIGN ELEMENTS

Urban ☐ Semi-urban ☐ Rural ☐

Road: Metric ☐ or Imperial ☐ Growth due to Improvement _____

Growth Factor _____ Design Year _____ A.A.D.T. _____ Design Class _____

Design Hour Volume _____ Directional Split _____ % Trucks _____

Design Speed _____ Maximum Gradient _____ Property Required _____

Maximum Horizontal Curvature or Minimum Horizontal Radius _____

Minimum Vertical Curves: Crest _____ Sag _____

Number and Width of Lanes: Traffic _____ Parking _____

Widths: Median _____ Boulevard _____

Shoulder _____ Roadway _____

Surface Type and Depth _____

Bridge: Type of Structure _____ Sidewalks _____

Width: Between Curbs _____ Between Protective Barriers _____

(C) GENERAL

Degree of Improvement: Spot Improvement ☐ To Tolerable Standards ☐ To Design Standards ☐

Type of Improvement: Complete Reconstruction ☐ Base and Surface ☐ Resurface and Minor Widening ☐

Estimated Cost: Road _____ Bridge _____ Total _____

Soils Investigation and Report to be provided: Yes ☐ No ☐

Design: Complete ☐ Partial ☐ None ☐
by Municipality ☐ by Consultant ☐

Planned Construction Phases and Estimated Costs (1) 19___ _____ $_____ (2) 19___ _____ $_____
(3) 19___ _____ $_____ (4) 19___ _____ $_____

Construction Planned by: Contract ☐ Day-Labour ☐

Construction Supervision: Municipality ☐ Consultant ☐

Remarks: _____

Date _____ Date _____

For Municipality For Ministry of Transportation and Communications

MR-A 10 82-04

2.2 (continued) GUIDE TO MUNICIPALITIES WHEN COMPLETING A MUNICIPAL ROAD AND BRIDGE DESIGN CRITERIA FORM

(A) EXISTING DATA SUMMARY

- Indicate if the Design Criteria is for road work or for the construction of a bridge. If both a road and bridge are involved; check both squares.
- Indicate if the road section to be improved is "urban", "semi-urban" or "rural".
- Inventory Section No. refers only to those municipalities which have had a needs study carried out. The Inventory Section No. is the same as that referred to on the pertinent appraisal sheet.
- The most recent A.A.D.T. (annual average daily traffic) and the year the count was taken is to be shown.
- The capacity of the existing road is expressed in terms of a two way hourly volume.
- The local road name together with Concession or Lot line to be shown after "Name".
- The length may be shown in kilometres or miles to the nearest one-tenth.
- The beginning and end of project to be shown, e.g. From Lot 10 to Lot 26 inclusive.
- All municipal bridges (span of 6 m (20 ft.) or greater) should have an MTC Structure Site No. A site number may be available for culverts under 6 m (20 ft.). The pertinent number is to be inserted.
- Likewise municipal bridges have a local number; this too should be included.
- The bridge location to be given by Lot and Concession in rural areas and by street name in urbans.
- The type of crossing such as creek, river, grade separation, railway overpass, etc. is to be shown.
- The existing load limit of the structure can be determined by a Professional Engineer or estimated. If estimated, it should be so indicated.

(B) PROPOSED DESIGN ELEMENTS

- Indicate whether the plans are to be prepared in Metric or Imperial units.
- Traffic data not required when it is not a design factor. (e.g. local residential)
- When a road has been improved, it usually attracts more traffic. This value may be estimated and entered as an A.A.D.T. for Growth due to improvement.
- The growth factor takes into account the amount the traffic volume will increase year by year until the Design Year.
- The Design Year A.A.D.T. is derived by adding the existing A.A.D.T., the growth due to improvement and the volume increase arrived at after applying the growth factor. The design year commonly represents the end of the economic life of the physical components of a road such as pavement structure, drainage structure, etc.
- For design class, show traffic volume range e.g., design year A.A.D.T. of 2550 would fit into the 2000 to 3000 design class.
- The Design Hour Volume generally should be the 30th highest hourly volume the roadway is expected to have during the future year selected for design. (It is deemed uneconomical to attempt to provide for a greater congestion than the 30th highest hour.)
- The Directional Split is the volume in each direction expressed as a percentage of the total Design Hour Volume.
- The % trucks (or commercial vehicles) is expressed as a percentage of the A.A.D.T. Generally the traffic composition determined from current traffic figures is sufficiently stable to be used for future design purposes. (Note: trucks tend to avoid peak periods of traffic.)
- Design Speed may be expressed in kilometres/hour or miles/hour.
- The Maximum Gradient is expressed as a percentage of the grade change to specified length of road, e.g. a 1% grade. There are recommended maximum gradients based on Design Speed and Traffic Volumes.
- Indicate whether or not property is required by a "Yes" or "No". If necessary, explain in the Remarks column of Section (C).
- If using metric units enter the minimum horizontal radius. If using imperial units, enter the maximum horizontal curvature.
- For the applicable minimum curves and width of lanes, please refer to the appropriate MTC Geometric Design Standards.
- A median may be used in a divided roadway and its width is considered to be the distance between the face of inside curb in one direction and the face of the inside curb in the opposite direction. If curbs are not involved, the median width is considered to be the distance between the inside travelled lane in one direction and the inside travelled lane in the opposite direction.
- The boulevard width is the distance between the face of the outer curb and sidewalk, in urban sections.
- The appropriate "shoulder width" may be obtained from MTC Geometric Design Standards and is applicable only to Rural and Semi-Urban Design. The normal 0.6 m (2 ft.) allowed for rounding is NOT to be included in the shoulder width.
- Roadway width is the travelled width between face of curbs in urban areas and the actual width from edge of rounding to edge of rounding if no curbs are involved. For divided roadways, it is the sum of such widths in each direction.
- An example of Surface Type and Depth is HL3 50 mm (2") in depth.

Bridges

- Examples of a type of Structure would be a "Single Span Concrete Rigid Frame" or "Two Span Prestressed Girders", etc.
- Sidewalks – give the number 1 or 2 and the width of each. If no sidewalk is required, indicate so by "N.A.".
- Width between curbs is the roadway width between the curbs.
- The width between Protective Barriers is the distance between base of parapet wall to base of parapet wall. Also it would apply to the distance from inside face of handrail or post to inside face of handrail or post. It should be noted that except for single lane structures, this width is not to be less than 8.5 metres (28 ft.).

(C) GENERAL

The information asked for in this section is considered self-explanatory. If you have any questions pertaining to this guide or the completion of Design Criteria form please contact the Municipal Staff in your local MTC District office, from whom additional forms are available.

2.3 Environmental Assessment of Project

(Reference should be made to the Class Environmental Assessment for Municipal Road Projects).

An environmental assessment may result in a re-evaluation of the road classification and the design criteria to be used. Collector streets are often most influenced by this review, for example, what might have been proposed as a 4 lane residential collector 13.5 m may be reduced to 2 lanes 9.5 m in order to protect a row of healthy mature trees.

Any such assessment will certainly involve a detailed site visit, taking along the design plans showing the existing topography and utilities. Check the information shown on the plan for missing or altered field information. Particular attention should be made with reference to any trees which may be affected by the work. Check and note drainage pipes which outlet into the existing ditches, and will require further attention. Environmental assessments will be dealt with further in a subsequent chapter.

2.4 Review of Proposed and Future Utility Requirements

All proposed utilities should be reviewed to ensure that they are compatible with the road proposals, and that their installation proceeds or is concurrent with the road construction.

All future utility needs should be assessed, and arrangements made for their installation, if possible, prior to or concurrent with the road construction. If it is not possible to arrange for the installation of a future utility, the design should be reviewed to allow for the future installation with a minimum of disturbance.

2.5 Review of Proposed and Future Development Proposals

Any proposed developments fronting the street, zoning changes, subdivision or submitted site plans, should be reviewed with particular reference as to their private access driveways, and their overall effect on the proposed road design. Access points to these developments do not come into conflict with bus bays, intersection channelization, or major drainage facilities.

2.6 Review of Road Allowance Requirements

An assessment should be made as to the desirable road allowance width in relation to the proposed road development, the existing road allowance, and the availability and cost of widenings. Road allowance widenings may be restricted to corner roundings and relatively narrow short widenings to facilitate the installation of channelizing an intersection, or may involve acquisition of a 2.5 m or wider strip down both sides of an existing road allowance.

Since property negotiations and acquisitions may take some time, the road allowance requirements should be carefully considered, and the necessary O.L.S. plans and financing of major acquisitions arranged well in advance of the road construction.

A review should also be made of the need for construction easements, slope rights, or permission to enter the properties adjacent to the road allowance.

2.7 Review of Existing Utilities

An evaluation should be made of any existing utilities which will be particularly affected by a proposed road widening or realignment. Since it may take some time for arranging the relocation of a major hydro or Bell facility, it may be desirable to arrange the financing for this work in advance of the road construction.

3.0 On Site Information Requirements

The following information is required before the designer attempts to establish the road profile.

3.1 A recent survey of all existing topography within the proposed road allowance and all pertinent topography adjacent to the roadway, together with details along intersecting roads, private access roads, and all developments adjacent to the roadway. The information may be obtained either by the use of aerial photography augmented by field information, or by conventional ground surveys. When possible, data should be tied into geodetic bench marks and the Ontario Co-ordinate System.

3.2 Cross sections and elevations should be taken every 10.0 m for urban work and every 25.0 m for rural work. 15.0 m minimum from centreline should be sectioned. Profile all private accesses to the house, garage or parking lot. Profile and cross section all intersecting streets for 100.0 m. Take elevations along all swales and low areas and any high spots.

3.3 Identification of Utilities

Identify the location and elevation of the storm and sanitary sewers, watermains, hydro cables, Bell cables, gas lines, and any other utility either below or above ground which may be expected to be encountered in the road allowance. It is also desirable to have the elevation of the associated manholes.

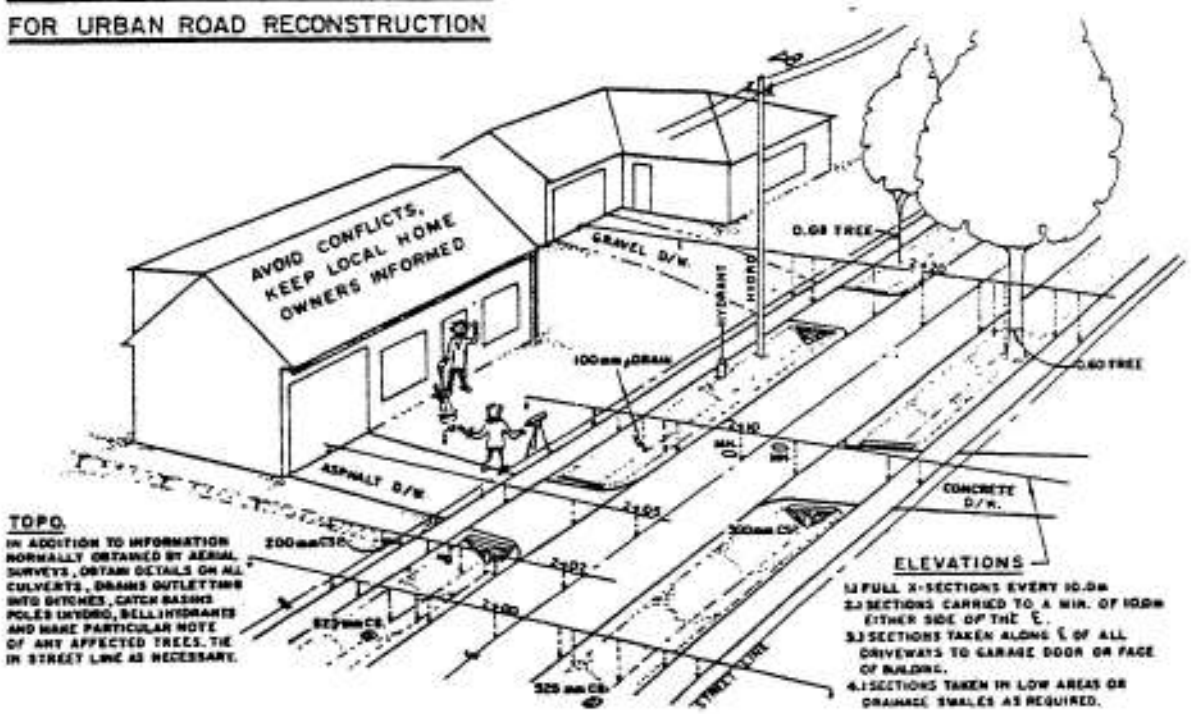
3.4 Soils information is required, including the determination of a rock profile where rock is encountered as consideration has to be given to the cost of installing utilities below the rock profile.

Suitable information may be available from previous work on an existing road allowance, or in the case of an unopened road allowance, from previous work in the immediate area. Otherwise, it may be necessary to have soil tests taken.

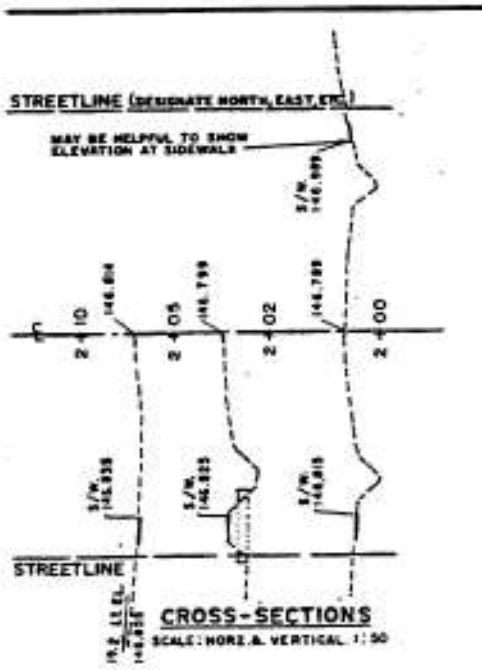
Consideration should be given to the transmission of vibrations, particularly in clay soils and with respect to collector or arterial streets. This problem can be modified by increasing the road bed depth.

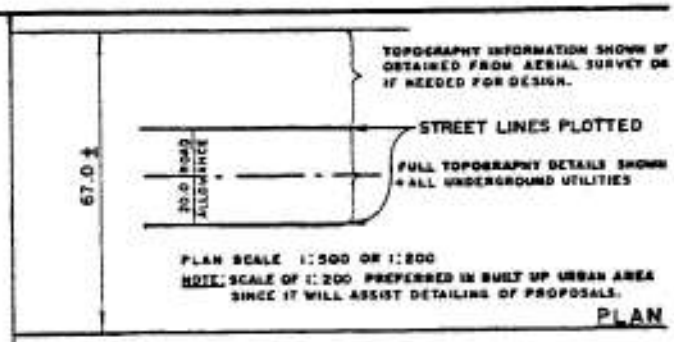
3.5

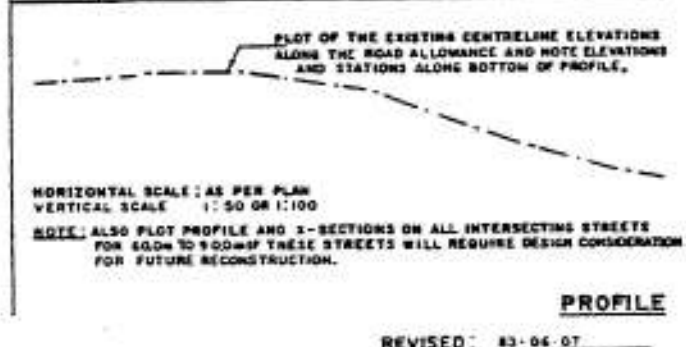
ON SITE INFORMATION REQUIREMENTS FOR URBAN ROAD RECONSTRUCTION



TOPOGRAPHIC INFORMATION IS TAKEN BY AERIAL OR GROUND SURVEYS AND TRANSFERRED TO A PLAN. IF GROUND SURVETS USED, USUALLY INFORMATION RESTRICTED TO ROAD ALLOWANCE LIMITS. ELEVATIONS ARE TAKEN BY GROUND SURVETS AND TRANFERRED TO THE PROFILE AND TO CROSS-SECTIONS. ALSO REFER TO 5.3.1 WHERE SIDEWALKS NOT EXISTING ON BOTH SIDES.







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4.0 Horizontal Alignment

The horizontal alignment and radius of curvature becomes more important as the normal vehicle operating speed increases. Generally the nature of a curve in a road should be readily anticipated by the driver.

The radius of curvature, pavement superelevation, tangent length between curves, etc. should all assist the driver to safely negotiate the turn.

It is most important that the road design classification is established at the planning stage of any new development in the urban area, and that the suitable minimum radius of curve is applied where necessary.

Too often it has been found necessary to upgrade a local road to the role of a collector, only to find that property purchases will be needed to ease an unsatisfactory bend in the road.

The "curvilinear design" philosophy provides the best combination of design standards and what looks right.

Some points to remember:

-Avoid minimum radii curves if possible.

-With small deflection angles use long curve radii.

-Long tangents should be joined by long radius curves.

-Avoid broken back curves. The absolute minimum tangent between curves in the same direction should be $\frac{4 \times \text{design speed}}{1{,}000}$

(ex. 80 km/h. Tangent .320 km or 320 m).

-Superelevated reverse curves should have a minimum connecting tangent of 100m to allow for change in superelevations.

A detailed discussion of the various aspects of horizontal alignment is readily available in the "Manual of Geometric Design Standards for Canadian Roads and Streets". The following general comments are made with respect to some of these.

4.1 Maximum Rate of Curvature

(Refer to Geometric Design Standards for Canadian Roads and Streets (B.3.2).

The curvature on a road should suit the design speed, with the running speed normally posted at 10 to 15 km/h less, except perhaps on local streets where the design and posted speed are often accepted as the same.

The following formula may be used for reviewing the design speed for a curve.

$$V = \sqrt{127R\ (e+f)}$$ where V = vehicular design speed in km/h

R = inside curb radius in metres.

e = rate of roadway superelevation in metres/metre

f = side friction factor

4.2 Maximum Superelevation (B.3.3) R.T.A.C. Manual)

Superelevation is not normally considered on local streets or on most collector streets. There may be the occasion where it is desirable to provide superelevation on these, particularly where the curvature is sharper than may normally be anticipated, or the curve is preceded by a steep gradient.

Superelevation on curves on arterial roads depends on the proposed operating speed along the section in question. In many locations where a maximum of a 50km/h posted speed will be imposed, superelevation is not needed, or in fact may not be practical due to the restrictions imposed by the properties fronting on the road.

Superelevation should be considered when the operating speed is proposed to be over 50 km/h.

To ensure that a vehicle can start from a stop on an icy pavement without side slipping, the maximum rate of superelevation used should be 0.06 m per m (6%).

In general "e" = 0 for intersections

e = 0.02 on most local and collector streets (i.e. normal pavement X-fall to inside of curve).

e = 0.06 on most Ontario rural roads

4.3 Side Friction Factor

The side friction factor varies with the speed, with one set of values normally used on through roads, and another at intersections due to lower approach speeds.

The friction factor is affected by the type of pavement and the condition of the surface. For example experience has shown that the surface asphalt using trap rock as an aggregate will provide a more skid resistant surface over a period of time than on with limestone. A concrete surface may be improved by cutting grooves.

The following are maximum safe side friction factors.

DESIGN SPEED KM/HR	15	25	30	40	50	60	70	80	90	100	110	120
"f" on through roads			.17	0.17	0.16	0.15	0.15	0.14	0.13	0.13	0.12	0.12
"f" at intersections	.32	.32	.27	.23	.20	.18						

Refer to Table B.3.2 in Geometric Design Standards.

4.4 Horizontal Sight Distance on Curves

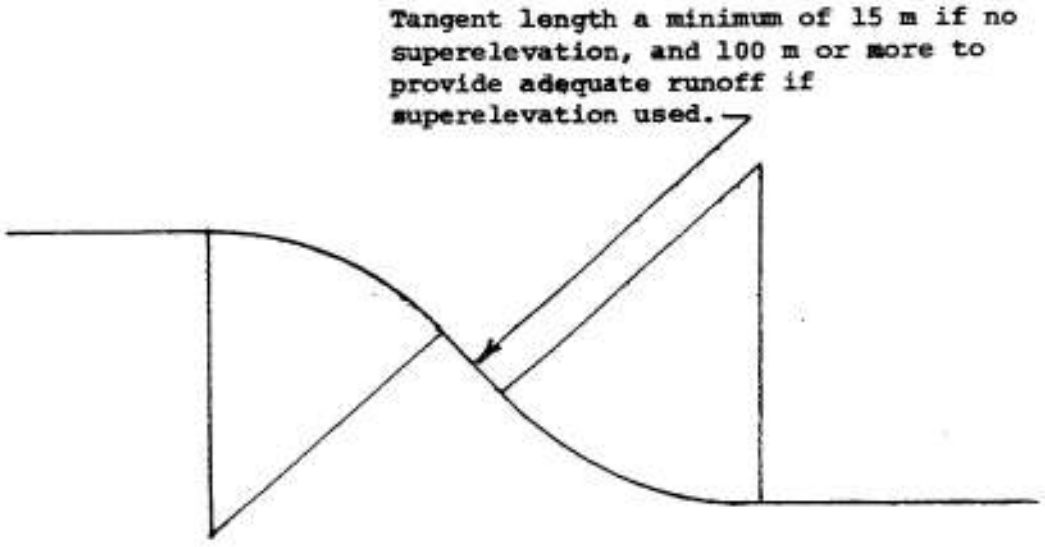
Particular care should be taken on sharp bends and at intersections to ensure that shrubs, trees or buildings do not interfere with the line of sight from one street to the other. Reference should be made to the sight distance charts available in the "Geometric Design Standards Manual".

4.5 Types of Curves

Simple curves, compound curves, spiral transition curves, and reverse curves may all be used in road design, however most curves are satisfactorily handled by the use of a simple curve.

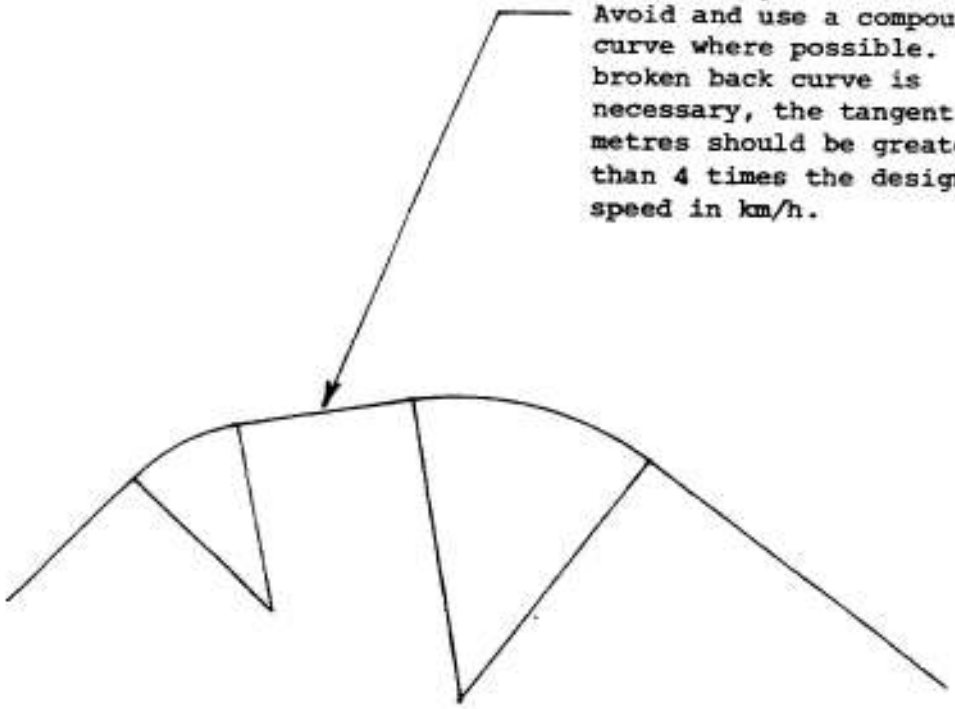
The need for general consistency in alignment is most important. If a sharp curvature is unavoidable on a through road, it is preferable that it is preceeded by a transition spiral curve.

Reverse Curve



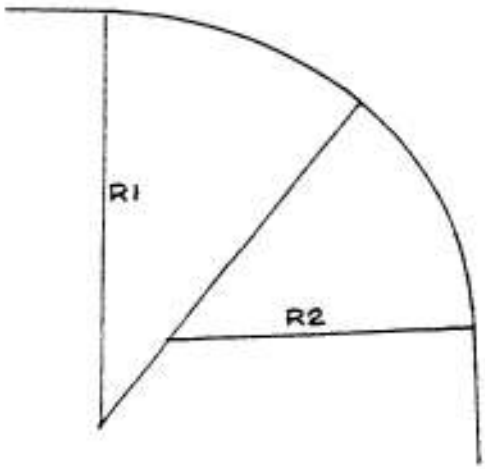
4.5 (continued)

Broken Back Curve



Avoid and use a compound curve where possible. If a broken back curve is necessary, the tangent in metres should be greater than 4 times the design speed in km/h.

Compound Curve



Avoid and use a simple curve where possible.

Use of this curve is often found desirable to facilitate the turning of trailer trucks at intersection, particularly if the angle of turn exceeds 105°.

When used

$\frac{R1}{R2}$ less than 1.5

4.6 Curves on Major Roads and Streets

On major (arterial and freeway) roads and streets, the upper design speeds in the "Geometric Standards" should be used wherever possible. The use of 100 km/h design (a 420 m radius curve) on an 80 km/h arterial road does not mean the running speed will be above the limit. The higher design standard is used to ease traffic movement by allowing the drivers to concentrate on the traffic and not have to worry about the geometry of the road.

4.7 Curves on Local Roads and Streets

Local rural roads are usually designed to the minimum acceptable standard. As the speed limit is usually 80 km/h a 250 m radius is to be used.

Local Urban streets because of the type of traffic, may be designed with tighter curves than the 90 m radius recommended for 50 km/h.

(a) Cul-de-sac or short crescents.

- Minimum inside radius 15 m.
- Pavement width should be increased through the bend area, particularly where the bend is in the vicinity of 90°. This widening facilitates vehicular turning movements and allows easier property access.
- To determine a design speed for such corners.

$$\text{Speed} = \sqrt{127R\ (e+f)}$$

$$= \sqrt{127 \times 15\ (0+0.32)}$$

$$= 24.7 \text{ km/h}$$

Therefore, a safe running speed for signing is about 20 to 25 km/h.

(b) Local urban streets with higher volume of traffic, industrial roads and bus routes in subdivisions, etc. require analysis as to traffic movement needs.

Normally an inside curb running speed of 40 km/h is desirable. Therefore, the inside curb radius should be.

$$40 = \sqrt{127 \times R\ (0.02 + 0.16)}$$

$$R = \frac{1600}{127\ (0.18)}$$

$$= 70 \text{ m}$$

4.7 (continued)

RECOMMENDED RUNNING SPEED	DESIGN RUNNING SPEED	USED ON	INSIDE CURB RADIUS
* 20 to 25 km/h	24.7	- intersections - cul-de-sacs - short crescents	15 m
20 to 25 km/h	25.8	- minor through streets	27.5 m
25 to 30 km/h	30.1	- through streets	37.5 m
30 to 35 km/h	34.7	- through streets	50 m
35 to 40 km/h	41.1	- through streets	70 m
40 to 45 km/h	45.4	- through streets	90 m
45 to 50 km/h	50.1	- through streets	110 m

* Use lower value on higher volume streets

4.8 Horizontal Alignment Examples

(a) Urban

ROAD CLASSIFICATION	DESIGN YR AADT	LANES THROUGH (PARKING)	ROADWAY WIDTH CURB TO CURB	ROAD USE	SPEED KM/H DESIGN	SPEED KM/H OPERATION	MINIMUM RADIUS	MAXIMUM SUPERELEVATION
Local	less than 1,000	2 (1)	8.5 (2.5)	residential cul-de-sac	25	25	15	None
				crescent	25	25	15	None
				through	40	40	70	None
		2 (1)	9.5 (3.0)	industrial through	50	45	90	0.06
Collector	1,000 to 10,000	2 (1)	9.5 (3.0)	through	50	50	110	None to 0.06
		2 (1)	10.0 (3.0) or 13.0 (3.0 & 3.0)	through	60	55	130	None to 0.06
		2 (2)		through				
Arterial	6,000 and up	4	14.0	auto	60	55	130	0.06
		4	15.0	mixed use	80	70	250	0.06
Freeway	120,000 and up	4 plus	3.5 to 3.75m per lane	mixed use	100	80	420	0.06

For values other than minimum see MTC Directive C-63 "Geometric Design Standards"

4.8 (continued)

(b) Rural

ROAD CLASSIFICATION	DESIGN YR AADT	LANES	ROAD WIDTH SHLD-SHLD	SHOULDER WIDTH	ROAD USE	SURFACE TYPE	SPEED KM/H		MINIMUM RADIUS	MAXIMUM SUPERELEVATION
							DESIGN	POSTED		
Local	less than 400	2	6.0	1.0	property access	LT	70	unposted	190	0.06
Collector	400 - 1,000	2	6.5	1.0	commuter	HT	80	80	250	0.06
	1,000 - 2,000	2	6.5	2.0	mixed	HT	100	80	420	0.06
Arterial	1,000 - 2,000	2	6.5	2.0	mixed	HT	100	80	420	0.06
	2,000 - 4,000	2	7.0	2.5	mixed with high truck useage	HT	100	80	420	0.06
	4,000 and up	as suits	3.5 to 3.75 per lane	2.5	"		110	80 or 90	525	0.06
Freeway		as suits	3.75 per lane	3.00	mixed with trucking		120	100	650	0.06

For values other than those shown see MTC Directive C-63 "Geometric Design Standards"

5.0 Vertical Alignment

Vertical alignment becomes more important as vehicle operating speed increases.

5.1 Gradient

The steeper the upgrade, and the longer the upgrade, the greater the reduction of vehicular speed. The graph on page 52 shows the effects of 2% to 8% gradients on trucks.

The maximum grades for various speeds are as follows:

DESIGN SPEED	MAXIMUM % GRADE
50 km/h	8-12
60 km/h	6-12
70 km/h	6-12
80 km/h	6- 8
90 km/h	6- 8
100 km/h	6- 8

For reasonable traffic flow maximum grades of 8% for local roads; 6% for collector roads; and 5% for arterials are recommended. Where possible the designer should use values less than the maximum.

Some special cases:

(a) Urban streets should have a minimum 0.5% (0.35% may be considered if curbed and guttered) grade to achieve drainage to catchbasins.

(b) Trucks and buses may have problems with 5% grades in iced conditions.

(c) Local cul-de-sacs should have maximum down grade of 3% to minimize exit problems in icy conditions.

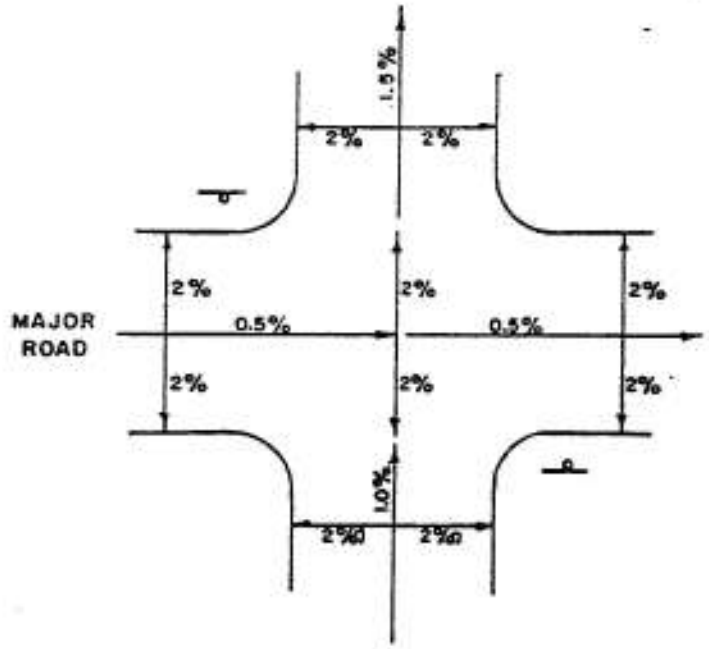
5.2 Gradient at Intersections

When designing intersections, think about traffic pattern and what should be the result of the design.

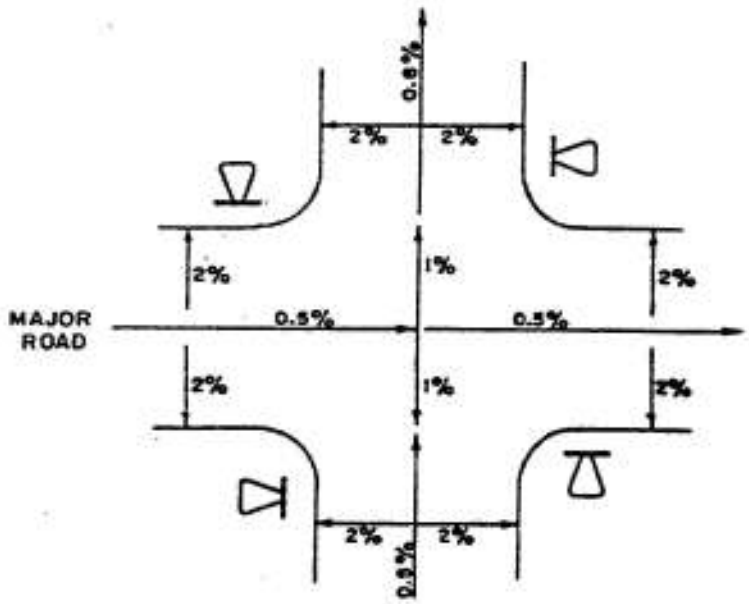
- The through road should have the continuous profile.
- The intersection should drain.
- Maximum and minimum grades should be 5% and 0.5%.
- A signalized interesection cross fall can be reduced to 1%.

5.2 (continued)

EXAMPLES:



THROUGH STREET / STOP STREET



SIGNALIZED INTERSECTION

5.3 Vertical Curves

A vertical curve is a parabolic transition from one tangent grade to another tangent grade. Vertical curves are usually inserted between grades changing more than 1%.

5.3 (continued)

Crest vertical curves are calculated to provide a stopping sight distance based on a height of eye of 1.15 m viewing a height of object also 1.15 m off the ground. Sag vertical curves are calculated to allow a vehicle dropping into the sag to see a reasonable distance up the other side. The sight distances have been calculated and are contained in tables by "K" factor. The "K" factor are specified for various design speeds.

Example:

DESIGN SPEED	K FACTOR	
km/h	CREST	SAG
50	8	12
60	15	18
70	25	25
80	35	30
90	50	40
100	70	45
110	90	50
120	120	60

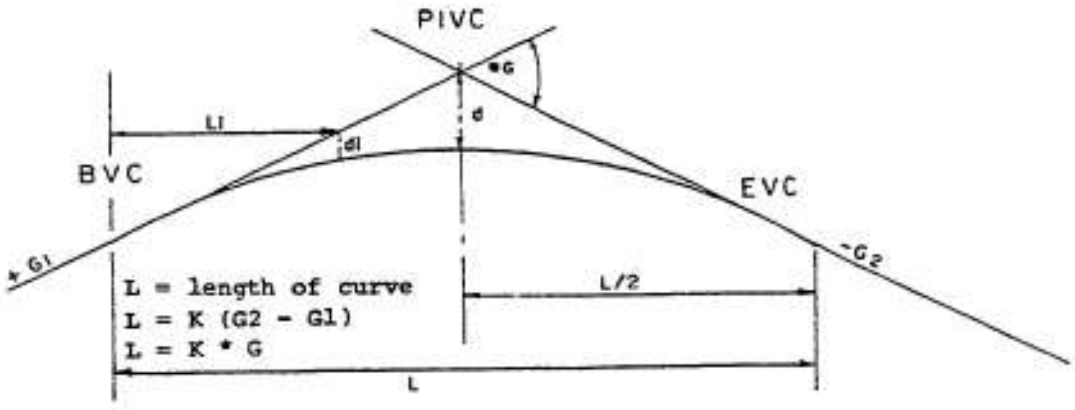
Vertical curves should where possible, be built to higher than the minimum standard. An analysis of the affects to topography; the smoothness of design and costs must be made before setting the K factor to be used.

In urban areas:

Particular attention should be made to ensure that the gutter drainage is not less than 0.35% through the flat area of the vertical curve, adjusting the road cross fall as may be necessary to achieve this.

On sag curves it is important to locate the low spots for the accurate placement of the catchbasins.

To avoid drainage problems along gutters, the maximum K factor for crests would be 60 and in sags 30.



5.3 (continued)

L = Length of vertical curve.

* = Algebraic difference in grade in %.

G = Tangent grade in % (uphill in direction of drainage if + downhills is -)

di = Tangent offset - the vertical distance from the tangent to the parabola in m at point i.

Li = The distance from the beginning of vertical curve to point i.

M.T.C's "Vertical Curve Tables" December 1977 explains and gives design aids for calculating ordinates of parabolic curves.

Usually grade points are calculated for the B.V.C., E.V.C., P.I.V.C. sufficient intermediate points to construct the curve. (in urban areas 10 m usually suffices while for long curves in rural areas 15 to 25 m gives enough control.)

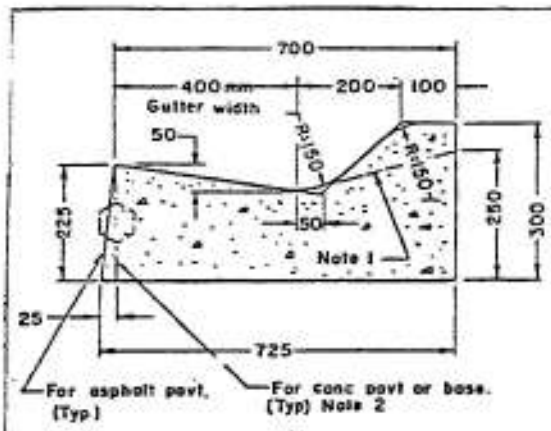
6.0 Road Cross Sections

6.1 Urban

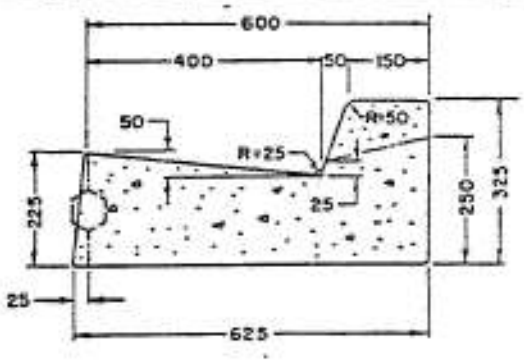
Curbs are used to delineate the outside edge of most urban roads. There are two types of curb drive through and barrier, built either as curb along or in combination with gutter.

When designing a cross section, road use and traffic are considered and the number of lanes and width selected. This, plus a 0.25 m curb clearance on each side establishes the road cross section from curbface to curbface.

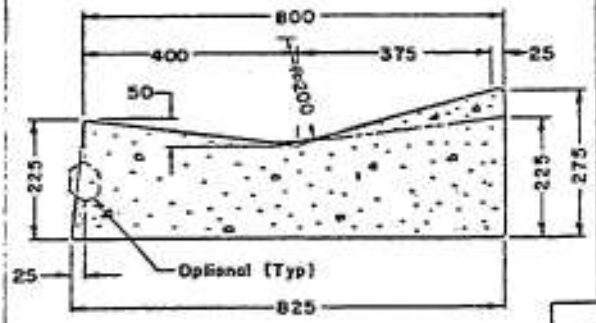
6.1 (continued)



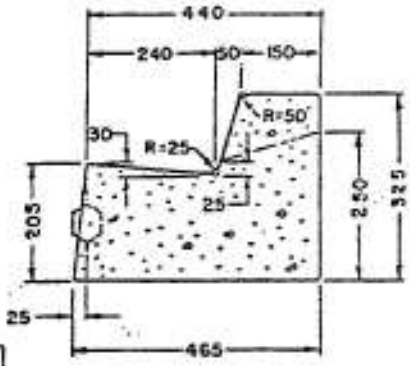
Type A : 0.156 m³/m



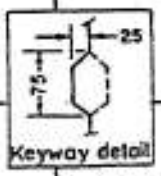
Type B : 0.143 m³/m



Type D : 0.172 m³/m



Type 6 : 0.109 m³/m



NOTES:

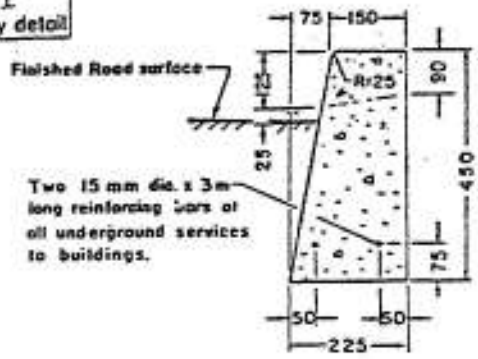
1. Length of dropped curb as shown on plans or as directed by the Engineer.
2. When curb & gutter is adjacent to concrete pavement or base, this Drawing to be used in conjunction with OPSD—

A. Top edges of front and back of curb & gutter to be rounded with a 6mm radius edging tool, except where front edge abuts concrete pavement with longitudinal joint.

B. The length of transition from one type to another shall be the greatest of:
   a. 50 x difference in overall curb heights
   b. 25 x difference in gutter widths
   c. 2.0 m

C. Minimum compressive strength of concrete to be 20 MPa at 28 days.

D. All dimensions are in millimetres unless otherwise shown.



Type H : 0.084 m³/m

ONTARIO PROVINCIAL STANDARD DRAWING

Date | Rev

CONCRETE CURB AND GUTTER

APPROVED Date _____

OPSD - 601

6.1 (continued)

GEOMETRIC DESIGN STANDARDS

FOR UNDIVIDED URBAN ROADS IN ONTARIO

DESIGN YR TRAFFIC		DESIGN SPEED	No of Lanes	LANE WIDTH	PARKING LANE WIDTH	CURB CLEARANCE	MINIMUM CURB to CURB DISTANCE	MINIMUM[c] GRADIENT
A.A.D.T.	D.H.V.	km/h		m	m	m	m	%
6000 and up	600 and up	80	4	3.5-3.75[a]		0.25	14.5	6-8
		60-70[a]	4	3.5		0.25	14.5	6-12
6000 to 3000	600 to 300	60-70[a]	4[b]	3.5		0.25	14.5	6-12
		80	2	3.5-3.75[a]	2.5-3.0[a]	0.25	10	6-8
		60-70[a]	2	3.5	2.5-3.0[a]	0.25	10	6-12
3000 to 2000	300 to 200	80	2	3.5	2.5-3.0[a]	0.25	10	6-8
		60-70[a]	2	3.25	2.5-3.0[a]	0.25	9.5	6-12
		50	2	3.0	2.5-3.0[a]	0.25	9	8-12
2000 to 1000	200 to 100	60-70[a]	2	3.25	2.5-3.0[a]	0.25	9.5	6-12
		50	2	3.0	2.5-3.0[a]	0.25	9	8-12
less than 1000	less than 100	40-50[a]	2	2.75-3.0[a]	2.5-3.0[a]	0.25	8.5	8-12

a Upper value is desirable, lower value is acceptable.

b Four lanes are appropriate in the upper limit of this traffic range where there is a measurable capacity deficiency with only two lanes.

c Lower value is desirable, upper value is acceptable.

- Horizontal and vertical geometric design standards shall conform to Alignment Standards.

- Auxiliary lane widths shall conform to criteria for Rural Highways.

6.1 (continued)

Boulevards

Boulevards are measured from the edge of pavement to the edge of sidewalk. Where boulevards are provided the desirable minimum boulevard widths are 3.0 m on arterial roads and 2.0 m on collector and local roads. The absolute minimum boulevard width should be 1.5 m.

Sidewalks

The desirable minimum sidewalk width is 1.5 m. Under restricted conditions an absolute minimum of 1.25 m may be used. Sidewalks of 2.0, 2.5 or 3.0 m may be considered in areas of high pedestrian activity.

The face of the curb should also be at least 0.5 m clear of any lighting or utility pole or any other vertical obstruction to ensure adequate clearance for bus or truck lean or sway.

Parking decreases capacity, impedes traffic flow and increases hazard. Parking when permitted should be parallel only. Parking lanes should be 3 to 3.5 m wide on major streets, so that they may be utilized as additional traffic lanes where necessary.

On local streets, a minimum width of 2.5 m should be provided for a residential street and 3.0 m on an industrial street.

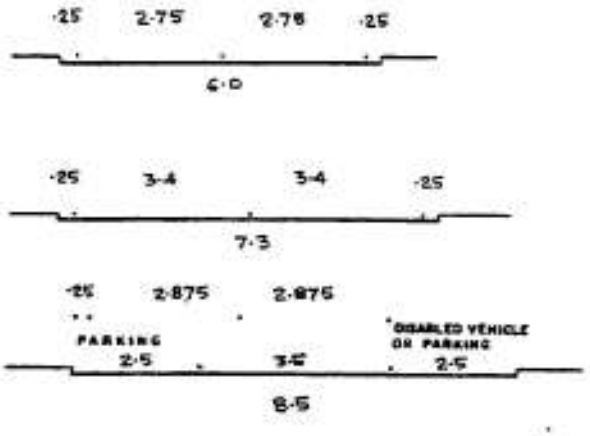
Turning lanes or bus pull in lanes should be a minimum of 3.0 m to 3.5 m.

Medians are sometimes used on major streets and are often used at major intersections. At intersections a 1.5 m median width is normally used adjacent to the left turn lane. Where the overall road is to be divided by a median, a 4.0 to 6.0 m width is desirable. This type of road is usually restricted to those with a minimum of street intersections and few if any private accesses.

6.1 (continued)

Examples Of Urban Road

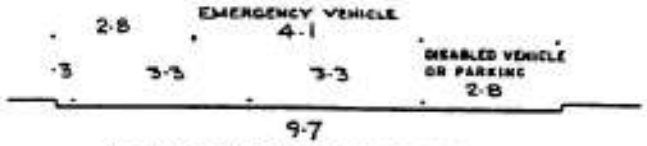
Cross Sections



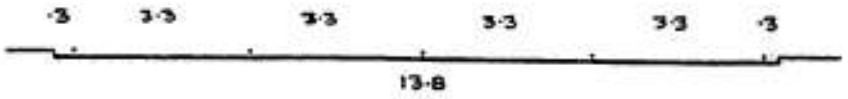
Normally restricted to lanes and private roads where additional off street parking provided

Passing of opposing vehicles normally facilitated by driveway openings.
On busier local streets, parking may be banned, restricted to one side or restricted within certain times

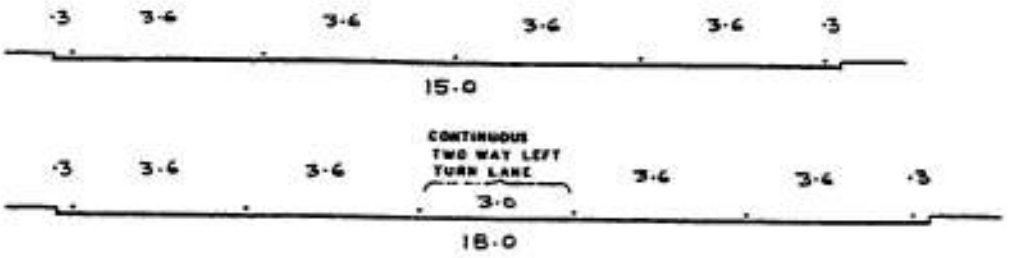
LOCAL STREET



2 LANE COLLECTOR OR LOCAL INDUSTRIAL STREET



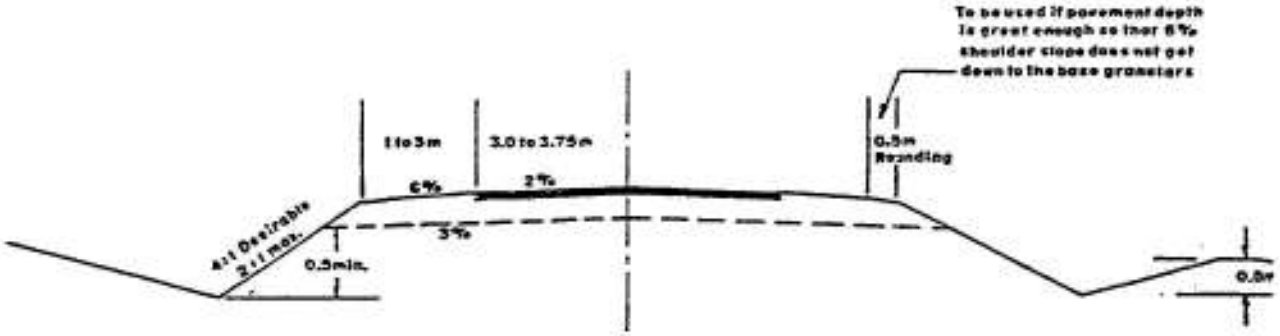
4 LANE RESIDENTIAL COLLECTOR STREET



ARTERIAL STREET

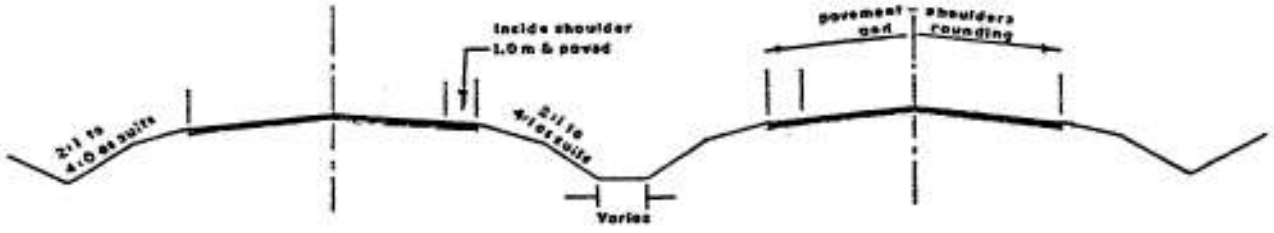
6.2 Rural

Where urban streets are usually bounded by curbs and vehicle escape and parking areas are often provided between the curbs, rural roads provide for vehicle escape and temporary storage with shoulders. Beyond that a foreslope, ditch and backslope complete the cross section.



2 LANE RURAL ROAD

Often the first 0.5 m of shoulder is paved to minimize edge cracking. If the shoulder or a part thereof is paved, the outside edge of the driving lane should be marked with a white line. Additional shoulder width from the minimums may be required in areas of guardrail.



4 LANE DIVIDED
RURAL ROAD

6.2 (continued)

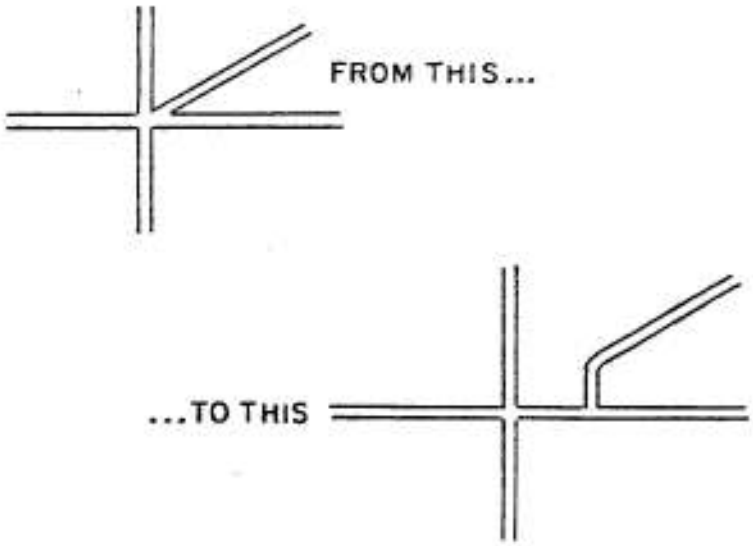
On divided rural freeways, the inside shoulder should be paved and the edge of the driving lane marked with a yellow line.

7.0 Intersections

Intersections are the meeting points of 3 or more roads or streets.

Five leg intersections are unsafe and inefficient and should be redesigned.

Example:



The secondary road should meet the major road at an angle between 70° and 90°. This allows drivers to get a good look in each direction before entering the intersection.

7.1 Continuous Two-way Left Turn Lane

Where there are a number of local intersecting side streets or high volume use driveways serving high density residential or commercial areas, it may be desirable to provide a continuous two-way centre left turn lane along an arterial road.

Such a lane should be 4.0 m wide (with a minimum of 3.0 m).

7.2 Major Intersection

Channelization is desirable at all major intersections. If the intersection is signalized, channelization assists traffic flow.

Right turn channelization is provided and if a significant number of right turns would cause delay (as in the case where pedestrian traffic reduces the time available for turning) or if it can be co-ordinated with a nearside bus bay.

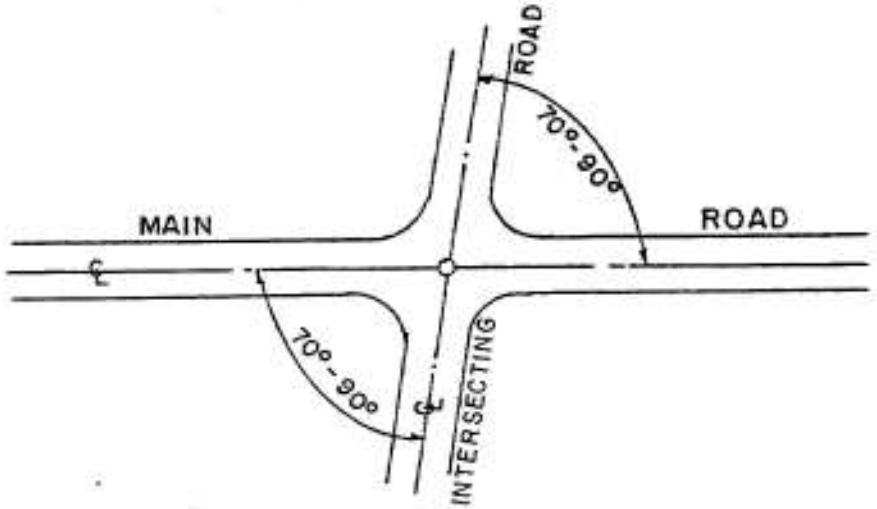
7.3 Angle of Intersection

It is most desirable to have intersections meet at or as near to a 90° angle as possible. The benefits of a near 90 degree angle of intersection are:

1. Reduces the angle of vehicle intersection
2. Reduces size of conflict area
3. Provides a more favourable condition for drivers to judge the relative position and relative speed of approaching vehicles and to change speed and/or to manoeuvre as required to avoid conflict.
4. A safer crossing area for pedestrians.

It is recommended, that where skew intersections are necessary, that the angle of intersection be not less than 70 degrees.

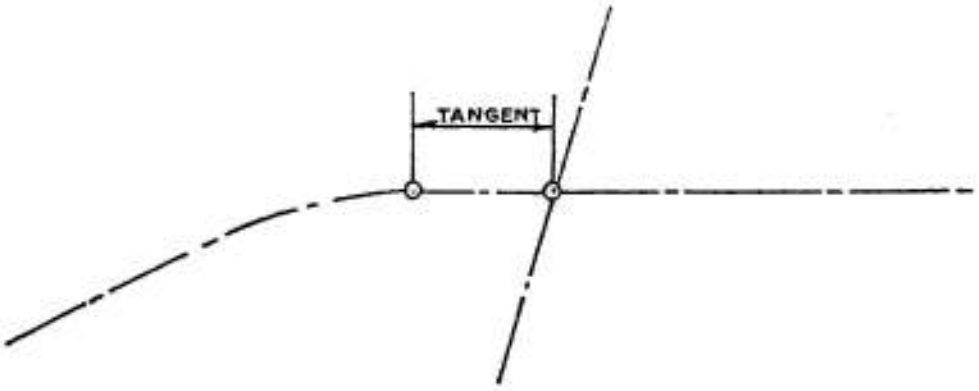
In the case of a cross intersection, the side road should intersect at the same angle on both sides of the main road.



ANGLE OF INTERSECTION OF INTERSECTING ROAD

7.3 (continued)

Where a curve is used approaching an intersecton, it is recommended that a minimum tangent length of 25 m be provided for a local road 40 m for a collector and 60 m for an arterial road.



Left turn channelization frees the through lane and is a must on all but low volume intersections. It is important that the storage length of the turn lane is adequate. Longer than necessary is desirable.

The following may be of some assistance.

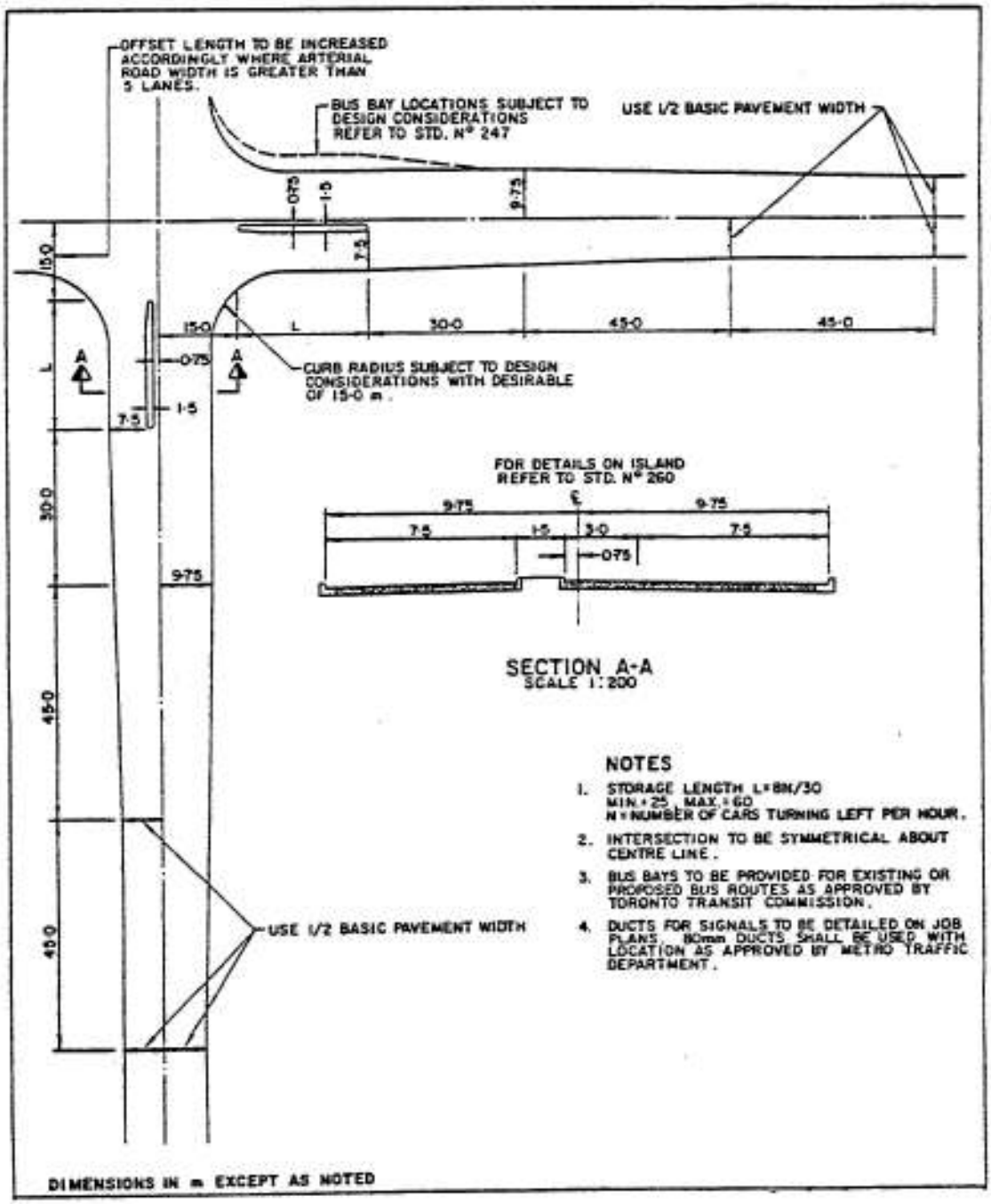
Length of left turn storage lane on Arterial Street	When intersecting with	Length of left turn storage lane on intersecting street
60 m	Arterial	60 m
60 m	4 lane industrial collector	60 m
60 m	4 lane residential collector	45 m
45 m	2 lane industrial collector	45 m
40 m	2 lane residential collector	30 m

Maximum 60 m, minimum 25 m.

In urban situations, median islands, particularly on 4 and 6 lane roads are desirable. They assist in traffic placement and provide pedestrian refuge. Care should be taken to make the curbs delineating the island free from sharp bends that will cause unpredictable traffic movements.

7.3 (continued)

INTERSECTION TWO PRIMARY COLLECTORS OR ARTERIAL ROADS



NOTES

1. STORAGE LENGTH L=8N/30
   MIN.=25, MAX.=60
   N=NUMBER OF CARS TURNING LEFT PER HOUR.
2. INTERSECTION TO BE SYMMETRICAL ABOUT CENTRE LINE.
3. BUS BAYS TO BE PROVIDED FOR EXISTING OR PROPOSED BUS ROUTES AS APPROVED BY TORONTO TRANSIT COMMISSION.
4. DUCTS FOR SIGNALS TO BE DETAILED ON JOB PLANS. 80mm DUCTS SHALL BE USED WITH LOCATION AS APPROVED BY METRO TRAFFIC DEPARTMENT.

7.4 Sight Distance at Intersections

From each approach to an intersection there should be sufficient sight distance along each intersection leg to allow vehicle operators to see approaching vehicles in time to avoid a collision.

At signalized intersections, this general requirement is not as critical, with the exception that signals should be visible at adequate distances.

At cross streets controlled by stop signs, sufficient sight distance along the major street should be provided to permit crossings from a stop position with safety.

For a full analysis reference should be made to the Manual of Geometric Design Standards for Canadian Roads and Streets.

Minimum Stopping Sight Distances for Turning Roadways:

Design Speed km/h	25	30	35	40	45	50
Minimum Stopping Sight Distance	25	30	35	45	55	65

Where parking is allowed along a major street, the parking lane should keep clear of parked cars at least 30 m to the left and 20 m to the right from the cross street centerline.

7.5 Intersection Curb Radii

Corner radii at intersecting streets depends largely on the classification of the intersecting roadways, and the volume and types of turning vehicles.

Smaller radii and encroachment on adjacent lanes may be considered at intersections where the turning vehicles are few in number, and where stop conditions are provided.

Where the angle of turn is greater than 90°, consideration should be given to the installation of a corner island, particularly where any appreciable volume of trucks turning movement may be involved. Reference should be made to the "Manual of Geometric Design Standards for Canadian Roads and Streets", with particular reference to the design of suitable compound circular curves.

The following minimum intersection curb radii are recommended for various street classifications.

8 m - at the intersection of local residential streets.

7.5 (continued)

9 m - at the intersection of local residential streets with collector or arterial streets.

11 m - at the intersection of local industrial streets.

- at the intersection of local industrial streets with collector or arterial streets.

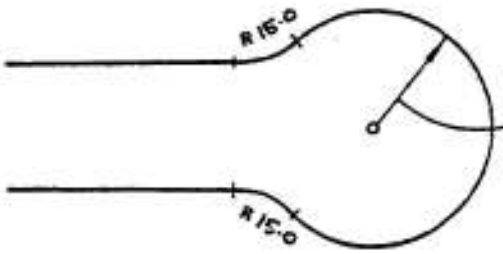
- at the intersection of collector streets.

15 m - at the intersection of arterial streets.

- at all intersections involving a high volume of turning movement by larger vehicles.

7.6 Culs-de-sac

It is desirable that all local roads which terminate at one end should be provided with a cul-de-sac of sufficient area to enable the turning of garbage and fire trucks as well as snow removal equipment and on industrial roads, semi trailer combinations.



Desirable cul-de-sac radii
9.0m R for residential streets
15.0m R for industrial streets

It may be desirable to consider a landscaped island where large turn around areas are to be provided on dead end residential streets. In these cases it is recommended that at least 6.0 m lane width be provided for the turning movement and snow clearing operations.

8.0 Driveways

Local municipalities may by the authority of a site plan requirement for new developments in the official plan, control the geometrics of driveways and private access. (Upper tier municipalities currently can control the water carrying conduit only but may, if proposed changes to the Public Transportation and Highway Improvement Act are approved, gain control of access geometrics and location). In some instances, entrance location can be controlled by utilization of .25 or .3 m reserve along the streetline. Another control in the turning movements of a road or street into a private entrance is the use of a continuous median island along the centre of the road. (Caution: when constructing this type of island, be sure the public has all the notice and opportunity to comment, required by the various statutes).

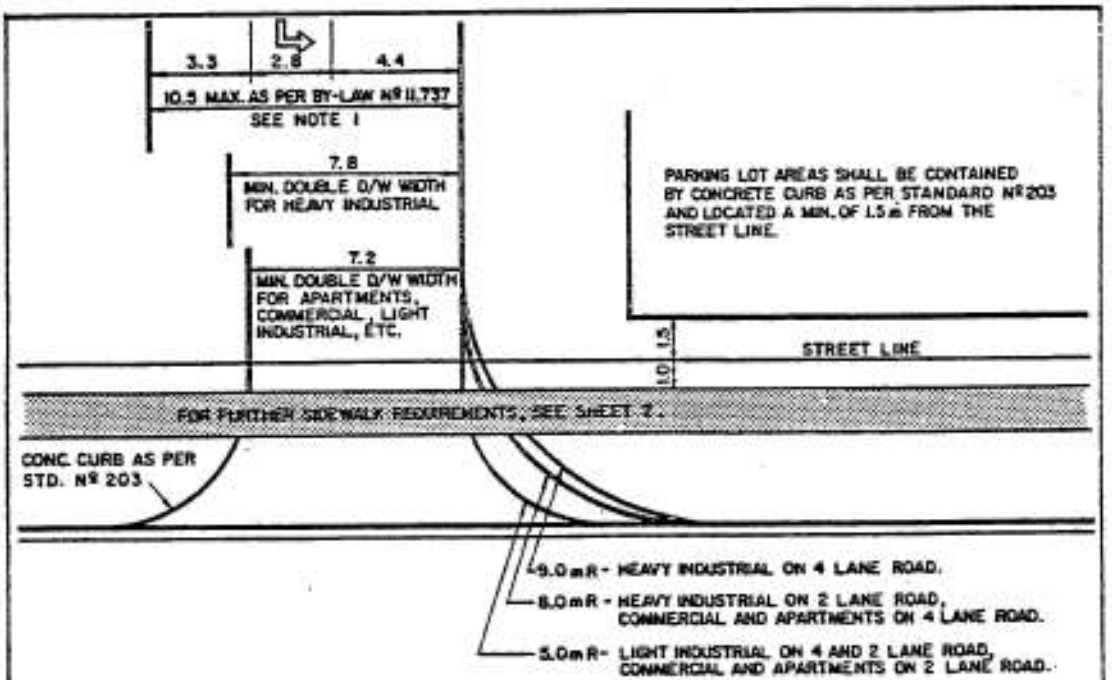
Driveway turning radii and pavement widths are usually reduced from those normally found on public roads, since the owner is prepared to accept some vehicle manoeuvreing problems for the sake of initial construction economics and overall land use requirements.

Particular attention should be paid to ensure that the proposed width and turning radii are compatable with the type and volume of vehicles anticipated to use the driveway. Fire trucks must always have ready access in and around the property. Moving vans and garbage pick up trucks must have adequate space to move on and off and within private property.

The following sample drawings, as well as reference to the drawings for various curbs as provided under section 5.2.2, Pavement Design, will provide some guidelines for the design of driveway accesses. (A set of turning templates is a valuable assistance when designing and checking driveway and turning radii for various vehicles. Templates may be purchased from the Roads and Transportation Association of Canada).

8.0 (continued)

INDUSTRIAL, COMMERCIAL AND APARTMENT DRIVEWAY REQUIREMENTS



TYPE	DEPTH OF SIDEWALK CONCRETE	DRIVEWAY PAVEMENT		
		ASPHALT	CRUSHED STONE	
HEAVY INDUSTRIAL	200 mm	40 mm HL3 150 mm (2 LIFTS) HL8	260 mm	
LIGHT INDUSTRIAL COMMERCIAL APARTMENTS	200 mm	40 mm HL3 100 mm HL8	260 mm	

NOTE: ALL DEPTHS ARE COMPACTED THICKNESSES IN ACCORDANCE WITH BOROUGH SPECIFICATIONS, AND ARE SUBJECT TO DESIGN REVIEW WITH RESPECT TO EXISTING SUBGRADE.

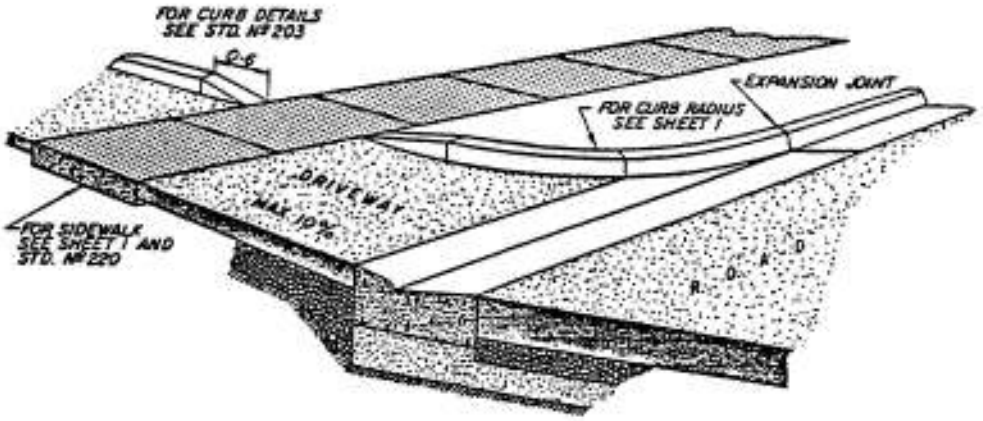
NOTES

1. FOR DRIVEWAYS WITH A HIGH VOLUME OF VEHICULAR MOVEMENT REQUIRING MORE LANES, MEDIAN ISLANDS, ETC., THE DETAIL DESIGN TO BE APPROVED BY THE ENGINEER.
2. SIDEWALK (WHERE REQUIRED) SHALL BE CONSTRUCTED IN ACCORDANCE WITH STANDARD Nº 220.
3. DRIVEWAY CURBS SHALL BE CONSTRUCTED IN ACCORDANCE WITH SHEET 2 AND STANDARD Nº 203.
4. RECONSTRUCTIONS OF EXISTING ROAD CURBS SHALL CORRESPOND WITH EXISTING CURBS IN ACCORDANCE WITH STANDARD Nº 201 TO 206.
5. FOR SERVICE STATION DRIVEWAYS REFER TO STANDARD Nº 250.

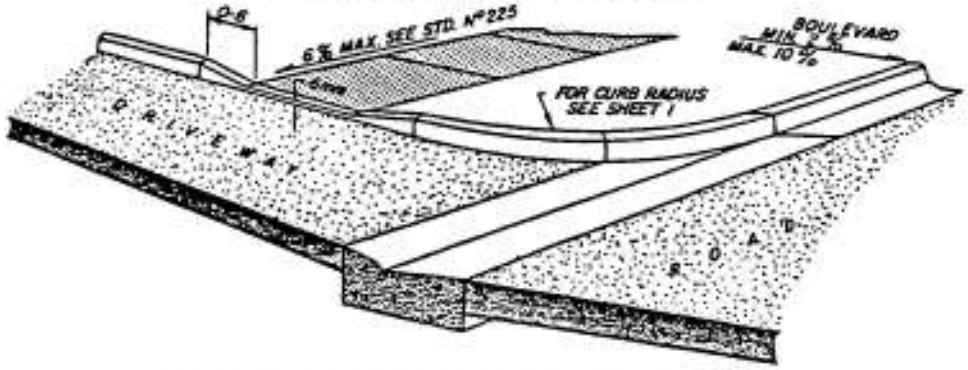
DIMENSIONS IN m EXCEPT AS NOTED

8.0 (Continued)

INDUSTRIAL, COMMERCIAL AND APARTMENT
DRIVEWAY REQUIREMENTS



NORMAL DRIVEWAY DETAILS



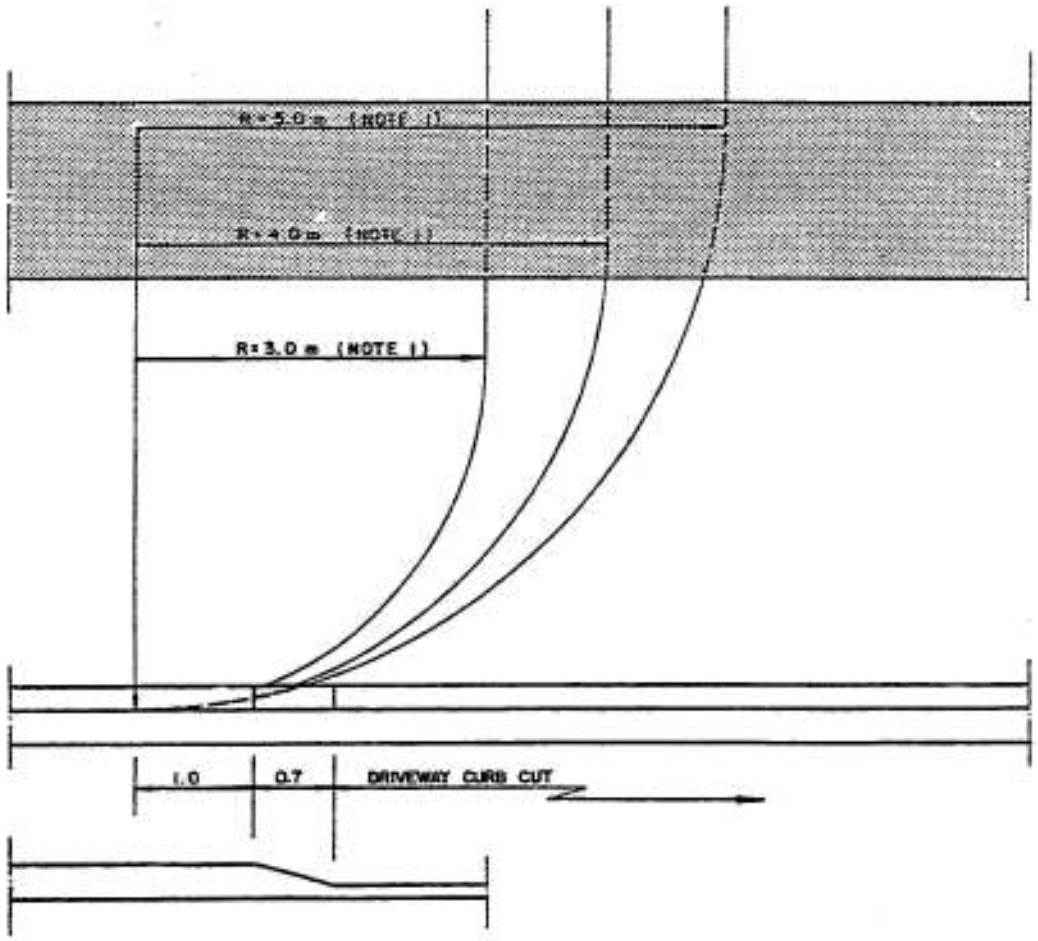
SIDEWALK TERMINATING AT DRIVEWAY FOR
SERVICE STATION OR MAJOR SHOPPING PLAZA ENTRANCE

NOTES

1. FOR PAVEMENT PURPOSES LENGTH OF DRIVEWAY CURB SHALL BE TAKEN FROM EXPANSION JOINT IN ROAD CURB.
2. FOR DRIVEWAY REQUIREMENTS, (CURB RADIUS AND DEPTHS OF CONSTRUCTION) SEE SHEET 1.
3. FOR DRIVEWAY CURB DETAILS SEE STD. Nº 205.
4. FOR SIDEWALK DETAILS SEE STD. Nº 220.
5. RECONSTRUCTION OF EXISTING ROAD CURBS SHALL CORRESPOND WITH EXISTING CURBS IN ACCORDANCE WITH STD. Nº 201 TO 206.

DIMENSIONS IN m EXCEPT AS NOTED

8.0 (continued)



NOTES

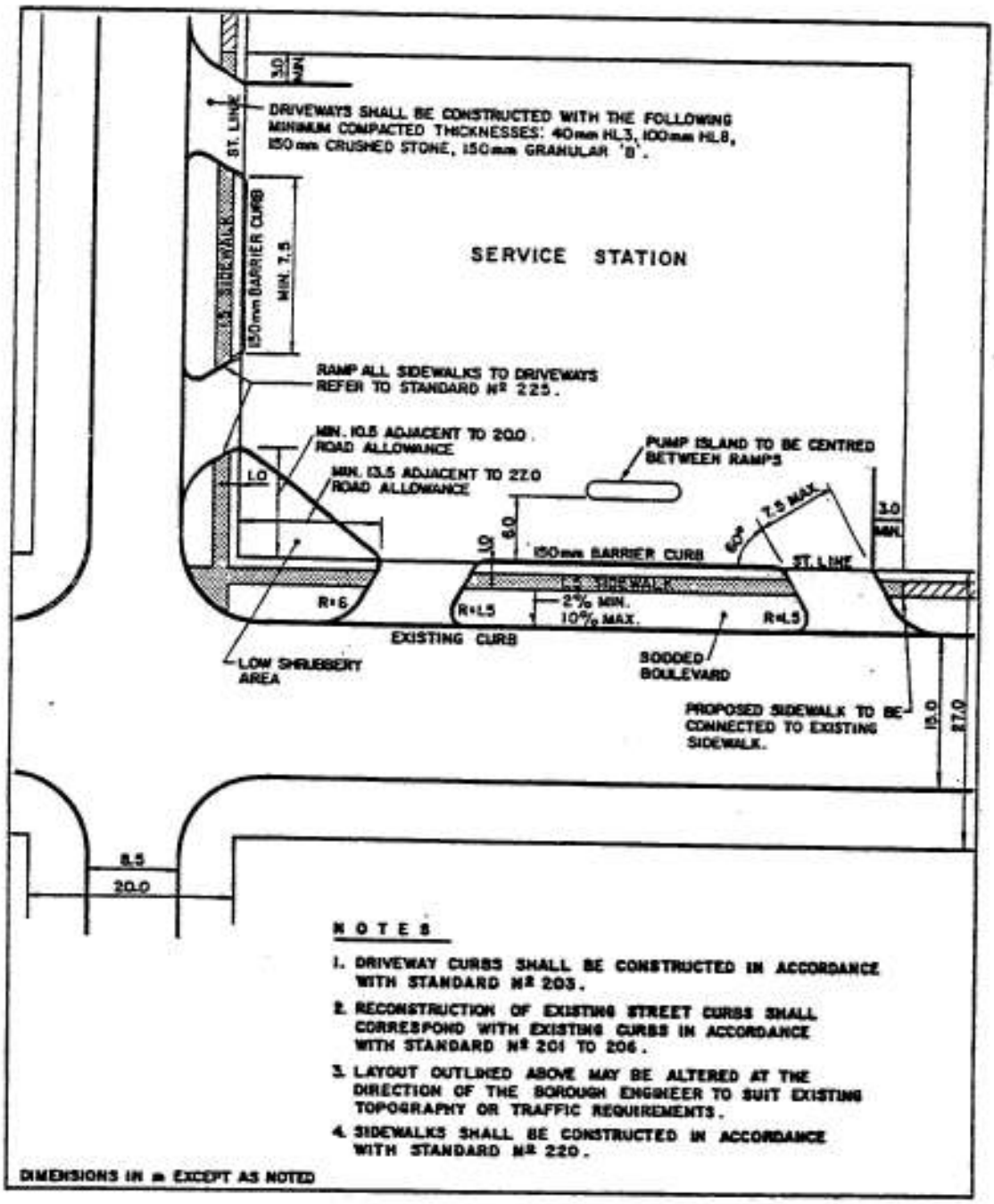
1. DRIVEWAY ASPHALT RADIUS AS REQUIRED BY THE ENGINEER OR AS SPECIFIED ON THE APPROVED DESIGN PLANS.
2. FOR FURTHER INFORMATION REFER TO SHEETS 1 AND 2.

INDUSTRIAL DRIVEWAY DETAILS WHERE DRIVEWAY CURBS NOT REQUIRED

DIMENSIONS IN m EXCEPT AS NOTED

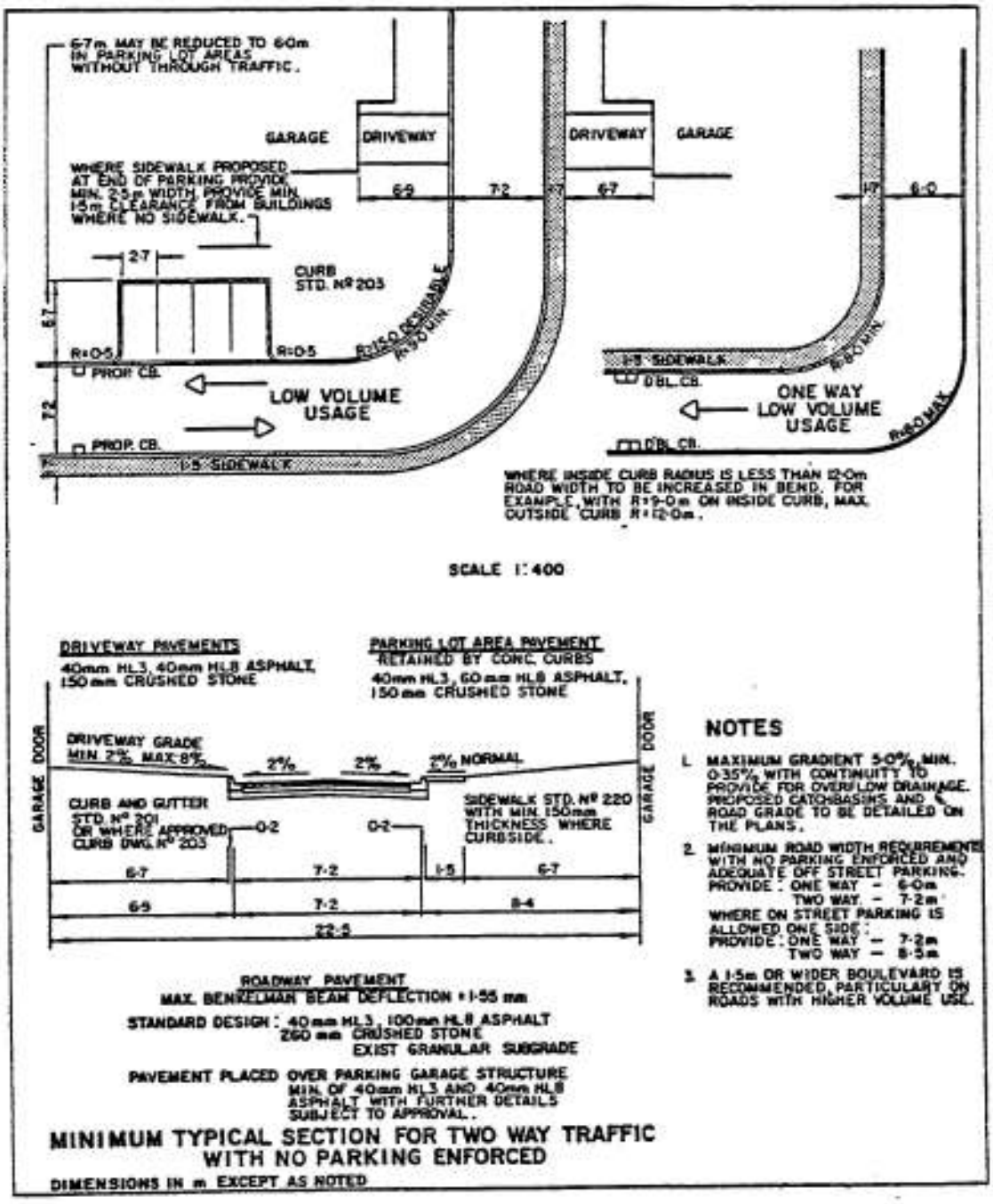
8.0 (continued)

SERVICE STATION LAYOUT



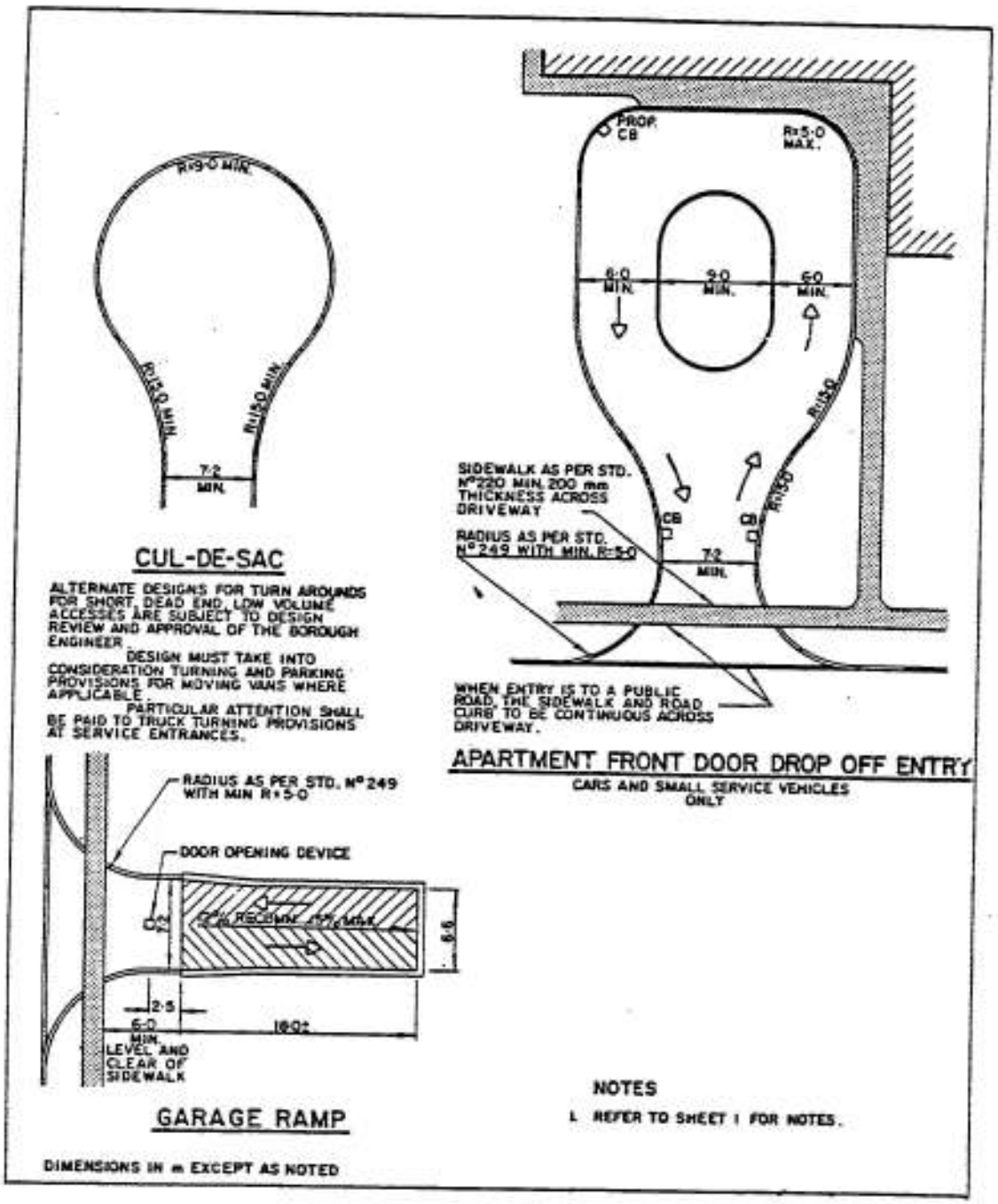
8.0 (continued)

TYPICAL VEHICULAR ACCESS DETAILS FOR CONDOMINIUMS & PRIVATE DEVELOPMENTS



8.0 (continued)

TYPICAL VEHICULAR ACCESS DETAILS FOR CONDOMINIUMS & PRIVATE DEVELOPMENTS



9.0 Vertical Design Anaysis

9.1 Urban

The vertical design grades on an urban road are often governed by the need to ensure adequate drainage gradients from adjacent properties, since the drainage generally must be carried over the proposed (or future) road curb. Urban horizontal alignments may be affected by existing buildings and property restrictions. Often the horizontal and vertical alignments must be considered together to achieve the most satisfactory design satisfying a particular set of circumstances.

The primary concern in the design of local streets is the affect on the fronting homeowner, since they will be the major user of the road, and the design is adjusted accordingly. Considerable adjustment to the grade in order to provide satisfactory vehicular access to the adjacent properties may be necessary.

The primary emphasis in the design of collector and arterial roads is the through traffic, and therefore certain of the recommended limits for various design factors are modified. This may result in a design on a major road which is not as satisfactory to the fronting properties as would have been a similar circumstance on a local road.

The factors of horizontal alignment, safety, sight distance, speed, construction costs, property available for the road allowance, and the location of utilities must also be considered.

The following procedure is suggested when establishing a proposed road profile for an urban road section. This procedure applies both where curbs are proposed in the immediate construction, or ditches are proposed with curbs to be installed at some future time.

a) On the existing centreline profile, indicate the elevation of all driveways at the streetline, or at the road side edge of existing sidewalks.

b) Develop a typical section for the proposed road, taking into consideration that the ideal condition for any road section is to be at an elevation so that the boulevards fall from the streetline to the curb is at a minimum slope of 2%.

9.1 (continued)

c) Mark on the profile a trial grade that will satisfy the proposed typical section, and is as smooth and as long as possible. The recommended minimum grade is 0.5%, although a minimum grade of 0.35% is sometime used together with a concrete curb and gutter section.

d) Review the trial grade with reference to driveway access gradients (not greater than 10%), drainage from low areas, curbed road grades on intersecting streets, utility conflicts, and topography (trees in particular). Adjustments may be made accordingly to the proposed road alignment, width, grade and pavement crossfalls, taking into consideration the road classification, to achieve the best overall design solution. Changes with reference to the road width and alignment, particularly if these will result in a road which will be less than the normal or standard requirements, should be documented and any necessary approvals received.

e) Review the proposed grade with reference to the continuity of drainage in the area. It is important that the grade is set to ensure that in the event of a major rainfall which exceeds the capacity of the storm sewer system, the drainage may be carried off on the surface without flood damages to buildings. This may involve adjustments to the proposed road grade, drainage easements, or storm sewer oversizing.

f) If a length of road grade requires dropping to pick up a very localized low area, review if it would be more economical overall to handle the drainage from the low area by means of a storm sewer inlet at the streetline. This may be done by a catchbasin at the road gutter with a culvert connection carried back to the streetline to pick up the drainage; or a catchbasin placed back at the streetline. Often the culvert - CB solution is easier to install. This is particularly true if there is already a culvert draining the area under an existing sidewalk.

g) At this point it is probably advisable to take a walk or slow drive along the road. Visualize the affect of your proposed road grade on the existing conditions. Review your proposals for handling low drainage areas. Check out driveways, particularly if steep gradients are involved, and if the existing road is to be cut. Check any affected trees for protection or special notes. Check pipes discharging into the ditches and note the action to be taken for connection to the storm sewer system, or to have them disconnected. Check intersecting streets and the limits of the proposed work for any special provisions to intercept surface drainage, length of proposed tapers, and end of proposed curbs.

9.1 (continued)

h) Review the proposed grade with particular reference to any underground utilities which may be left with less than the required minimum cover. Evaluate if an adjustment in the grade or road alignment is possible, or practical, to avoid the conflict. It may be necessary to carry out a cost analysis of several possible solutions to achieve the best results. Also consider the long term maintenance and road use in the solutions. Watermains particularly may be affected, requiring from 1.5 m to 2.0 m of cover, depending on the frost penetration in the area and the local experience with maintenance problems. Gas mains should have sufficient cover below the proposed subgrade elevation to ensure safety from damage by the construction equipment.

Similarly hydro and Bell cables should be well below the subgrade, although not as critical as the gas lines.

In all cases of anticipated conflict review the proposals, minimum cover requirements and any utility relocations with the local utility authorities.

i) Insert vertical curves between grades changing more than 1% on collector and arterial roads, and 1.5% on local roads, with the length of the vertical curve being dependent on the design speed.

j) Establish pavement design (see 5.2.2) and then plot proposed detail sections. This provides a further chance to review the proposed grades and the crossfalls in particular.

- remember that access to the road from the driveways should be as smooth as possible. Any difficulty experienced negotiating driveways after construction will be a constant source of annoyance. Generally where a 3.5 m boulevard is maintained between the curb and the sidewalk, a driveway grade of at least 5% is required to ensure a 2% boulevard grade.

- driveway grade should be kept as close as possible to this gradient, with a maximum gradient of 10% suggested if problems are to be avoided.

- any residential driveway with a gradient in excess of 10% or any steep heavily used driveway should be carefully checked in relation to the vehicles using the driveway, with every possible design adjustment made to assist the access.

- in any location where sidewalks are not existing or immediately proposed, take into consideration their future installations when establishing the grades on the boulevards. Steep boulevards make later sidewalk installations difficult, and may cause problems where a new driveway access is required.

9.1 (continued)

- a further site visit may be desirable, particularly if there are any aspects of the design which may require further clarification. For example a driveway in excess of 10% (perhaps reconstructing an existing sidewalk at a lower grade would assist?), or a proposed severe change in crossfalls (can the change be spread over a greater distance and still avoid damaging the large trees?).

9.2 Rural

Basically a rural road is built up ("High"-Way) base drainage, snow removal (and storage in the resultant ditches) all benefit from a high grade line. Vertical alignment is controlled mainly by topography, with a concern to balance cut and fill quantities. The factors of horizontal alignment, safety, sight distance, speed, property, location of utilities, construction costs and the performance of heavy vehicles on a grade must also be considered.

In flat terrain the grade line is usually controlled by soil conditions and drainage considerations. Since the drainage off the road surface is normally shed to ditches, the longitudinal gradient may be decreased to a minimum of 0.15%.

Ensure that the grade line set will allow adequate cover for any underground utilities, and particularly drainage culverts. Adequate clearance should be provided over swamp areas.

The effect of a grade on driveways, intersecting roads, adjacent roadside developments and scenic locations require thorough review.

From a truck operation standpoint, a profile with sections of maximum grade broken by lengths of flatter grade is preferable to long sustained grades.

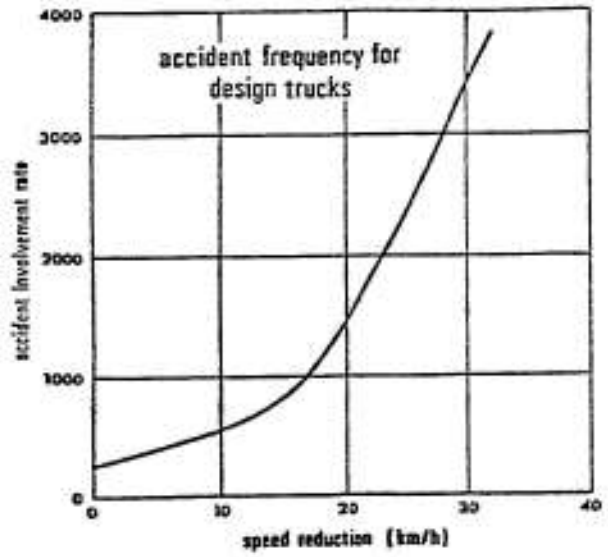
When critical lengths of upgrades cannot be avoided, consideration should be given to providing climbing lanes, particularly where trucks volume is high.

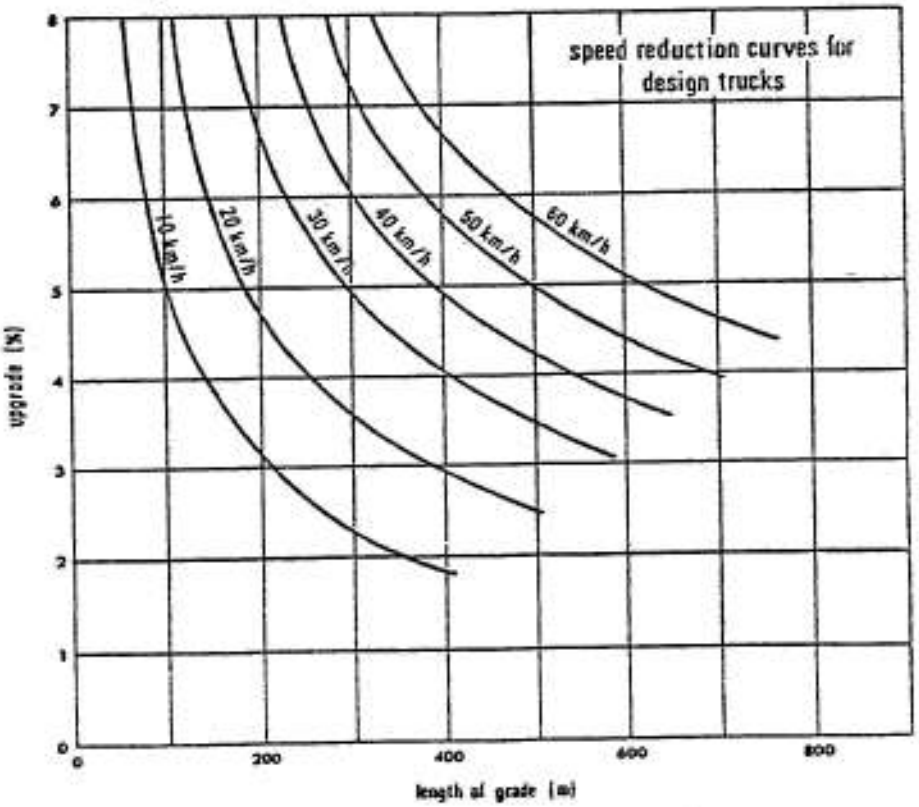
Upgrade Percent	3	4	5	6	7	8
Critical Length of Upgrade in feet	500 m	350 m	250 m	200 m	175 m	150 m

Rural horizontal alignment has similar considerations to those outlined for urban roads, with perhaps some increased design restrictions imposed where higher vehicle operating speeds are anticipated.

9.2 (continued)

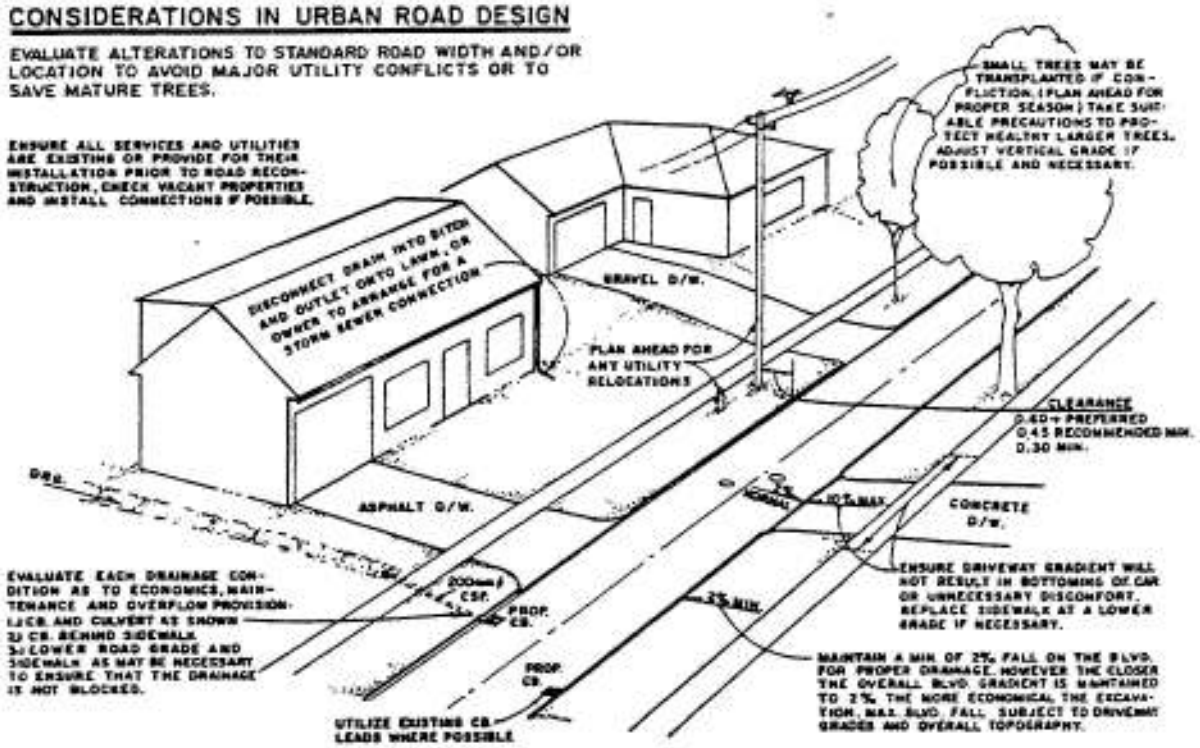
truck speed reduction B.2.4a



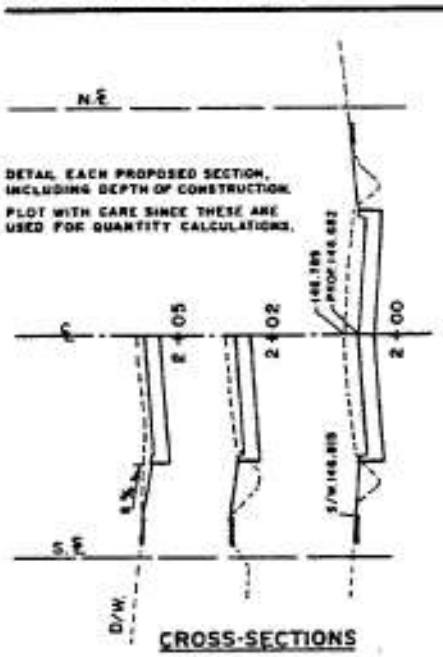


10.0

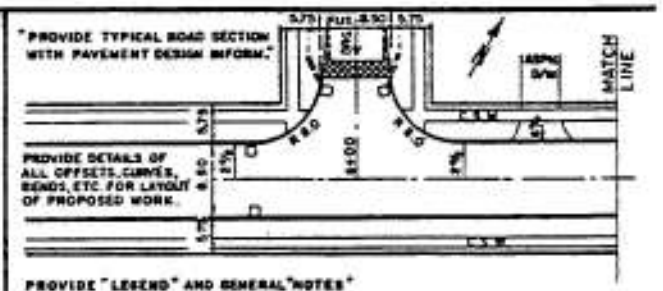
CONSIDERATIONS IN URBAN ROAD DESIGN



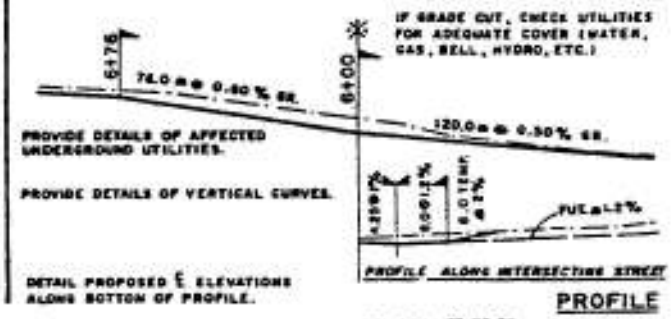
WHERE SIDEWALKS ARE NOT EXISTING ON ONE OR BOTH SIDES OF THE STREET, THE DESIGN AND BOULEVARD GRADING SHOULD BE CARRIED OUT TO PROVIDE FOR THEIR FUTURE INSTALLATION. REFER TO 5.3.1 (PEDESTRIAN FACILITIES). PARTICULAR REFERENCE SHOULD ALSO BE MADE TO THE CHAPTERS ON PAVEMENT DESIGN (5.2.2) AND ROAD DRAINAGE (5.2.3).



CROSS-SECTIONS



PLAN



PROFILE

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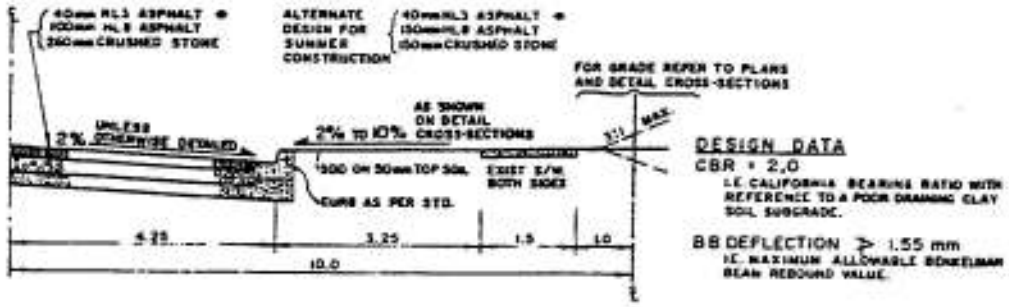
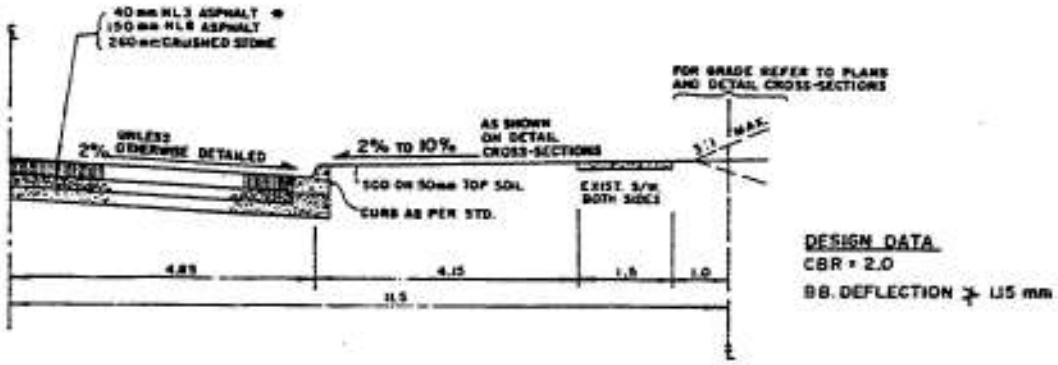
10-D

10.0 (continued)

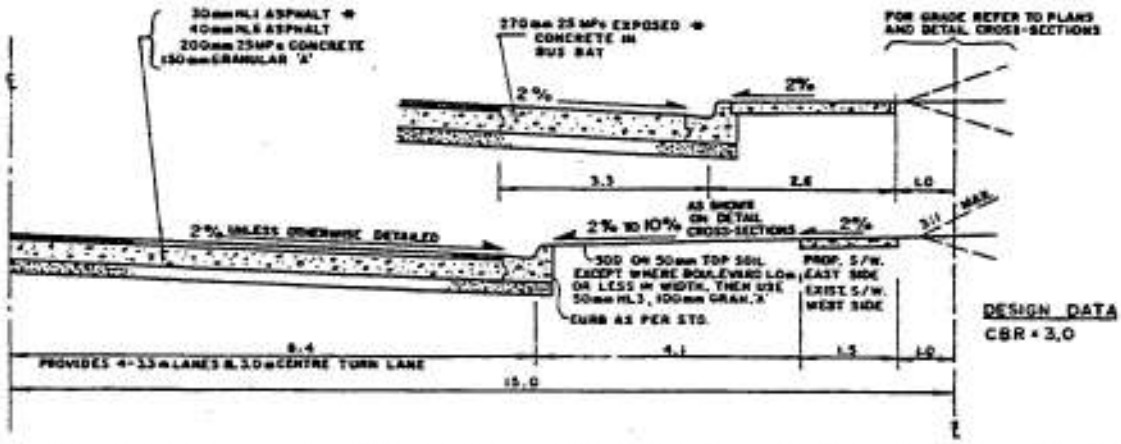
EXAMPLES OF TYPICAL ROAD DESIGN SECTIONS



RESIDENTIAL LOCAL STREET



RESIDENTIAL COLLECTOR STREET
OR INDUSTRIAL LOCAL STREET



ARTERIAL STREET

* PAVEMENT DESIGNS INDICATED ARE PROVIDED TO ILLUSTRATE INFORMATION WHICH IS DESIRABLE ON A TYPICAL SECTION, REFER TO TEXT FOR ACTUAL PAVEMENT DESIGN CONSIDERATIONS.

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11.0 Preparation of Drawings

Plan and profile drawings shall be prepared for road projects (either rolled plan and profile paper or a book of A-1 sized paper are common).

Drawing shall include:

a) A key plan (scale 1:100,000 or 1:50,000)

showing
- limits of project;
- project identification number;
- town boundaries, townships, lots and concessions, side and major roads and streets as required to locate the site;
- railways;
- a north arrow.

b) Project identification

example:
- Project 18-007-83
- Grading, Drainage and Structure.
- Main Street, from 1st Street across the Black River to 2nd Street.
- Town of Finchburg.

c) The plan and profile including:

- existing features;
- features to be removed;
- design curb line, edge of pavement, sewers, culverts, toe of slopes;
- notes as required:
  a) to specify curb type
  b) to designate sidewalk thickness;
  c) to warn of utilities to be avoided, supported or adjusted;
  d) to direct attention to desired standard drawings, etc.;
  - heading, north arrow and scale. (usually 1:1000 and 1:10 or 1:500 and 1:10 or 1:200 and 1:50)

d) Summary sheets giving design quantities for major project items:

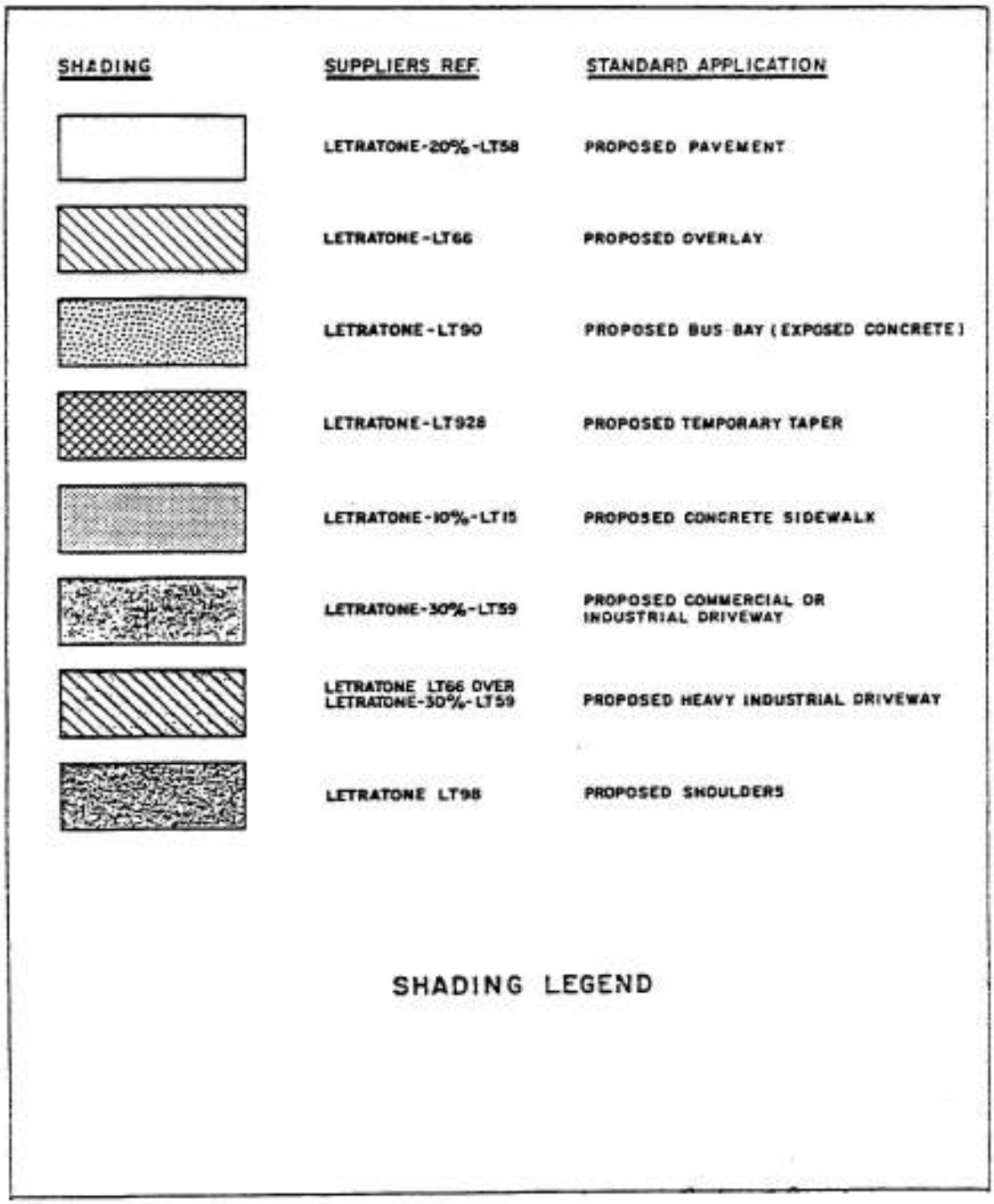
- culverts;
- sewer pipe by diameter and length;
- excavation, borrow;
- grubbing;
- guide posts;
- concrete;
- etc.

11.0 (continued)

e) Typical cross-sections with stations identifying location.

f) Side road and special case private entrance profiles.

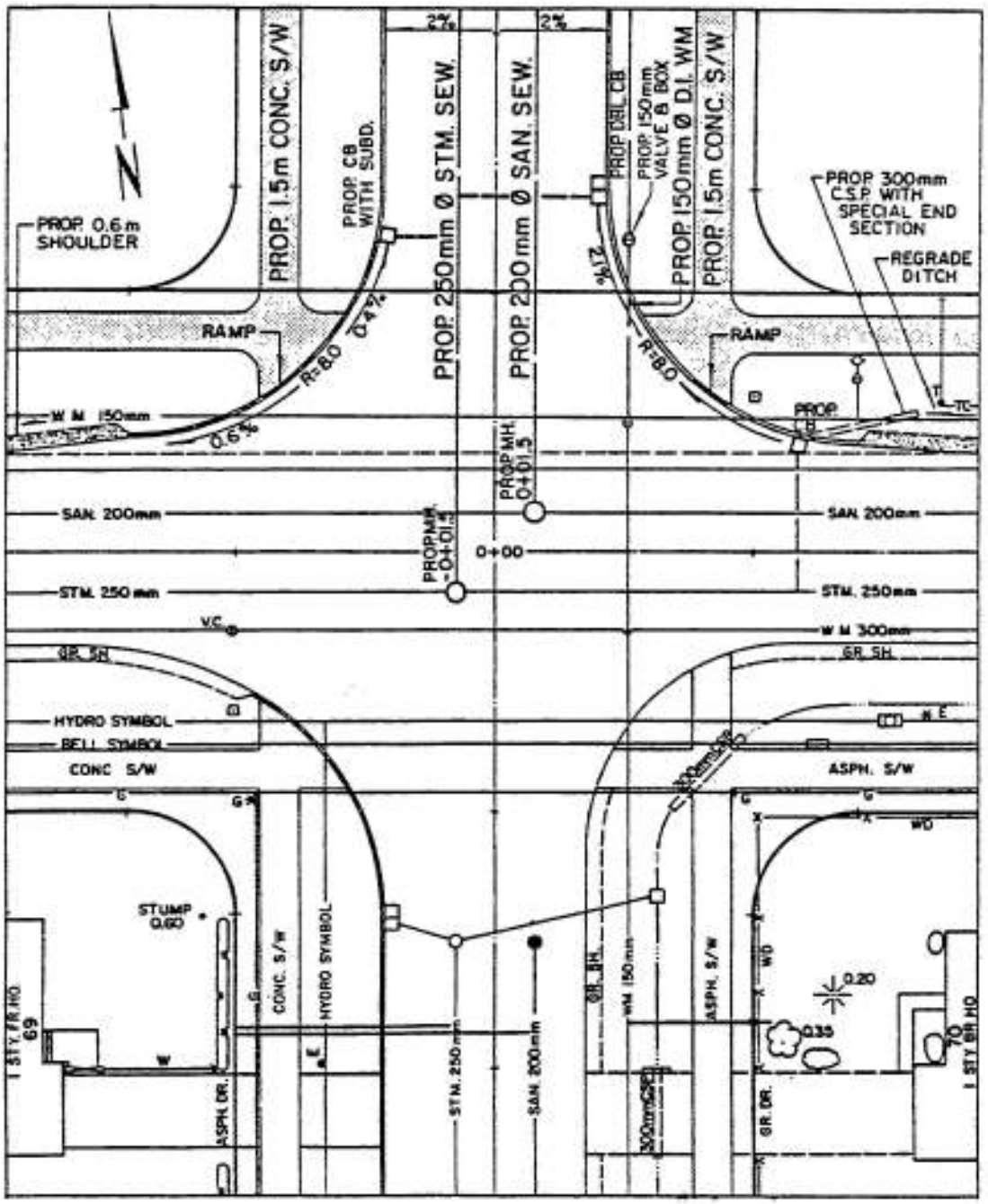
g) Standard drawings as required. (The Ontario Provincial Standard Drawings are recommended).



SHADING LEGEND

11.0 (continued)

SAMPLE PLAN



5.2 VEHICLE FACILITIES

5.2.2 PAVEMENT DESIGN INDEX

1. Introduction

2. Pavement Design Factors
- 2.1 Traffic
- 2.2 Subgrade Soils
- 2.3 Environmental Conditions
- 2.4 Performance of Similar Pavements
- 2.5 Economic Costs

3. Pavement Selection

4. Pavement Design Guidelines
- 4.1 Flexible Pavements
- 4.2 Rigid Pavements
- 4.2.1 Exposed Concrete Pavements
- 4.2.2 Composite Rigid Pavements
- 4.3 Pavement Examples

5. Curbs
- 5.1 Barrier Concrete Curb and Gutter
- 5.2 Barrier Concrete Curb
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- 5.6 Integral Barrier Concrete Curb & Sidewalk
- 5.7 Typical Curb Sections
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6. Frost Treatment
- 6.1 Road Construction
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7. Swamp Treatment
- 7.1 Methods of Treatment Under Roads
- 7.2 Excavation
- 7.3 Consolidation and/or Gravity Displacement
- 7.4 Underfill Blasting
- 7.5 Toe Shooting and Cross-section Blasting
- 7.6 Geotechnical Use of Fabrics
- 7.7 Disposal of the Excavated Swamp Material

8. Pavement Management

5.2 VEHICLE FACILITIES

5.2.2 PAVEMENT DESIGN

1.0 Introduction

For design considerations and details, reference should be made for the following publications in particular:

"Pavement Management Guide", Roads and Transportation Association of Canada, 1765 St.-Laurent, Ottawa, Canada, K1G 3V4.

"Pavements, Structural Design, Guidelines", Ministry of Transportation and Communications, Downsview, Ontario.

"Pavement Management Systems", Ministry of Transportation and Communications, Downsview, Ontario.

2.0 Pavement Design Factors

Five principal factors must be taken into account in pavement design. They are: Traffic, Sub-grade soils; environment conditions; performance of similar pavement in the area; and economics.

These factors are dealt with in some considerable detail in the above-noted publications and similar reference material. The following comments are therefore, provided to further emphasize or enlarge on certain aspects.

2.1 Traffic

Information may not be as readily available as would be desirable, particularly in a fast growing urban environment. Changes in zoning will often increase the volume and change the type of vehicles travelling on the street.

Widening of roads, improvement of intersections, new bridges, etc., may tend to increase the volume of traffic using a particular route.

2.1 (continued)

Since urban roads are, in most cases, ultimately constructed with curbs and gutters, it is important that the new pavement be designed to carry the anticipated future loading. Any future overlays to improve the riding qualities and road strength are limited by the curbs.

Therefore, traffic volumes should be estimated on the high side, taking the worst situation or anticipated developments into account. The extra centimetre of asphalt added to handle an anticipated high future volume or heavy truck traffic at the design stage is often money well spent, ensuring longer pavement life and less traffic interruptions for maintenance repairs.

Traffic forecasts that only take normal growth factors into account, particularly in fast growing urban areas, can often be misleading. Special attention should be given to factors that can influence traffic volumes such as land use, recreational facilities, industrial parks, etc. Development of a "Road Classification" plan in co-operation with your Planning Authority will provide considerable assistance. With this, major traffic routes, both present and future, may be established, together with collector roads, and roads designated for heavier truck traffic or bus routes.

2.2 Subgrade Soils

Where available, reference should be made to the soil maps available from the Ministry of Agriculture and Food, Parliament Buildings, Queens Park, Ontario. M7A 1A5.

Although originally prepared to assist agricultural use, more recent additions are providing considerable engineering information. Even the agricultural maps may provide good guidelines for use in road design analysis.

As with traffic analysis, soil testing, such as that done to establish a California Bearing Ratio, (C.B.R.), should be reviewed with caution.

Tests on the same subgrade soil type over a length of road will often provide a range of C.B.R. values.

Use the lowest value found for design analysis of the pavement, and if in doubt of this value because of past poor experience in the area, pick a lower value.

2.3 Environmental Conditions

Take into consideration the weather conditions in the area and the depth of the frost penetration in the Winter. Particularly in a road in a low-lying area, an area with a high water table, or a subgrade which tends to hold moisture.

Increase the design depths accordingly to help compensate for the effects of poor local environmental conditions.

2.4 Performance of Similar Pavements

Past performance of similar pavements in the area is always a valuable guide for the future performance of future pavements.

If the depth of the existing pavement structure is not available, cores should be taken or test holes dug to establish same.

Often older ditched roads have been built up over a period of years, and what would appear to be a very serviceable road at present, has been the result of many layers of various types of surfacings.

On Flexible type pavements, a structural analysis should be carried out as part of the evaluation using a Benkelman Beam, Dynaflect, or similar measuring device.

2.5 Economic Costs

This aspect of the design analysis can be quite involved, with local conditions, material supply and facilities playing a considerable part.

The following outline is provided as a guide as to some of the considerations that should be evaluated before a final design selection is made.

i) upgrade existing pavement structure

- if curbs are to be added to a ditched road, the drainage of adjacent properties becomes a prime consideration; i.e. economics of providing satisfactory drainage, both by means of a storm sewer system and by overland flow to ensure flood damage does not result if the storm system becomes plugged or overloaded.

- review existing pavement structure re economics of upgrading - i.e. salvage value of existing pavement (may be badly deteriorated and require major overlay to upgrade).

- review aesthetics of overall finished project

2.5 (continued)

ii) salvage of materials from existing pavement structure to use in new pavement

- review quantity of materials available and their suitability (i.e. may be contaminated, may have insufficient quantity to justify setting up a salvage operation).

- review with respect to availability and costs of new materials; e.g. crushing of asphalt may be economically justifiable in some instances for use as granular base in new road.

iii) review type of new pavement to be constructed

- cost and availability of materials; e.g. granular, crushed stone, asphalt and concrete.

- availability of competitive bidding with respect to construction (e.g. contractors available may only be equipped to handle certain size or type of construction).

- a cost comparison of pavement types should be made to determine the total cost, having regard for initial cost and future cost of the pavements. The cost comparison and consideration of the criteria mentioned in the section dealing with Pavement Selection will determine the pavement type most suitable to the project.

3.0 Pavement Selection

In an urban area and particularly on a road which is designated to carry a high volume of traffic, such as an arterial road, or perhaps an industrial collector road, a major factor in the pavement selection will be long life with a minimum of future traffic interruptions for maintenance operations, as well as maintaining reasonable routes for traffic during the construction period.

For example, it may not be possible to provide a reasonable alternate route for the heavy volume of truck and automobile traffic on a major arterial road, and therefore, this traffic must be maintained along the right-of-way during the road reconstruction.

If the existing road is to be left in place, with only lane widenings and an overlay involved, the traffic may be readily handled on the present pavement, and a flexible type road construction provided for on the new lanes.

3.0 (continued)

However, if the complete road requires reconstruction, temporary pavement may be required on part of the existing road shoulder to provide for two lanes of traffic over one side of the right-of-way centerline, while construction of the new half of the road continues on the other.

If there are few, if any, driveways to contend with, probably either flexible or rigid type pavements may still be given consideration.

However, if a multitude of driveway accesses, and particularly industrtial type accesses, are to be maintained, then a rigid type base may have preference. The rigid base can be constructed with relative ease over short sections of road, and with the normally quick strength gain of today's concrete (often 3 days), traffic may be readily provided access to part of a driveway, first over the granular road base, then over the new concrete base as the missing section is placed.

It is in this type of construction in particular, as well as in major intersection construction where through-traffic must be maintained, that the rigid type of pavement design appears to have merit.

Flexible pavement design such as granular base and asphalt surfacing, is the most common pavement design and is universally used. A key advantage to this pavement design is that the necessary equipment is readily available throughout the Province.

4.0 Pavement Design Guidelines

In addition to the references already provided, additional information may be obtained in the following publications:

"Thickness Design - Asphalt Pavement Structures for Highways and Streets", The Asphalt Institute.

"Thickness Design for Concrete Pavements", The Portland Cement Association.

"Joint Design for Concrete Pavements", The Portland Cement Association.

4.1 Flexible Pavements

There are three basic design considerations; Conventional, Deep Strength, and Full Depth; with the design usually closely based on economics and local experience, and other considerations.

The required thickness is calculated as a total thickness of granular base; the pavement make-up is then calculated on an equivalent basis, using the following formula:

1 mm asphaltic concrete = 2 mm crushed stone or granular 'A' base

1 mm asphaltic concrete = 2.7 mm granular 'B' sub-base

The actual thickness of each material is determined according to local experience and preferences and other considerations, job nature, availability of materials, relative cost of materials, etc.

The use of the Benkelman Beam provides a quick, reliable method for the design of flexible pavements. By relating existing pavements and recently-constructed pavements, the designs for new pavements with similar conditions may be closely evaluated. Commonly accepted maximum Benkelman Beam rebounds are as follows:

Arterial Roads	0.75 to 0.90 mm
Collector Roads	1.05 to 1.15 mm
Local Roads	1.55 mm

In evaluating the Benkelman Beam rebound results, the most critical concern is to establish the "Spring Ratio". For this purpose, it is desirable to have test sections which compare with the proposed road as to subgrade type, design depths of pavement materials, and construction control. Where such test sections are not available, a "Spring/Fall" rebound ratio may be selected from the R.T.A.C. publication referred to in the Introduction to this chapter. This ratio is selected with reference to the type of subgrade soil, and if there is any doubt, a fairly high value should be chosen.

The peak deflections for pavements with 90 mm or less asphalt depth generally occur in April, whereas pavements of 100 mm and greater asphalt thickness generally have their peak deflections during the hot weather of June and July.

If deflection measurements are obtained at times other than late September to mid November, the "Spring Ratio" used to obtain the maximum anticipated rebound must be estimated from available data, engineering judgement and experience.

4.1 (continued)

The following chart is provided as an example. The "Spring Ratios" shown resulted from an evaluation of an extensive Benkelman Beam test program carried out in the City of Etobicoke during 1975, as well as evaluation of a number of control test sections carried out over several years.

Also a conversion table for use with Benkelman Beam rebound values is included.

BENKELMAN BEAM MAXIMUM REBOUND RATIOS FOR 1975

Pavement Type	Subgrade Soil	Pavement Temperature	May	June	July	Aug.	Sept.	Oct.	Nov.
CONVENTIONAL 90 mm asphalt maximum	Clay	up to 90°	1.4	1.4	1.5	1.6	1.7	2.0	2.5
	CBR=3+	90° +		1.3	1.3	1.3			
	Sand	up to 90°	1.3	1.3	1.3	1.4	1.6	1.7	1.8
	CBR 10+	90° +		1.3	1.3	1.3			
DEEP STRENGTH 100mm + asphalt 300mm ± granular	Clay or Sand	up to 90°	1.2	1.1	1.1	1.2	1.4	1.6	1.8
		90° +		1.0	1.0	1.0			
FULL DEPTH 150 mm + asphalt 150mm maximum granular base	Clay or Sand	up to 90°	1.2	1.1	1.1	1.2	1.4	1.6	1.8
		90° +		1.0	1.0	1.0			

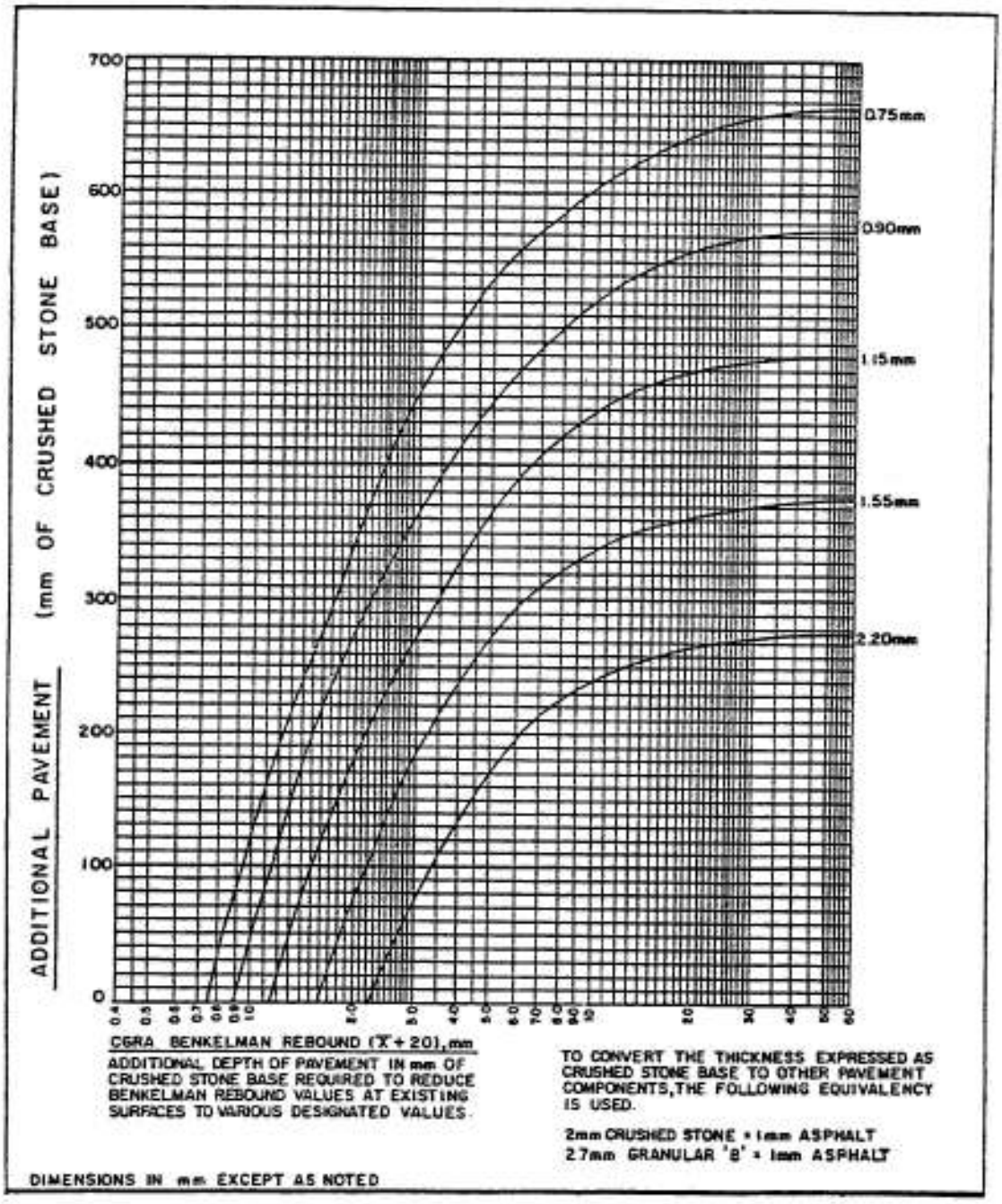
Note: RATIOS GIVEN ARE APPROXIMATELY MID MONTH AND SHOULD BE INTERPOLATED ACCORDING TO THE ACTUAL TIME OF THE BENKELMAN BEAM TESTING

: RATIOS DEVELOPED FROM REVIEW OF 1975 CONTROL TESTING, WITH REFERENCE TO PREVIOUS CONTROL TESTING, AND OVERALL REVIEW OF 1975 BENKELMAN BEAM TESTS

4.1 continued...

CONVERSION CHART

FOR USE WITH BENKELMAN BEAM REBOUND VALUES



4.2 Rigid Pavements

4.2.1 Exposed Concrete Pavements

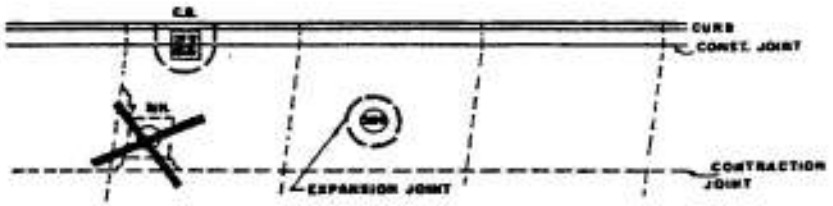
The number of heavy axle loads expected is the major factor affecting the thickness design of concrete pavements. Once the number and weights of the loads expected during the design life of the pavement are known, the designer can calculate the required depth following the method outlined in the Portland Cement Association booklet referred to earlier.

Expansion Joints are required only at fixed objects (manholes, catchbasins, etc.), and at unsymmetrical intersections provided that:

(a) The pavement is constructed when the temperature is well above freezing and from materials with normal expansion properties; and

(b) Contraction joint spacing is short enough to control transverse cracking.

Whenever possible, the expansion joints should be installed with rounded corners as shown below. This greatly reduces the incidence of cracks radiating from the corners of square cornered joints.

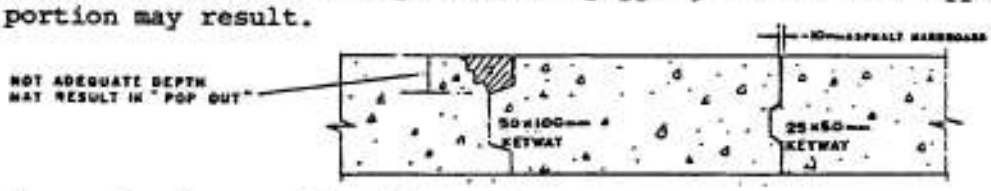


Contraction Joints should be placed at intervals short enough to control transverse cracking (i.e. 5.0 to 6.0 m). They may be formed by sawing or by placing 10 mm thick asphalt hardboard into the fresh concrete. To be effective, they must be at least 1/4 of the depth of the pavement. In urban areas, it is often found that the placement of asphalt hardboard for the contraction joints is more practical, since the optimum time for saw-cutting often will be during the night, and if it is left until the next day's operations, uncontrolled cracking is often the result.

Skewing of the joints to a skew of about 1 in 6 will ensure that truck wheel loads move across the joint one at a time, and will generally improve the riding characteristics of pavement considerably.

4.2.1 (continued)

Longitudinal joints are required wherever the width of the pavement exceeds 5.0 m. As with the contraction joints, they can be formed either by sawing or by inserting strips of asphalt hardboard in the fresh concrete, and the same considerations regarding saw-cutting apply. They are designed as a hinged joint, tied together by means of deformed bars to hold the faces in contact, thus ensuring load transfer across the joint by means of aggregate interlock or a keyway. The keyway, if used, should be carefully designed and placed, or popping off of the upper portion may result.



In most urban roads, where the pavement is restrained at edges enough to prevent separation of the slabs, the tiebars can be eliminated.

Surveys carried out by the Portland Cement Association, in conjunction with various highway departments in the United States, showed satisfactory performance of undowelled joints in all areas where frost penetration was less than one foot 90% of the time, and unsatisfactory performance where frost penetration is deeper and more frequent and where soil and climate conditions are similar in Ontario; except on roads carrying small volumes of heavy trucks. This would seem to indicate that the use of undowelled joints should be limited to roads that meet this criteria; however, undowelled joints have been used in Ontario on roads carrying large volumes of trucks at speeds of 60 km/h or less, and pavement performance has been satisfactory.

Where the subgrade soil is of a granular nature, base materials such as Granular "A", "B", etc., are not required. A granular material, 100 to 150 mm in depth, will facilitate construction when weather conditions and soils conditions combine to make work difficult. In some cases, a granular layer may prevent a med-pumping action.

4.2.2 Composite Rigid Pavements

Composite rigid pavements, such as concrete base with asphalt surface, are used extensively by many road authorities in the Province. This type of pavement appears to retain all the advantages of exposed concrete pavements with none of the disadvantages. In may areas, the initial cost is less than exposed concrete pavements because of the simplicity of the operation. Specialized equipment is not needed, and in most cases, the concrete can be deposited direct from the ready-mix truck.

4.2.2 (continued)

There is also a savings in maintenance costs for composite pavements versus exposed pavements, particularly with respect to expansion and contraction joints.

Design details for concrete based pavements should be the same as those given in the last section for "Exposed Concrete Pavements", with the exception of finishing the concrete base, which does not require anywhere near the care used in finishing exposed pavements.

A surface course asphalt, HL1, 2, 3 or a slight variation therefrom, depending on the road classification a depth between 40 mm and 75 mm is the surfacing material generally placed on a concrete base pavement.

The concrete surface is usually tack coated, and 40 mm of HL8 asphalt placed as a leveling course. The surface course asphalt, usually 40 mm of HL1 or HL3, is then placed.

Variations to this include a single lift of surface course asphalt, using HL1, 2, 3 or a slight variation therefrom, of a depth between 40 mm and 75 mm.

Also surface treatments have been used effectively as a temporary surface.

4.3 Pavement Examples

The following pavement design examples are provided as a guide. These designs are based on a poor draining clay subgrade with a C.B.R. of 2.0 to 3.0. From construction, maintenance and Benkelman Beam testing experience in the Etobicoke area of Metropolitan Toronto they have performed satisfactorily over the past 10 years.

Road Classification	Pvmt Width	Max. B.B. Rebound	Flexible Deep Strength Asphalt Design	Composite Rigid Design
Arterial	15.0 m+	0.90 mm	40 mm HL3 & 200 mm HL8 Asphalt	40 mm HL3 & 40 mm HL8 Asphalt
			260 mm Crushed Stone	200 mm 25-MPa concrete 150 mm Crushed Stone
Collector & Local Industrial	9.7 m	1.15 mm	40 mm HL3 & 150 mm HL6 Asphalt 260 mm Crushed Stone	

4.3 (continued)

Road Classification	Pvmt Width	Max. B.B. Rebound	Flexible Deep Strength Asphalt Design	Composite Rigid Design
Local * Residential & Major Access Driveways	8.5 m	1.50 mm	40 mm HL3 & 100 mm HL8 Asphalt 260 mm Crushed Stone	40 mm HL3 & 40 mm HL8 Asphalt 150 mm 25-MPa concrete 150 mm Crushed Stone
Parking Areas, Minor Access Driveways	Vari-able	2.00 mm	40 mm HL3 & 60 mm HL8 Asphalt 260 mm Crushed Stone	
Private Residential Driveway	Vari-able	Not Appli-cable	40 mm HL3A & 40 mm HL8 Asphalt 150 mm Crushed Stone	150 mm 25-MPa exposed concrete on compacted subgrade

* If the local residential road construction takes place during the warm dry Summer months, the following "full depth" asphalt type design is sometimes used, taking advantage of less excavation and sometimes salvaging suitable on site materials.

40 mm HL3 and 150 mm HL8 asphalt on 150 mm Crushed Stone or suitable salvaged Granular 'A' type material.

5.0 Curbs

5.1 Barrier Concrete Curb and Gutter

The concrete curb and gutter section is the one recommended for use in most urban street locations, since it provides a number of benefits over other curb sections.

5.1 (continued)

The concrete gutter can be constructed to closer tolerances than an asphalt gutter, therefore allowing gradients down to 0.35% while still obtaining reasonable drainage characteristics.

With a gutter crossfall of 1:12, this curb section assists the road drainage. This may be of particular benefit when it is desirable to intercept drainage off the boulevard from weeping across a superelevated road surface.

The curb and gutter section provided an improved delineation to the edge of the road in comparison to other curb sections.

In new developing areas a concrete haunch is sometimes placed in conjunction with the road base asphalt. The balance of this curb and gutter section is placed later when the development is nearing completion, and prior to the placement of the final surface asphalt. The haunch and curb sections are usually tied together by a key way and dowels.

With the development of the concrete saw to cut driveway accesses in the barrier curb, it is not as great a concern to establish the location for any new driveways prior to the curb.

From a cost point of view the standard curb and gutter section is generally comparable to other curb sections, when the equivalent road area to the concrete gutter area is included.

5.2 Barrier Concrete Curb

The concrete curb section is generally recommended for use in road median islands, driveways and parking areas, particularly where the drainage is carried away from the curb face.

Since it is dependant on an asphalt gutter, it should not be used where the gutter gradient will be less than 0.5%

5.3 Barrier Precast Concrete Curb

This type of curb has been used with considerable advantage in new developing areas. Only the haunch is installed at the initial stage, with the curb section added later when the development is nearing completion.

Using this method of construction the driveway cuts may be properly aligned with the driveways. Also relatively inexperienced workers can achieve a product of satisfactory appearance and quality.

It is not normally used on urban road reconstruction projects since it usually has a higher cost to the curb and gutter section and does not have the advantage of a concrete gutter.

5.4 Roll Type Concrete Curb

This type of curb is often used along the side of the road where boulevard parking is provided.

Although it is also often used on new roads in developing areas, since it avoids the problem of improperly placed driveway cuts, it does not provide a suitable barrier for the protection of pedestrians. Also greater problems may be anticipated with the boulevard area maintenance and general appearance.

5.5 Barrier Asphalt Curb

Asphalt curbs are usually installed where a temporary curb section is required, since it can be readily removed and disposed of.

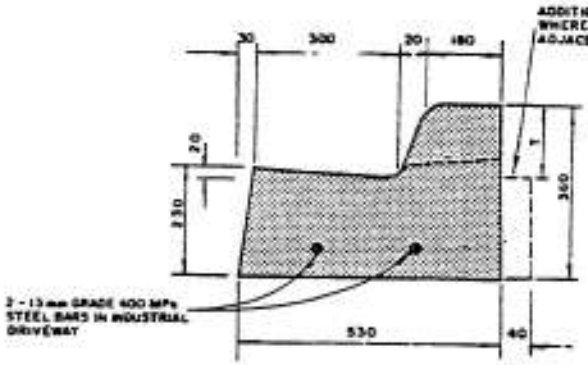
The asphalt curb tends to be easily damaged, particularly if in an area with truck or bus traffic. It also has the disadvantage of an asphalt gutter for drainage, and may require painting to provide a suitable colour differentiation to the asphalt road surface.

5.6 Integral Barrier Concrete Curb and Sidewalk

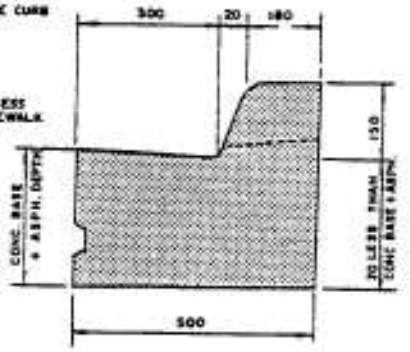
In many new urban areas the sidewalk, when placed curb side, is usually placed on a step in conjunction with the standard concrete curb and gutter section.

In older downtown areas, an integral curb and sidewalk section is often used. This may be of some value, particularly in commercial areas where a minimum of a 150 mm depth of sidewalk is used. The two sections may be combined without future problems of cracking just behind the curb which sometimes develops when a 100 mm sidewalk is placed integrally with the curb.

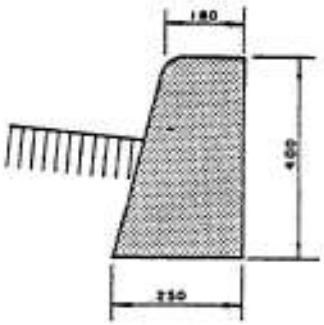
5.7 Typical Curb Sections



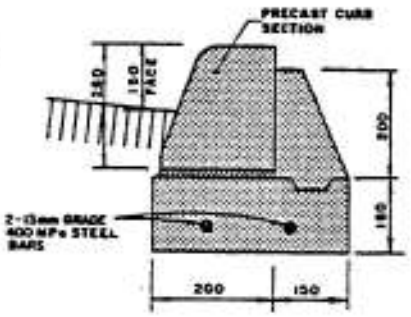
BARRIER CONCRETE CURB AND GUTTER FOR FLEXIBLE TYPE PAVEMENT



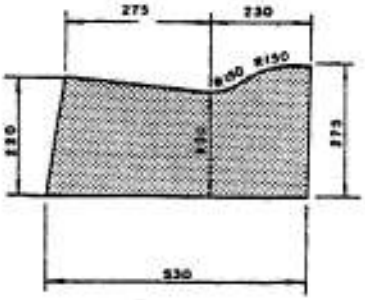
BARRIER CONCRETE CURB AND GUTTER FOR RIGID TYPE PAVEMENT



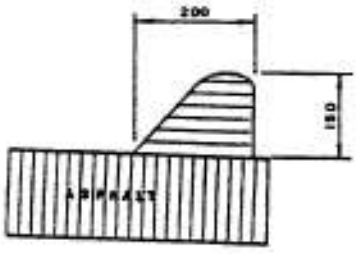
BARRIER CONCRETE CURB



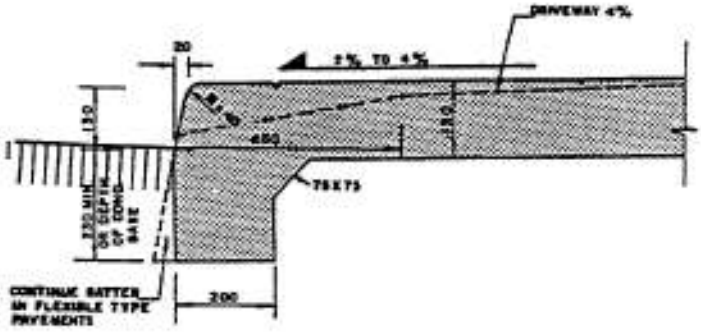
BARRIER PRECAST CONC. CURB



ROLL TYPE CONC. CURB



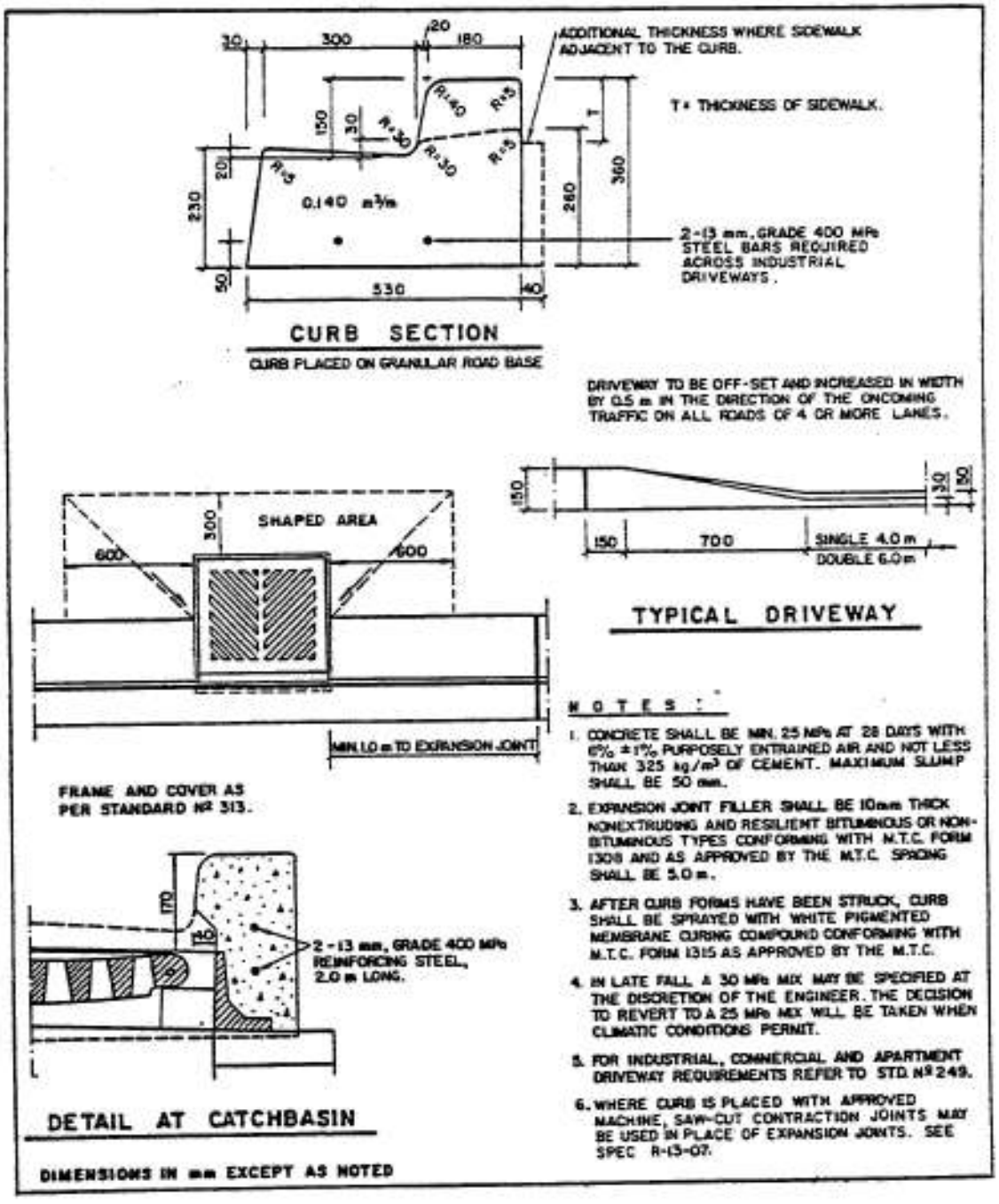
BARRIER ASPHALT CURB



INTEGRAL BARRIER CONCRETE CURB AND SIDEWALK

REVISED: 83-05-31

5.8 Details for Concrete Curb and Gutter



6.0 FROST TREATMENT

One of the special soil problems of our country is that of damage caused to roads and streets by the freezing and thawing of subgrades and bases during the winter and spring seasons. Under certain conditions, very severe damage to subgrades, bases, and pavements, both rigid and flexible, may result from frost action. In simplified terms, these conditions are;

(1) Frost susceptible soils.
(2) Freezing soil temperature.
(3) Water supply at or near the freezing front.

6.1 ROAD CONSTRUCTION

There are three basic techniques used in road construction to minimize frost action.

(1) Removing the frost susceptible soil and replacing with a design depth of granular material which is not frost susceptible.

(2) Providing base drainage to remove excess free water.

(3) Placing layers of insulation below the asphalt surface to minimize the depth of frost penetration.

6.3 UNDERGROUND UTILITY PROTECTION

Underground utilities are subject to damages from frost action when they are installed at shallow depths and in areas of frost susceptible soils. The resultant movement of the subgrade and/or utility bedding can distort or rupture the utility pipe. In order to protect the utilities three basic techniques are used.

6.3 (continued)

(1) Install or relocate utilities to areas not directly beneath the roadway where frost penetration is the deepest. One such area could be the boulevards.

(2) Construct granular soil frost tapers in the utility trench to minimize the effect of differential heaving.

(3) Place insulation around the utility thus keeping its surrounding soil temperature above freezing.

Sufficient care and consideration should be given before the use of insulation is chosen. This is due to the fact that a non-uniform frost penetration depth versus the roadway may cause differential heaving thus while the utility is being protected, the road surface may be destroyed. For more information on the use of rigid insulations one should contact the various companies who manufacture and promote its use.

7.0 Swamp Treatment

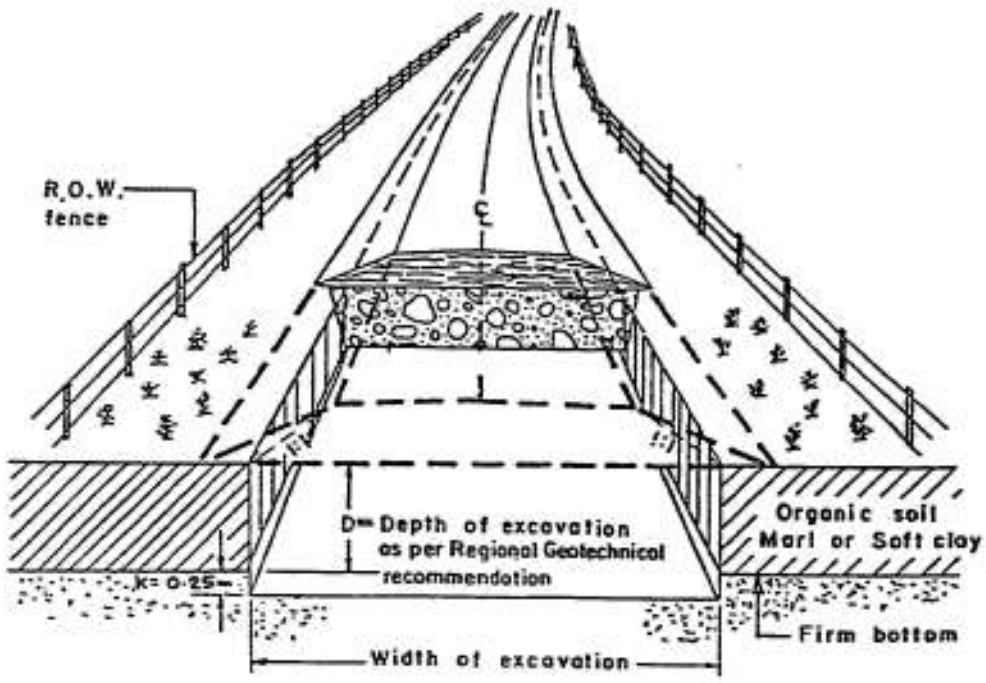
Swamp - often referred to as muck, peat, marsh, muskeg etc., is a deposit of organic material resulting mainly from the decomposition of roots and plants which have grown and died in place. This material usually has a high moisture content and a low bearing capacity.

7.1 Methods of Treatment Under Roads

- Excavation;
- Consolidation and or gravity displacement;
- Underfill blasting;
- Toe shooting and cross-section blasting;
- Geotechnical use of Fabrics.

7.2 Excavation

(a) Full excavation (the usual method for swamp material with a depth of 4 m or less). The full depth of muskeg plus 0.25 m is excavated to a width equivalent to the driving surface plus shoulders plus twice the design depth of granulars.



SWAMP EXCAVATION

7.2 (continued)

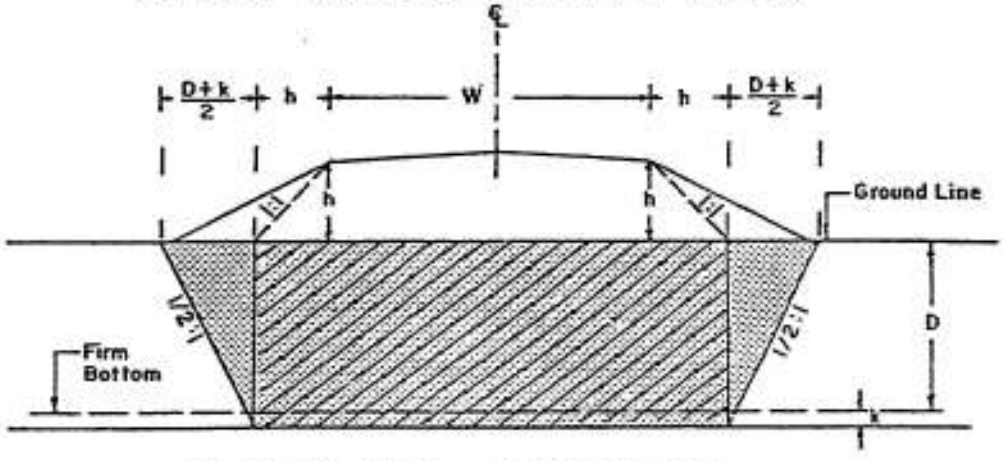
(b) Partial excavation, also called floating or riding (when the depth of swamp material is greater than 4 m).

The results of a soils investigation will determine the depth of excavation when the muskeg is greater than 4 m. Again, the excavation will be of sufficient width to accommodate the road surface, shoulders and twice the granular depth.

(c) Backfill. The material selected to backfill the volume of excavated muskeg is usally selected earth borrow. It should be placed as a simultaneous combined operation with the excavation.

It is expected that the backfill will displace muskeg thus in full depth swamp excavation approximately 15% more material (by volume) is placed than removed. In partial excavation, the extra volume of replacement backfill required (due to consolidated of the remaining muskeg under the excavation) will be determined by the soils investigation.

COMPLETE EXCAVATION, DEPTH 4.0 m OR LESS

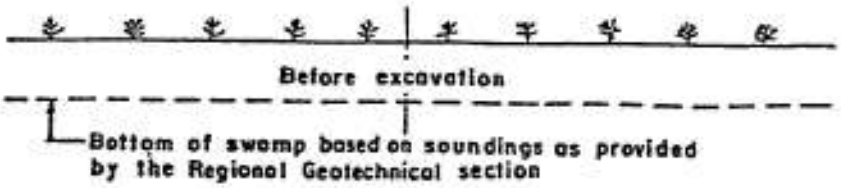


EXCAVATION AREA = $(W+2h)(D+k)$

BACKFILL AREA = $\left[(W+2h)+\frac{(D+k)}{2}\right](D+k)$

$k = (0.25\ m)$

7.2 (continued)



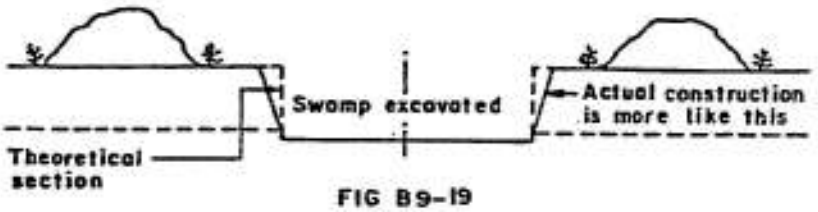


FIG B9-19

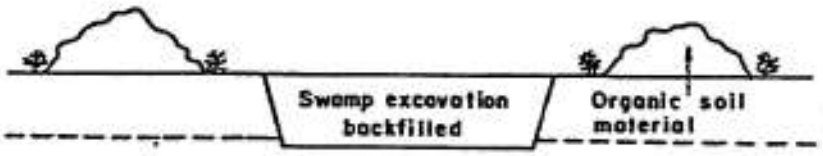


FIG B9-20

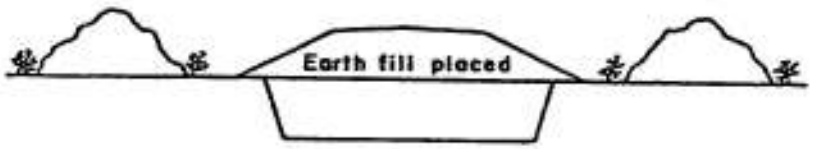


FIG B9-21

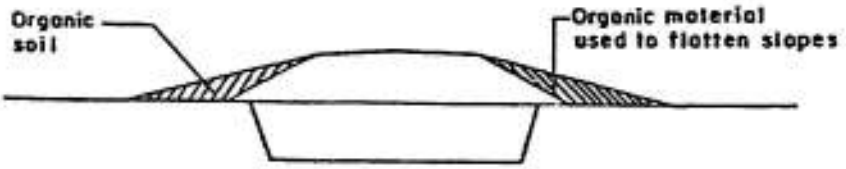
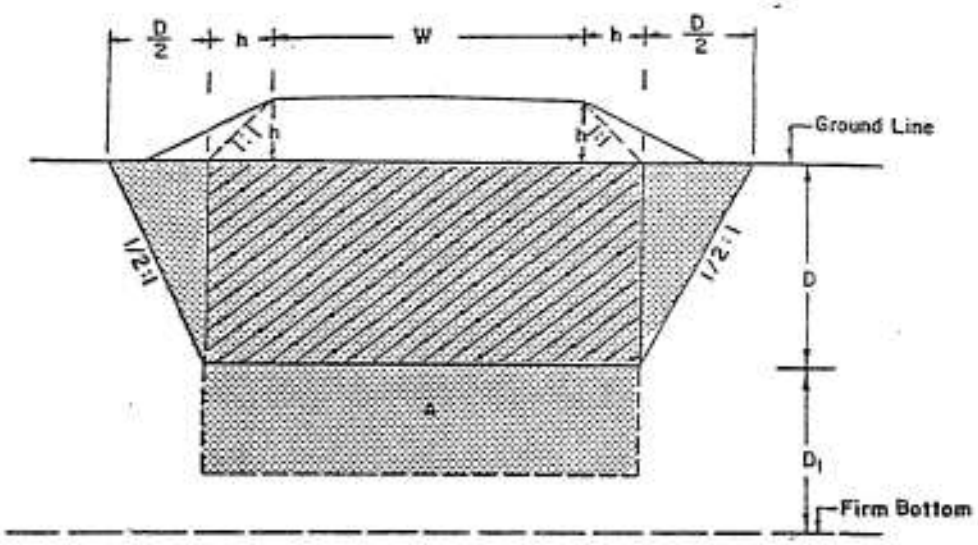


FIG B9-22

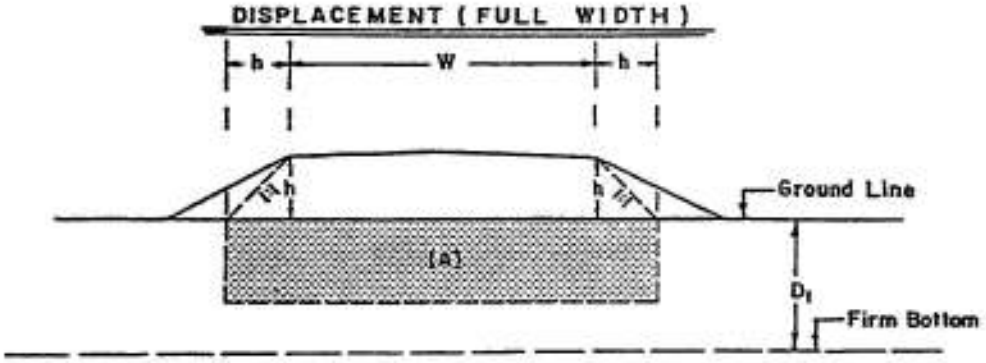
7.2 (continued)

PARTIAL EXCAVATION AND DISPLACEMENT, DEPTH GREATER THAN 4.0m



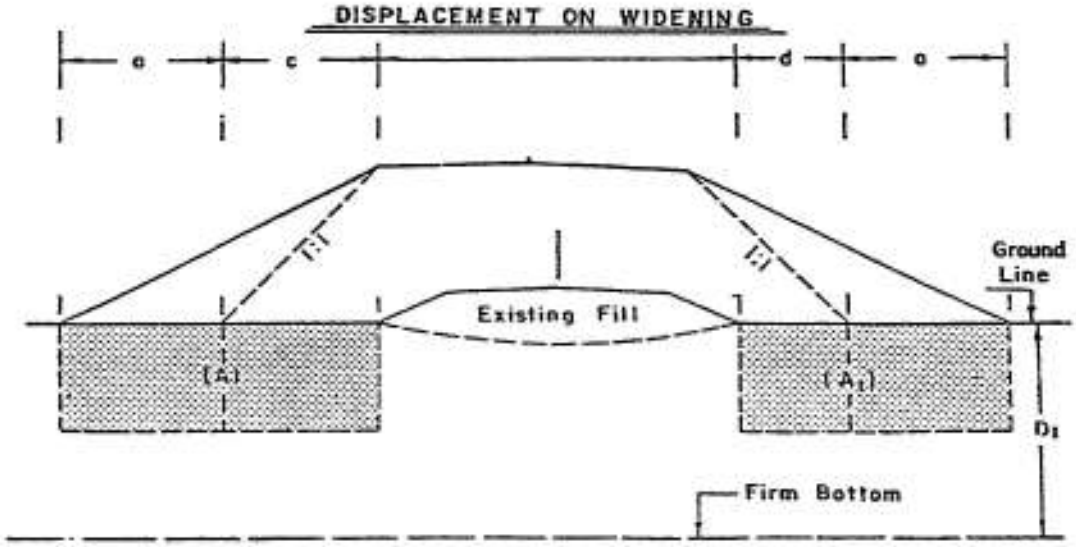
EXCAVATION AREA = D(W+2h) BACKFILL AREA = D(W+2h+D/2)

DISPLACEMENT AREA A = $pD_1(W+2h)$



FIGB9-29

DISPLACEMENT AREA (A) = $pD_1(W+2h)$



7.2 (continued)

(d) Partial excavation. Often when there is a normal existing road requiring widening over swamp material, the existing road bed is left untouched and excavation of muskeg is performed in the area of widening only.

7.3 Consolidation and or Gravity Displacement

In this method (based on the recommendations from a soils investigation), an embankment is built over the swamp. The weight of the embankment will consolidate or displace (or a combination of the two) the muskeg. After the recommended consolidation and displacement has occurred, the roadway can be built on the embankment.

7.4 Underfill Blasting

This method shall consist of placing the necessary explosives in muskeg below existing fills so that when exploded, the muskeg will be displaced and the embankment properly settled into the swamp.

7.5 Toe Shooting and Cross-Section Blasting

These methods, involve placing explosives immediately at the toe of slope at either the front or the sides of the V-shaped fill section. The blast will destroy the swamp surface root mat and liquify the muskeg prior to placing additional fill.

Both methods require the loading or advancing of end of the embankment to be plow shaped to facilitate the sideways displacement of the swamp material.

7.6 Geotechnical Use of Fabrics

Another method of constructing low volume roads over soft ground and swamp is the geotechnical use of filter fabric. It is used when the savings in time and material costs more than cover the cost of the fabric.

A non-woven fabric is recommended. Basically the fabric is rolled onto a cleared and close cut subgrade (joints are overlapped 1.m or sewn) then, the granular base courses placed over the fabric.

The fabric restrains the subgrade and spreads the load over a larger area than with a conventional construction. The subgrade thus can better withstand local shear and rutting. The fabric also acts as a barrier between the subgrade and granulars preventing contamination of the base by fine materials from below and the loss of granular by punching into the soft subgrade. Two reports with design information are:

7.6 (continued)

- Ministry of Transportation and Communications of Ontario Report EM-13.

  Geotechnical use of filter fabrics in Highway Construction.

- U.S. Department of Transportation Federal Highway Administration Offices of Research and Development Report #FHWA-TS-78-205.

  Guidelines for Use of Fabrics in Construction and Maintenance of Low-Volume Roads.

7.7 Disposal of the Excavated Swamp Material

Most of the excavated material should be disposed of within the right-of-way a large quantity of muskeg can be wasted by building the side slopes up from 1:1 to whatever is possible.

8.0 Pavement Management

The planning or programming phase includes the acquisition of traffic information, an assessment of deficiencies on a network basis, the establishment of priorities, the development of a schedule for carrying out the needed work and the determination of any extra property required. It is in this phase that the basic investment decisions, recognizing budget constraints, are made.

The design phase includes several major activities, starting with the acquisition of a variety of information on materials, traffic costs, etc., followed by the development of alternative designs. These alternatives are then analyzed, compared with respect to their costs and benefits, and the best one is selected for construction.

The construction phase of pavement management can include such activities as work scheduling, actual construction operations, quality control, and processing of data.

The maintenance phase includes the establishment of a program and schedule of work based largely on budgetary considerations, the actual operations of crack filling, patching, etc., and the acquisition and processing of data.

The evaluation of pavement includes the establishment of control sections, the periodic measurement of such factors as pavement structural capacity, roughness, condition, skid resistance, traffic, etc. It also usually includes the analysis of the data for use in the other phases of the pavement management system.

8.0 (continued)

The basic purpose of a pavement management system is to achieve the best value possible for the available public funds and to provide safe comfortable and economic transportation. This is accomplished by comparing investment alternatives at both the network and project levels, coordinating design, construction, maintenance and evaluation activities, and making efficient use of existing practices and knowledge.

The overall structure of a pavement management system is a broad encompassing structure that allows for considerable variation of details within these major phases or classes of activities. It assumes that route selection, feasibility studies and functional planning of the highway or street have already been conducted.

There are many references available for pavement management. The Roads and Transportation Association of Canada Pavement Management Guide is one of the best references.

5.2 VEHICLE FACILITIES

5.2.3 ROAD DRAINAGE INDEX

1. Introduction

2. Continuity of Drainage

3. Catchbasins

4. Pavement Cross Fall

5. Gutter Drainage

6. Ditch Drainage

7. Boulevard Drainage

8. Sub-grade Drainage

5.2 VEHICLE FACILITIES

5.2.3 ROAD DRAINAGE

1.0 Introduction

The road centreline gradient, rates of pavement cross slope, and the gutter gradient should provide for rapid surface drainage to avoid pavement icing due to intermittent thawing and freezing.

This is of particular importance on collector and arterial roads, at all road intersections and at curves and bends in the road alignment.

On urban roads, the road alignment often provides the route for the continuity of the overall drainage. Provision should be made in the road grade to ensure that this drainage does not flood into buildings in the event that the ditches or storm sewer system becomes overloaded in a severe storm.

2.0 Continuity of Drainage

Where possible the overall longitudinal road gradient should provide for overflow drainage. This will ensure that the runoff from storms beyond the design capacity of the storm sewers may be safely carried to open water courses without flooding of buildings.

In established road patterns, the maintaining of the overflow drainage provisions may involve easements for drainage ditches between or through private properties.

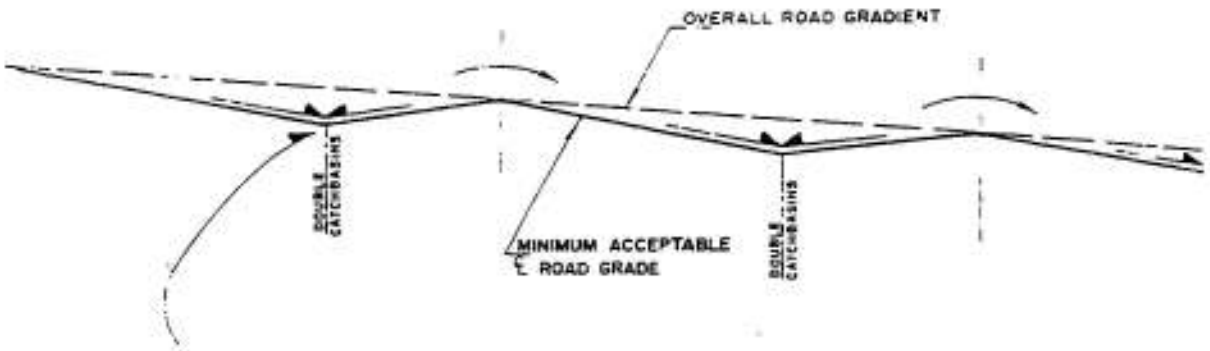
Drainage easements are a source of continual annoyance to the property owners and present continuing maintenance difficulties for the municipality. This becomes increasingly true as the density in an area increase.

It is most desirable that the engineer has input in the planning stage of a developing areas so that the road pattern may facilitate the continuity of drainage, and avoid any need for easements.

With a satisfactory provision for the overflow drainage along the road allowance, it may be possible to achieve some economics in the storm sewer design.

In areas of flat gradients it may be necessary to summit the road, as outlined in the following example.

2.0 (continued)



It is recommended that the maximum depth of ponding in the low point areas retain any flooding to within the road allowance, and preferably within the curbs.

3.0 Catchbasins

Sufficient catchbasins should be provided along the gutter line to intercept the water and to prevent flooding of the road surface. The gutter at the catchbasin should be depressed to assist in the interception of the water into the catchbasin.

Catchbasins should be located upstream of pedestrian crossing areas, and 1.5 m clear of any driveway curb depression.

Double catchbasins are required in valleys, and the maximum length of gutter contributing to each side should be 75% of the maximum spacing for the approaching road gradient.

With a normal 2% road crossfall either side of the centreline the following maximum catchbasin spacings are recommended.

ROAD GRADIENT	ROAD WIDTH	MAXIMUM SPACING
0.35 to 0.5%	8.5 m	45 m
	15.0 m	40 m
0.6 to 3.0%	8.5 m	90 m
	15.0 m	70 m
3.1 to 5%	8.5 m	75 m
	15.0 m	55 m
5.1 to 6%	8.5 m	60 m
	15.0 m	45 m

Where changes in gradient occur, the average gradient should determine the maximum spacing.

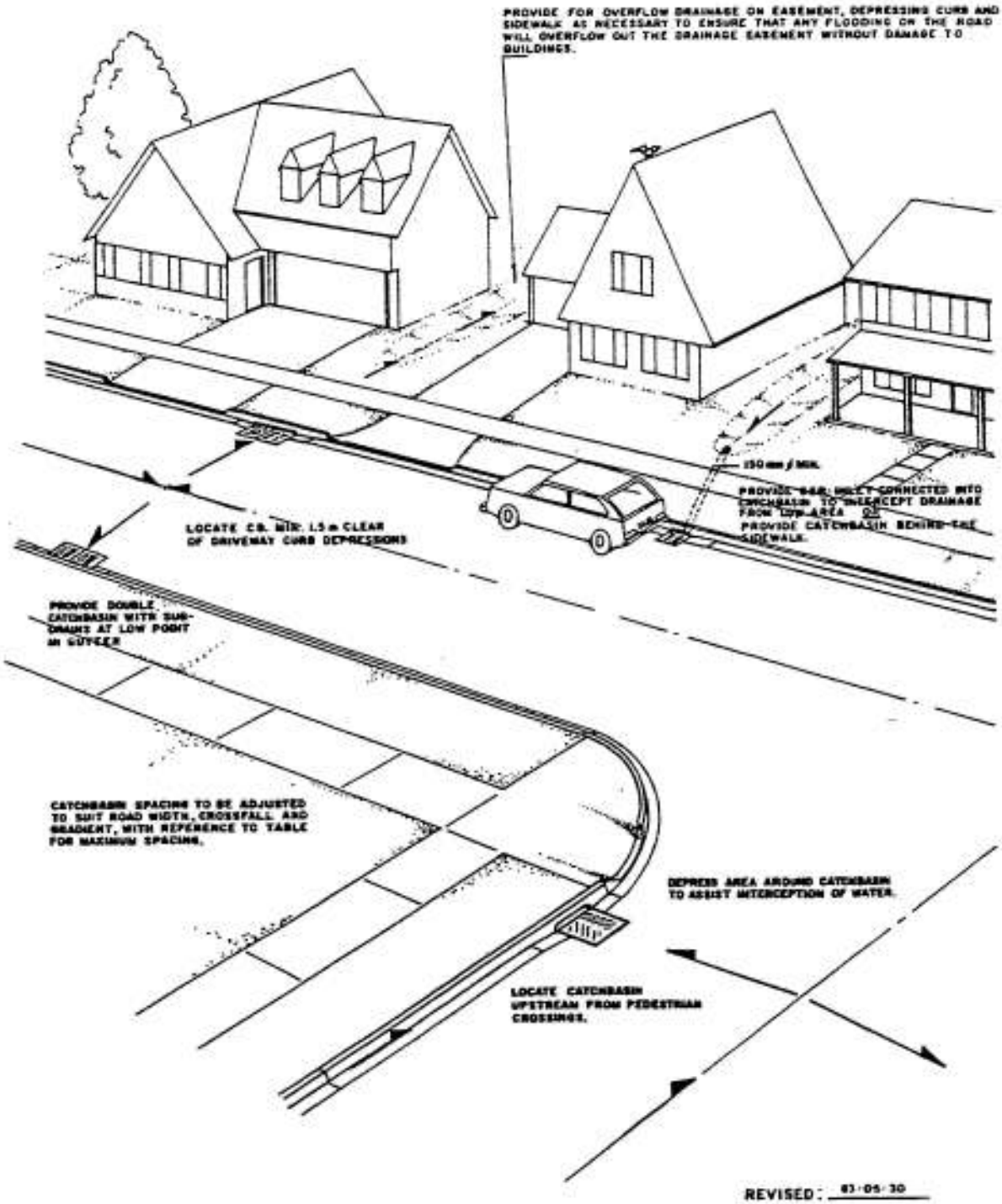
3.0 (continued)

The catchbasin spacing may be altered to take into consideration the amount of drainage reaching the road surface from the adjacent properties. If all the buildings have a direct connection into the storm sewer, then the need for road catchbasins may not be as great.

Additional catchbasins may be required to intercept some surface drainage before it reaches the road area. This may involve a culvert connected into a curb side catchbasin, or a separate catchbasin back at the street line.

3.0 (continued)

CATCHBASINS



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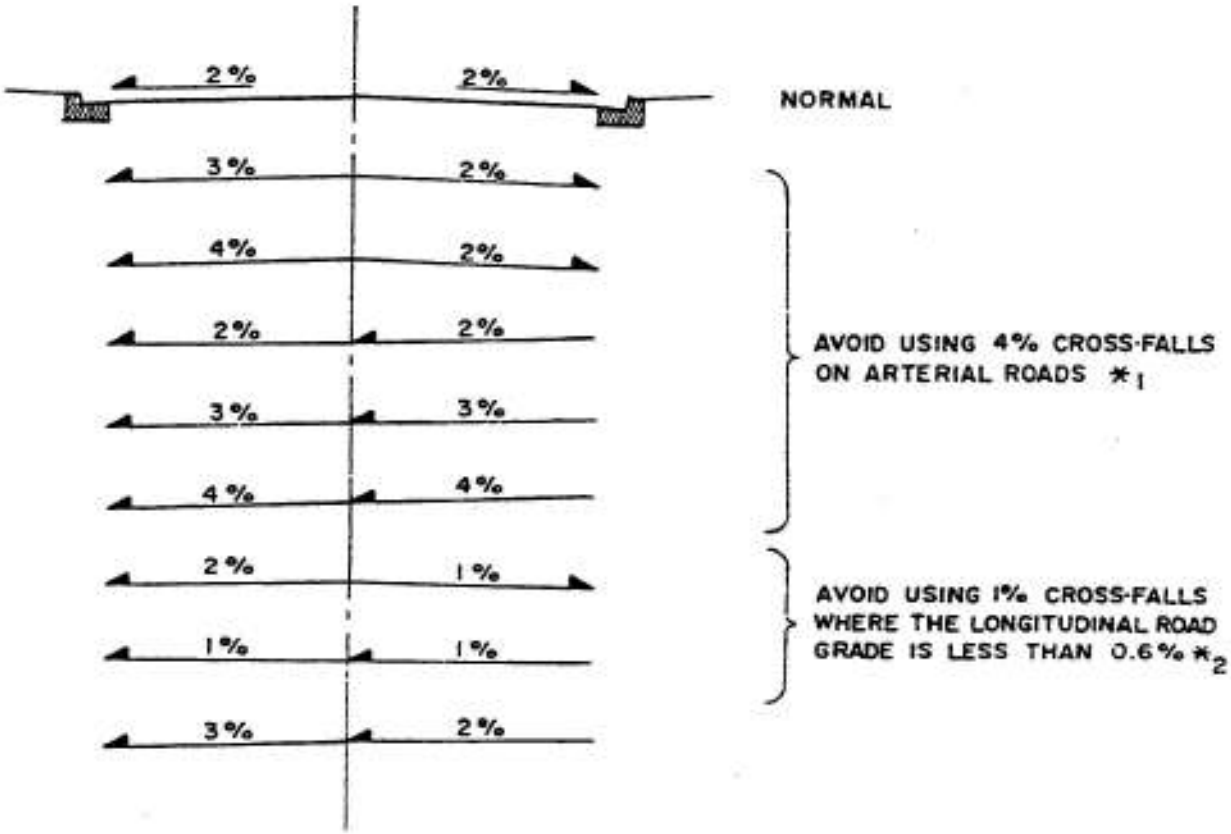
4.0 Pavement Cross Fall

On 40 to 55 km/h local and collector streets the pavement cross fall is established to provide drainage off the road surface, and to blend the road cross section into the surrounding terrain.

The cross fall may require alteration from the normal 2% to pick up the drainage from a low lying property, or to avoid too steep a driveway gradient on a high property.

The following are some cross fall variations that may be considered:



*1 On arterial roads, with buses and trucks at higher running speeds, an uncomfortable sway may result with 4% cross fall. Also, poles close to the curb may be in danger.

*2 With flat road grades the increased cross fall is required to assist drainage.

4.0 (continued)

The transition section between changes in cross fall should be an adequate length to maintain vehicle driver comfort as well as ensure a satisfactory gutter drainage.

For driver comfort a 1% change in cross fall in 8.0 m of road provides a reasonably comfortable transition. On local streets this may be decreased to a minimum of 3.0 m.

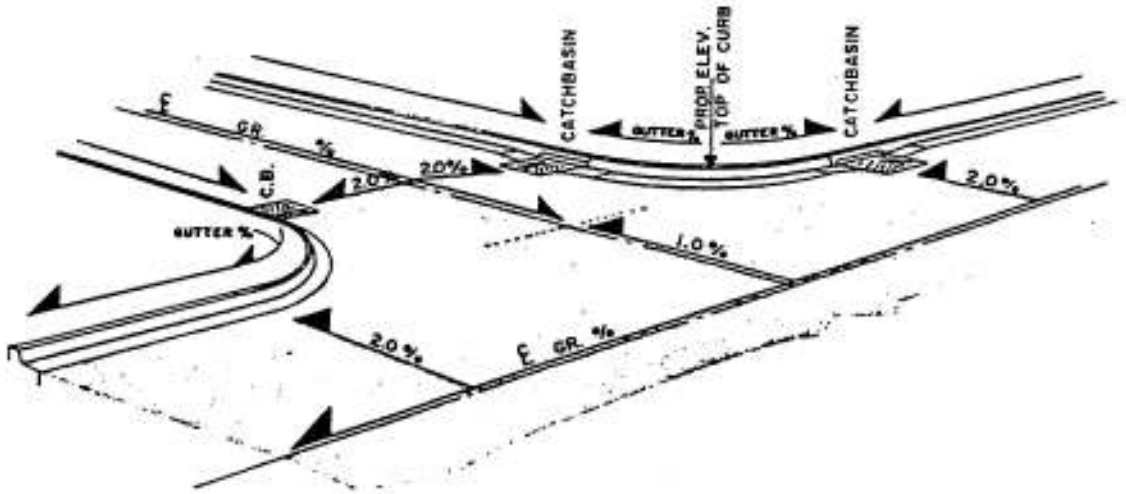
Cross fall changes along roads with a minimum longitudinal gradient may be tied into catchbasins for drainage.

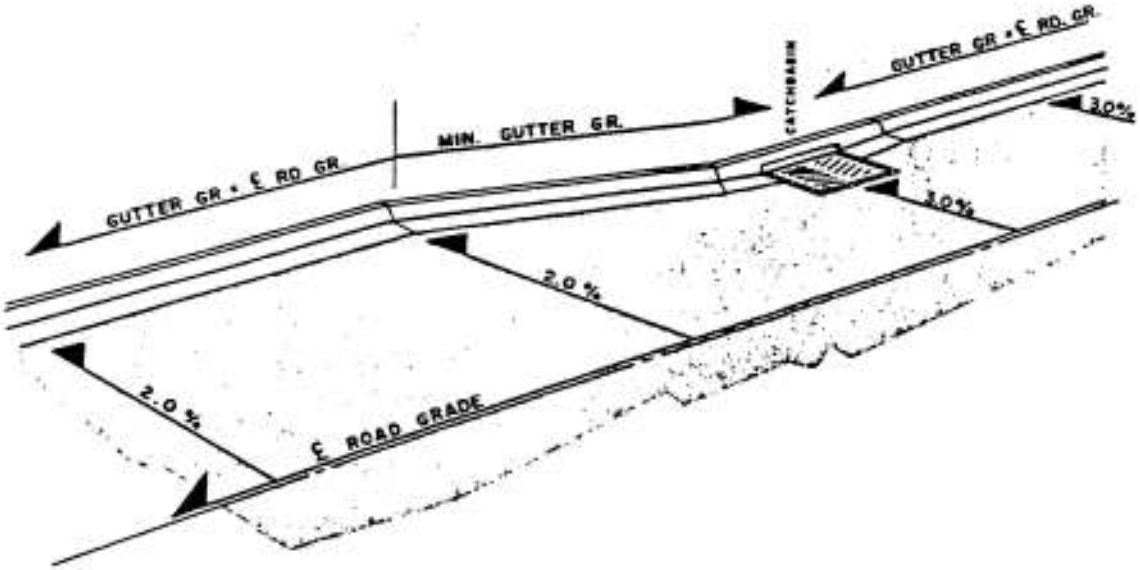
Cross falls are often adjusted at intersections particularly where the intersection is controlled by traffic lights, to improve the vehicle ride. Particular attention should be paid to the road surface drainage on 'flat' road gradients to avoid "bird baths" on the pavement area.

4.0 (continued)

PAVEMENT CROSS FALL

PROVIDE DETAILS OF GUTTER DRAINAGE AROUND CORNERS AND ENSURE THAT THE MINIMUM GRADIENTS ARE MAINTAINED.





WHEN THE CROSS-FALL IS ALTERED ON A ROAD WITH A FLAT ℄ ROAD GRADIENT, CHECK TO ENSURE A MINIMUM GUTTER GRADIENT IS MAINTAINED OR BACKFALL GUTTER AT A MIN. GRADE OVER THE TRANSITION SECTION TO A CATCHBASIN.

5.0 Gutter Drainage

The pavement gutters normally follow the established centre-line road gradient, except in a transition section between changes in pavement X-fall or where the pavement width changes. The normal range for the pavement longitudinal grade is from 0.5 to 5.0%. In order to economically provide a road to suite particular site conditions these limits may be exceeded. It is recommended that the minimum gutter gradient be not less than 0.35%. Gutters of less than 0.5% grade should be formed in concrete as part of the road curb.

Where crest or sag vertical curves are involved, attention should be paid to the gutter gradient to ensure that ponding conditions do not result.

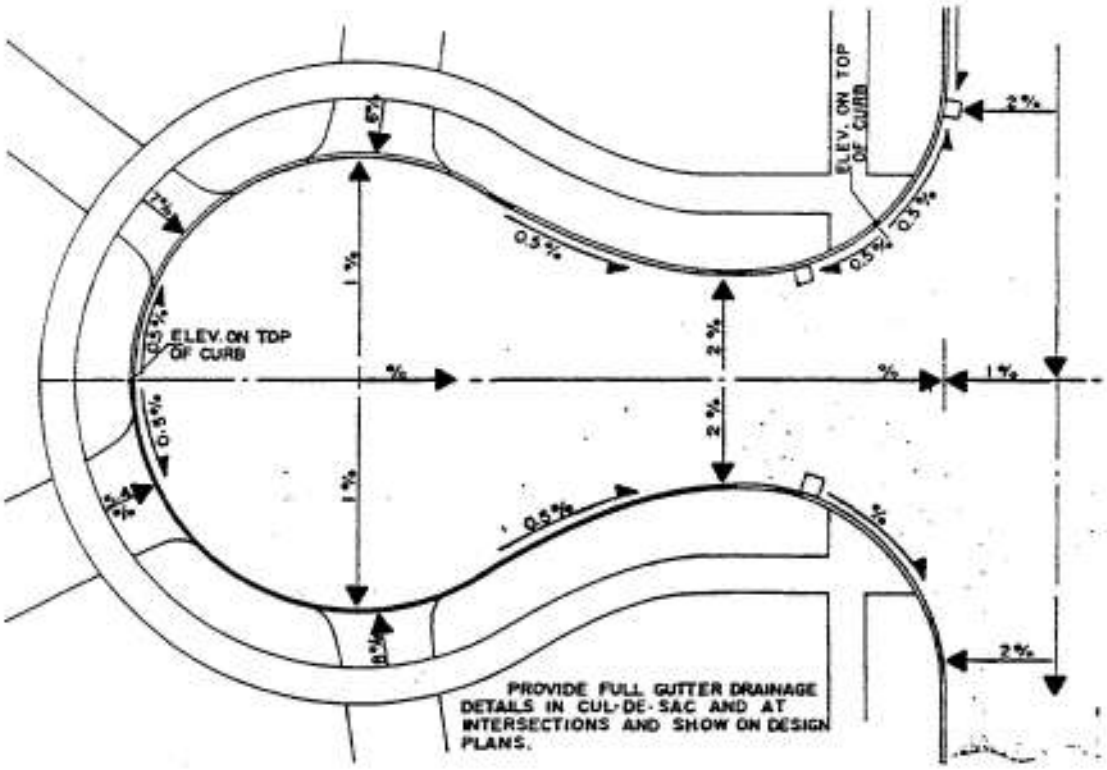
Particular attention should be paid to ensure satisfactory gutter grades around the curb radius at road intersections and in road culs-de-sac. These gutter grades should be detailed on the design plans to provide a guide for the layout for construction by field staff.

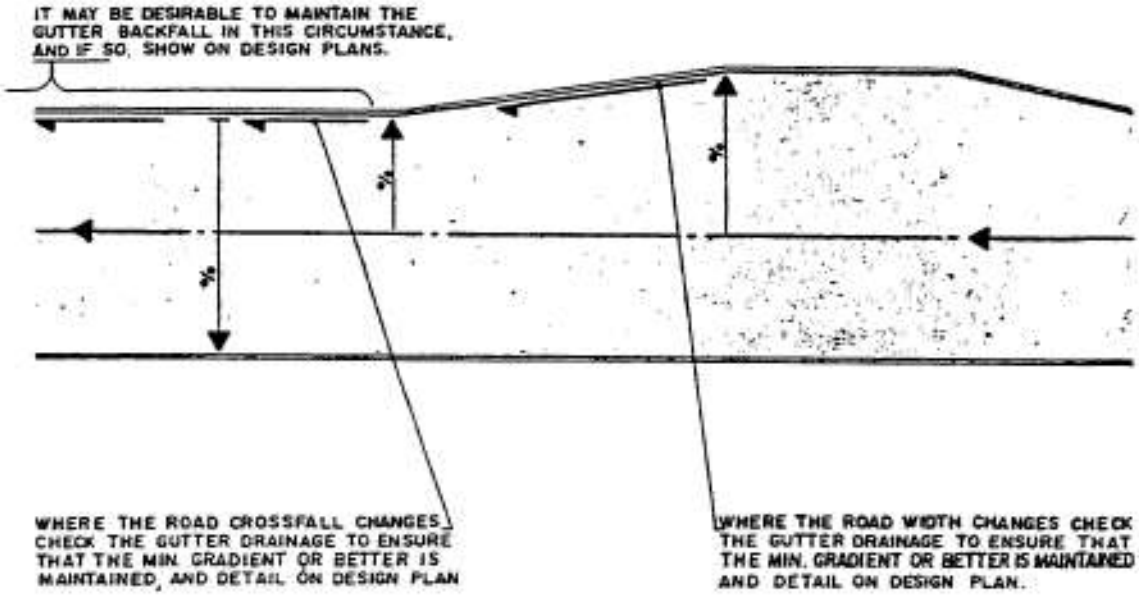
A concrete curb and gutter section is generally recommended even when the road gradient is 0.5% or more. When this type of curb section is used, it is desirable that an increased cross fall be provided in the gutter to assist in collecting the drainage from the road surface and carrying it to the nearest catchbasin. If a 300 mm concrete gutter width is used then a 20 mm fall is suggested.

This gutter backfall is also of assistance to collect the drainage off the high side boulevard and carry it to a catchbasin rather than let it drain across a superelevated road width.

5.0 (continued)

GUTTER DRAINAGE





6.0 Ditch Drainage

Where a storm sewer system is not available, or where an "interim" type of road development is desired, ditches may be used to carry the drainage down both sides of the road.

In a sandy soil situation, and the availability of a storm sewer system or relatively low flows from the immediate area, very shallow swales beside the road shoulder may serve the purpose. In this case probably culverts will not be required at driveway crossings.

Otherwise a full ditch section will be required, with sufficient depth to allow for the installation of a suitably sized culvert at each driveway, as well as at road intersections.

If storm sewers are available, or may be installed with either a satisfactory temporary overflow outlet or connected into an existing storm sewer system, it is most desirable that a curbed pavement be considered rather than any ditched interim development. In many instances it may be found that the cost of constructing shoulders, ditches and installing the necessary driveway culverts will be close to the initial cost of the curbs, without taking into consideration the overall increased maintenance cost of maintaining ditches particularly during winter conditions.

Although a road with adequate cross fall and using ditch drainage may be constructed with a flat gradient, it is well to remember that in an urban environment sooner or later it will become desirable to eliminate the ditches and install curbs. The road grade should therefore be designed accordingly.

The ditch gradient will generally follow the road gradient, however, it is desirable to maintain a minimum ditch gradient of 0.50% where possible to avoid problems with water ponding along the ditch line.

It is recommended that a minimum of 300 mm diameter culvert be used to carry the ditch drainage under driveways and across intersections. It may be necessary to increase the culvert size used depending on the anticipated flow and the tributary drainage area involved.

Culverts are generally a minimum of 1.6 mm thick galvanized corrugated steel pipe.

7.0 Boulevard Drainage

In order to avoid that the drainage from the road area spill back onto private properties, the road allowance is normally graded to provide a fall from the street line to the road curb.

This slope is generally recommended to be from 2 to 6%. A slope in this range ensures satisfactory drainage off the boulevard area, as well as any driveway access to the road being not greater than 10%. Driveways slopes greater than 10% may cause a problem with vehicles backing out of the driveway hitting the pavement with their tail pipe or bumper.

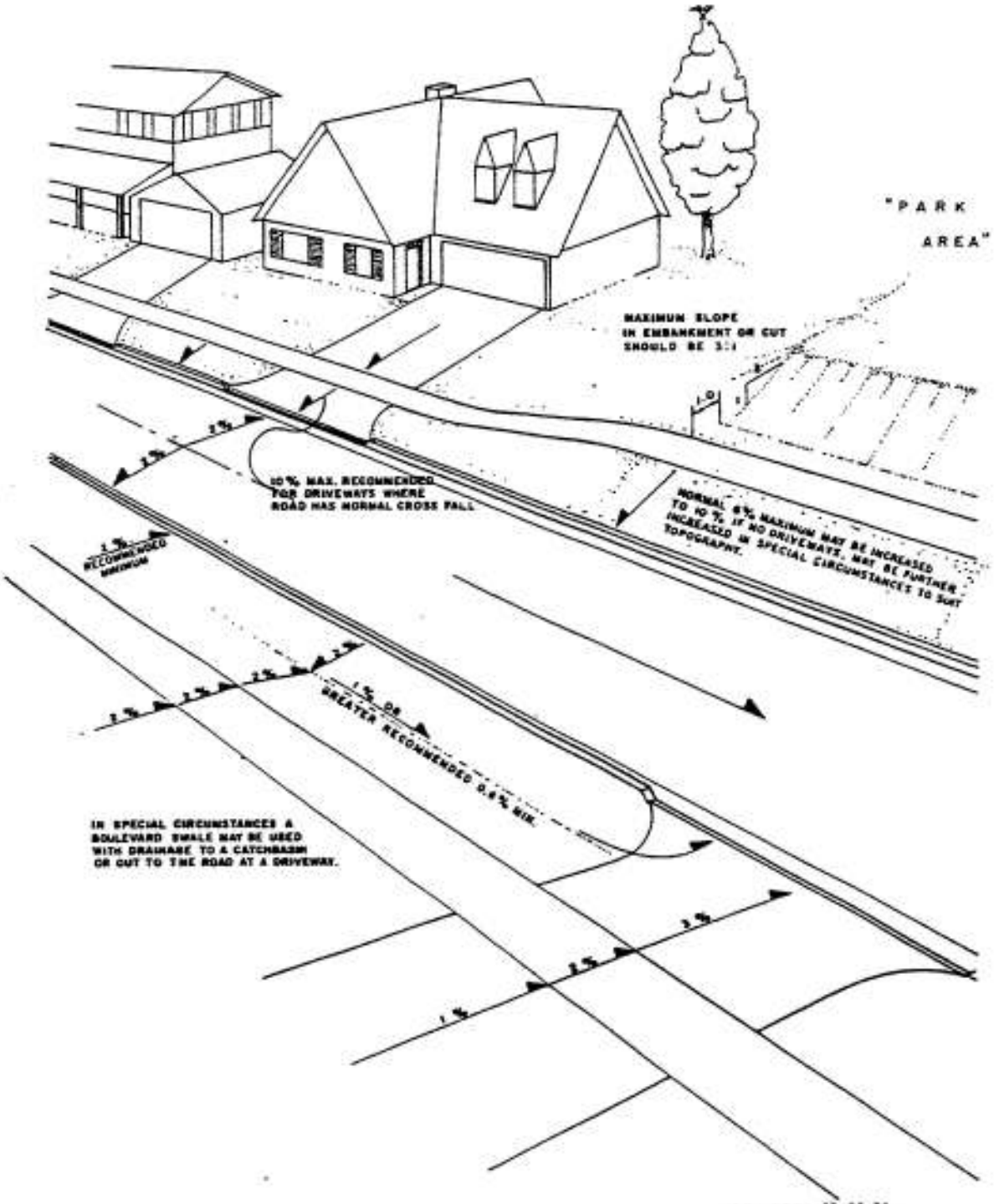
In sections where a driveway will not be required in the future, the boulevard slope may be increased to 10% or in special circumstances greater to suit the topography (e.g. to save a large tree). However, consideration should be given to the location and gradient of any future sidewalk as well as the maintenance of the boulevard and the slope restrained accordingly.

Occasionally it may be necessary to provide a drainage swale in the boulevard area between the curb and sidewalk. In these circumstances the drainage may be carried out a driveway, or to a catchbasin in the boulevard. Particular attention should be taken to ensure an adequate fall along the swale. It is desirable that this grade be 1% or greater, however, in no case should it be less than 0.6%.

In cut or embankment conditions, the maximum slope should be 3:1, in order to avoid erosion problems as well as provide reasonable slopes for grass mowing. Retaining structures should be considered where difficulties are experienced obtaining a slope gradient.

7.0 (continued)

BOULEVARD DRAINAGE



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8.0 Sub-grade Drainage

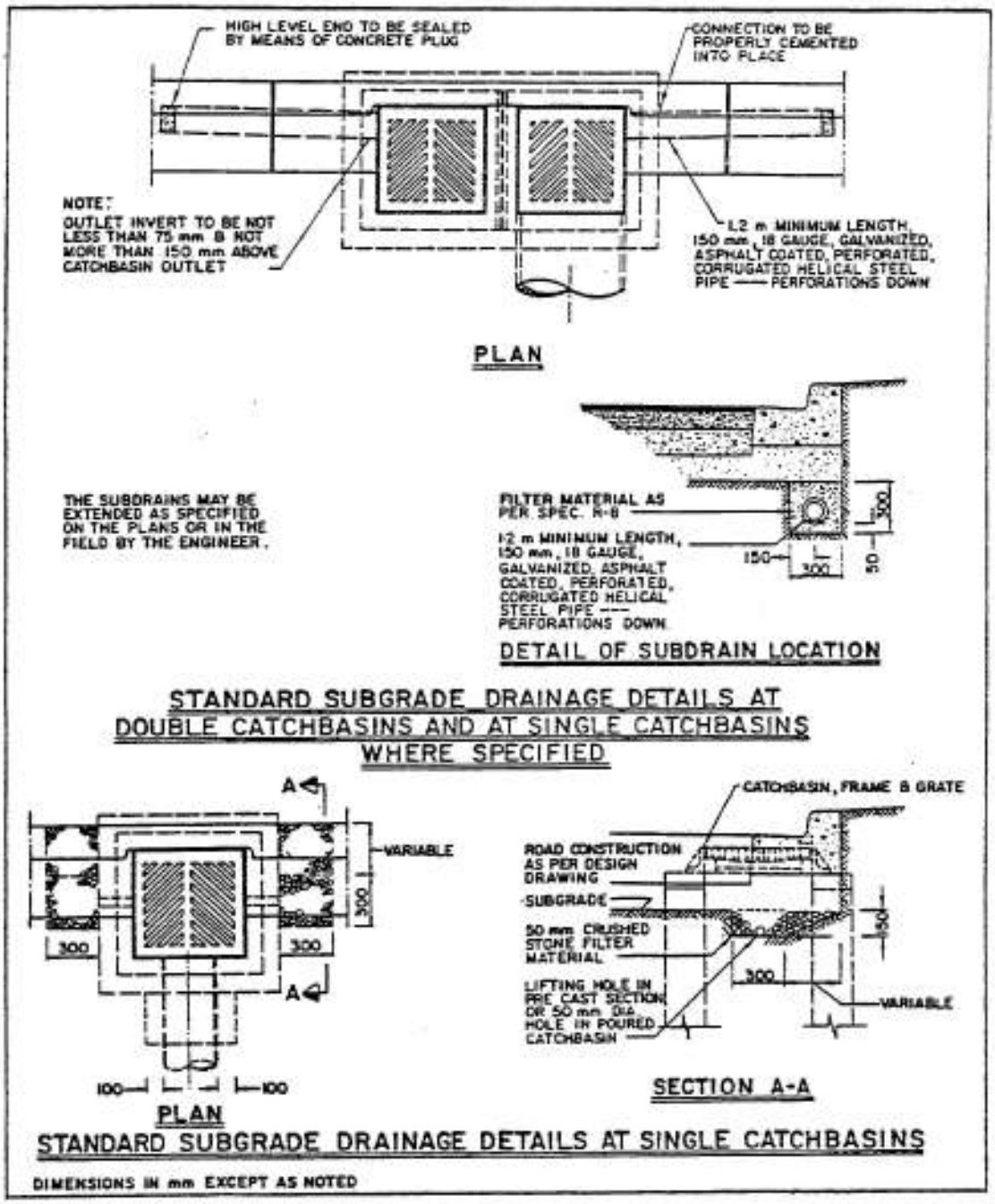
Sub-drains are perforated pipes placed under the curbs below the sub-grade level of the road draining into the catchbasins.

They are generally used to carry off the drainage from a high water table or other sub-surface drainage condition or as to maintain an unsaturated condition in the road base materials.

In urban areas they are usually restricted to use in depressed road conditions such as in an underpass, or for short lengths on either side of low point double catchbasins.

The following drawing will provide some details for the installation of sub-drains.

8.0 (continued)



5.2 VEHICLE FACILITIES

5.2.4 PARKING

1.0 Introduction

Parking operations within a municipality should be properly planned to give a balance of on-street and off-street parking facilities.

Parking should be at locations where the wishes of the public are fulfilled. They must be convenient for the user and should not create an undue hardship on the non-parking public.

Access to the off-street parking can best be located on minor streets and not near the intersection of major streets because it adversely affects the flow of all nearby traffic. The width of entrances must be sufficient for a vehicle to turn easily into it. It is suggested that a maximum width of 9.0 m be used so that the driveway does not become unsafe for pedestrian use but is sufficient for two lines of vehicles and permit a pedestrian safe passage. An auxiliary turning lane can be constructed as part of a public street where entrances are built on collector or arterial streets by necessity.

Parking problems usually arise within a central business district. A planned approach can only be obtained by establishing certain parking policies. A mixture of short, medium and long term parking should be provided to serve both the needs of the shoppers and the workers. Parking near shopping areas should be for short and medium time limits to serve the shoppers and discourage all day parking by workers. Long term parking can be provided on the periphery of a district since people are generally willing to walk a few blocks to their final destination when they intend to park for more than two hours. On-street parking should be prohibited at intersections for a minimum of 30.0 m on the approach side and 18.0 m on the departure side on arterial collector streets of the business area. Where turning volumes are low on the side streets this requirement can be reduced to a minimum of 15.0 m on the approach side and 7.5 m on the departure side of an intersection. Parking should be prohibited from a street on a block basis where the number of parking spaces is minimal (3 or less). On-street parking directly in front of off-street parking areas should be eliminated with the possible exception of where more than 3 parking spaces can be provided. This restriction should also apply for a distance of 3.0 m on either side of all driveways.

1.0 (continued)

By taking along term view on a parking plan it is suggested that a mixture of short, medium and long term parking be provided. Shopper parking should be provided within a distance of 90 m or approximately one block from a shopping destination. Off-street can be developed around the periphery of a core area for long term parking. This can be for 5 hour duration and located some 300 m from an activity centre. Rear pedestrian access to stores and general policy of improving rear store entrances should always be encouraged where practical. Adequate off-street parking can be provided as an integral part of all new developments based on the ratios specified usually under a land use. This requirement varies as the number of parking spaces per 90 m^2 of floor space; say for banks equal to 5.4 and decreasing to 4.0 for libraries, grocery stores, medical buildings; 3.6 for city or county offices; 2.8 for retail stores; 2.1 for restaurants; 2.0 for motels; 1.5 for offices 1.2 for auto sales; 0.6 for hotels; 0.5 for industrial sites, churches and schools. Parking meters and rates should always be considered as a means of regulating parking usage to satisfy the greatest number of parkers and not simply as a source of revenues. Revenues from parking can be used to defray the costs of development, maintenance and operation of parking facilities.

In general parking should be considered as an adequate terminal system in the retail/commercial areas of the community. Objectives should be established and maintained to improve the parking service in a business area; - to provide guidelines for parking needs related to specific land uses; to provide a program of parking system improvements; - and to establish acceptable rate structures for parking fees that will allow a solvent operation.

2.0 <u>On Street Parking</u>

Street parking decreases capacity, impedes traffic flow, and increases hazard. Preferably, parking should be eliminated on major streets.

If parking is to be provided on major streets, it should be parallel rather than diagonal. Diagonal parking results in much greater hazard in the unparking maneuvers.

With parallel curb parking there is appreciable interference to traffic movement as vehicles stop in the through lane to back into parking spaces. Interference of this nature may not be serious where few vehicles park and where the volume of through traffic is low, but it requires only a slight increase in either to cause serious delays and congestion.

2.0 (continued)

In many cases objection to street parking arises only during the peak hours, and often eliminating the curb parking on one or both sides of the street during these periods results in an increased efficiency of operation.

Where parallel parking is to be permitted on a major street, there should be sufficient parking width to allow the through traffic to move with reasonable efficiency and safety. Although the parked vehicle is a fixed object, it is also subject to motion, perhaps without warning. A driver leaving the parking space may edge out to have clear vision, a left door may open suddenly, or a pedestrian may appear from behind or between parked vehicles.

For cars, a parking lane width of from 3.0 to 3.6 m is recommended, and for buses and commercial vehicles, 3.6 to 4.2 m, where parallel parking is to be provided adjacent to arterial traffic flows. These widths do not represent the space actually occupied by the vehicles, but the width of pavement needed for proper operation.

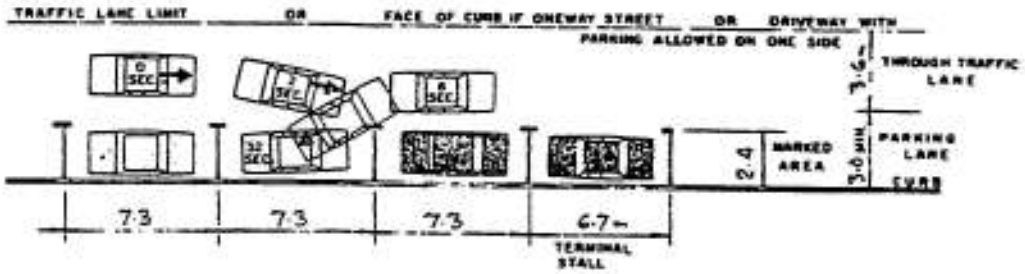
To ensure proper use of the parking area, the marking stripe should be located 2.4 m from the curb face. Where few commercial vehicles are expected to park 2.1 m may be considered, and on industrial streets, 2.7 m.

The recommended length of each stall is 7.3 m, with 6.7 m used at terminal stalls.

In order to provide for adequate site lines at the intersections with major streets, the parking lane should be kept clear of parked cars for a distance of 30 m to the left and 21 m to the right from the cross street centerline.

2.1 On Street Parking Schemes

Conventional Parallel Parking

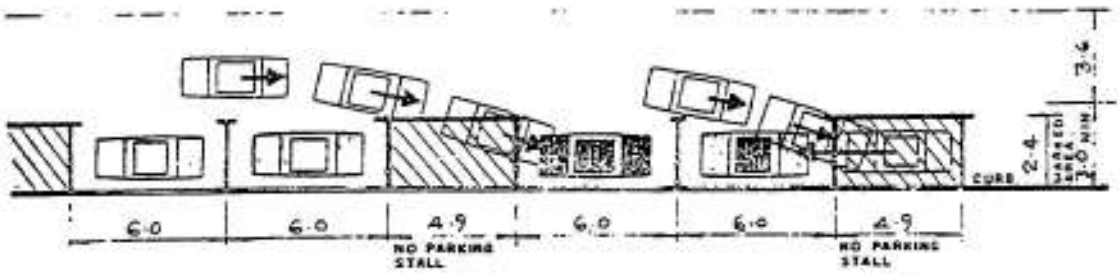


Approximate average parking time for car maneuvering as shown.

Approximate average parking time for car maneuvering as shown.

2.1 (continued)

Tandem Parallel Parking



Where 'no parking' maneuvering areas are provided, the approximate average parking time is reduced to 4 seconds for a car pulling into a parking space from a downstream stall to 6 seconds where a car pulls in from an upstream stall. This arrangement considerably improves through traffic movement.

3.0 Parking Lots

Parking lots should be designed to accommodate the larger cars frequently used. The accompanying diagrams provide some guidelines for parking lot layout.

When parking at right angles to a curb or buffer, the overhang needs consideration when planning the width of a sidewalk affected by the overhang. The overhang may be as much as 1.2 m for a car backing into a stall, and a little over 0.6 m for the front overhang.

The front overhang may be calculated as 0.45 m when figuring closely the minimum feasible spacing between buffers for a minimum width lot. For example with 90 degree parking off both sides of a central driveway, the space normally required is 18.9 m, however the space between buffers may be 17.7 m because of the overhang.

Parking stalls should be at least 2.7 m wide, and should be increased to 3.0 m where space is not too restricted.

End stalls, particularly where the parking maneuvers may be restricted by a building or wall, should be about 0.3 m wider than usual.

A central driveway with two rows of cars and 90 degree parking gives the best economy. Diagonal parking is easier for the driver, reduces the necessary driveway width, but requires more total space.

3.0 (continued)

Careful thought should be given to ensure a satisfactory parking pattern, including the location of access drives.

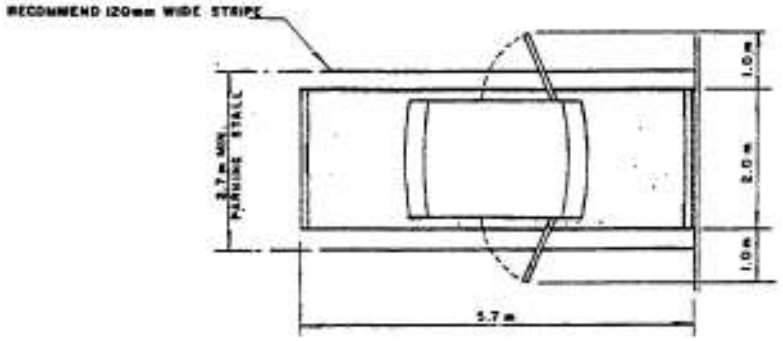
Where ramps are required, the maximum recommended gradient is 15% with 12% generally suggested. They should be treated to avoid winter icing problems.

Care should be taken in the design of the parking area drainage, with adequate provision for catchbasins, in order to avoid ponding or icing conditions which would make walking difficult for the person crossing the parking area to and from a car.

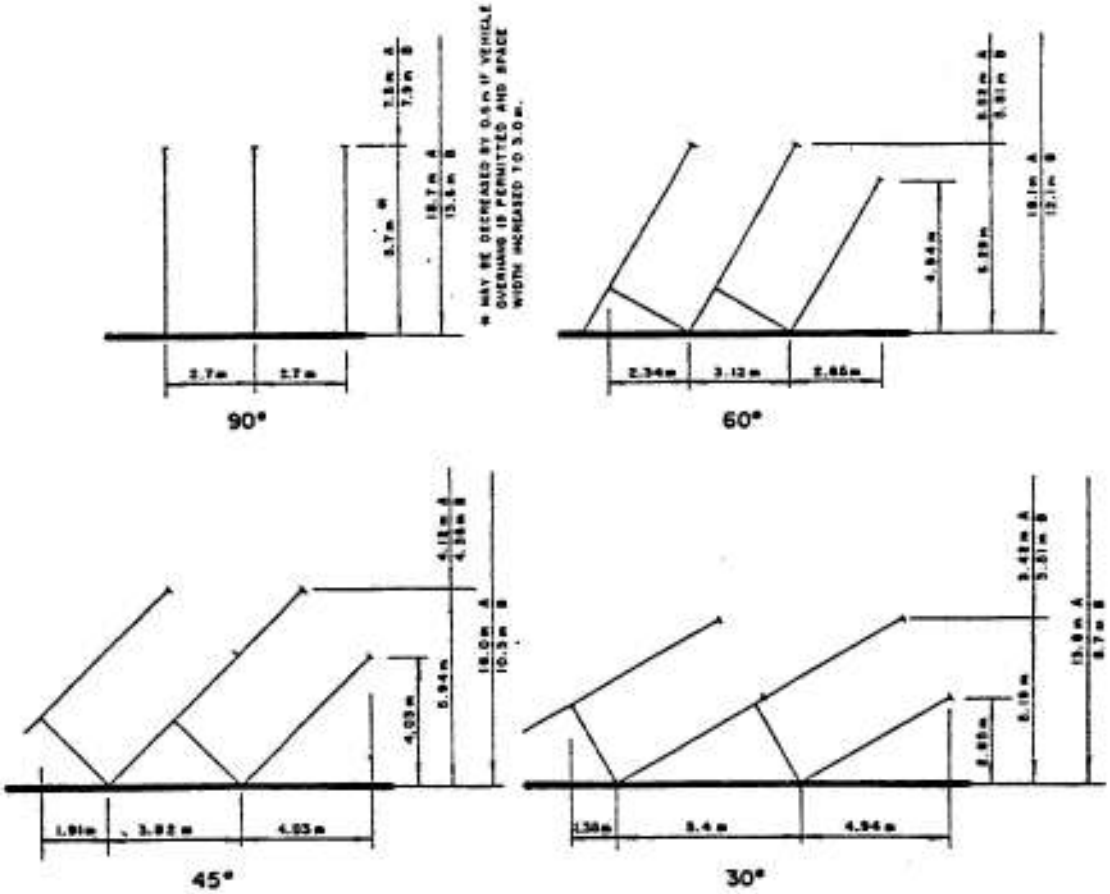
It is recommended that the slopes be at least 1%, since drainage is usually carried along a depressed area in the asphalt rather than by concrete gutters or adjacent to a curb.

3.0 (continued)

PARKING LOTS



STANDARD FULL SIZE CAR DIMENSION

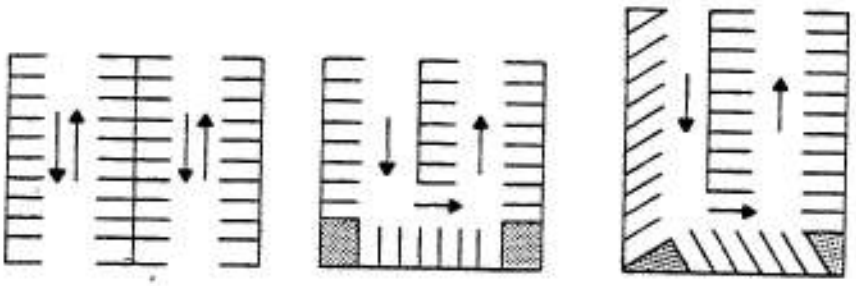


NOTE: DRIVE WIDTH A = PARKING ON TWO SIDES OF DRIVE
B = PARKING ON ONE SIDE OF DRIVE

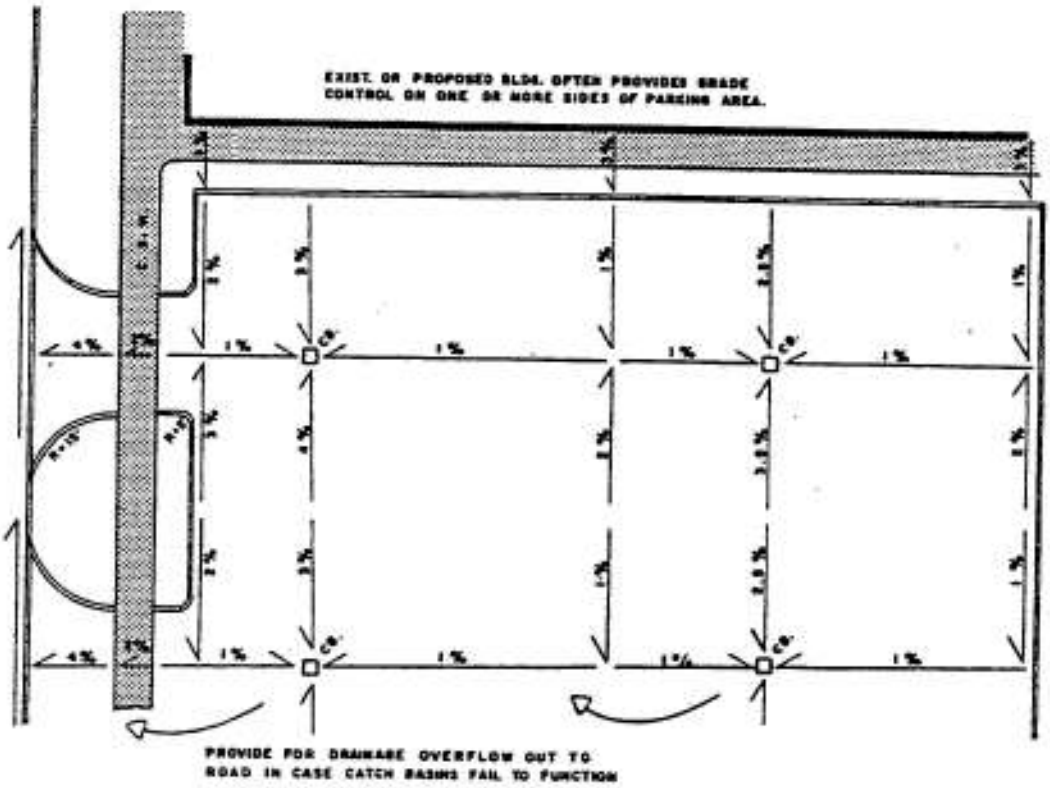
RECOMMENDED MINIMUM PARKING STANDARDS

3.0 (continued)

PARKING LOTS



EXAMPLES OF PARKING LOT PATTERNS



EXAMPLE PARKING LOT DRAINAGE

5.2 VEHICLE FACILITIES

5.2.5 RAILROAD LEVEL CROSSINGS

1.0 General Considerations

For at grade intersections, adequate protection must be provided to safeguard crossing of the tracks by motor vehicles. There are several methods of providing protection at these hazardous locations. They are: -

a) Grade Separation (This is the most expensive treatment, but guarantees complete protection)

b) Automatic Signals and Gates

c) Automatic Signals only

d) Warning Signs only

The factors influencing the choice of one of these protective measures are many. Some of these are: -

- Roadway Classification
- Number & Speed of trains
- Number of tracks
- Number of lanes on roadway
- Orientation of Highway & Railway alignment
- Adjacent Property Costs
- Exposure Factor: - Number of trains per day multiplied by the Annual Average Daily Traffic.

Regulations concerning crossings and crossing protection are administered by the Canadian Transport Commission. The type of crossing protection required at a specific location may be dictated by the Commission, however, the following guidelines may assist in identifying hazardous locations. Exceptional conditions should, of course, receive special consideration. For example, on a rural road with an exposure factor of less than 3500, but where the trains are all high speed, protection may be considered. Conversely on a local road or street with an exposure factor of more than 3500, but where the trains are slow moving on a siding or where the use of a flagman is mandatory, further protection may be unnecessary.

Exposure Factor (Trains per day X A.A.D.T.)	Type of Protection
0. - 3500	Advance warning sign only
3500 - 7500	Automatic Signals
75000 and above for rural areas and 150000 and above for urban areas.	- Grade separation where possible to build.

1.0 (continued)

In addition to the above, where visibility is less than that shown in the table below, automatic signals shall be used. When the crossing has two or more tracks, signals and gates shall be used to prevent a motorist being struck by a train that is hidden from view by another train travelling in the opposite direction.

Design Speed of Highway	Dimensions of Sight Along Highway	Distance Triangle Along Railway
50 km/h	65 m	365 m
60 km/h	90 m	365 m
80 km/h	110 m	365 m
100 km/h	150 m	425 m
110 km/h	190 m	490 m

2.0 Road Design Considerations

Avoid a change, if possible in the proposed road centreline gradient in the vicinity of the level railroad crossing.

If the railroad crossing is at a break or crest of a road grade, as may often be the case, maintain as flat a road centreline grade as possible on the section across the tracks (i.e. 0.35% at centreline).

Pay particular attention to the road gutter drainage and detail it on the design plans.

5.2 VEHICLE FACILITIES

5.2.6 LIGHTING INDEX

1. Introduction

2. Light Quality for Effective Seeing

3. Planning the Lighting System

4. Design Criteria

5. Lamps and Luminaires

6. Illumination Calculation

   6.1 Data Collection
   6.2 Select Criteria
   6.3 Select Equipment
   6.4 Select Pole Luminaire and Lamp Combinations
   6.5 Calculate Pole Spacing

7. Typical recommended layouts for lighting installations

8. Appendix AT&K

5.2 VEHICLE FACILITIES

5.2.6 LIGHTING

1.0 Introduction

The purpose of roadway lighting is to provide visual environment that will reduce the possibility of night-time accidents, and provide a feeling of security, comfort and convenience for motorists and pedestrians.

The lighting systems should be designed to be efficient in energy use and provide an adequate level of illumination for the task at hand.

The Illuminating Engineering Society of North America, IES Lighting Handbook (Available in most public libraries as a reference) lists recommended levels of illumination for most locations and activities (tennis courts, railway stations, factories, etc.). This discussion is restricted to roadway lighting. For a more detailed explanation than this space allows reference should be made to:

"Guide for the Design of Roadway Lighting", a Roads and Transportation Association Publication sponsored by Transport Canada 1983, available at 1765 St. Laurent Blvd., Ottawa, K1G 3V4.

"Design Manual for Highway Illumination" 1978, Ministry of Transportation and Communications, Downsview, Ontario.

The definitions of some words used in this Chapter Section are as follows:

Lumen (lm)

Luminous flux is the time rate of flow of light. It is expressed in lumens. Radiant energy in the visual region of the spectrum varies in its ability to pruduce visual sensation, the variation depending on the wavelength. The ratio of the luminous flux to the corresponding radiant flux is know as the "Luminosity (visibility) factor" and is expressed in lumens per watt. This varies with wavelength, having a maximum at approximately 5,550A Angstroms.

A lumen is the Sl unit of luminous flux; the flux emitted within a unit solid angle (one steradian) by a point source having a uniform luminous intensity of one candela light. Sources are rated in lumens.

Luminaire

Luminaire is a complete lighting unit consisting of a light source, globe, reflector, reactor and housing and the support which is part of the housing.

1.0 (continued)

Lux (lx)

Lux is the practical unit of illumination in the Sl system and is equivalent to the metre candle. It is the illumination on a surfce of one square metre on which there is a uniformly distributed flux of one lumen; or the illuminance produced at a surface, all points of which are at a distance of one metre from an uniform point source of one candela.

Mounting Height (MH)

The vertical distance between the roadway surface and the centre of the apparent light source of the luminaire.

Luminance (L)

The luminous intensity of any surface in a given direction per unit of projected areas of the surface as viewed for that direction $L = dl/(dA \cos \theta)$ where θ is the angle between the direction of observation and the normal to the surface (cd/m^2).

Candela (cd)

The Sl unit of luminious intensity (l). One candela, is one lumen per steradian.

Candela/m^2 (cd/m^2)

The Sl unit of luminance (L) equal to the uniform luminance of a perfectly deffusing surface emitting or reflecting light at the rate of one lumen per square metre or the average luminance of any surface emitting or reflecting light at that rate.

Illuminance

The density of luminous flux incident on a surface; $E = \theta/A$ when the area is uniformly illuminated (lux = lumens/m^2) θ = vertical angle at the point of the observer's eye.

Isolux Diagram

A diagram plotted on any appropriate set of co-ordinates to show all points on a surface for which the illuminance is the same as represented by series of isolux line curves.

2.0 Light Quality for Effective Seeing

Before the eye can establish contact with the surrounding objects, sufficient quantities of light should be available. A common method of describing the light quantity is based on the assessment of incident light density falling on a surface and is measured in lux, that is, it is measured before it strikes the object.

Negative contrast (silhouette seeing) is the principle which enables us to detect objects under low luminence levels.

Incident light density has little relationship to the visual process since light flux reaching the surface from a source does not help to assess the contrast sensity between the object and its background.

A comparison between the illuminance values and the road surface luminance which the driver experiences was made by the MTC on Hwy 401, Toronto By-pass, and without reproducing the complete data here, the information obtained from two locations was as follows, horizontal illuminance 6.5 lux, luminance from concrete surface 0.6 cd/m^2, luminance from the same area paved with bituminous asphalt 0.3 cd/m^2, at another location horizontal illuminance 2.7 lux, luminance 0.35 cd/m^2, and the same area paved with bituminous asphalt 0.2 cd/m^2. Since object detection can be assessed by calculating the contrast difference between the object and its background, the light quantity assessment based on luminance is related much more closely to the visual process than the density of incident light or illuminance.

A preliminary lighting design is simpler to carry out using illuminance methods, followed by a luminance calculation which is tedious manually. There are computer programmes available. While the eyes are capable of adapting to luminances in the order of 10,000 cd/m^2 (summer sunlight) to 0.001 cd/m^2 (starlight) the process of adaption takes up to 40 minutes. While driving at night the driver is subjected to other cars headlights, bright service station areas, etc. in the urban area. A roadway that is lit to give a luminance close to the background lighting level will modify these bright contrasts, will increase the driver's visual contact with the highway and adjacent objects.

Objects on the roadways can be identified, by direct vision by vehicle headlights. The range of vehicle's dipped headlights is restricted to about 75 m, which is inadequate for speeds in excess of 65 km/h.

The increase or decrease in the reflecting power of the road surface has a much greater effect on revealing power than a corresponding percentage increase or decrease in the light

2.0 (continued)

output of the lighting installations. This is because the increase in light output brightens both the light patch on the road and objects equally and the apparent contrast is little changed. Thus, in our design criteria, it is necessary not only to establish a level of illumination and/or luminance, but a uniformity ratio. This is established by a number of agencies such as CIE, RTAC as 3:1 average to minimum road luminance, ratio.

Illuminance established as $E = \frac{\text{incident lumens}}{\text{area}}$

$$= \frac{\not{\Phi}}{A \cos \theta}$$

where E = illuminance in lux
$\not{\Phi}$ = luminous flux in lumens
A = area in m^2
θ = the angle between the incident intensity and the normal to the plane of the area

$$\underline{\text{Luminance}}\ L = \frac{\text{reflected intensity}}{\text{area}} = \frac{Ie}{A \cos \theta}$$

where L = luminance in cd/m^2

A = area of surface perpendicular to incident flux in m^2
I = emitted luminous intensity in candela
e = the angle between the normal to the area and the emitted intensity

3.0 Planning the Lighting System

Warrants

Lighting and upgrading of a low level of illumination has been found to significantly decrease the frequency and proportion of night traffic collisions, between vehicles, and vehicles and pedestrians, there are many documented studies for this conclusion. There is, however, little that has been accomplished by way of defining a relationship between light intensity and accidents, that is, above a certain level of illuminaton at night, there appears to be no significant change in the accident rate reduction. In rural areas it has been determined that 30% to 40% of night-time accidents occur at intersections about half of these could be eliminated by partial lighting.

In general terms, warrants can be categorized as geometric factors, operational factors, environmental factors and accident factors.

3.0 (continued)

Appendix 'A' at the end of this section is a table extracted from the 1983 RTAC Manual illustrating a method for warranting lighting for a non-controlled access roadway, the factors used are self-explanatory.

It will be found that in applying this table, that horizontal curvature, operating speed, pedestrian movements and adjacent lighting are the major considerations.

There are special locations where lighting is always warranted:

a. signalized intersections,
b. roadway underpasses of a greater length than 19 m,
c. lighting of roadway underpasses or tunnels greater than 124 m including daytime lighting, and
d. all pedestrian tunnels greater than 23 m long.

Partial Lighting

a. the warrant for traffic signals are at least 80% fulfilled,
b. the accident rate exceeds 1.5 accidents per million vehicles entering the intersection,
c. the night/day accident ratio exceeds 1.5,
d. the night accident rate is 3 per year over the last three years, and
e. adjacent properties have high lighting levels.

At entrance and exist ramps to freeway interchanges, at pedestrian crossings of entrance and exit ramps and at railway crossings where the exposure index exceeds 5,000.

Partial lighting at various hazardous locations such as substandard roadway geometry, temporary work, temporary detours, and at lane terminations, such as the change from a divided roadway to an undivided road and a merging from two lanes to one lane.

Priorities

Priorities for the installation or retrofit of existing roadway lighting should be based on an overall policy that has taken into consideration, energy conservation and night accident prevention.

Regarding energy conservation, an analysis of existing systems and their cost as compared with the benefit desired from a new level of illumination and revised operating cost for a new system can be carried out. Incandescent lighting is not a efficient system and it can in most cases, be easily proved that replacement is warranted.

4.0 Design Criteria

The definitions of roadway class referred to in the following text are those defined in the RTAC Manual, "Geometric Design Standards for Canadian Road and Streets".

Levels of Lighting

Studies have been carried out by many agencies to determine the most desirable level of illumination on roads of various characteristics. There are standards established by the Canadian Standards Association (CSA), British Standards Association (BSA), Illuminating Engineering Society of North American (IES), Lighting Handbook and RTAC to name a few. Attached as Appendix 'B' is the Table of Design Criteria from the RTAC 1983 "Guide for Roadway Lighting". This establishes both luminance and illuminance criteria, which are similar to the levels established in other guides and are recommended minimum average levels.

5.0 Lamp and Luminaires

Appendix 'C' is a Table prepared for the RTAC Manual based on the IES Handbook. It illustrates lamp data, and in brief commentary on some aspects in selecting an appropriate lamp is as follows.

The most energy efficient system should be used consistent with an appreciation for the colour rendition. While low pressure sodium lamps (LPS) are efficient, their monochromatic colour is not always acceptable in commercial or residential areas. High pressure sodium (HPS) lamps are normally acceptable in most locations. There are areas where better colour rendition is required and colour corrected mercury vapour (MV) lamps may solve the problem.

Appendix 'D', a Table from the RTAC Manual recommending luminaires and mounting lights. Adherence to the recommended levels will reduce the problem of glare, in the area of intended illumination. Note should be taken of possible glare on adjacent roadways, such as commonly occur in interchange areas.

Appendix 'E', a Table from the RTAC 1983 Manual recommending luminaire pole combinations on various road classes.

6.0 Illumination Calculation

6.1 Data Collection

(a) Plan of roadway, including underground plant location.

(b) Cross-section of roadway.

6.1 (continued)

(c) Possible layout arrangement of light poles; i.e. opposite, staggered, centre mounted where a centre median available.

(d) Luminaire overhand, from face of curb, or outside edge of pavement.

(e) Luminaire data from manufacturer, illustrating I (intensity values).

(f) Initial lumen rating of lamp.

(g) Light loss factors are a combination of the following:

(1) Lamp lumen depreciation.
(2) Luminaire dirt depreciation is dependent on the environment and the dirt that accumulates on the luminaire and can be remanded by cleaning.
(3) Luminaire component depreciation, is the depreciation of the reflection and refractor due to staining, corrosion, etc. that cannot be removed by cleaning.

6.2 Select Criteria

(1) Average illuminance required, see Appendix B.

(2) Uniformity ratio, Appendix B.

(3) Veiling luminance, Appendix B.

6.3 Selection of equipment, this may be done by reference to Appendix 'E' and 'D'.

Select Luminaires

Mounting Light

Distribution Type

Reference to Appendix 'E' illustrates, long, medium and short range distribution and Type I, II, III and IV laterial distribution.

Normal situations, use short and medium and Type II, III and IV.

Light Loss Factor

In surburban areas with moderate traffic, AHPS lamp would have a factor of 0.70.

6.3 (Continued)

Pole Type

A normal selection could be from concrete, steel or aluminum, with variations as to type of finish, cross-section. There can be considerable variation in the cost depending on the type.

Pole Offset

From edge of pavement to face of pole, preferably at least 2 m at 50 km/h rising to 5 m at 80 km/h. In an urban environment these desirable levels cannot always be achieved. Clearance to allow for door opening then becomes the most desirable minimum.

Bracket Length

These are normally standardized to some degree in municipalities, usually in the range 2 m to 3 m.

6.4 Select the possible combinations to those that appear practical, maximize pole spacing by choosing the highest lamp wattage allowed, to maximize uniformity and minimize glare choose the highest mounting height i.e. 14 m, choose a staggered pole arrangement in preference to an opposite arrangement to minimize cyclic glare effects.

6.5 Calculate Pole Spacing

This example is repeated from the RTAC Manual in abbreviated form as a general guide to illustrate the method of illumination calculation. Using the manufacturer's graphical photo metric data, Appendix 'F' and 'G' (Figures 4.6.2.4a and 4.6.2.4b).

Determine the street side ratio (SSR) on the street side.

SSR = W/h where W = width to be illuminated from the ratio of the luminaire across the roadway (say 18 m in this case)
h = mounting height (say 14 m in this case)

Enter the SSR = 18/14 = 1.29 on the CU graph as indicated on Appendix 'F' to obtain Point 'X' project upwards to the curve to Point 'Y'. Read the co-efficient of utilization as the street side, CU's at the Point 'Z' as 43.2%.

Calculate the spacing from the formula:

$$S = \frac{\phi 1 \times CU \times LLF}{Eav \times W}$$

6.5 (Continued)

ϕ = 27,500 see Appendix 'C' for 250 W HPS
LLF = light loss factor determined as 0.69
EAU = average illuminance see Appendix 'B'
W = width as above

$$\text{Item S} = \frac{(2 \times 27{,}500) \times 0.432 \times 0.69}{17 \times 18} = 54 \text{ m}$$

6.6 Calculate the uniformity

The proposal is sketched as illustrated on Appendix 'A'.

Uniformity requires determination of the illuminance at selected points for comparison against the average. The most likely low point is 'A' and the calculations for it is as follows. Other points can be determined in a similar fashion. For Point A the horizontal distances must be calculated from each luminaire considered to Point A in terms of mounting height ratios.

The longitudinal mounting height ratio (LMHR) or the distance along the roadway from luminaires 1 and 2 is LMHR = (54/2)/14 = 1.93.

The transverse mounting height ratio (THMR) or the distance across the roadway from luminaires 1 and 2 is zero since the luminaires are in line with point 'A'.

For point 'A' luminaire 3: LMHR = 0; THMR = 18/14 = 1.29.

For point 'A' and luminaires 4 and 5, LMHR = 54/14 = 3.86; THMR = 18/14 = 1.29.

Using Appendix 'F' we can now plot 'A' on the isolux diagram, as points A1, A2, A3, A4 and A5, with reference to the luminaires 1, 2, 3, 4, etc.

The contribution to each point from each luminaire is now evaluated as follows.

Measure and calculate the proportional distance from the point to the adjacent isolux lines, in this example:

	Point 'A'	Point 'B'
Luminaire 1	6.67	11.836
Luminaire 2	6.67	3.286
Luminaire 3	10.98	11.836
Luminaire 4	0.86	3.386
Luminaire 5	0.86	Nil
Total	26.04 lux	30.244 lux

6.6 (Continued)

The correction factors to be applied to the isolux diagram are as follows:

Mounting height correction factor (MHCF), since the horizontal illumination at any point varies enversely as h^2, the correction factor applies as follows:

$$MHCF = \frac{ht^2 \text{ (as tested)}}{h^2 \text{ (actual)}}$$

Lamp lumen correction factor (LLCF) since lamp lumens to be used may differ from test conditions.

$$LLCF = \frac{\text{Øi actual}}{\text{Øi as tested}}$$

Light loss factor (LLF) - as previously referred to as we are concerned with average maintained values.

The total ICF becomes $\frac{12.19^2}{14^2} \times \frac{27500}{27500} \times 0.69 = 0.52$

The actual value at Point 'A' = 13.54 lux.

Assuming after a number of points are calculated this point is still low.

Average/minimum ratio = 17/13.54 = 1.26:1.

Computer programmes are available from the Ministry of Transportation and Communications and others, as computer aided design techniques. Each programme should have the appropriate user manual as all input data is not identified in the same manner.

7.0 Typial Recommended Layouts For Lighting Installations

Some recommended layouts for various locations in the urban area re attached hereto as:

Appendix 'H' -	Recommended layout for normal crosswalks.
Appendix 'I' -	Rocommended layout for normal crosswalks signalized.
Appendix 'J' -	Recommended layout for crosswalks at intersections.
Appendix 'K' -	Recommended layout for crosswalks at mid-block.

8.0 Appendix "A"
Table 2.4.2.1a
Warrants for non-controlled access roadways.

CLASSIFICATION FACTOR	RATING FACTOR 'R' 1	2	3	4	5	Weight 'W'	Score R x W	
GEOMETRIC FACTORS								
1 No. of lanes	≤4	5	6	7	≥8	0·15		
2 Lane width (m)	>3·6	3·4-3·6	3·2-3·4	3·0-3·2	<3·0	0·35		
3 Median openings / km	<2·5 or 1-way	2·5-5·0	5·0-7·2	7·2-9·0	>9·0 or no med.	1·40		
4 Entrances / km	<20	20-40	40-60	60-80	>80	1·40		
5 Horiz. curve radius (m)	>600	450-600	225-450	175-225	<175	5·55		
6 Vert grades (%)	<3	3-4	4-5	5-7	>7	0·35		
7 Sight distance (m)	210	150-210	90-150	60-90	60	0·15		
8 Parking	prohibited	loading	off peak	one side	both sides	0·10		
								G
Sub-Total Geometric Factors								
OPERATIONAL FACTORS								
9 Signalized intersect (%)	80-100	70-80	60-70	50-60	0-50	0·15		
10 Left-turn lane	all major ints. or 1-way	substantial no. of major ints.	most major intersections	half major intersections	infrequent no. or TWLTL[3,4]	0·70		
11 Median width (m)	>10	6-10	3-6	1·2-3	0-1·2	0·35		
12 Operating speed km/h	≤40	50	60	70	≥80	0·60		
13 Pedestrians / km / night [3]	nil	nil-30	30-60	60-120	>120	0·70		O
Sub-Total Operational Factors								
ENVIRONMENTAL FACTORS								
14 % Development	nil	nil-30	30-60	60-90	>90	0·15		
15 Area Class	rural	industrial	residential	commercial	downtown	0·15		
16 Setback distance (m)	>60	45-60	30-45	15-30	<15	0·15		
17 Advertising or area ltg.	nil	sparce	moderate	distracting	intense	1·38		
18 Raised curb median	none	continuous	all ints.	signalized ints.	few locations	0·35		
19 Crime rate	very low	<average	average	>average	very high	0·35		E
Sub-Total Evironmental Factors								
ACCIDENT FACTORS								
20 Night / day accident ratio	<1·0	1·0-1·2	1·2-1·5	1·5-2·0	>2·0 [1]	5·55		A
Sub-Total Accident Factors								
G + O + E + A = TOTAL WARRANTING POINTS								
WARRANTING CONDITION								60
'D' - DIFFERENCE (±)								

Notes:

1. Continuous lighting warranted.
2. Operating speed ≤ 100 km/h.
3. Number of pedestrians crossing the section of roadway.
4. Two-way left turn lane.

8.0 Appendix "B"
Table 3.1
Design criteria.

ACCESS CLASS	ROADWAY CLASS	AREA CLASS	LUMINANCE CRITERIA			ILLUMINANCE CRITERIA			
			AVERAGE Luminance Lav (cd/m²)	UNIFORMITY Maximum Avg/Min Ratio	UNIFORMITY Maximum Max./Min Ratio	GLARE Maximum Lv/Lav (%)	AVERAGE Illuminance Eav (lux) ③	UNIFORMITY Maximum Avg/Min Ratio	UNIFORMITY Maximum Max./Min Ratio	GLARE Average Ly Max (cd/m²)
Fully Controlled	Freeways	Downtown/Commercial	0·6	3·5:1	6:1	25	9	3:1	6:1	0·15
		Industrial/Residential	0·5	3·5:1	6:1	25	7	3:1	6:1	0·13
		Rural	0·4	3·5:1	6:1	30	6	3:1	6:1	0·12
	Expressways Parkways	Downtown/Commercial	1·0	3:1	5:1	20	15	3:1	6:1	0·20
		Industrial/Residential	0·8	3:1	5:1	25	13	3:1	6:1	0·20
		Rural	0·6	3·5:1	6:1	30	11	3:1	6:1	0·18
Partially Controlled	Parkways Arterials	Downtown/Commercial	1·0	3:1	5:1	20	15	3:1	6:1	0·20
		Industrial/Residential	0·8	3:1	5:1	25	13	3:1	6:1	0·20
		Rural	0·6	3·5:1	6:1	30	11	3:1	6:1	0·18
Non-Controlled	Arterials	Downtown	1·4	2·5:1	4:1	20	22	3:1	4:1	0·28
		Commercial	1·2	3:1	5:1	25	17	3:1	6:1	0·28
		Residential	0·8	3·5:1	6:1	30	12	3:1	6:1	0·24
		Industrial	0·6	3·5:1	6:1	30	9	3:1	6:1	0·18
		Rural	0·6	3·5:1	6:1	30	9	3:1	6:1	0·18
	Collectors	Downtown	1·2	3:1	5:1	20	17	3:1	6:1	0·24
		Commercial	0·8	3:1	5:1	25	13	3:1	6:1	0·20
		Residential	0·6	3·5:1	6:1	30	10	3:1	6:1	0·18
		Industrial	0·4	4:1	8:1	40	6	3:1	6:1	0·16
		Rural	0·4	4:1	8:1	40	6	3:1	6:1	0·16
	Locals	Downtown	0·8	3·5:1	6:1	25	12	3:1	6:1	0·20
		Commercial	0·6	4:1	8:1	30	10	3:1	6:1	0·18
		Residential	0·4	5:1	10:1	40	6	6:1	12:1	0·16
		Industrial	0·4	6:1	10:1	40	6	6:1	12:1	0·16
		Rural	0·3	6:1	12:1	40	4	6:1	12:1	0·12
	Bikeways	Type A	0·25	10:1	20:1	Criteria to match adjacent roadway		10:1	20:1	0·12
		Type B				50	3·5			
	Alleyways	Commercial/Industrial	0·4	6:1	10:1	40	6	6:1	10:1	0·16
		Residential	0·3	6:1	12:1	40	4	6:1	12:1	0·12

NOTES:
① Use either Luminance Criteria or Illuminance Criteria, not both (see Section 4.3)
② Recommended values are meaningful only when used with supplementary recommendations for averaging values, coverage and visual quality do not apply to special situations such as tunnels, crosswalks or insections etc.
③ Values are given for asphaltic pavements of type R2 or R3. For other types multiply the values shown by $(0.07/Q_0)$

8.0 Appendix "C"
Table 2.2.5.1
Lamp data

Lamp Type & Designation	Lamp Watts	Approximate Ballast Watts (1)	Total Watts	Initial Lumens	Initial Lamp lm/w	Mean Lumens	Mean lm/w (2)	Average Life (3)	Annual Energy Use kWh (3)
INCANDESCENT									
202 PS-25	202	N/A	202	2800	14	2500	12	6000	808
327 PS-35	327	N/A	327	4850	15	4360	13	6000	1308
FLUORESCENT									
F48T10 /CW	105	30	135	6700	64	4100	39	9000	540
F72T10 /CW	150	35	185	9400	63	6850	37	9000	740
MERCURY VAPOUR									
H38HT-100	100	35	135	4040	40	3415	34	24000+	540
H38JA-100/DX	100	35	135	4425	44	3620	36	24000+	540
H39KB-175	175	35	205	7975	46	7430	42	24000+	820
H39KC-175/DX	175	35	205	8600	49	7640	44	24000+	820
H37KB-250	250	35	285	11825	47	10625	37	24000+	1140
H37KC-250/DX	250	35	285	12775	51	10790	43	24000+	1140
H33CD-400	400	55	455	21000	53	19150	42	24000+	1820
H33GL-400/DX	400	55	455	23125	58	19840	50	24000+	1820
H35NA-700	700	65	765	40500	58	36250	52	24000+	3060
H35ND-700/DX	700	65	765	42750	61	36045	51	24000+	3060
H36GV-1000	1000	90	1090	56150	56	48400	48	24000+	4360
H36GW-1000/DX	1000	90	1090	63000	63	48380	48	24000+	4360
HIGH PRESSURE SODIUM									
S68MS-50	50	18	68	3650	73	3285	48	24000	272
S62ME-70	70	20	90	5800	83	5220	58	24000	360
S54SB-100	100	35	135	9500	95	8550	63	24000	540
S55SC-150	150	45	195	16000	107	14400	74	24000	780
S66MN-200	200	50	250	22000	110	19800	79	24000	1000
S50VA-250	250	60	310	27500	110	24750	80	24000	1240
S50VA-250/S	250	60	310	30000	120	27000	87	24000	1240
S67MR-310	310	70	380	37000	119	33300	88	24000	1520
S56WA-400	400	85	485	50000	125	45000	93	24000	1940
LOW PRESSURE SODIUM									
L69RA-18	18	14	32	1800	100	1720	54	10000	128
L7DRB-35	35	25	60	4800	137	4570	76	18000	240
L7IRC-55	55	30	85	8000	145	7640	90	18000	340
L72RD-90	90	35	125	13500	150	12850	103	18000	500
L73RE-135	135	40	175	22500	167	21400	122	18000	700
L74RF-180	180	50	230	33000	183	31400	137	18000	920

Notes:

(1) Shown as the highest loss found for commonly used ballast type.

(2) Mean lm/w is 'Mean Lumens' divided by 'Total Watts' representing the available light output per watt of lamp/ballast combination input.

(3) Based on 4000 hours burning time per year.

(4) Typical lamps are shown. For others refer to the IES Lighting Handbook, reference volume, chapter 8 (104).

8.0 Appendix "D"
Table 2.2.6.1
Recommended luminaires and mounting heights

ROADWAY CLASS (1)	AREA CLASS (1)	ROADWAY WIDTH(m) (Note 2)	MOUNTING HEIGHT MIN.	MOUNTING HEIGHT PREF	MOUNTING HEIGHT MAX.	RECOMMENDED LUMINAIRES(4)	ALTERNATIVE LUMINAIRES(4)
Freeways, Expressways, Parkways	All	≤ 14	10	12	15	135-180W LPS(6)	150-250W HPS
		14-22	12	15	17	180W LPS(6)	200-400W HPS
		22-30	15	17	20	180W LPS(6)	400W HPS
		≥ 30	15	20	20	400W HPS	180W LPS
Arterials, Collectors	Downtown Commercial Residential (Note 5)	≤ 14	8	10	12	70-100W HPS	
		14-18	10	12	14	100-250W HPS	
		18-22	12	14	16	150-310W HPS	
		22-26	14	15	16	200-400W HPS	
		26-30	14	15	16	310-400W HPS	
		≥ 30	15	15	16	400W HPS	
	Industrial Rural	≤ 14	8	10	12	55-90W LPS	70-100W HPS
		14-18	10	12	14	90-180W LPS	100-200W HPS
		18-22	12	14	16	135-180W LPS	150-310W HPS
		22-26	14	15	16	180W LPS	200-400W HPS
		26-30	14	15	16	310-400W HPS	180W LPS
		≥ 30	15	15	16	400W HPS	180W LPS
Locals	Downtown Commercial (Note 5)	≤ 9	6	8	10	50-100W HPS	
		9-12	8	10	12	70-150W HPS	
		12-16	10	12	14	150-250W HPS	
	Residential (Note 5)	≤ 9	6	8	10	50-100W HPS(3)	35-55W LPS
		9-12	8	10	12	70-150W HPS(3)	90-135W LPS
		12-16	10	12	14	150-250W HPS(3)	135-180W LPS
	Industrial Rural	≤ 9	6	8	10	35-55W LPS	50-100W HPS
		9-12	8	10	12	55-90W LPS	70-150W HPS
		12-16	10	12	14	135-180W LPS	150-250W HPS
Alleyways	Downtown Commercial Residential(5)	≤ 9	4.5	6	8	50-70W HPS (Note 3)	35-90W LPS
	Industrial	≤ 9	4.5	6	8	35-135W LPS	50-100W HPS
Bikeways, Walkways	All	≤ 5	4.5	6	8	18-135W LPS	50-70W HPS

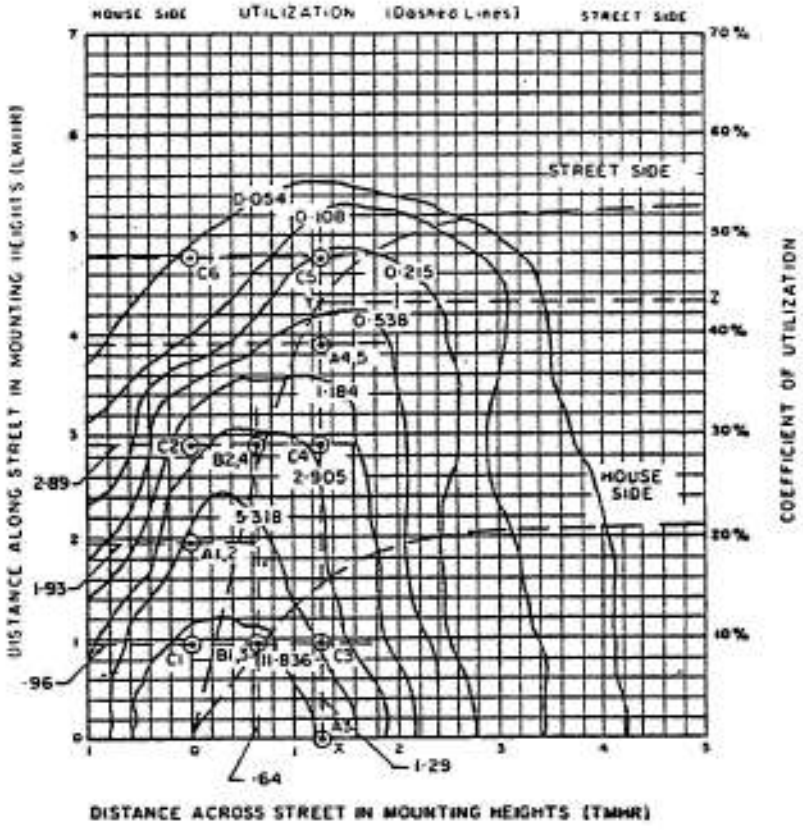
Notes:

1. For definition of classes see Sections 3.3 & 3.4
2. For divided hightways with centre-mounted luminaires, use the width corresponding to the widest approach on one side.
3. LPS luminaires recommended where 'socially acceptable'.
4. Data given is for planning purposes only. Luminance and quality criteria may over-rule the requirements shown for any particular luminaire.
5. Mounting height should consider general height of adjacent buildings.
6. LPS luminaires recommended for median mounting HPS luminaires may be more economical for outside mounting.

8.0 Appendix "E"
Table 2.2.6.3
Suggested luminaire - pole combinations

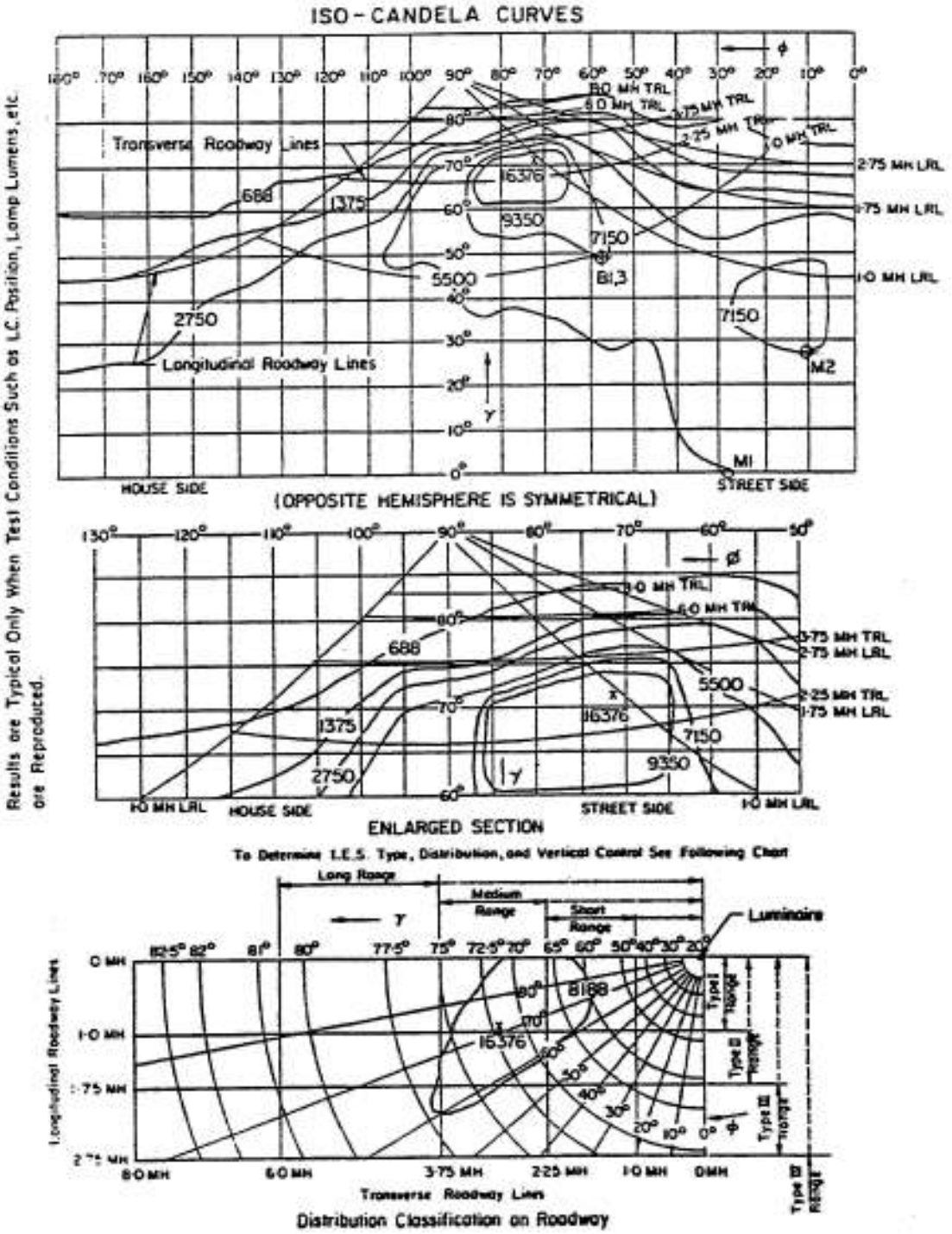
JURISDICTION SIZE	ROAD CLASS	AREA CLASS	ROADWIDTH (m) NOTE 1	MOUNTING HEIGHT (m)	LUMINAIRES (LAMP WATTS)	COMBINATION TOTALS
1. Towns up to 25,000 Population	All (NOTE 2)	All	℄ 9	10	100 W HPS	3 Luminaires/Lamps 3 Mounting Heights
			9 – 22	12	250 W HPS	
			22 – 30	15	400 W HPS	
2. Cities of 25,000 to 300,000 Population	Expressways, Parkways	Downtown Commercial Residential	℄ 9	12	250 W HPS	5 Luminaires/Lamps 3 Mounting Heights
			9 – 22	12	250 W HPS	
			22 – 30	15	400 W HPS	
		Industrial Rural	℄ 9	12	135 W LPS	
			9 – 22	12	135 W LPS	
			22 30	15	180 W LPS	
	Arterials Collectors Locals Alleyways Bikeways Walkways	Downtown Commercial Residential	℄ 9	10	100 W HPS	
			9 – 22	12	250 W HPS	
			22 – 30	15	400 W HPS	
		Industrial Rural	℄ 9	10	135 W LPS	
			9 – 22	12	135 W LPS	
			22 – 30	15	180 W LPS	
3. Cities over 300,000 Population	Expressways, Parkways	Downtown Commercial Residential	℄ 9	12	250 W HPS	8 Luminaires/Lamps 4 Mounting Heights
			9 – 22	12	250 W HPS	
			22 – 30	15	400 W HPS	
		Industrial Rural	℄ 9	12	135 W LPS	
			9 – 22	12	135 W LPS	
			22 – 30	15	180 W LPS	
	Arterials Collectors	Downtown Commercial Residential	℄ 9	10	100 W HPS	
			9 – 22	12	250 W HPS	
			22 – 30	15	400 W HPS	
		Industrial Rural	℄ 9	12	135 W LPS	
			9 – 22	12	135 W LPS	
			22 – 30	15	180 W LPS	
	Locals, Alleyways	Downtown Commercial Residential	℄ 9	8	70 W HPS	
			9 – 16	12	150 W HPS	
		Industrial Rural	℄ 9	8	55 W LPS	
			9 – 16	12	135 W LPS	
	Walkways, Bikeways	All	All	8	55 W LPS	
4. Provincial (NOTE 3)	Freeways Expressways Parkways Arterials Collectors	Downtown Commercial Residential	℄ 9	12	250 W HPS	4 Luminaires/Lamps 2 Mounting Heights
			9 – 22	12	250 W HPS	
			22 – 30	15	400 W HPS	
		Industrial Rural	℄ 9	12	135 W LPS	
			9 – 22	12	135 W LPS	
			22 – 30	12	180 W LPS	
5. Any	Partial Lighting	Rural	℄ 9	12	135 W LPS	4 Luminaires/Lamps 2 Mounting Heights
			9 – 22	12	135 W LPS	
			22 – 30	15	180 W LPS	
		Other than Rural	℄ 9	12	200 W HPS	
			9 – 22	12	135 W HPS	
			22 – 30	15	250 W HPS	

8.0 Appendix "F" 1 of 2
Figure 4.6.2.4a
Photometric report for luminaire code no. 4



ISOLUX & UTILIZATION CURVES
400 W HOUSING
TYPE M-C-III
ϕ_T = 27,500 lm.
h_T = 12·19 m

8.0 Appendix "F" 2 of 2
Figure 4.6.2.4b
Photometric report for luminaire code no. 4



8.0 Appendix "G"
Figure 4.6.2.5a
Example illuminance calculation form

TABLE 4.6.1

Illuminance Calculation Form

Roadway: RTAC ARTERIAL Project No: 1163-81 By: C.E.S. Date 82-06-10

Location: Street 'A' to N Limit Chk'd. By: H.C. Date: 82-06-15

Road Class: ☐ Freeway ☐ Expressway ☐ Parkway ☒ Arterial ☐ Collector ☐ Local
Area Class: ☐ Downtown ☒ Commercial ☐ Residential ☐ Industrial ☐ Rural ☒ Mixed
Access: ☐ Fully Controlled ☐ Partially Controlled ☒ Non-Controlled ☐ Mixed

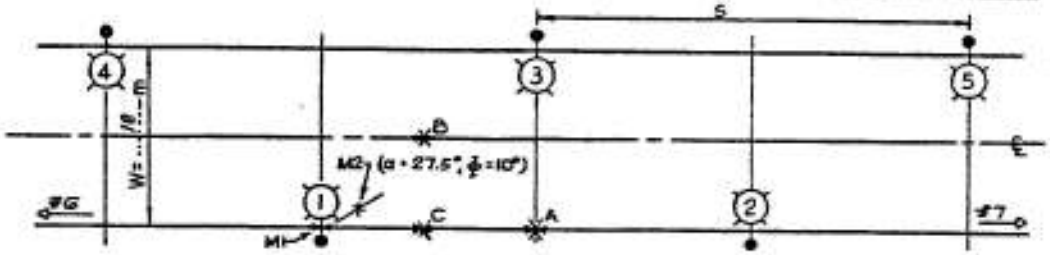
Average Maintained Illuminance, E_{av} = 17 lux Average Max. Veiling Luminance, L_v = 0.28 cd/m² max
Uniformity: Average/Minimum (U1) = 3 :1 max. Uniformity: Maximum/Minimum (U2) = 6 :1 max.

Roadway Width, w = 18 m; Mounting Height, h = 14 m; Overhang o = 0 m

Luminaire Manufacturer Code 4h4 Type = 400 W/M-C-III Test No. = XXX
Test Height, h_t = 12.19 m; Lamp = 250W HPS; Test Lumens, Φ_t 27,500; Initial Lumens, Φ_i 27,500

LLF = 0.69; LLCF = Φ_i/Φ_t = 1.00; MHCF = h^2_t/h^2 = 0.76; ICF = LLF x LLCF x MHCF = 0.52

Sketch Arrangement = 2S



SSR = (W-o)/h = 18/14 = 1.29 Cus = 0.432
HSR = o/h = 0 Cuw = 0.000
Cu = 0.432

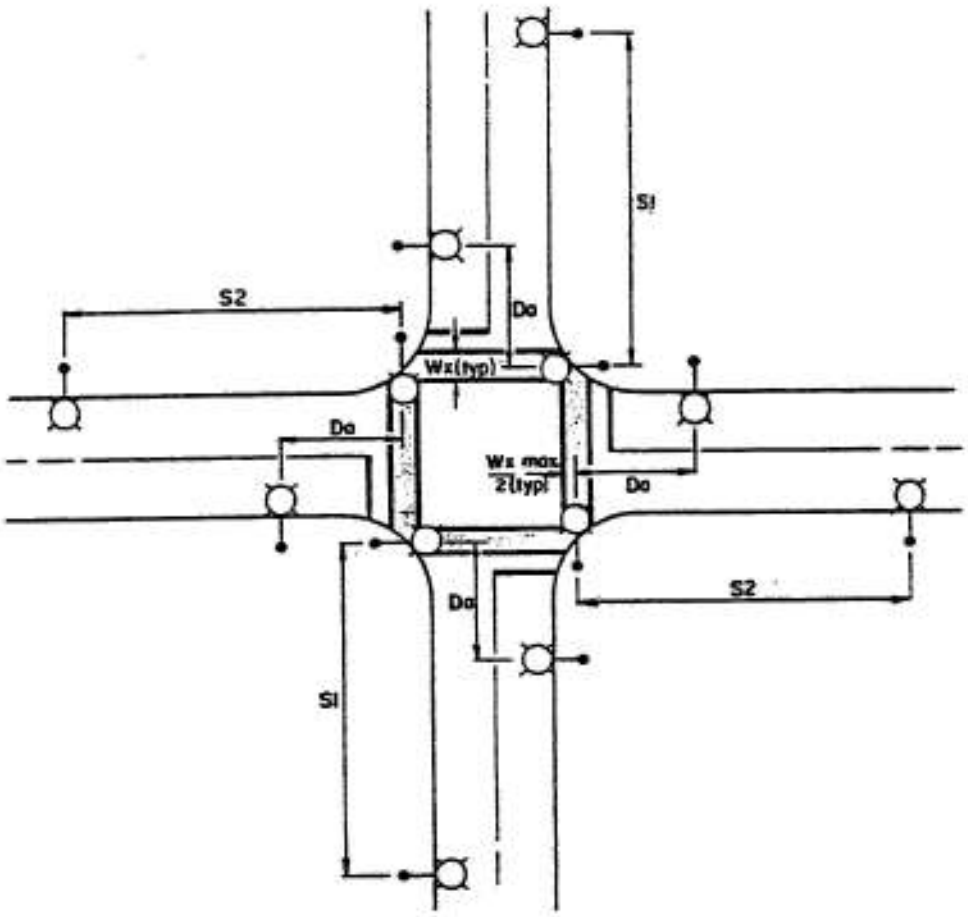
$S = \frac{(n \times \Phi_i) \times Cu \times LLF}{E_{av} \times W} = \frac{(2 \times 27{,}500) \times 0.432 \times 0.69}{17 \times 18} = 53.6$ m = 54 m.

Luminaire	Point A			Point B			Point C			Point M1			Point M2		
Number	LMHR	TMHR	E_{HA}	LMHR	TMHR	E_{HB}	LMHR	TMHR	E_{HC}	LMHR	TMHR	E_H	LMHR	TMHR	E_H
1	1.93	0	6.670	0.96	0.64	11.836	0.96	0	15.060	0	0				
2	1.93	0	6.670	2.89	0.64	3.286	2.89	0	2.040	–	–				
3	0	1.29	10.580	0.96	0.64	11.836	0.96	1.29	5.460	1.93	1.29	2.90			
4	3.86	1.29	0.860	2.89	0.64	3.286	2.89	1.29	2.580	1.93	1.29	2.90			
5	3.86	1.29	0.860	5.79	0.64	nil	4.75	1.29	0.220			6.80			
6				5.79	0.64	nil	4.79	0	nil		x	0.52			
7											=	3.02			
8										#1	(calc)	19.36	#1	(calc)	17.57
Totals			26.040			30.244			25.060			22.38			
Corrected x ICF			13.54			16.72			13.04			22.38			< 22.38

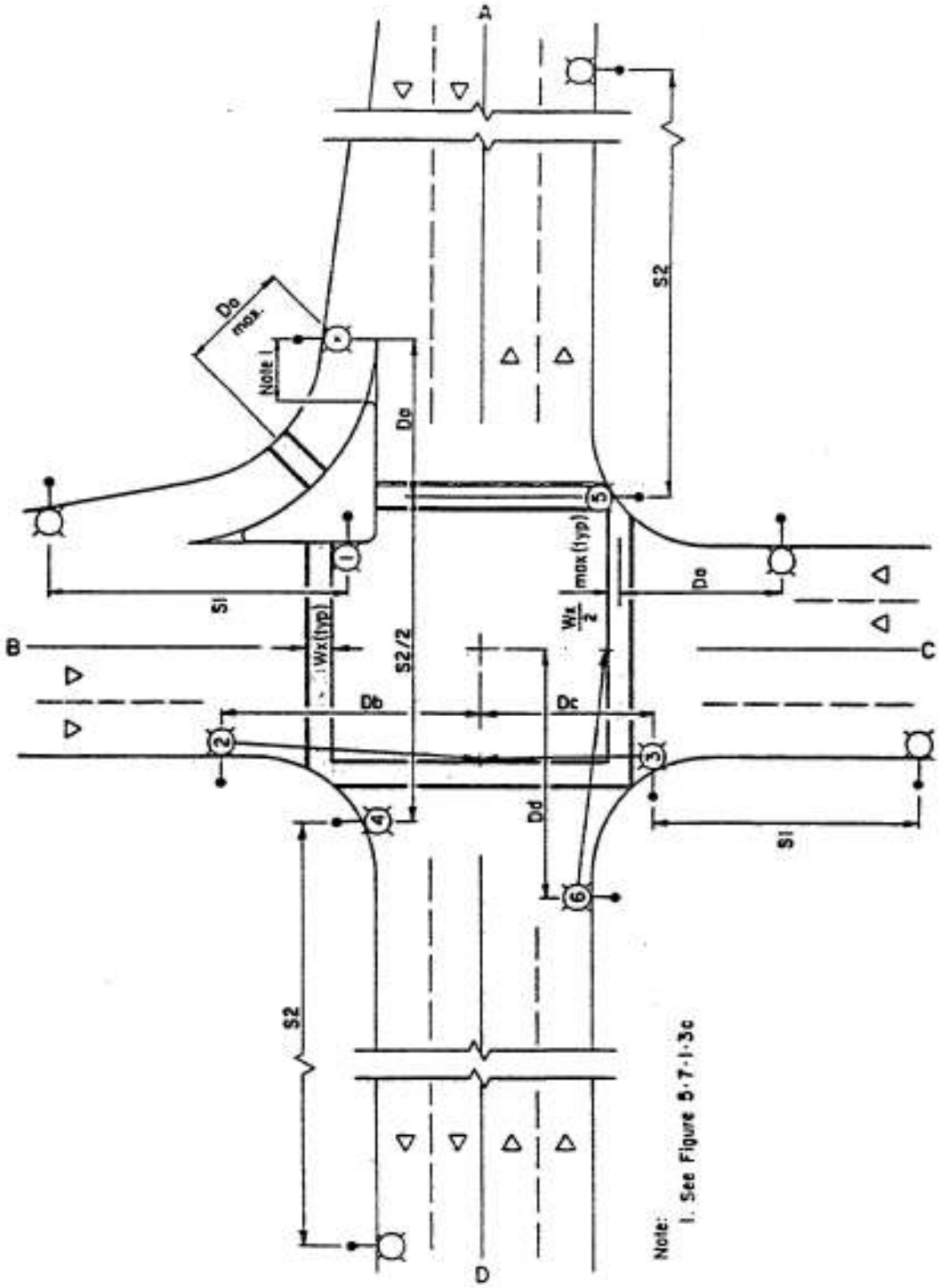
U1 = Average/Minimum = 17/13.04 = 1.30 :1 $E_{av} = \frac{(n \times \Phi_i) \times CU \times LLF}{S \times W}$ = _____ = _____ lux

U2 = Maximum/Minimum = 22.38/13.04 = 1.72 :1 L_v (est) = 0.12 cd/m²

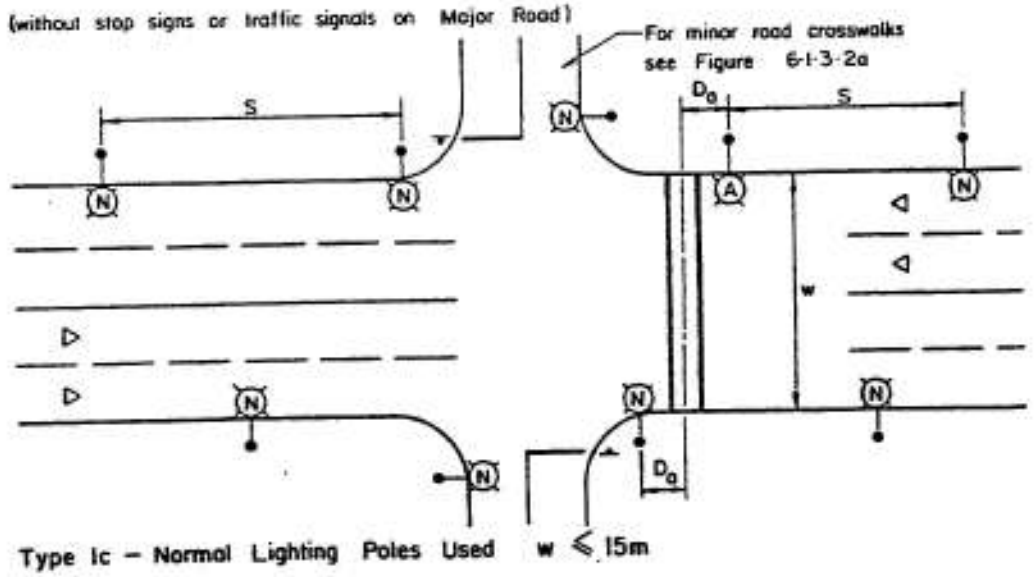
8.0 Appendix "H"
Figure 6.1.3.2b
Recommended layout for normal crosswalks (major roads)



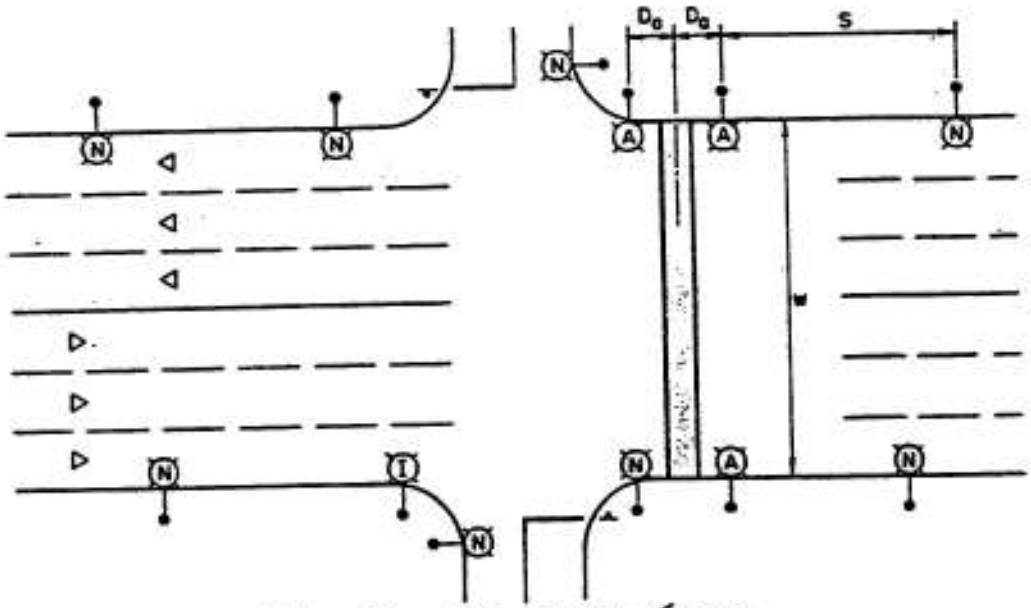
8.0 Appendix "I"
Figure 6.1.3.2c
Recommended layout for normal crosswalks (signalized)



8.0 Appendix "J"
Figure 6.1.3.3a
Type 1 crosswalks at intersections



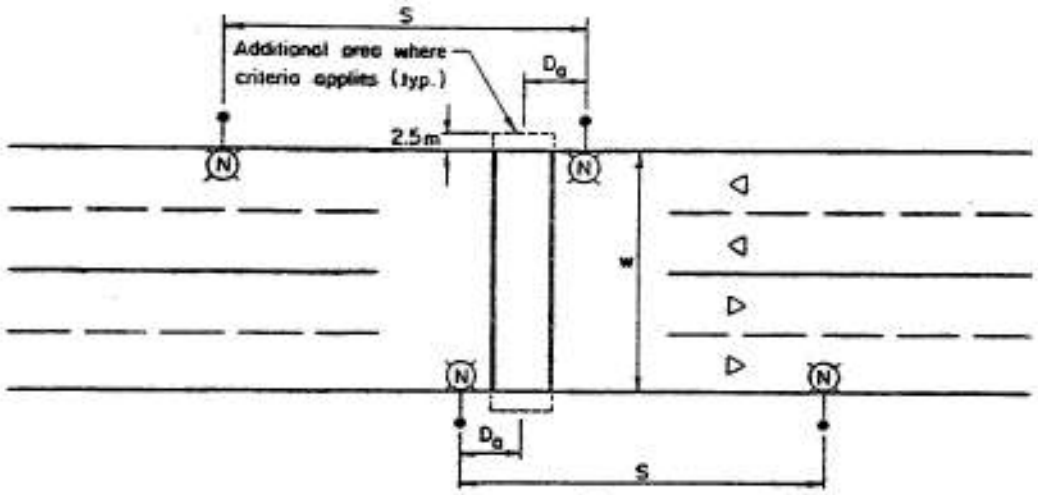
Type Ic – Normal Lighting Poles Used $w \leq 15m$



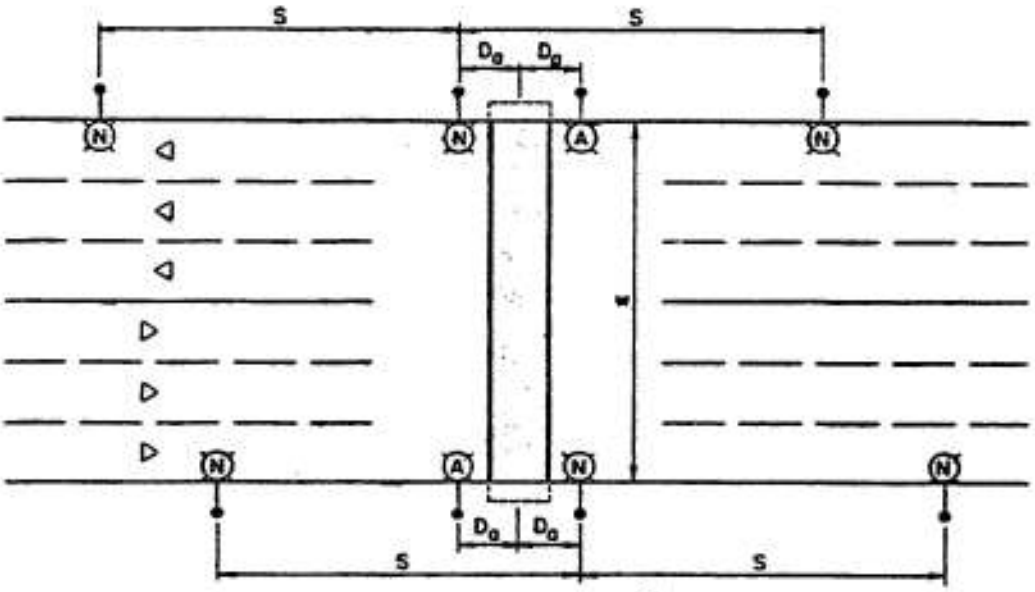
Type Id Normal Lighting Poles Used $15 < w \leq 48m$

Notes:
'N' = normal luminaires required for roadway lighting
'A' = auxiliary luminaires required for crosswalk
'I' = auxiliary luminaires required for additional intersection lighting.

8.0 Appendix "K"
Figure 6.1.3.3b
Type 1 crosswalks mid block



Type 1a - Normal Lighting Poles Used $w \leq 15m$



Type 1b - Normal Lighting Poles Used $15 < w \leq 48m$

Notes:
'N' = normal luminaires required for roadway lighting
'A' = auxiliary luminaires required for crosswalks

5.2 VEHICLE FACILITIES

5.2.7 TRAFFIC INDEX

1. Introduction
2. Traffic Counts
3. Capacity
4. Signs
5. Pavement Markings
6. Traffic Signals
7. Traffic Islands
8. Pavement Speed Control Devices

5.2 VEHICLE FACILITIES

5.2.7 TRAFFIC

1.0 Introduction

For further consideration and details reference should be made to the following publications:

(1) "Manual of Uniform Traffic Control Devises for Canada":

c/o Roads and Transportation Association of Canada, 1765 St. Laurent Blvd., Ottawa, Ontario, K1G 3V4.

(2) "Transportation and Traffic Engineering Handbook": by The Institute of Traffic Engineers, published by Prentice-Hall.

(3) "Manual Of Uniform Traffic Control Devices": c/o Ontario Ministry of Transportation and Communications.

(4) "Traffic Engineering; Theory & Practice" by Louise J. Pignataro, published by Prentice-Hall.

2.0 Traffic Counts

The Design Hour Volume (D.H.V.) is the volume to which a municipality designs any facility. Other terminology that is used in traffic reference is the A.D.T. - Average Daily Traffic and is used with the week or the month specifically and the A.A.D.T. is the Annual Average Daily Traffic and is referenced to the year, namely the year total divided by 365. A volume reference in traffic is to the number of vehicles passing a particular point in a given length of time.

Traffic flow varies with the hour of the day, the day of the week, and the month or season of the year. The actual hours or time period used for the determination of road needs has a significant influence on the extent to which a road is improved. Designing the road system to accommodate the yearly highest hour of traffic flow would be unrealistic. Similarly the design for the lowest hour of travel for the year would also be unrealistic.

In communities that have a summer inflow of tourist traffic it has generally been found that the peak hour of the average summer week day is selected as the design hour volume (D.H.V.). A traffic count survey must be established within the municipality and the peaking hours identified. From statistics obtained it is generally found that for the purpose of planning road improvements a peak hour volume of 10% of the 24 hour volume provides a good estimate of the D.H.V. Procedures for counting have to be established realizing the overall background data that is available. Generally speaking counts on a roadway or intersection are taken and factored against the records of a permanent counting station that gives the variation on the days of the months of the year.

3.0 Capacity

Capacity is the measure of the ability of a facility to carry traffic. It can be said that the practical capacity is the maximum number of vehicles that can pass a given point on a roadway or on a designated lane in one hour without the traffic density being so great as to cause unreasonable delay, hazards or restriction to a driver's freedom to manoeuvre under the prevailing roadway and traffic conditions. A density reference is to the number of vehicles per unit distance or, in other words, vehicles per mile.

3.0 (continued)

Under ideal conditions the capacity of a two lane, two way roadway is about 2000 passenger vehicles per hour, regardless of volume distribution by direction. Ideal conditions include uninterrupted flow, freedom from side interferences, 3.6 m lanes and no lateral obstructions within 1.8 m of the edge of the pavement. In an urban area the ideal conditions are rarely encountered and the actual traffic volumes are much lower. Typically the capacity per lane of an urban arterial road between 450 and 750 vehicles per hour. These lower values are due to inter-section capacities that in urban areas are usually much lower than the sectional street capacity.

Many factors influence the interrupted flow of traffic through at-grade intersections and, therefore, it is difficult to define ideal conditions as in the case of uninterrupted flow. Consequently the criteria used in determining the intersection capacities is developed around typical or average conditions with adjustments to reflect special conditions which may exist. Some factors which effect intersection capacity are the width of the approach, parking, one-way or two-way operation, the location within the urban area, the number of turning vehicles and the amount of green signal time per hour given to each intersection approach.

The roadway capacity can be expressed in terms of Level of Service whose descriptive values are listed below;

Level A - free flow
Level B - stable flow
Level C - stable flow
Level D - approaching unstable flow
Level E - unstable flow
Level F - forced flow

The average travel speed is a good measure of determining the level of service obtained on a particular street.

Level C is generally considered a desirable design value for establishing standards of road improvements on a residential street whose speed limit is 50 km.p.h., Level C would have an average travel speed of 35 km.p.h.

Level C is generally considered a desirable design value for establishing standards of road improvements.

4.0 Signs

The purpose of traffic signing is to assist in the safe and orderly movement of traffic. Signs give information about routes, directions, destinations and points of interest. They are essential to inform drivers about hazards that are not self-evident. The effectiveness of a sign depends mainly on the attention value given to the sign, the legibility, the comprehension value of the message and the appropriateness of the sign message. Improper or excessive use of signs weakens the value of the sign in the eyes of the travelling public.

There are three general classifications of signs based on its function. Regulatory signs give highways users notice of traffic regulations, Warning signs call the attention to conditions in or adjacent to the street that are potentially hazardous to traffic operation and Guide signs show route designations, destinations, directions, distances, and points of interest and other geographical or cultural information.

To help in avoiding misunderstanding a standardization has been adopted. This achieved by basic colours and shapes. As examples: Red is used for STOP indications; Yellow for WARNING; Green gives direction or destination; and White is designated for regulation signs. These signs have also a shape designation and are controlled by legislation as to their size. The erection of signs is also defined by legislation.

5.0 Pavement Markings

Pavement markings should only be used after all facts have been established by survey. Markings are used to designate Stop Lines, Lane Lines, Pedestrian Crosswalks and convey direction or warning without distracting a driver. There are certain limitations to their use. Snow will obliterate the markings, wet pavement distracts from the effectiveness of the marking and wear on heavily travelled streets soon decreases their purpose. Pavement markings can be colour coded with the use of yellow or white paint and the standard colouring of the province should be used to maintain conformity.

6.0 Traffic Signals

Traffic signals should only be used to assign the right-of-way between conflicting streams of vehicular and pedestrian traffic with efficiency. As a criterion signals should ony be installed when their operation would result in a minimum delay or less delay than would occur before their installation. The installation of a traffic control signal at any particular location can be justified and will probably prove beneficial when any one of a vehicular warrant, pedestrian warrant, accident warrant, and a progressive signal warrant is completely satisfied.

Because the principal function of traffic control signals is to permit crossing streams of traffic to share the same intersection by means of time separation, the major criterion for signal control is the volume of traffic entering the intersection. Generally, traffic control signals are not needed if traffic is sufficiently light, that adequate gaps appear at frequent enough intervals to permit all intersection traffic to enter and cross the intersection with little delay. The need for a traffic control signal at any particular location must, however, be carefully evaluated in relation to several warrants (discussed in detail in the "Manual on Uniform Traffic Control Devices").

Warrant 1 - Minimum vehicular volume
Warrant 2 - Interruption of continuous traffic
Warrant 3 - Minimum pedestrian volume
Warrant 4 - School crossings
Warrant 5 - Progressive movement
Warrant 6 - Accident experience
Warrant 7 - Systems
Warrant 8 - Combination of warrants

Warrants should be considered as a guide in the determination of the need for traffic control signals instead of absolute criteria. For example, such factors as physical roadway features, age of pedestrians, effect of adjacent signalized intersections, etc., may modify a decision based on the warrants. Warrants must be used in conjunction with professional judgment based on experience and consideration of all related factors.

7.0 Traffic Islands

Islands can be constructed at intersections where the angle of the intersection or the width of a particular street requires this consideration. Islands provide a storage area for pedestrians crossing a wide street and they also provide a location for hydro pole lines where the span is too great to cross the complete intersection. Islands can also be used for the installation of the signal plant required at the interesection. Although these benefits are obtained from islands it must be understood that installation of an island creates a maintenance problem especially in the winter months when snow and ice has to be cleared following a road plough operation.

8.0 Pavement Speed Control Devices

Rumble strips of different types varying from a coarser pavement surface to grooving of the pavement and, in some instances, the use of a series of small bumps have a long history of application on rural highways predating the implementation of traffic plans in certain urban areas. They effectively act as warning devices on high speed highways at such locations as approaches to stop signs, severe curve conditions and at ramp exits.

In urban areas, and particularly on local residential streets, there may be concern expressed over speeding vehicles which the residents feel should be controlled to a greater degree and in other cases, there is simply a desire to reduce the amount of through traffic using local residential streets by discouraging motorists from driving on them.

Often the residents request the use of some sort of pavement speed control device, similar to those used on highways, but more severe.

Rumble strips on local streets have been experimented with in Toronto, Ottawa and Calgary, as well as other areas.

In both Ottawa and Calgary the strips were removed very shortly after their installation due to complaints by the directly affected residents. The complaints generally resulted from the noise and the vibrations felt in the adjacent homes from vehicles crossing the rumble strips.

The interpretation of the speed data collected from the Calgary test was complicated by the fact that a few motorists had discovered that it was easier to negotiate the bumps at a higher speed.

8.0 (continued)

The technical reasons usually mentioned against the installation of this type of speed control device are as follows:

a) According to section 109 of the Highway Traffic Act, speed limit on any highway under municipal jurisdiction cannot be reduced below 40 km per hour, therefore speed signing on streets which accommodate bumps, and other devices to reduce speed to less than 40 km per hour, can only be of a warning nature which is non-enforceable. (The speed limit on a road in a public park may be reduced to 20 km/hr.).

b) The speed control device, be it a 'speed bump' (vertical) or narrowing of the road by the installation of bumper curbs at certain places so as to alter its alignment (horizontal) can definitely be looked upon as traffic hazards, especially in light of the fact that the legal speed limit cannot be reduced at the same time. Private legislation may exempt a municipality from legal responsibility but certainly not from a degree or moral responsibility.

c) The signing required to warn motorists of the obstacle has to be very extensive to be effective. This was one of the chief complaints of the residents on Balmoral Avenue and Farnham Avenue in the City of Toronto where rumble strips have been used. They felt that the heavy concentration of signs and flashers was very unsightly.

d) In most of the cases when there is a real speeding problem on a local residential street, effective enforcement by the Police Department takes care of the problem. The reduction of traffic might be the aim of some but from the viewpoint of the whole urban community this can be considered a very selfish attitude. Public roads are designed to expedite the movement of vehicles and the flow of traffic should not be restricted at the wish of a small number or residents.

8.0 (continued)

"It was resolved at the 1974 annual meeting of the Ontario Traffic Conference that:

'Traffic control devices should never be used for discriminatory purposes. All municipal streets are public thoroughfares, and should be recongnized as such. Public officials should exert every effort to make such streets as efficient and safe as possible, however, PRACTICES WHICH ATTEMPT TO OBSTRUCT AND DISCOURAGE TRAFFIC FROM ANY STREET ONLY TO DETOUR TRAFFIC TO OTHER STREETS ARE DISCRIMINATORY, UNSAFE AND CONDUCIVE TO ABUSE OF THE PRINCIPLE OF PUBLIC STREETS".

5.3 PEDESTRIAN FACILITIES

5.3.1. SIDEWALKS INDEX

1.0 Introduction

2.0 Location

3.0 Alignment

4.0 Width

5.0 Gradient

6.0 Drainage

7.0 Surfaces and Construction

8.0 Decorative Finishes

9.0 On Site Information Requirements for Sidewalk Design

10.0 Considerations in Sidewalk Design

5.3 PEDESTRIAN FACILITIES

5.3.1 SIDEWALKS

1.0 Introduction

The following publication provides a reference to the considerations which should be given to all pedestrian facilities, so that convenient access to the outdoor environment may be made by all people in our society.

'Barrier Free Site Design'

by The American Society of Landscape Architects Foundation, and The U.S. Department of Housing and Urban Development c/o The Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 stock number 023-000-00291-4 Price $2.30 (1975).

2.0 Location

Reliable quantitative estimates of pedestrian volumes are difficult to obtain and assume. Quantitative estimates of vehicular volumes are obtainable and can be assumed with a much higher degree of accuracy. It is from a combination of these volumes that it is decided whether sidewalks should be constructed on both sides of a street, one side or not at all. There are numerous tables in technical literature setting out exact pedestrian-vehicular volumes warranting sidewalks on one or both sides of a street.

While ideally from a purely safety point of view sidewalks should be required on both sides of every street, for the sake of economy this is not standard practise. Generally speaking it is recommended that sidewalks be provided on both sides of Arterial Streets, Collector Streets, Local Streets with through traffic and generally on one side of Local Streets. They are often recommended only on Minor Local Streets, Culs-de-sac, etc., when they form a well defined route to high actvity centres such as schools, recreational centres, etc.

The location of the sidewalk on the road allowance is also subject to a number of variables generally relating to the expected use of the Street; the width of the road allowance and historical Community Acceptance. In the older neighbourhoods and downtown areas of Cities sidewalks have generally been built immediately adjacent to the curbs. While this type of construction provided good

2.0 (continued)

access for people getting out of their cars and in residential neighbourhoods longer landscaped areas in front of the homes it minimizes the pedestrian safety from vehicular movement on the street.

In Residential Areas and other areas where Zoning By-Laws provide adequate provision for building set back from property lines sidewalks should be constructed in close proximity to the Street Lines. Generally speaking this means 0.5 to 1.0 m from property lines. This location, depending, of course, on the right-of-way width and pavement width, usually provides the following advantages:

(a) A boulevard width between the sidewalk and curb adequately sized to provide a degree of safety between vehicular and pedestrian movement.

(b) Provides space both on the property side of the sidewalk and in the boulevard to enable the location here of some of the underground utilities rather than under the paved road surface with encruing values of ease of construction and maintenance.

(c) Provides boulevard space suitable for landscaping and possible tree planting generally making the street more aesthetically pleasing.

(d) Provides space for snow storage which can greatly reduce snow removal costs in areas so affected.

3.0 Alignment

Sidewalks can be adjusted from their normal alignment paralleling the street line or road curb to avoid healthy mature trees, hydro poles, or other surface topography which it is felt desirable to preserve.

The bends in the sidewalk in these locations should be smooth; long enough to provide for the movement of sidewalk snow clearing equipment within the sidewalk width.

For the same reason it is also desirable that the sidewalk have a 0.5 m lateral set-back for trees, light and sign posts, etc. This clearance also assists pedestrian flow along the walk, and is of particular assistance to blind people.

4.0 Width

Although a 1.0 m walk width has been used for minor pedestrian movement in some areas, it is generally recommended that a sidewalk be at least 1.2 m wide. This width allows two adults to walk side by side or to pass in reasonable comfort.

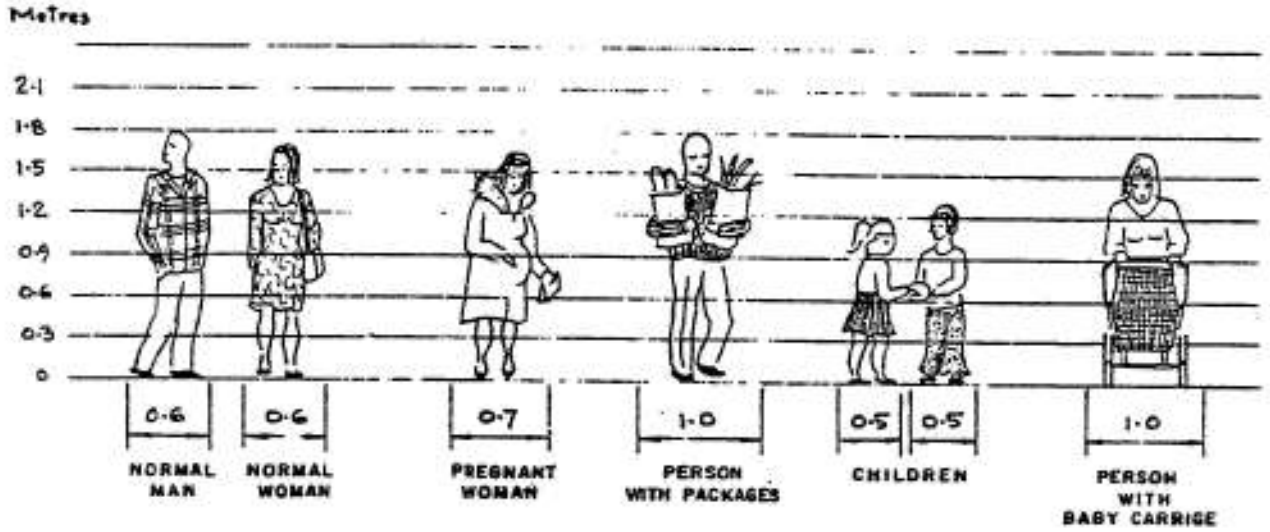
Many areas install a minimum sidewalk width of 1.5 m. This additional width provides for passing room between an adult and a person pushing a baby carriage, a person in a wheelchair, child on tricycle, etc. (refer to following "Dimensions for People Outdoors")

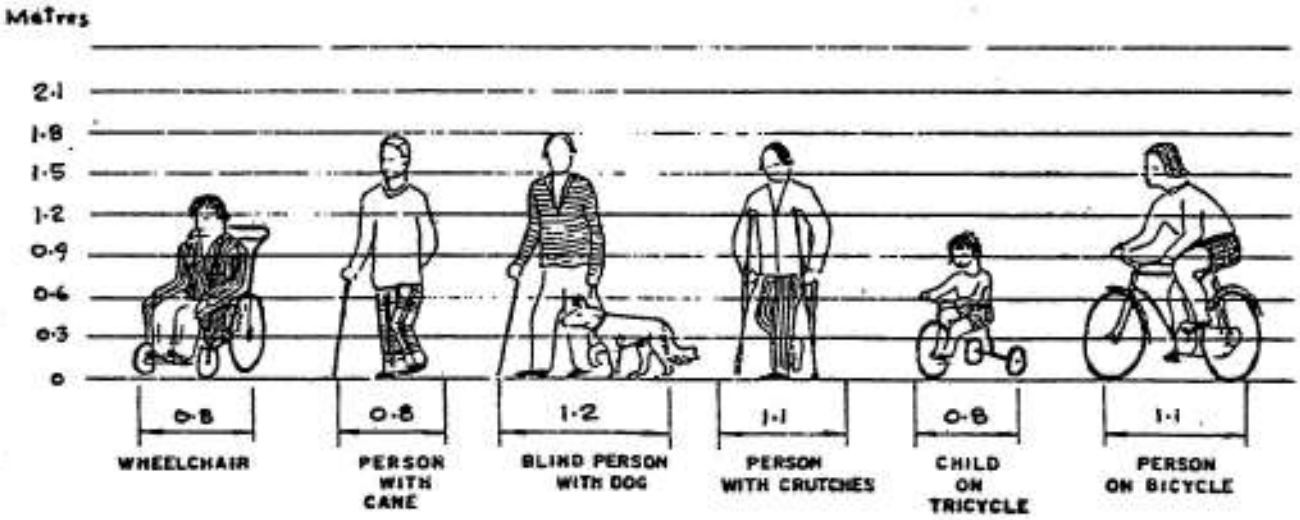
Also in areas where the municipality clears the snow off the sidewalks, the 1.5 m minimum width assists in cutting down damage to adjacent landscaped areas by the snow clearing equipment.

In general, the intensity of land development measured in terms of people per unit area must be quite high to require a sidewalk of higher capacity. Extra width sidewalks should be provided near schools, offices and industrial buildings where large pedestrian volumes occur for short periods of time. Sidewalk widths of 1.8 to 2.4 m can be considered in areas of this nature. Shopping Areas and Entertainment Areas require even wider widths and 2.4 to 3.6 m plus should be considered.

4.0 (continued)

DIMENSIONS FOR PEOPLE OUTDOORS





5.0 Gradient

Generally the sidewalks will follow the road gradient, with the maximum sidewalk grade controlled accordingly.

When the sidewalk gradient is in excess of 5% it should be generally considered as a ramp and may require a coarser surface texture as an aid against slipping during icy weather, as well as other aids such as a handrail.

The maximum recommended gradient is 8%, however, in some locations it is not possible to achieve this due to the existing topography, and a section of grade at 12% or even 15% may be necessary. Under these circumstances, steps should be considered. However, in areas of heavy snowfall the difficulties in ensuring the proper clearing of the steps could result in a greater hazard than a steeper grade. This choice can only be reconciled from area experience.

6.0 Drainage

The recommended cross slope on a sidewalk is 2%. Experience in our northern climate has shown that this fall is desirable to provide reasonable drainage off the sidewalk during freezing weather.

An absolute minimum fall of 1% should only be considered for short sections of walk where particular design conditions may warrant its use. The cross slope may be increased, but it is generally recommended that it be not greater than 4%, due to the increased slipping hazzard. An absolute maximum fall of 6% should only be considered for very short sections (at a driveway) and preferably only on low volume usage sidewalks.

Particular care must be made in the design and installation of a sidewalk to ensure that drainage from the private properties onto the road allowance is not blocked.

Where the drainage from several lots is carried down a swale to the road, it is desirable to intercept this drainage by means of a catchbasin or a culvert. Drainage of this nature, particularly from larger areas, may cause winter icing problems if allowed to drain over the sidewalk.

Normally where the drainage is of a local nature, the sidewalk grade should be adjusted to allow the drainage to pass over it.

6.0 (Continued)

Care should also be taken to ensure an adequate boulevard cross slope, so that the drainage may be carried clear of the sidewalk and over the road curb. A minimum boulevard slope of 2% should be maintained for this purpose.

7.0 Surfaces and Construction

Concrete is the most prevalent material used in municipal sidewalk construction, since it has proven to provide a long life with generally low maintenance, together with a pleasing walking surface.

Asphalt is more generally used as a temporary facility both as a new patch or as a temporary repair of an existing sidewalk.

Interlocking bricks and precast slabs are also used, however, these are generally applied where a commercial area or public facility will gain the benefit of their decorative values, since the initial installation cost is normally higher than the plainer concrete facility.

Attached are several drawings providing typical details for the installation of concrete sidewalks. As noted, the sidewalk is placed directly on a thoroughly compacted subgrade. This situation should be satisfactory for most soil conditions, however, should be reviewed if a wet clay subgrade conditions is prevalent, which may require the placing of some granular base materials.

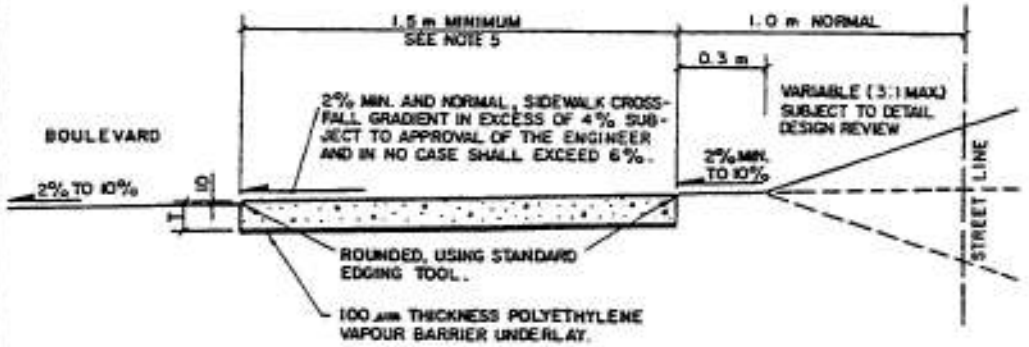
It is recommended that when an asphalt walk is to be constructed, that it consist of a minimum asphalt depth of 50 mm HL3A over a minimum of 100 mm of Granular 'B' material.

Interlocking brick walks are also placed on a granular base and the installation should follow the manufacturer's specifications and recommendations.

Particular reference should be made to the construction of pedestrian ramps between the sidewalk and the road surface on curbed streets. The details shown are typical of the ramps presently being installed in the Metropolitan Toronto Area.

7.0 (continued)

CONCRETE SIDEWALK



THICKNESS TABLE	
LOCATION OR USE	THICKNESS T (mm)
NORMAL SIDEWALK	100
ACROSS RESIDENTIAL DRIVEWAYS, AT CURBSIDE AND OTHER LOCATIONS AS SPECIFIED	150
ACROSS COMMERCIAL AND INDUSTRIAL DRIVEWAYS	200

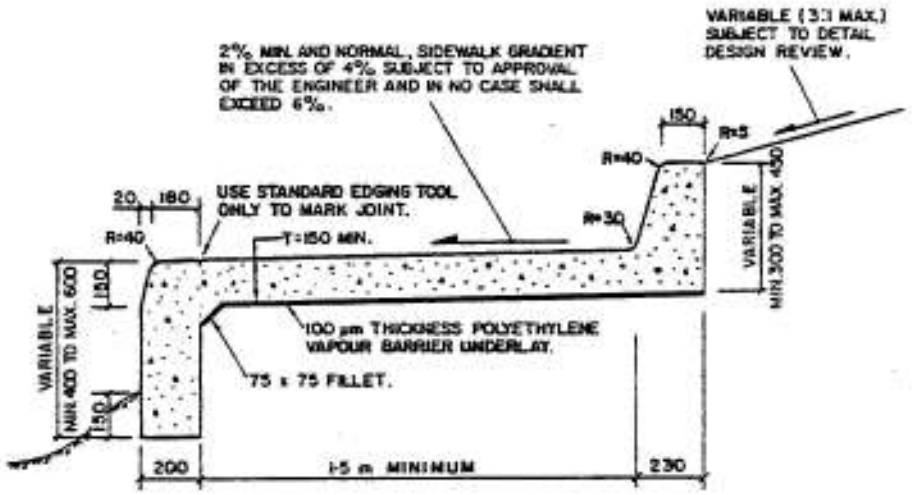
NOTES

1. CONCRETE SHALL BE MIN 25MPa AT 28 DAYS WITH 6% ± 1% PURPOSELY ENTRAINED AIR AND NOT LESS THAN 325 kg/m³ OF CEMENT. MAXIMUM SLUMP SHALL BE 75mm. IN LATE FALL A 30MPa MIX MAY BE SPECIFIED AT THE DISCRETION OF THE ENGINEER. THE DECISION TO REVERT TO A 25MPa MIX WILL BE TAKEN WHEN CLIMATIC CONDITIONS PERMIT.
2. IMMEDIATELY AFTER FINISHING WITH WOODEN FLOAT ALL WALKS, CURBS AND WALLS SHALL BE SPRAYED WITH WHITE PIGMENTED MEMBRANE CURING COMPOUND CONFORMING WITH M.T.C. FORM 1315 AND AS APPROVED BY THE M.T.C.
3. EXPANSION JOINTS SHALL BE LOCATED ADJACENT TO EXISTING STRUCTURES (BUILDINGS, WALKS, HYDRANTS, ETC.) AS DIRECTED BY THE ENGINEER IN THE FIELD. REFER TO STANDARD № 224.
4. FOR LOCATION AND DETAIL OF JOINTS SEE STANDARD № 223.
5. THE SIDEWALK WIDTH IS SUBJECT TO DESIGN REVIEW WITH A MINIMUM WIDTH OF 1.8 m RECOMMENDED WHERE INSTALLED CURB SIDE ON COLLECTOR OR ARTERIAL ROADS AND 2.4 m ADJACENT TO SCHOOLS, BUS STOPS AND OTHER AREAS OF HIGH PEDESTRIAN MOVEMENT.

DIMENSIONS IN mm EXCEPT AS NOTED

7.0 (continued)

CONCRETE SIDEWALK RETAINING CURBS



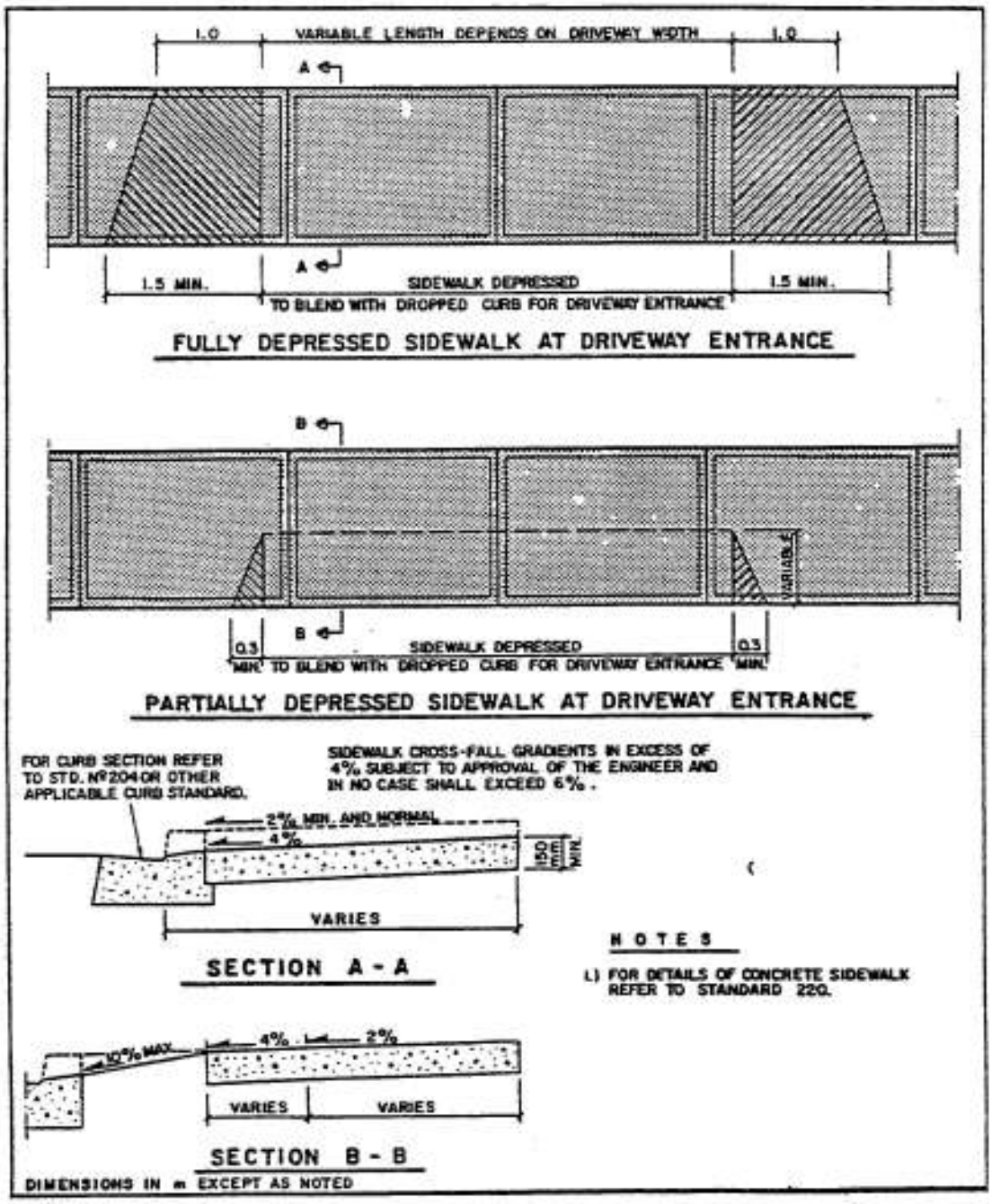
THICKNESS TABLE	
LOCATION OR USE	THICKNESS T (mm)
SIDEWALK WITH RETAINING CURB	150
ACROSS COMERCIAL AND INDUSTRIAL DRIVEWAYS	200

1. CONCRETE SHALL BE MIN. 25MPa AT 28 DAYS WITH 6% ± 1% PURPOSELY ENTRAINED AIR AND NOT LESS THAN 325 kg/m³ OF CEMENT. MAXIMUM SLUMP SHALL BE 75mm. IN LATE FALL A 30MPa MIX MAY BE SPECIFIED AT THE DISCRETION OF THE ENGINEER. THE DECISION TO REVERT TO A 25MPa MIX WILL BE TAKEN WHEN CLIMATIC CONDITIONS PERMIT.
2. IMMEDIATELY AFTER FINISHING WITH WOODEN FLOAT ALL WALKS, CURBS AND WALLS SHALL BE SPRAYED WITH WHITE PIGMENTED MEMBRANE CURING COMPOUND CONFORMING WITH M.T.C. FORM 1315 AND AS APPROVED BY THE M.T.C.
3. EXPANSION JOINTS SHALL BE LOCATED ADJACENT TO EXISTING STRUCTURES (BUILDINGS, WALKS, HYDRANTS, ETC.) AS DIRECTED BY THE ENGINEER IN THE FIELD. REFER TO STANDARD Nº 224.
4. FOR LOCATION AND DETAIL OF JOINTS SEE STANDARD Nº 223.

DIMENSIONS IN mm EXCEPT AS NOTED

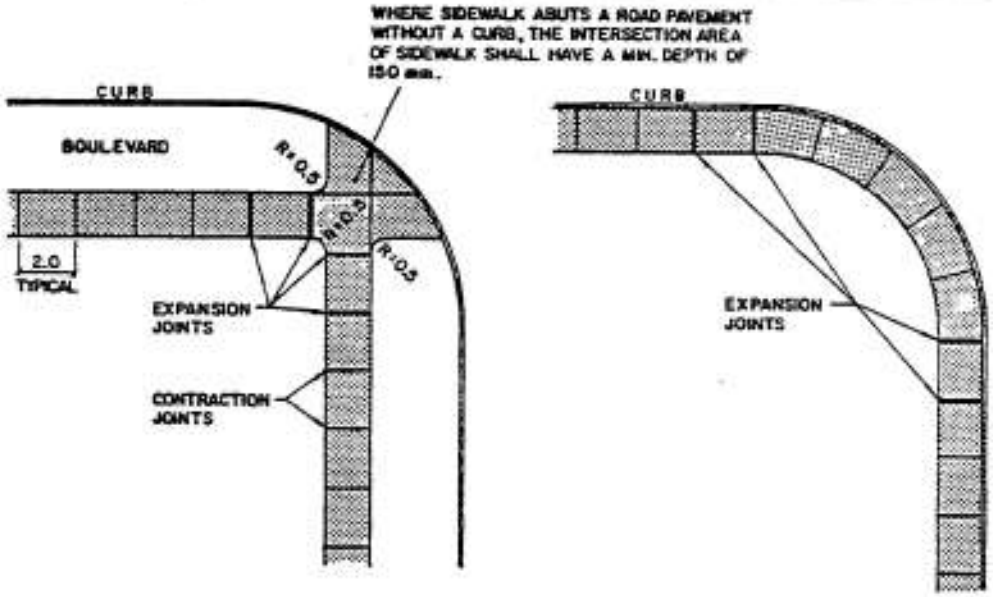
7.0 (continued)

SIDEWALK DRIVEWAY ENTRANCE DETAILS

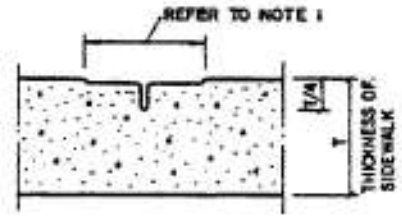


7.0 (continued)

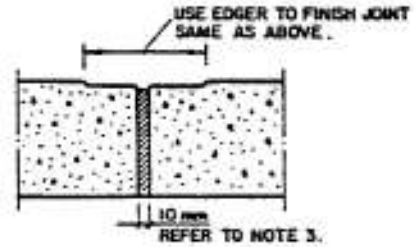
LOCATION AND DETAIL OF JOINTS FOR SIDEWALKS



TYPICAL SIDEWALK INSTALLATIONS



CONTRACTION JOINT



EXPANSION JOINT

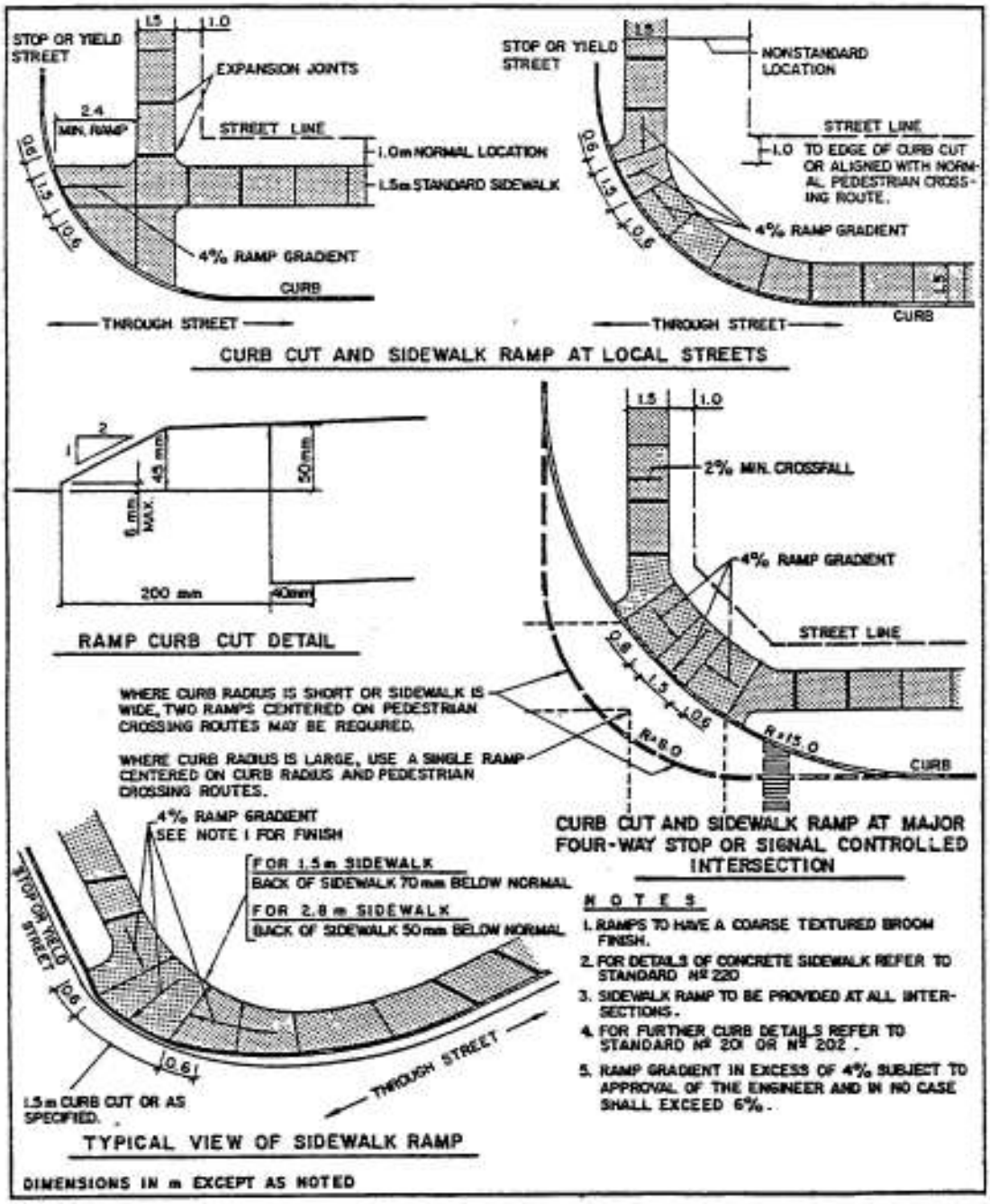
DIMENSIONS IN m EXCEPT AS NOTED

N O T E S :

1. CONTRACTION JOINTS SHALL BE FORMED BY CUTTING A SLOT 1/4 OF THE DEPTH OF THE FRESH CONCRETE. AFTER THE CONCRETE HAS PARTIALLY HARDENED THE JOINT SHALL BE EDGED WITH A DOUBLE EDGER TO MAKE A CLEAN STRAIGHT JOINT OF 25 mm MINIMUM DEPTH.
2. CONTRACTION JOINTS SHALL BE LOCATED AS DETAILED ABOVE. WHERE SIDEWALK WIDTH IS 2.5 m OR GREATER, A CONTRACTION JOINT SHALL BE FORMED ALONG THE CENTER LINE OF THE WALK.
3. EXPANSION JOINT FILLER SHALL BE 10 mm THICK NON-EXTRUDING AND RESILIENT BITUMINOUS OR NON-BITUMINOUS TYPES CONFORMING WITH M.T.C. FORM 1308 AND AS APPROVED BY THE M.T.C.
4. EXPANSION JOINTS SHALL BE LOCATED ADJACENT TO EXISTING STRUCTURES (BUILDINGS, WALKS, HYDRANTS, ECT.) AS DIRECTED BY THE ENGINEER IN THE FIELD. EXPANSION JOINTS SHALL BE LOCATED AT INTERSECTIONS AS DETAILED ABOVE. EXPANSION JOINTS SHALL BE LOCATED WHERE THE PLACING OF CONCRETE MUST BE STOPPED FOR A PERIOD IN EXCESS OF ONE HOUR.
5. REFER TO STANDARD Nº 224 FOR FURTHER DETAILS OF JOINTS AT SIDEWALK OPENINGS.

7.0 (continued)

CURB CUT AND SIDEWALK RAMP AT INTERSECTION



8.0 Decorative Finishes

Before discussing decorative finishes one important point should be stressed and that is any sidewalk surface whether strictly functional or decorative should be done in a neat workmanlike manner to attract usage as attracting usage is really what sidewalk construction is all about.

However, the surface chosen must meet its prime purpose that of conveying pedestrian traffic in a safe manner providing primarily a non skid surface free from tripping hazards.

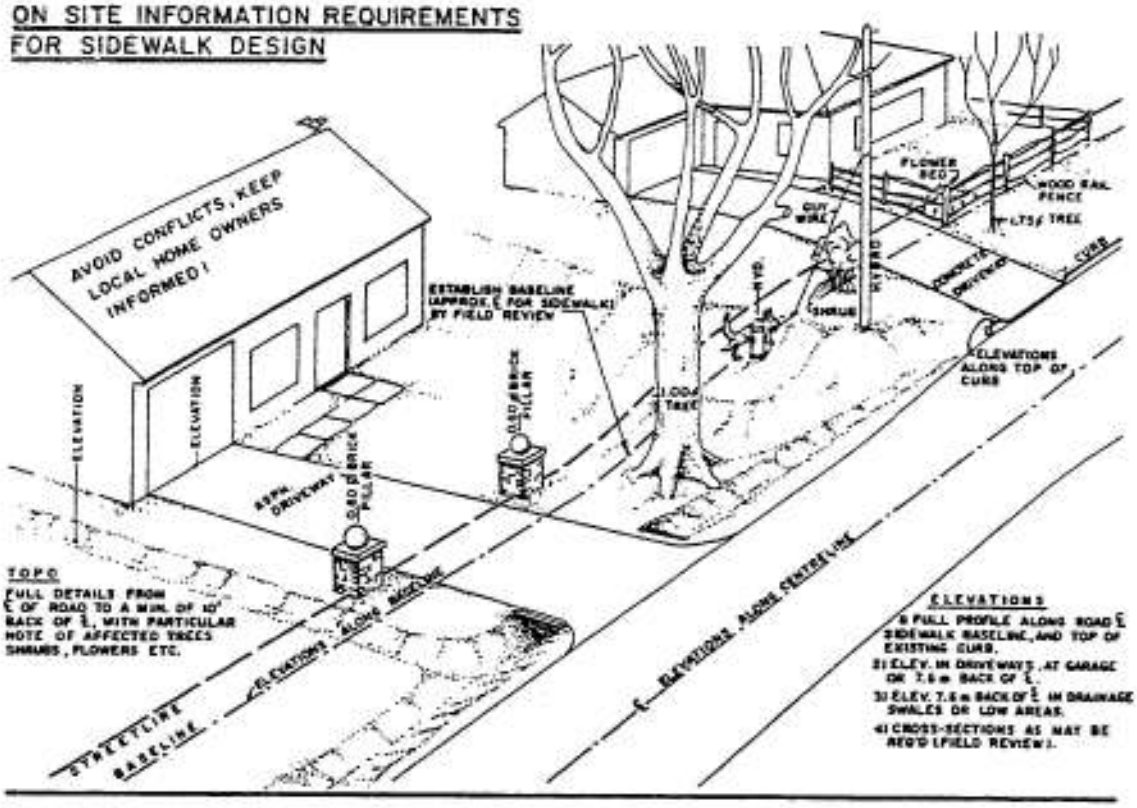
Sidewalk finishes need not be all repetitious to be functional. Textural patterns lend visual interest and can be obtained without sacrificing quality of the sidewalk either structurally or from safe footing. There are various methods for obtaining textural patterns. They are not necessarily expensive to obtain but best results are obviously obtained by using experienced finishes in the various techniques. Por example a pleasing effect can be obtained by simply drawing a brush across the surface in wavy lines alternating directions of the lines between sections of the walk. The manner in which the surface itself is floated or the type of float used can produce attractive variations of texture. Coarse textures may be made by magnesium or wood floats. Very smooth matte finishes can be obtained by using a steel trowel by holding it in a flate manner as with a float and finishing with swirling motions. Patterns can be introduced by using the various floats in a serious of arcs or twists.

Highly smooth surfaces are acquired with the use of a steel trowel. Final trowelling in this instance takes place when the surface of the concrete is quite hard. To reduce the gloss of such a surface it is normally finished with a soft hair brush. This provides a very light textured surface. Heavier textured surfaces are obtained with progressively heavier finishing in progressively softer concrete. A good surface for areas of high usage can be obtained by brushing newly floated concrete with a fibre broom.

Additional time and effort is of course required for integral geometric and artistic patterns. The variety and type of these are endless really limited again only by the designer's imagination. However, repeating once again sidewalks are for usage and attractive finishes attract usage.

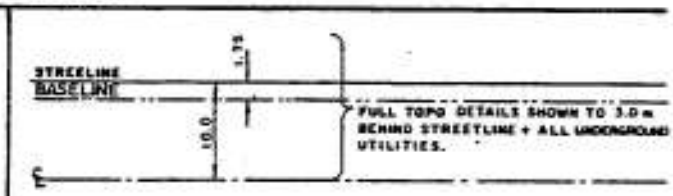
9.0

ON SITE INFORMATION REQUIREMENTS FOR SIDEWALK DESIGN



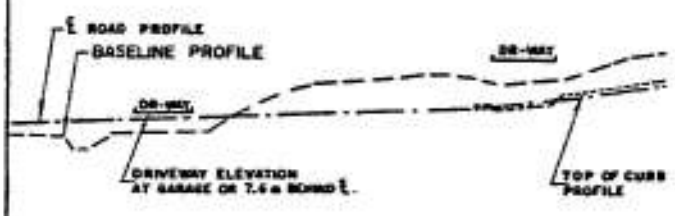
TOPOGRAPHIC INFORMATION IS TAKEN BY AERIAL OR GROUND SURVEYS, AND TRANSFERRED TO A PLAN. IF GROUND SURVEYS USED, USUALLY INFORMATION RESTRICTED TO AREA BETWEEN THE ROAD CENTRELINE TO 3.0 m BEHIND THE STREET LINE. ELEVATIONS ARE TAKEN BY GROUND SURVEYS AND TRANSFERRED TO THE PROFILE, WITH CROSS-SECTIONS TAKEN WHERE REQUIRED.

CROSS-SECTIONS NOT NORMALLY REQUIRED, HOWEVER MAY BE DESIRABLE WHERE STEEP SIDE SLOBES INVOLVED; ALSO TO REVIEW DRIVEWAY GRADIENTS OR DRAINAGE SWALES.



PLAN SCALE 1:200'

PLAN



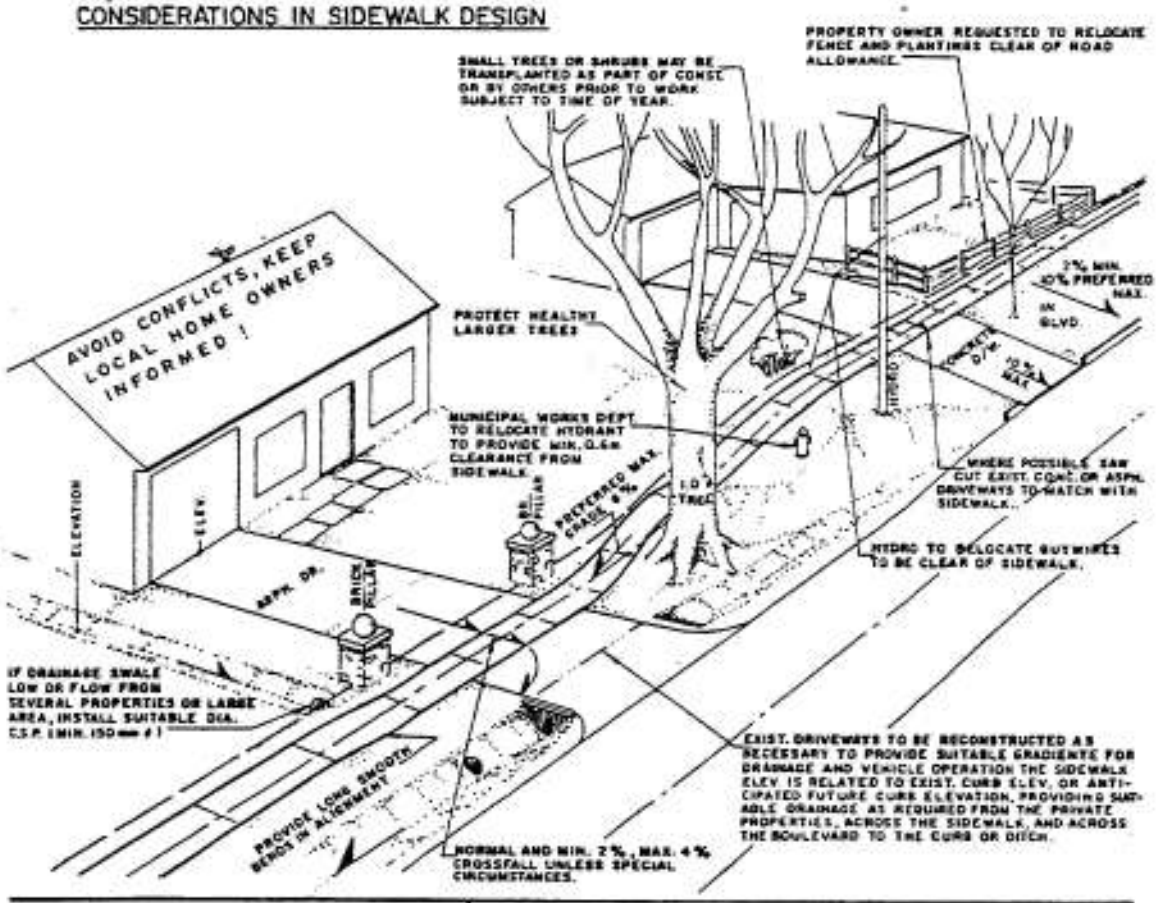
CROSS-SECTIONS

PROFILE

REVISED: 83-05-30

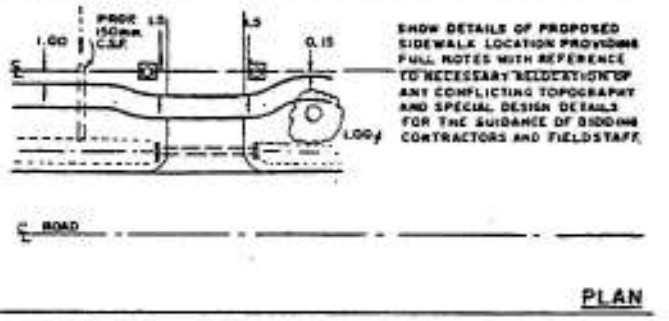
10.0

CONSIDERATIONS IN SIDEWALK DESIGN

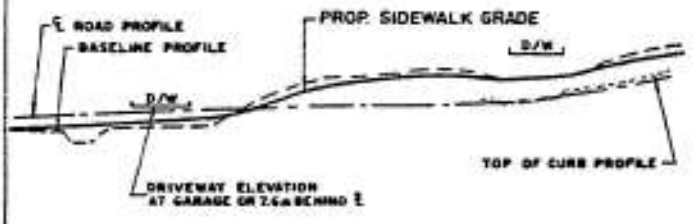




CROSS-SECTIONS



PLAN



PROFILE

REVISED: 1983-06-07

5.3 PEDESTRIAN FACILITIES

5.3.2 WALKWAYS INDEX

1.0 Crosswalks

2.0 Pedestrian Overpass and Underpass

3.0 Walkways between Private Properties

- 3.1 Location and Width
- 3.2 Drainage and Surfacing
- 3.3 Fencing and Lighting
- 3.4 Typical Walkways Details
  - i) Between Streets
  - ii) Park Entrance

4.0 Walkways for Recreation Areas.

5.3 PEDESTRIAN FACILITIES

5.3.2 WALKWAYS

1.0 Crosswalks

Probably the most simple and useful of pedestrian aids is the crosswalk. They are used to delineate and concentrate pedestrian movement within intersections and other needed locations within the road systems. There are no defined or accepted warrants for the use of a crosswalk. However, consideration should be given to their use at the following locations.

(1) Intersection of Major Arterial Streets with other Major Arterial Streets;

(2) Intersections of Major Arterial Streets with Collector Streets;

(3) Intersections of Collector Streets with other Collector Streets;

(4) Intersections or mid-block locations controlled by vehicular or pedestrian traffic signals;

(5) Intersections or mid-block locations adjacent to school and along home-school routes.

The necessity of a crosswalk can be based on:

(1) Pedestrian volumes crossing the street;

(2) Vehicular volumes, speed and turning movements;

(3) Accident frequency at intersections;

(4) Width and shape of intersections;

(5) Use as school crossings.

"Natural Paths" of pedestrian movement is the prime factor to be considered in selecting the proper location for crosswalks. They should be located, when practical, where the crossing is most convenient for the pedestrian. They are in fact most frequently located as extensions of existing sidewalks. Their width should be at least as wide as and preferably wider than those of sidewalks.

1.0 (continued)

Proper marking of crosswalks encourage use by the pedestrian and discourage vehicles from stopping in them. Clearly defining crosswalks can be accomplished by painting, use of plastic and metallic markings or multi-coloured concrete. It is important that such markings be seen and understood during both daylight and night hours.

Supplementary signs and signalization which direct the pedestrian to the proper crossing and the driver's attention to it along with controlling the crossing may also be erected. Reasonableness of such signing must be observed for both pedestrian and vehicles to obey such. It should be remembered that standardization of crosswalks and driver and pedestrian observance and usage of them are not the same from City to City.

2.0 Pedestrian Overpasses and Underpasses

Just as the crosswalk is the most simple and useful of pedestrian aids the pedestrian underpass and overpass is the most complex. However, sometimes the physical separation of pedestrian and vehicular traffic is warranted if indeed not mandatory. Pedestrian overpasses and underpasses are expensive facilities and their usage by pedestrians is not readily accepted due to the pedestrian's innate resistance to walking up or down when crossing roadways. However, where large volumes of pedestrian and vehicular traffic are in direct conflict or where the topography and other physical conditions are favourable pedestrians will find it attractive and expedient to use a well designed overpass or underpass.

Pedestrian overpass and underpass construction is considered in areas where institutions (pedestrian generators) such as schools, factories or sports arenas are in close proximity to arterials, expressways or other similar vehicular generators. With the increased development of major expressways in Municipalities increased development of pedestrian overpasses and underpasses can be expected.

2.0 (continued)

Similar to crosswalks the location of these facilities should if possible be in the most convenient location for pedestrian usage as pedestrians generally like to travel "as the crow flies". As indicated, getting pedestrians to accept usage of overpasses and underpasses is difficult and proper design, lighting and maintenance of them play a major role in obtaining this acceptance.

The use of pedestrian barriers is quite commmon if not almost a necessity in overpass and underpass construction. These are necessitated to insure use of the facility by the pedestrian particularly where there is added distance to walk or a change in grade which almost invariably occurs. A ramp approach to these structures where feasible is usually more satisfactory and acceptable than stairs. The ramp provides easier access and exit for the elderly, handicapped, cyclists and baby carriages, etc. They are also easier to maintain.

The topography of an area usually dictates whether an overpass or underpass is required. Overpasses are generally more advantageous than underpasses in that they can usually be constructed at considerably less expense. They are more readily accepted by the pedestrian in that many people do not like the feeling of being underground as in the case of an underpass nor are the criminal problems associated with underpasses as prevalent with overpasses.

In overpass construction protection for the vehicles passing under must also be considered just as in protection for the pedestrian using the overpass. This usually leads to a minimum of high side screening. Overpasses are exposed to the elements and sometimes complete enclosure is considered, or necessary.

Underpasses while generally being protected from the elements have their own special problems relating to drainage, ventilation and lighting and as mentioned earlier in some instances criminal activity. Tunnel work is expensive and particularly so in the Municipality where conflict between the excavation and the various utility lines on the street is usually unavoidable.

2.0 (continued)

Underpass or overpass construction is complicated and expensive and should only be considered when strongly warranted. It should be proceeded with only after detailed engineering studies considering both the location and type of construction.

3.0 Walkways Between Private Properties

Walkways between private properties fall primarily into two categories (1) aerial walkways between buildings such as one sees in the larger municipalities and (2) walkways between individual lots such as is common in most modern subdivision development. Both types of walkways have pedestrian convenience and safety in common. However, aerial walkways are generally built by private enterprise with the prime purpose of transferring customers from one area to another be it from a parking installation to a store or from one place of business to another. Their problems are most closely associated with overpass and underpass construction be it location, heating, lighting, ventilation or safety. They usually do not come under the jurisdiction of the Municipal Engineer except perhaps through the issuance of the building permit and overall planning. This section will deal exclusively with the second type of walkway that is between individual lots at ground level.

Once the old gridiron patterns of subdivision development was abandoned in favour of the modern day loop and cul-de-sac design the need for walkways from subdivisions joining areas of high pedestrian activity such as schools, parks and shopping centres increasing tresspass or private property occurs with its attendant problems.

Pedestrian walkways between private properties must be designed not only for the convenience of the pedestrian but also for the protection of and to lessen the inconvenience to abutting properties. There are a number of areas of concern in the proper design of walkways between private properties namely location and width, drainage and surfacing; fencing and lighting. Each of these sub-topics will be dealt with individually.

3.1 Location and Width

In new subdivision development locations for walkways can be carefully planned taking into consideration the most beneficial and direct pedestrian routes to the desired locations. The homes adjoining these walkways can be designed and sited to provide the least amount of conflict. In older already developed areas this is much more difficult and considerable care must be exercised in choosing a site location to ensure minimum amount of conflict between the abutting properties and the walkway. There is no standard criteria to apply in siting walkways of this nature and success can only be obtained by diligent evaluation and good public relations. Unopened lane allowances quite often found in older subdivisions are logical locations but still sometimes difficult because abutting property owners have come to regard them as their own. With respect to width a 3.0 m width is generally acceptable. This will allow machine maintenance and yet discourage illegal vehicle usage although sometimes the erection of vehicle barriers are necessary at the entrances to the walkway. A minimum 2.0 m paved surface is recommended where a fence is installed on both sides immediately adjacent to the walkway.

3.2 Drainage and Surfacing

Drainage in walkway construction as in road construction is an important component. A good rule to follow where possible is that walkway drainage should be self contained causing no adverse drainage from the walkway to abutting properties nor blocking the drainage from these properties. Surfacing of walkways is equally as important as drainage and a paved walkway surface similar to those for sidewalks should be provided. Walkway surfaces left in gravel provide not only dust and direct problems but also lend themselves to providing ammunition for stone throwing children with the usual undesirable consequences.

3.3 Fencing and Lighting

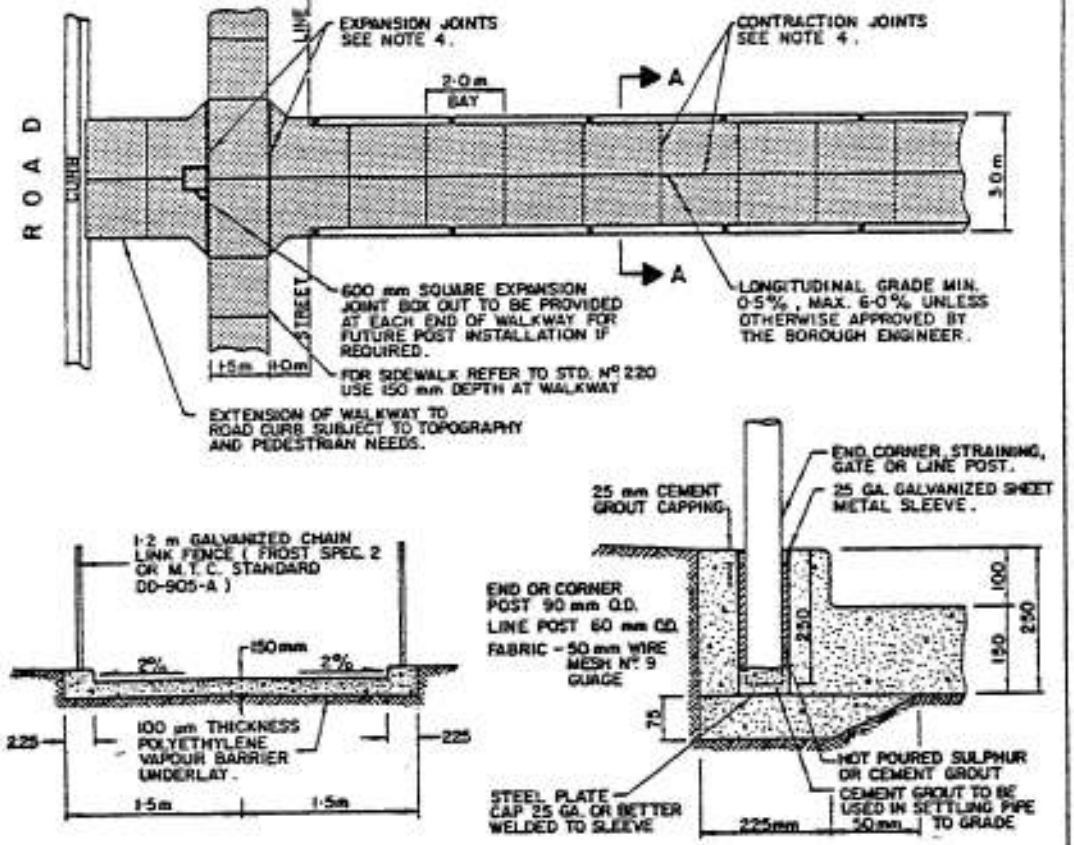
Fencing of footpaths is necessary to prevent tresspass of the users of the walkway unto the abutting properties. There are many varieties of fencing that can be used with 1.2 m chain link fencing probably being the most common. In

3.3 (continued)

choosing fencing a number of criteria should be considered namely (a) cost, (b) aesthetics, (c) length and (d) maintenance and possible vandalism. Wooden fences while generally more aesthetically pleasing than chain link fences are much more expensive from a maintenance and vandalism point of view. They are also generally much more climbable by children with the resulting problems considering both safety and neighbour annoyance. Depending on the length of the walkway sometimes the ordinary street lighting on the abutting streets is sufficient particularly if light standards are located near the entrances. On longer walkways intermediate lighting should be considered. The design of this intermediate lighting should consider both the safety of walkway users and the possible inconvenience to the abutting properties. The lighting should also be considered from an aesthetic point of view as well as functional.

3.4 Typical Walkway Details

(i) Between Streets



NOTES

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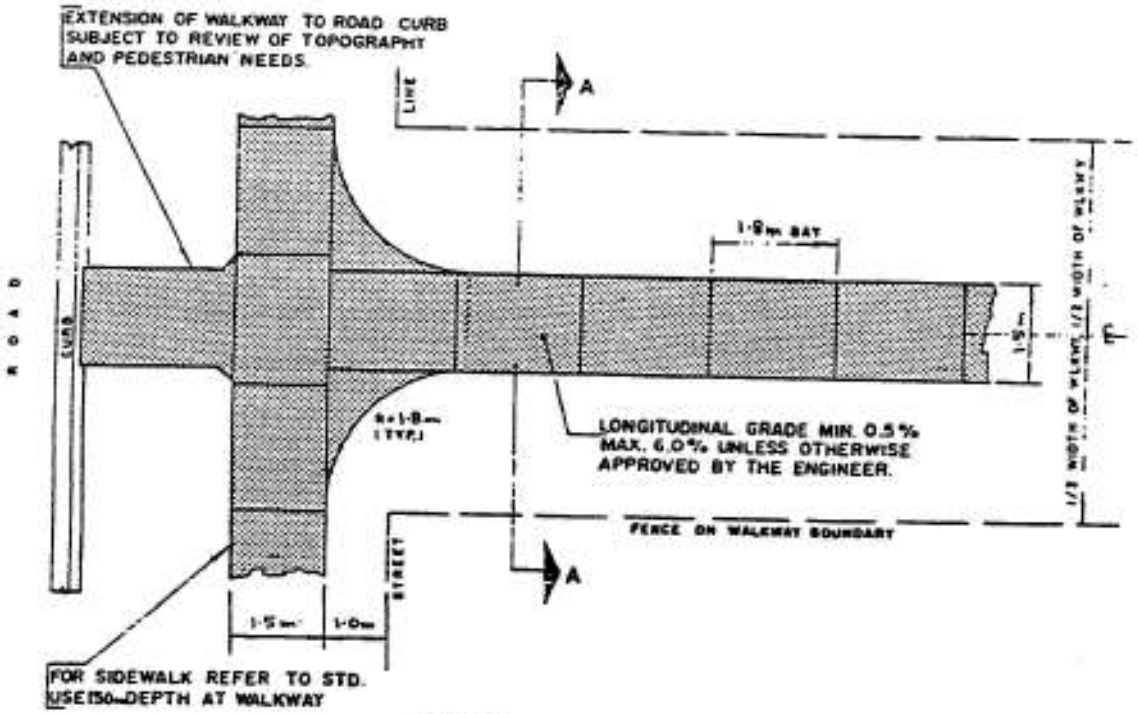
3. EXPANSION JOINTS SHALL BE LOCATED ADJACENT TO EXISTING STRUCTURES (BUILDINGS, WALKS, HYDRANTS, ETC.) AS DIRECTED BY THE ENGINEER IN THE FIELD. REFER TO STANDARD Nº 224.
4. FOR LOCATION AND DETAIL OF JOINTS SEE STANDARD Nº 223.

DIMENSIONS IN mm EXCEPT AS NOTED

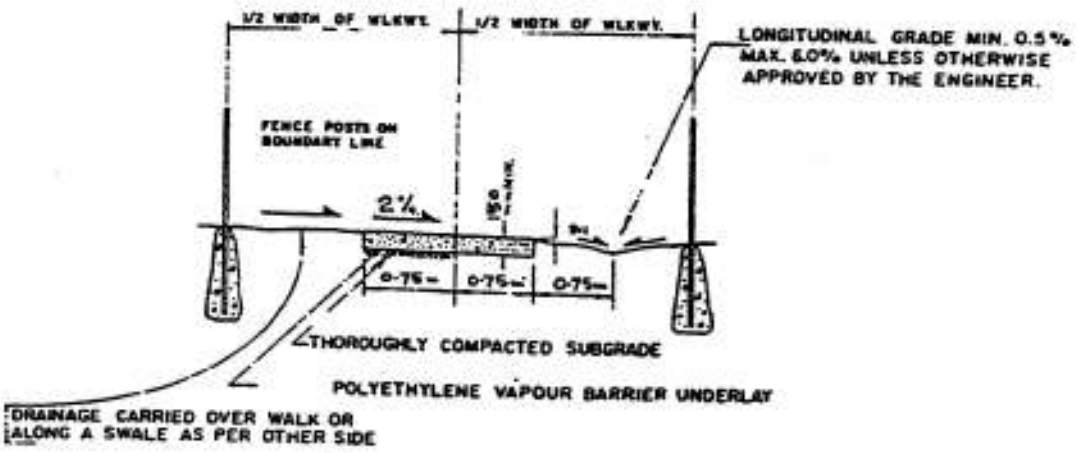
3.4 (continued)

Typical Walkway Details

(ii) Park Entrance



PLAN

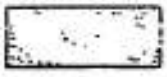
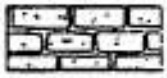
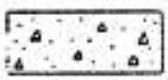
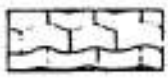


SECTION A - A

4.0 Walkways for Recreation Areas

Many of the comments previously made under sidewalks similarly apply to walks in recreation or park facilities, with the exception that a greater variety of surfaces may be acceptable for certain conditions.

Particular care must be taken in the location of the walkways. The existing trees, shrubs and ground covers should be preserved as much as possible while at the same time providing the needed pedestrian facilities through the park area.

SURFACES FOR WALKWAYS		COMMENTS
		Soft Surface Characteristics
SOFT: Crushed Rock		■ Irregular and soft surfaces make walking extremely difficult for people with mobility handicaps; ■ Poor surfaces for wheelchairs and other small-wheeled vehicles; ■ The blind have difficulty with orientation; ■ Surfaces are susceptible to erosion; ■ Surfaces will withstand only light traffic; ■ Surfaces are useful for areas where light pedestrian traffic will need a moderately firm surface, i.e. recreation areas, parks, nature areas, etc. ■ High maintenance requirements, low installation costs.
SOFT: Earth		
SOFT: Lawn-grass		
SOFT: River rock		
SOFT: Soil cement		
SOFT: Tanbark		
		Variable Surface Characteristics
VARIABLE: Cobble stones		■ Irregular surfaces and wide joints make walking extremely difficult for people with mobility handicaps; ■ Joints easily trap crutch and cane tips, heels, narrow wheels; joints should be filled and no wider than 15 mm, ■ Irregular surfaces make movement difficult for wheelchairs and other small-wheeled vehicles; ■ Ice and snow can be a problem by damaging the surface or being difficult to remove; ■ Moderate maintenance requirements, moderate to high installation costs.
VARIABLE: Exposed aggregate		
VARIABLE: Flagstones		
VARIABLE: Sand-laid brick		
VARIABLE: Wood Deck		
VARIABLE: Wood disks in sand		
		Hard Surface Characteristics
HARD: Asphalt		■ Firm and regular surfaces for walking and moving wheeled vehicles; ■ Joints are kept to a minimum, less than 15 mm wide and filled; ■ Ice and snow removal possible without extensive damage to surfaces; ■ High installation costs, lowest maintenance costs.
HARD: Concrete		
HARD: Interlocking brick or brick on concrete		

5.3 PEDESTRIAN FACILITIES

5.3.3 RAMPS, STAIRS AND HANDRAILS INDEX

1.0 Outdoor Ramps

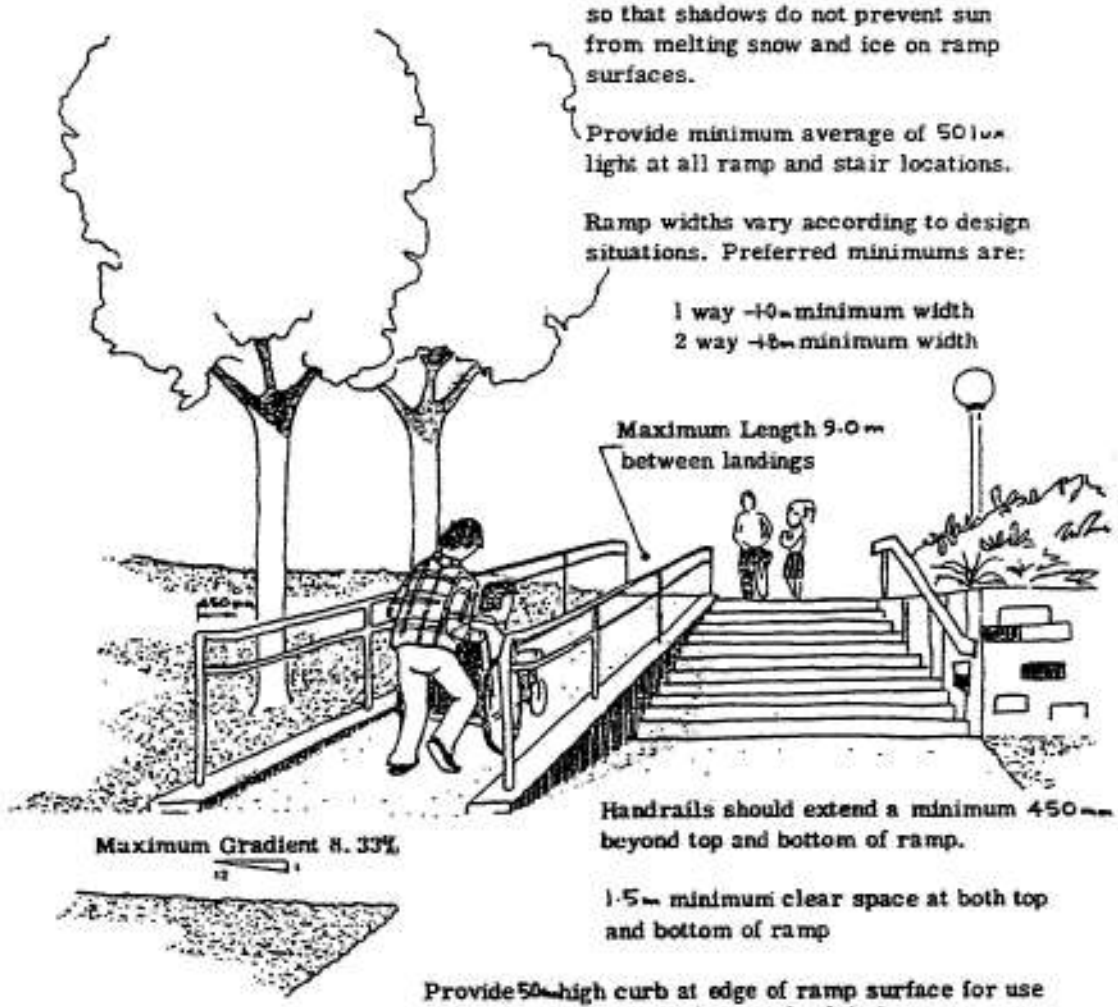
2.0 Outdoor Stairs

3.0 Handrails

5.3 PEDESTRIAN FACILITIES

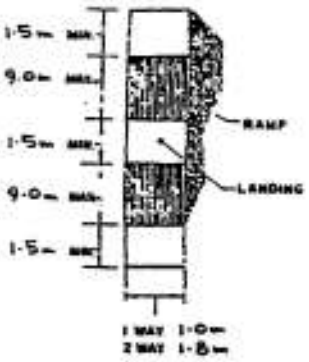
5.3.3 RAMPS, STAIRS AND HANDRAILS

1.0 Outdoor Ramps

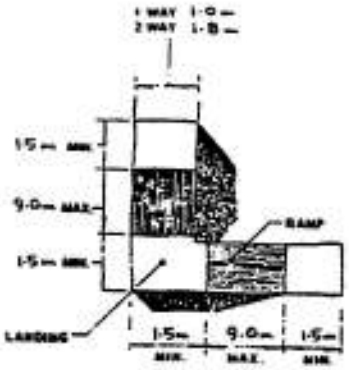


1.0 (continued)

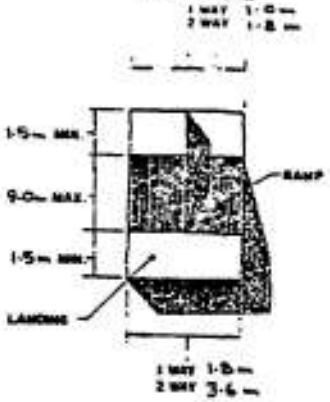
1. STRAIGHT-RUN



2. ANGLED LANDING

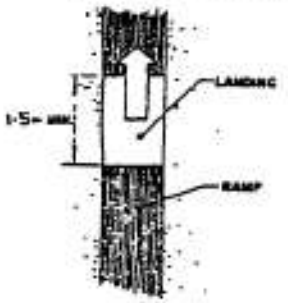


3. INTERMEDIATE/ SWITCH-BACK LANDING

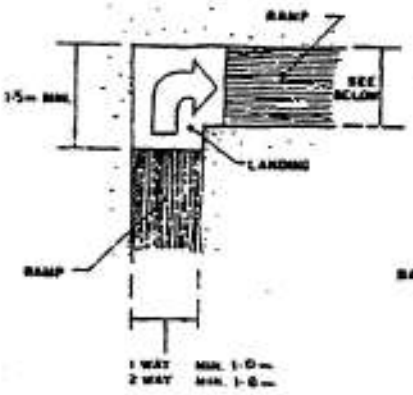


CONDITIONS AT TOPS & BASES OF RAMPS

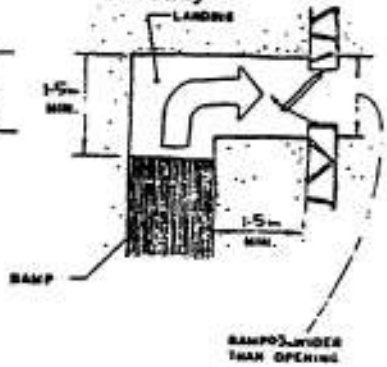
1. Traffic Goes Straight



2. Traffic Turns



3. Traffic Turns to Gate/ Doorway



2.0 Outdoor Stairways

SHADOWS FROM ADJACENT PLANTING SHOULD NOT PREVENT THE SUN FROM MELTING ICE AND SNOW.

PROVIDE A MINIMUM AVERAGE OF 50 LUX LIGHT AT ALL STAIRWAY LOCATIONS.

SURFACE OF ALL TREADS SHOULD BE NON-SLIP AND PITCHED FORWARD AT 1 % TO DRAIN SURFACE WATER. PROVIDE CHAMFER OR 25 ROUNDING TO NOSING OF ALL TREADS.

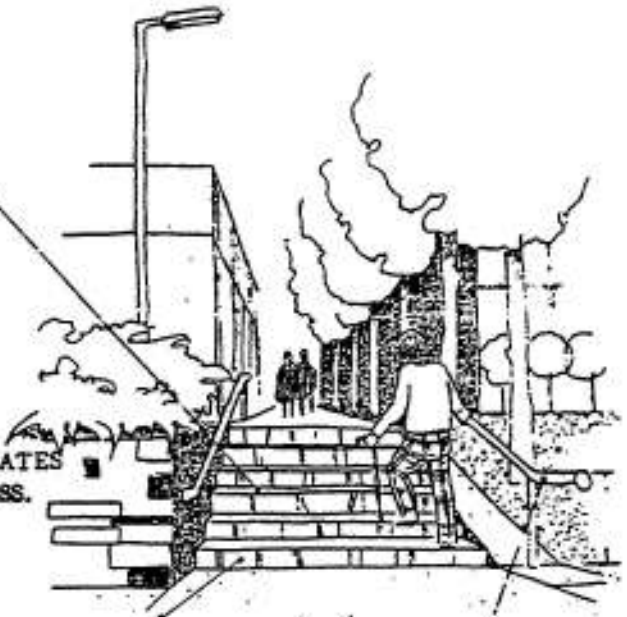
COLOR OF STAIRS SHOULD CONTRAST WITH ADJACENT PAVING.

CHEEKWALLS AT SAME GRADE LEVEL AS ADJACENT LAWN ELIMINATES NEED FOR HAND TRIMMING OF GRASS.

SEE HANDRAILS FOR ADDITIONAL INFORMATION.

STAIRWAYS WIDTHS SHOULD BE DETERMINED BY THE PROJECTED AMOUNT OF PEDESTRIAN TRAFFIC AND THE WIDTHS OF APPROACHING WALKWAYS. PREFERRED MINIMUMS ARE:

1 WAY 10 MINIMUM WIDTH
2 WAY 15 MINIMUM WIDTH

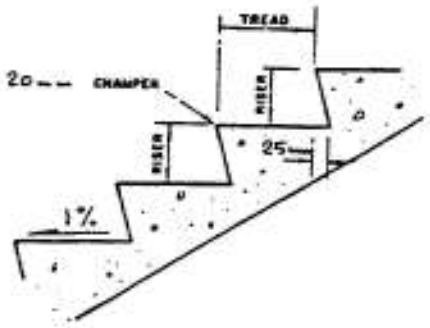


HANDRAILS SHOULD EXTEND BEYOND THE TOP AND BOTTOM STEP A MINIMUM OF 450 --.

CHEEKWALLS SHOULD EXTEND BENEATH HANDRAILS AN EQUAL DISTANCE.

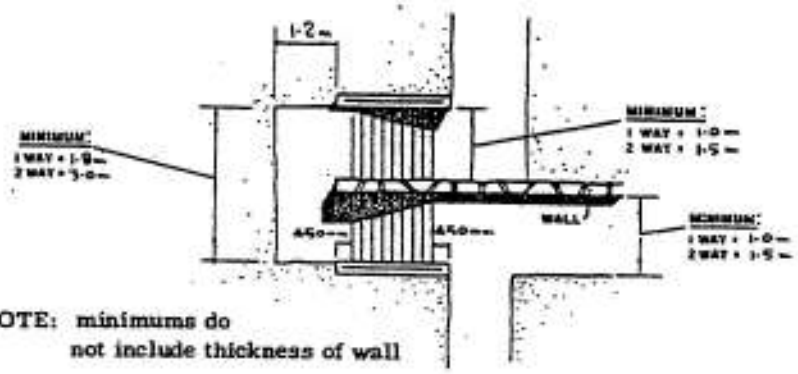
2.0 (continued)

OUTDOOR STEPS RULES-OF-THUMB



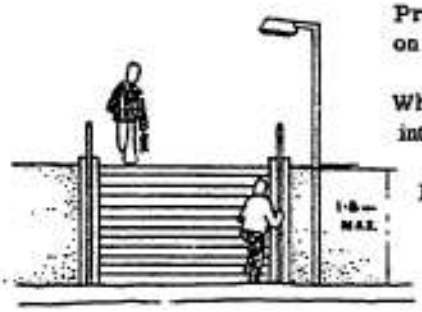
Acceptable Combinations in mm	
Risers	Treads
178	280 290
165	280, 290, 305, 320
152	280, 290, 305, 320, 330, 340, 355
140	280, 290, 205, 320, 330
127	280, 290, 305

OUTDOOR LANDINGS



*NOTE: minimums do not include thickness of wall

HEIGHT BETWEEN LANDINGS



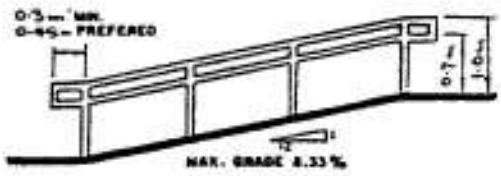
Provide 50 lux lighting on stair and landing areas.

Where total grade change exceeds 1-8 — intermediate landings are necessary.

Provide landing at 1-2 — intervals.

3.0 Handrails

HANDRAILS FOR RAMPS

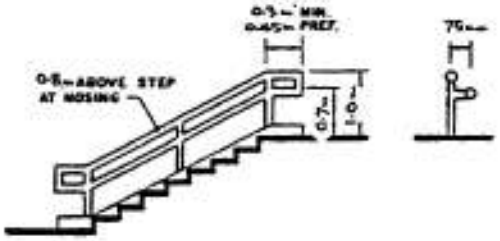


1·0m is the most comfortable height for handrails on ramps.

A second handrail, useful to people in wheelchairs and children should be placed at 0·7m.

Handrails should extend a minimum 0·3m beyond both ends of a ramp.

HANDRAILS FOR STAIRWAYS

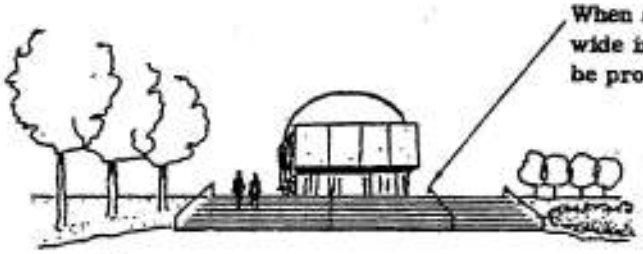


1·0m is the most comfortable height for railings at both ends of stairways. 0·8m is the accepted height on stairways.

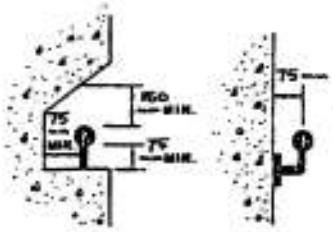
A second handrail, useful to children should be placed at 0·7m.

Handrails should extend a minimum of 0·3m beyond stairways.

HANDRAILS FOR EXTRA-WIDE STAIRWAYS



When steps become exceptionally wide intermediate handrails should be provided at 6.0 to 9.0m intervals.



All edges should be chamfered or rounded off, also keep wall surfaces smooth to limit cuts and scrapes.

HANDRAIL CROSS-SECTIONS

Do not allow hands to use natural opposing grip; usually because rail is too wide.



Allow hands to use natural, opposing grip.

5.4 PUBLIC TRANSIT FACILITIES INDEX

1.0 Buses

2.0 Bus Bays

3.0 Bus Loops

4.0 Bus Lanes

5.0 Bus Passenger Shelters

6.0 Subways and Commuter Trains

5.4 PUBLIC TRANSIT FACILITIES

1.0 Buses

The most common form of public transit facility which the urban engineer will become involved with is the bus.

Bus routes will be of prime concern both in the selection of of the pavement design and the road geometrics. The looping of a bus route on local residential streets will result in premature pavement failure and higher maintenance costs unless the original pavement design is upgraded to suit. Also road bends and corner radii may require increasing to avoid vehicle conflicts.

2.0 Bus Bays

On major collector and arterial streets it may be advisable to provide bus bays, particularly at locations where a bus makes frequent stops, or is normally delayed by the volume of passenger movement on and off the bus. Bus bays will assist the through traffic movement and reduce the accident prone weaving movement on four lane roads.

The location of the bus bay must be reviewed with the transit authority to ensure that it suits their operations. It will also require a careful site review to ensure that conflicts with driveways or other topographic features are minimized.

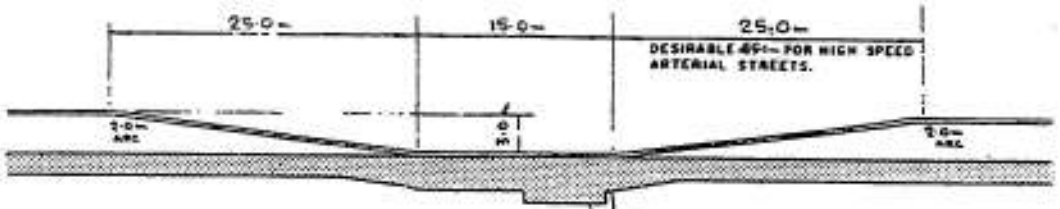
A near side bus bay, particularly where the bus usage is infrequent and of short duration, may be used to advantage as a right turn lane.

There are other minor advantages in the use of bus bays in that they may be used for deliveries to adjacent stores in the time period when transit vehicles are not in operation. They may also be used as pick up and drop off areas for taxis and cars.

Care in the design must be taken allowing long enough entrances and exists to discourage serious disruption of traffic movement in the adjacent lanes. Design characteristics of the bus bay are shown on the following page.

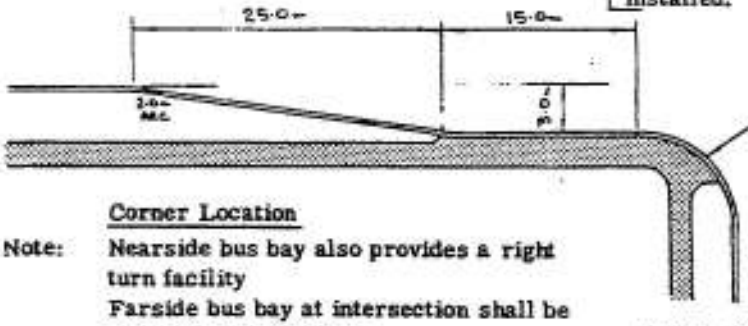
2.0 (continued)

BUS BAYS



Remote from Intersection.

It is recommended to widen the sidewalk to 24" at the bus stop. A further widening should be provided if a bus shelter will be installed.

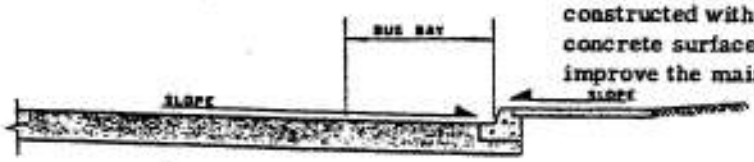


radius as per design plans (30" min.)

Corner Location

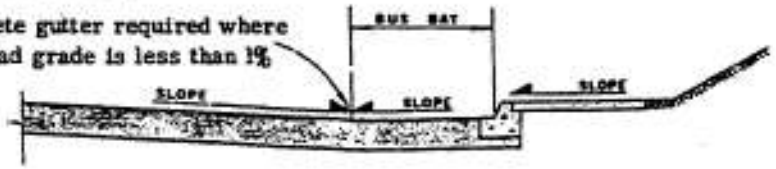
Note: Nearside bus bay also provides a right turn facility
Farside bus bay at intersection shall be opposite hand to above

Note: On concrete base pavements the bus bays may be constructed with an exposed concrete surface, which may improve the maintenance of bus stop area.



Recommended Section

concrete gutter required where the road grade is less than 1%



Alternate Section

used where existing topography cannot be altered practically.

3.0 Bus Loops

The Bus Loop in its simplest definition is a form of construction for the effcient and economical handling of passengers and buses at transfer and end of route locations. The construction and operation of Bus Loops is of prime importance and most beneficial at high use transfer points. There are a number of advantages to the use of Bus Loops besides the increased efficiency to the transit operation that can result. With their use passengers are able to complete their transfers in comparative safety not being subjected to the hazards associated with adjacent street traffic nor interfering with this traffic. Bus Loops free inportant curb space and operational lanes thereby reducing congestion which normally would occur with the buses standing at the curb to load and discharge passengers and maintain schedules.

The most appropriate location for the Bus Loop is dependent upon a detailed analysis of headways during peak hour operation on all the routes proposed to be serviced by the Loop. Normally the Loop is located on the route with the shortest headway.

In designing Bus Loops the most directly related consideration is the operating characteristics of the transit vehicles using the facility. The Loop exit drive should always be located on the shortest headway route. It should also be located as far as possible from an adjacent signalized intersection. Schematic examples of a Single and Double loop are shown on the following page.

The simplest form is the Single Loop with a width in the 4.5 to 6.0 m range. The minimum length of the Loop must be related to the turning radii of the transit vehicles being used and more importantly to the combined lengths of the maximum number of transit vehicles that will be using the Loop at any one time. To take full advantage of the additional length required because of designing for maximum usage the loading platform must be located at the exit end of the loop. In single loop design where required storage conflicts with scheduled operations due to multiplicity of route transfers it may be designed with an additional inner lane used for passing. Under this arrangement the loading platform must be designed to accommodate the number of transit vehicles required to use it at any one time. This length must also include maneuver space.

There are a number of variations of the double loop usually involving a centre loading platform. The physical design again giving consideration to the basics such as characteristics of user vehicles, storage requirements and maneuverability. As shown in the schematic drawing given certain conditions the concentric Loops can operate in a counterclockwise fashion on each Loop adjacent to the central loading platform.

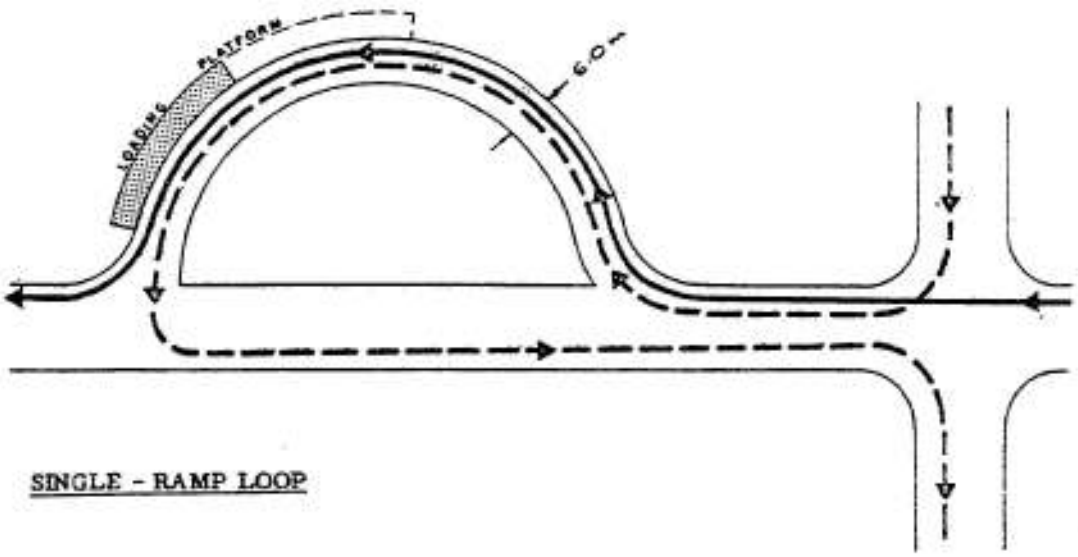
3.0 (continued)

Loops are initially expensive due to property and construction costs and as in all engineering endeavours their use should be economically justified. The location of the Loop must avoid conflicts with the area residence, and to reduce the property requirements. The Loops are often designed to minimum standards.

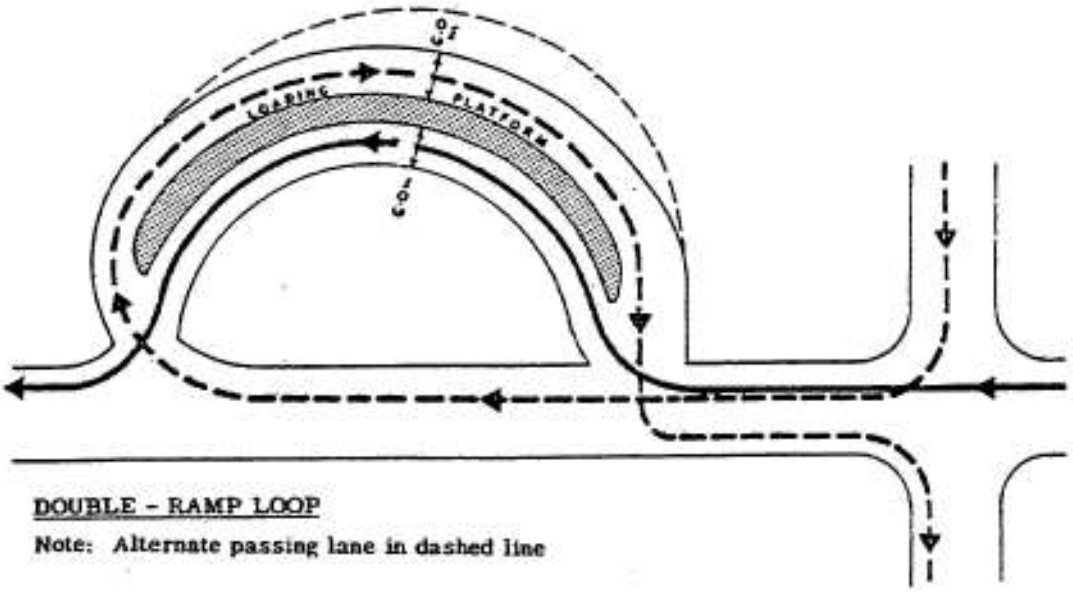
Examples outlining typical minimum requirements for a 12.2 m bus are provided.

3.0 (continued)

BUS LOOPS



SINGLE - RAMP LOOP



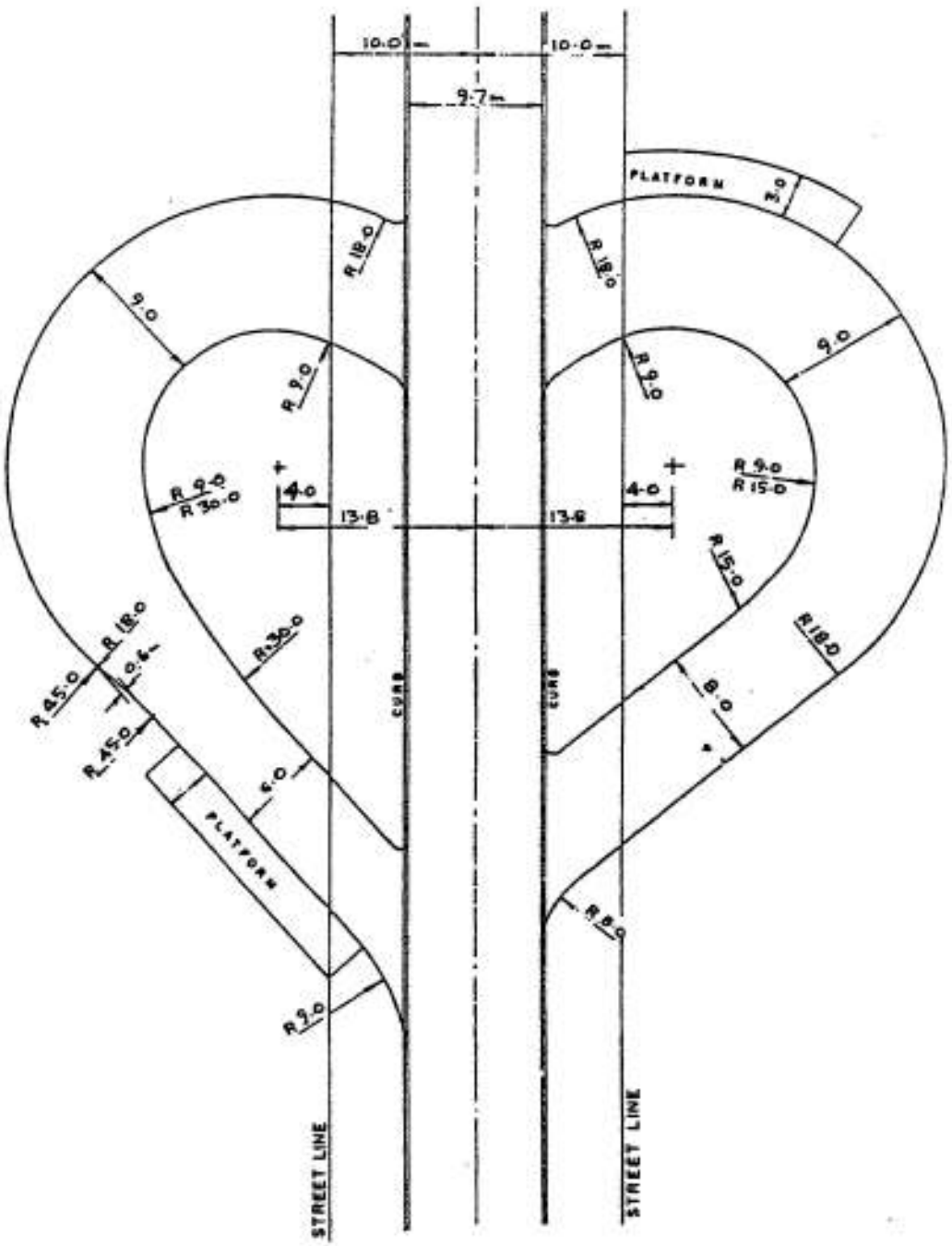
DOUBLE - RAMP LOOP

Note: Alternate passing lane in dashed line

3.0 (continued)

BUS LOOPS

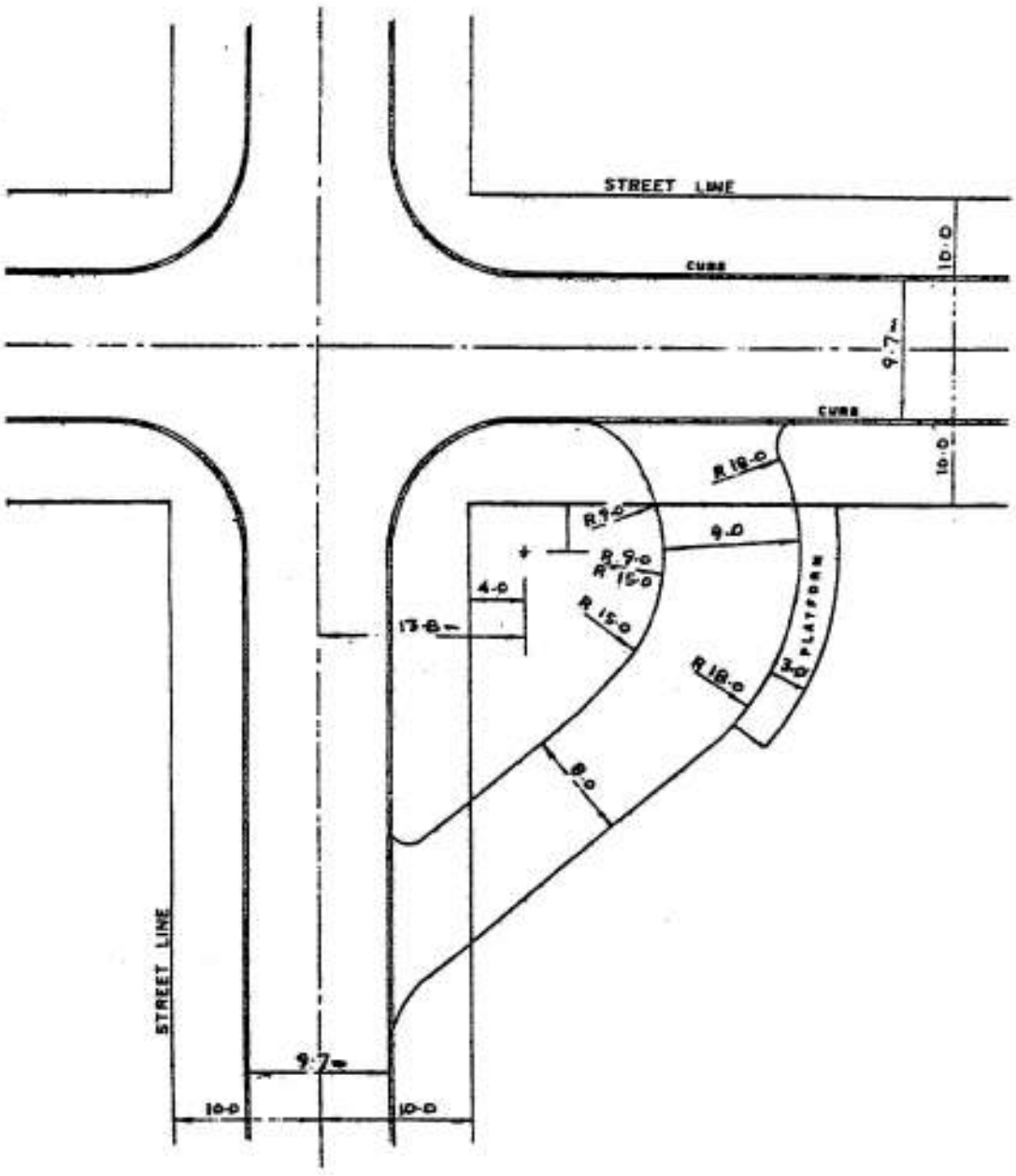
12 m Bus



3.0 (continued)

BUS LOOPS

12 m Bus



4.0 Bus Lanes

A Bus Lane in its most simple form is one or possibly two lanes of a municipal street reserved for buses only. The use of a Bus Lane is a recognized method of providing additional capacity in the peak direction on streets with heavy bus volumes. There are a number of variations in the actual use of the bus lane. In some instances, taxis, carpools and emergency vehicles or some combination thereof are also allowed to use the Bus Lane. Whereas some are exclusively used for buses. The lanes may be exclusively used for such traffic or only at certain times of the day depending on the circumstances. Their use is dependent on their ability to move at least as many people as a normal traffic lane would. They should not interfere with normal traffic movements nor access to street frontage where existant. They are generally considered a low cost technique of providing additional street capacity. They are usually created by the passing of appropriate By-Laws, proper signing and barriers where necessary and most important proper and consistant enforcement.

The prime objective of the Bus Lane is to provide a more efficient bus service by improving bus speeds through congested areas. Hopefully this improved service will attract increased ridership thereby reducing the number of cars using the road. Although it does not always provide increased capacity for other road users it can help reduce congestion by separating from the rest of traffic the slower moving and frequently stopping buses. The obvious disadvantage of the Bus Lane is it reduces or usually decreases the number of moving lanes available to the rest of traffic. If the Bus Lane is immediately adjacent to the curb or street frontage it makes right hand turns difficult and street access loading zones virtually impossible during the hours of the Bus Lane's operation. If a centre lane is used for the Bus Lane it can create a hazardous situation for the pedestrian in the loading zones. While use of the centre lane facilitates right turn movements if used on a two way street will cause problems or make difficult the left turn movement.

Bus Lanes are usually carefully delineated by overhead signs or signs or utility poles immediately adjacent to the lane and with special pavement markings including printed messages. If Bus Lanes are located in the centre of the street pedestrian safety islands are usually constructed carefully delineated to provide some safety to the waiting passengers.

4.0 (continued)

Another variation of the Bus Lane is the Contraflow Bus Lane. As the same implies it is the use of an exclusive lane for Buses in the opposite direction to normal flow. In many cases a typical divided street will have a peak traffic periods a congested traffic condition in one direction with limited flow in the opposite direction. The Contraflow Lane is temporarily placed on the low density side opposing normal traffic flow. Effective temporary signing, barriers and signalization must be used along with adequate enforcement during the hours of operation to ensure non interference between the bus lane and normal traffic flow.

Whatever variation of the bus lane is used it will only be successful with constant and consistent police enforcement and a good program to acquaint the motoring public with its use.

5.0 Bus Passenger Shelters

Bus shelters are fast becoming an integral part of the urban bus transit picture. A combination of the inflationary aspects of the economy; the increased crowding of our streets and highways and the dwindling accessible parking and cost thereof have caused more people to look to the urban bus system as a major means of their daily transportation. With the swing to bus transportation more public demands, particularly in areas of severe climate, are being made for some type of shelter at bus stops. Particularly at the more heavily used transfer points and in locations of high bus user generators such as hospitals and shopping plazas. Also no longer is the public satisfied with makeshift shelters. Attractive, functional and well maintained facilities are required.

There are a number of factors used in determining the need for such shelters including the most obvious the number of users benefited; also the average duration of wait between buses; severity of climate; availability of other nearby shelter and by no means the least important public relations value.

The siting or location for each shelter should be carefully studied including the geometrics of the bus stopping locations; the distances from the shelter to waiting buses and possible conflicts with other activities in the immediate area. Even the prevailing winds should be considered in site location if possible. Some diligent engineering at this particular phase can save a multitude of problems at a later date which can be expensive and sometimes impossible to resolve.

5.0 (continued)

The shelters themselves must be acceptable to the public. They are an important source of public relations or advertising to the transit company. If they are readily accepted by the public and do provide comfortable waiting they can attract increased ridership. They should be well constructed and preferably portable in order that they can be readily dismantled and relocated if for example a major route change occurs eliminating the necessity of a shelter at a specific location. One of the most functional types are those made out of a light metal framework and metal panelling incorporating sufficient area of unbreakable glass or plexiglass in their design.

There are a number of small yet important points to be considered in the overall functional design of bus shelters. One of the most important is good visibility. The two end walls and front should contain large window areas to provide ample view of route approaches and as a form of security for the user. Adequate lighting should be provided both interior and exterior. This lighting should be protected either through height or security meshing. Access to the shelter should be adequate to provide easy exist and entry, light and visibility. If seats are to be provided they should be located on the side and near walls and in most cases should be minimized as they are often subject of vandalism and other uses than the intended purposes. Similarly washrooms should be avoided unless at terminal or main transfer stations where constant supervision is available. Easily accessible refuse containers should be located in close proximity to the shelter. The colour of the shelters should be closely considered. There are two divergent schools of thought on this. Some felt the shelters should be colour coded the same as the transit system's buses; while others feel they should blend into the surrounding neighbourhood.

Policing and maintenance of bus shelters are important factors to take into consideration. Bus shelters tend to be targets of vandalism and as such their design should be as simple and functional as possible leaving little to attract vandals. Periodic checks by Police on their normal rounds is also of assistance. The shelters cannot simply be constructed and forgotten. They must be systematically cleaned and maintained. This, in itself, tends to lessen attention by vandals. As stated earlier, they are a form of advertisement for the transit operation and their condition can influence people's thoughts towards using the system.

5.0 (continued)

BUS PASSENGER SHELTER EVALUATION FACTORS

FACTOR	RELATIVE IMPORTANCE IN PERCENTAGE OF TOTAL POINTS	
A. PHYSICAL ASPECTS		
Exposure to weather conditions	20%	
Lighting	5%	
Obstruction to visibility	5%	
All physical aspects		30%
B. PASSENGER USAGE (Determine by T.T.C. on-passenger count)		
Low on-passenger count	10%	
Medium on-passenger count	20%	
High on-passenger count	30%	
Total usage		30%
C. LENGTH OF WAIT		
During rush hours	5%	
During off-peak	15%	
Total length of wait		20%
D. AREA BEING SERVED		20%
ALL FACTORS		100%

5.0 (continued)

WARRANTS FOR
BUS PASSENGER SHELTER EVALUATION

LOCATION _____ and _____

CORNER _____ ROUTE _____ DIRECTION _____

A. PHYSICAL ASPECTS	
Exposure to weather conditions	Minimum 5 / Average 10 / Full 20
Lighting	Poor 0 / Good 5
Obstacle to visibility	Yes 0 / No 5
B. PASSENGER USAGE (Determined by T.T.C. on-passenger count)	Low 10 / Average 20 / High 30 — TOTAL ON COUNT
C. PASSENGER WAITING TIME (Half of headway)	

Rush Hour		Off-peak	
Less than 2.0	0	Less than 4.0	0
2.1 - 4.0	1	4.1 - 8.0	3
4.1 - 6.0	2	8.1 -12.0	6
6.1 - 8.0	3	12.1 -16.0	09
8.1 -10.0	4	16.1 -20.0	12
10.1 - UP	5	20.1- UP	15

D. AREA BEING SERVICED	
Undeveloped	0
Commercial-Industrial	5
Residential-High density	10
Transfer point	20
TOTAL OF A to D	ORDER OF PRIORITY ON PROGRAMME

5.0 (continued)

SPECIFICATIONS FOR BUS PASSENGER SHELTERS

Size 1.2 m x 3.65 m x 2.1 m (small)
Size 2.4 m x 3.65 m x 2.1 m (large)

(1) Shelters to be constructed to the size and design illustrated.

(2) Shelters to be constructed of anodized aluminum and glass except for the use of 13 mm plywood in accordance with item 3.

(3) All lower panels to consist of two sheets of 0.032 gauge minimum aluminum with reinforcing sheet of 13 mm plywood between the panels.

(4) All upper panels to consist of a single sheet of .032 gauge minimum aluminum.

(5) All frame members and base channels to be .050 gauge aluminum and corners to be .062 gauge aluminum.

(6) All upper and lower panels to be anodized and finished in blue enamel.

(7) All glass used must be tempered and have a minimum thickness of 5.5 mm.

(8) Glazing - aluminum angle glazing with self tapping stainless steel screws, for easy accessibility - plastic glazing stops not acceptable.

(9) Roof - 60 mm 0.17 kg corrugated white fibre glass fastened with tapping screws at one foot centres to aluminum under-structure, with sufficient supports in centre to give minimum load capacity of 195 kg/m^2 or 0.17 kg pen type white fibre glass with rafters @ 400 mm centres. Aluminum rafter ties are to be installed along the length of the shelter - two sections above the wall on each small shelter with added sections in centre of each large shelter, riveted to each rafter and to the outside frame. In addition, the rafter ties and rafters are to be bolted at the four corners of each shelter with 63. mm x 6.4 mm cadium plated steel bolts. The roof must be manufactured with a full perimeter flange to attach to the wall panels, brackets not acceptable. Roof pitch must be 50 mm minimum and have overhang of 150 mm on all sides.

5.0 (continued)

(10) At the base to securely hold the shelter in place a 25 mm x 50 mm base channel to be supplied for the full perimeter of the shelter, excepting only at the entrance. Brackets or outside angles not acceptable.

(11) To securely hold the panels together, aluminium "I" shaped extrusions to be supplied.

(12) Tenders must include drawings illustrating all construction detail including dimensions and gauge of aluminium used.

(13) Delivery date _____ weeks.

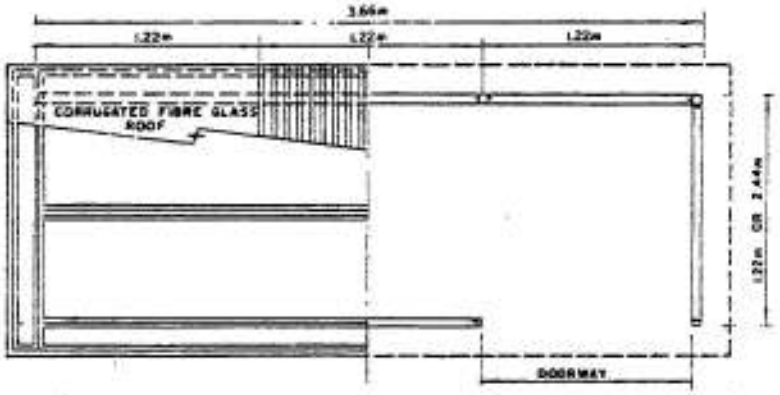
(14) The concrete pads for shelters will be constructed by the City of Etobicoke, Works Department, Roads Division.

Warranty:

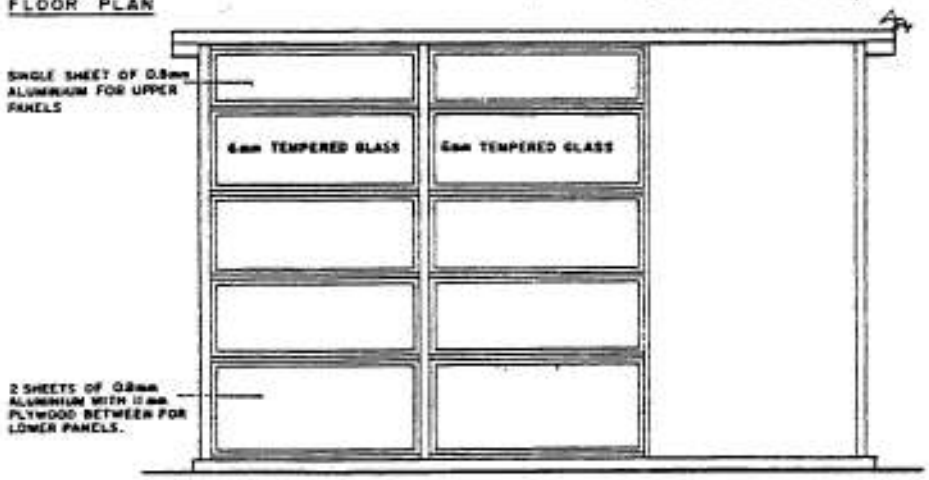
The Manufacturer is required to repair or replace, as necessary, free of charge any items found to be defective due to faulty manufacture or workmanship, within two years of the supply of any individual shelter.

5.0 (continued)

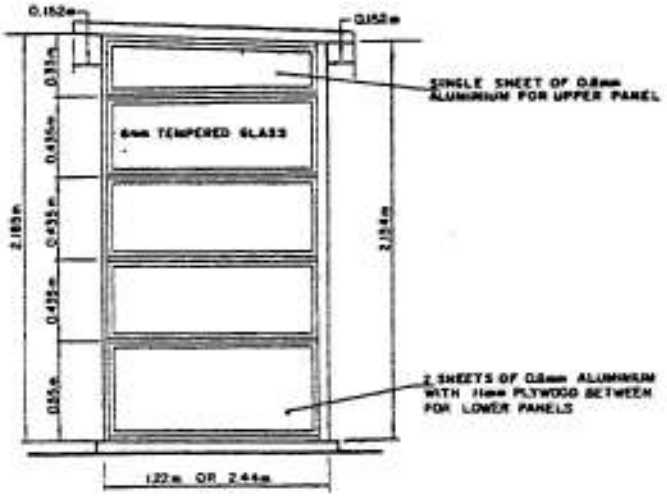
BUS PASSENGER SHELTER



FLOOR PLAN



FRONT ELEVATION



SIDE ELEVATION

6.0 Subways and Commutor Trains

The subway or commutor trains station will increase the vehicular movement in its area. Consideration should be given to the road pattern, the road widths, and particular attention to vehicle turning movements and affected intersections.

Parking should be provided close to the station with long term provisions. Also "Kiss and Ride" facilities (area for pick up and drop off for car passengers) should be considered particularly in suburban areas.

When the station acts as a transfer point for buses, adequate looping facilities should be provided.

Particularly careful consideration should be given to all these facilities at the terminal point of subway or commutor train service, since these often develop as the most congested points for both other transit facilities and for cars.

5.5 BICYCLE FACILITIES INDEX

1.0 On Public Road Allowances

1.1 General Guidelines

2.0 In Recreation Areas

3.0 Bicycle Path Design Standard

3.1 Alignment

3.2 Grades

3.3 Width and Clearance

3.4 Surfaces and Construction

3.5 Drainage

3.6 Other Services

3.7 References

3.8 Typical Sections

BICYCLE FACILITIES

1.0 On Public Road Allowances

There are three basic methods of dealing with bicycle traffic on public road allowances:

(a) Outright banishment of bicycle traffic;

(b) The part-time use of roads exclusively for bicycle and other non-motorized traffic;

(c) Sharing of facilities between all types of traffic.

Alternative (a), Banishment, is the logical step to take in certain instances such as urban controlled access freeways and other high-speed, high-volume roadways where the danger to cyclists is high.

Alternative (b), the Part-Time Use of Roadways exclusively for Bicycles and other Non-Motorized Traffic, is preferable from a safety viewpoint. There may, however, be vigorous opposition from those people who reside on these streets and from businesses that would be affected by the closure of these roadways to automotive traffic. Even businesses in districts served adequately by back-lanes for motor vehicle access may depend upon automobile traffic for drive-in trade. However, precedents have been set for the banning of motor traffic from public streets on a fulltime basis; i.e. pedestrian and shopping malls, and some municipalities have banned motor vehicles from certain streets on specified days for cyclists.

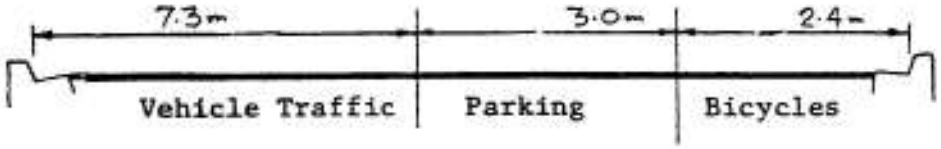
Alternative (c), Shared Facilities, would seem to be the most widely used and therefore the most acceptable method. Since bicycles are not entirely compatible with either motor vehicles or pedestrians, there are safety problems in the sharing of road facilities. As the number of bicycles is increasing rapidly, these safety problems arising out of their operation in close proximity to other traffic can also be expected to increase. The provision for "bike lanes" or "bike paths" is the most feasible way of giving an acceptable degree of protection to the cyclist sharing the roadway with vehicular traffic. Similarly, bike lanes and bike paths eliminate the conflicts with pedestrian traffic.

1.1 General Guidelines

As sharing of the road allowance is the most common method employed, it is worth noting some of the guidelines and techniques now being used. The usual method of providing bike lanes on existing streets has been to mark out an area about 6 to 8 feet (2 metres) wide adjacent to the curb and identify it accordingly.

1.1 (continued)

No barriers are erected to prevent vehicles from encroaching on the bike lanes, and on local streets where speed and traffic volumes are low, this method is functional. If bike lanes are to be provided on existing streets where traffic volumes are high or where the speed limit is higher than 35 m.p.h. (60 Kilo. per hour), some consideration should be given to the provision of a barrier to prevent motor vehicles from straying into the bike lanes and visa versa. One method which has been used in some American cities has been the placing of concrete bumper curbs along the edge of the bike lanes with the area being utilized for snow storage where bicycle traffic is seasonal. A solution such as this would require a minimum pavement width of 42 feet (approximately 13 metres) allowing for two lanes of traffic with parking on one side plus the bike lane.



Although this method provides adequate safety protection for the bike rider, it would require either widening of the roadway or reducing the number of lanes available for traffic, or both, on most roads where this degree of protection is required. On new roads and existing roads where major reconstruction is contemplated, serious consideration should be given to the construction of a separate bike path as part of the roadway. The decision on a separate path would consider financial limitations and space availability on that particular road.

A pathway exclusively for bicycles, physically separated from both vehicular and pedestrian traffic by a grassed or planted boulevard is, without a doubt, the best method of providing for cyclists on busy, high-speed roads. The separation should, of course, be as great as possible. In urban areas adequate provision must be made for them at intersections (dropped curbs, signing, pavement markings, etc.) to enable the cyclist to use the bike paths safely. Where bike paths must cross freeways, rivers, wide ravines, etc., roadway structures should be designed to accommodate this need.

In rural areas, some effort should be made to avoid long, straight sections. This can be accomplished by changing sides occasionally at culverts or bridges, by closely following the natural topography rather than the highway grades, or by other innovative ideas depending on the natural features available. Adequate rest areas, with at least minimal facilities, must be provided.

1.1 (continued)

BIKEWAY CLASSIFICATIONS

CLASS I



TOTAL SEPARATION / DIVIDING STRIP BETWEEN RIGHT OF WAYS ON SEPARATE SURFACES.

CLASS II



TOTAL OR PARTIAL SEPARATION / ADJACENT, BUT SEPARATED RIGHT OF WAYS ON SAME SURFACE.

CLASS III



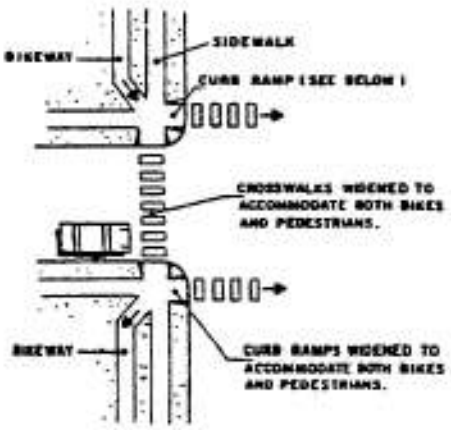
PARTIAL OR NO SEPARATION / SHARED RIGHT OF WAY ON SAME SURFACE

POSSIBLE LOCATIONS FOR BIKEWAYS

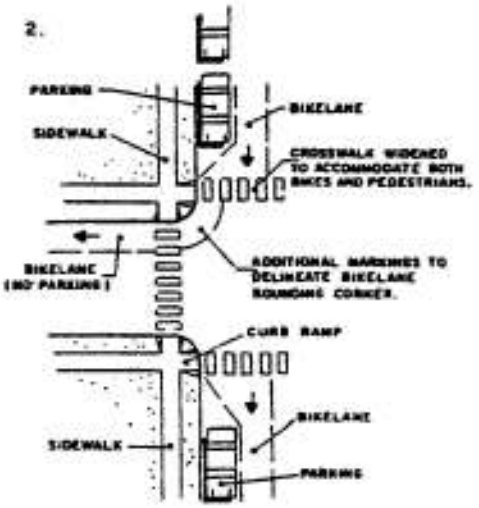
Abandoned R.R. right-of-ways;
Electric and pipeline right-of-ways;
River banks;
Dry washes
Beach fronts, lake fronts;
Flood control dikes and levees;
Irrigation canal banks and dikes;
Fire breaks

INTERSECTIONS FOR BIKEWAYS

1.



2.



2.0 In Recreational Areas

Bicycle paths in public recreational areas should be:

(a) Kept separate from pedestrian and vehicle traffic whenever possible.

(b) Constructed wide enough to permit two-way traffic and the access of maintenance and emergency vehicles.

(c) Surfaced with a material that is compatible with the thin tires popular on present-day bicycles.

(d) Blend in with the natural features of the ground to be traversed; and

(e) Of sufficient length and in locations which encourage their use. A comprehensive survey should be carried out prior to the development of any bicycle path to ensure that the proposed bicycle path will get the intended usage. Unless it is of a sufficient length and is properly located, usage will likely not justify the development costs.

3.0 Bike Path Design Standards

3.1 Alignment

In general, the use of long tangents should be avoided. It is generally acknowledged that a bicyclist can become discouraged by visible stretches of bike path in excess of 400 m. The alignment, therefore, should be carefully chosen in the field to take optimum advantage of the natural topography and surface features. A curvilinear alignment is preferable to one with short tangents connected with curves. The curves can be quite sharp, 10 m radius, as long as superelevation is provided. On steep uphill sections, the curves can be sharper, 3 m radius, and superelevation is not required.

3.1 (continued)

Table I Bicycle Path Superelevation

Design Speed	Radius (m)	Superelevation (m/m)
20 km/h	11 m (min.)	0.10
	12 m	0.08
(Level Sections)	14 m	0.05
	15 m	0.03
	17 m	0.02
	18 m	0.02
	20 m	0.02
	21 m	-0.02
	23 m	-0.02
30 km/h	24 m (min.)	0.10
	27 m	0.08
(Downhill Sections)	30 m	0.06
	34 m	0.04
	37 m	0.02
	40 m	0.02
	43 m	0.02
	46 m	-0.02

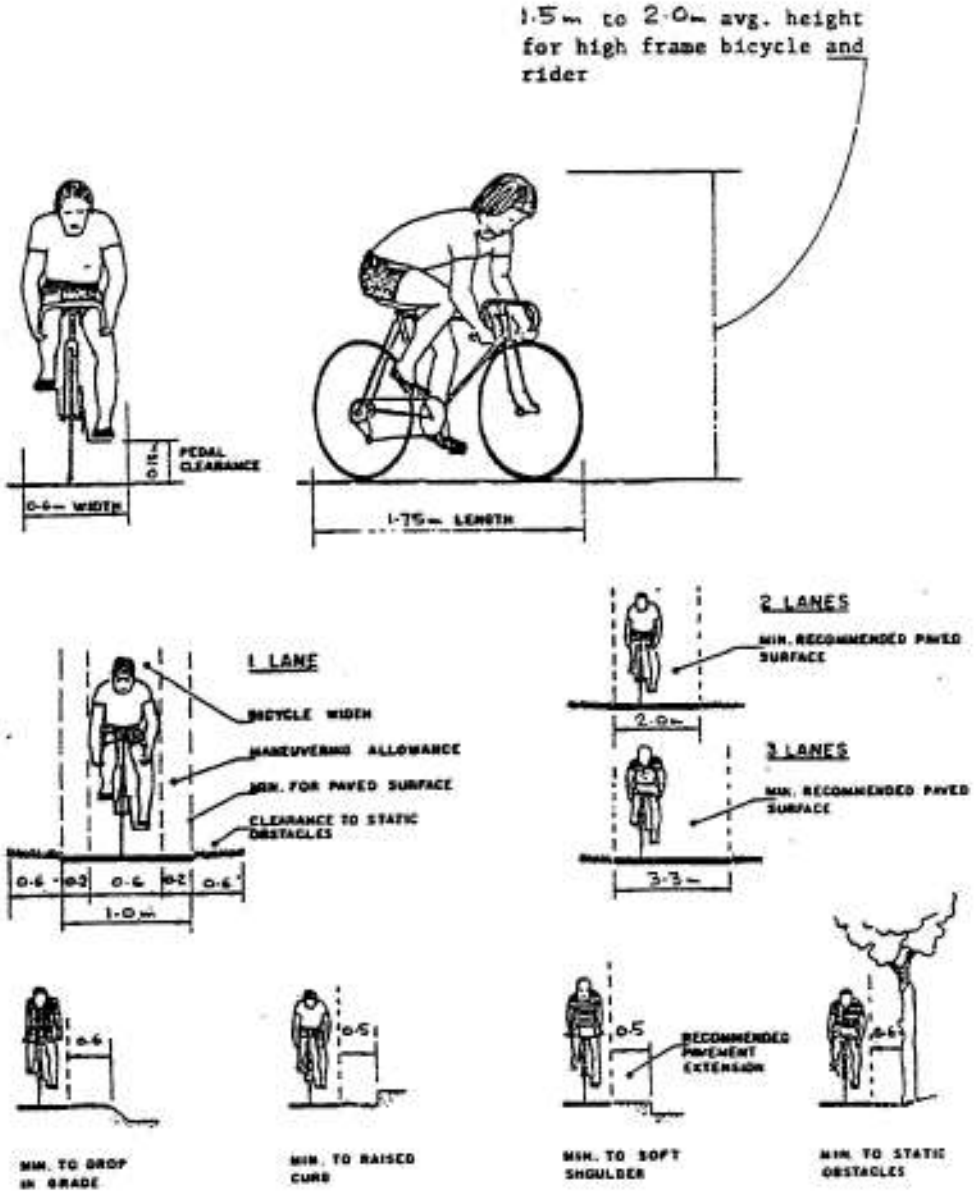
Negative Superelevation indicates that advere banking for drainage purposes may take precedence over banking to counteract centrifugal forces.

3.2 Grades

Long grades should not exceed 3%. Short lengths (up to 75 m) with grades as high as 8% can be tolerated. Rolling grades are preferable to steady uphill sections, as they give the cyclist a time to coast and catch his breath. Grades steeper than noted will force riders to dismount and push their bikes, and if these sections are numerous, will likely result in the pathway not being used to its capacity. If the topography requires the construction of paths with grades steeper than indicated, separate one-way paths should be considered. The downhill path should not be steeper than about 6% with the curves superelevated for a design speed of 30 km/h. The uphill portion can be considerable steeper (20%) and shorter, reducing the distance to push the bike proportionately.

Gradient	Length	
	Norm.	Max.
1.5%	490 m	
3 %	120 m	240 m
4.5%	45 m	90 m
10 %	9 m	18 m

3.3 Width and Clearance



3.3 (continued)

One-way paths, when constructed, should have a minimum width of 1 m with a slightly wider width 1.2 m preferred. Two-way paths should have a width of 2.1 m minimum and preferably 2.5 m. Where traffic is heavy, consideration should be given to providing for a minimum of 3 lanes by widening the pathway to 3 m or more. One-way paths should be avoided wherever possible, as the narrow width does not allow for the passing of slower riders.

Fixed objects should be kept clear of the edge of the path by 0.5 m. A minimum overhead clearance of 3 m is desirable and in no case should the overhead clearance be less than 2.1 m. Bridge, rails, guard rails, fence posts, etc., should have a horizontal clearance of at least 0.3 m.

3.4 Surfaces and Construction

A survey conducted by the American Institute of Park Executives showed that over 75% of people preferred the use of asphalt as a surfacing material for bicycle paths. Second choice was either dirt, gravel or soil cement with approximately 5% each. Other types of surfaces gained negligible support. The type of bicycle currently in vogue with its thin, high-pressure tires is very difficult to operate on anything but a smooth hard surface. The National Capital Commission standard for bicycle paths is 50 mm HL3 asphalt surface over 150 mm of compacted granular "A" base, and this would seem to be suitable for most areas of the province. A full depth asphalt surface 75 to 100 mm thick placed in one lift, on a compacted natural earth subgrade may be more economical and would probably be suitable in most areas also.

3.4 (continued)

Recommended Surfaces



Bituminous Conc. mixes



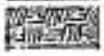
Concrete



Stone Chips



Soil Cement



Stablized Earth



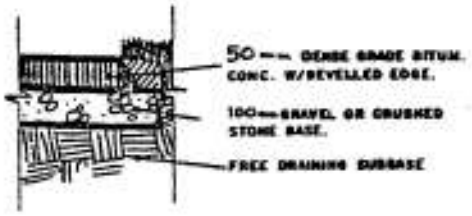
Wood Planking

All weather, permanent surfaces:
(most widely used; highest installation costs, longest wearing life; especially good for heavy use in urban areas.)

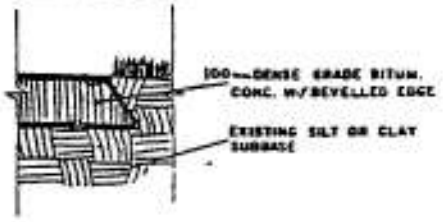
Loose aggregate, compositional, natural surfaces:
(lower installation costs; high maintenance requirements, susceptible to poor natural drainage; will not stand up to continual heavy use.)

Wood Surfaces:
(use only when laid perpendicular to direction of travel, usually on light bridges, boardwalks, etc.)

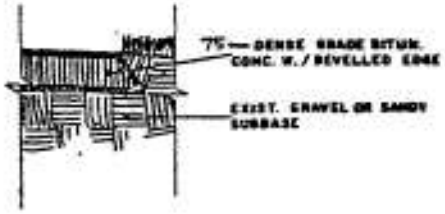
Standard Construction



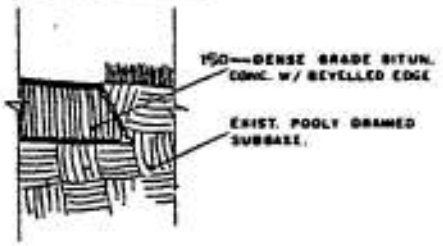
Silts & Clays



Gravels & Sands



Poorly Drained



3.5 Drainage

Surface run-off from small areas can be allowed to drain directly across the pathway and the pathway should be sloped to either side with a crossfall of 2% to 3% to facilitate this drainage. Where large amounts of water must be accommodated, culverts should be installed. Side ditches, where required, should have flat side slopes or be constructed at distances of 1 m or more from the edge of the path to the top of the ditch slope.

3.6 Other Services

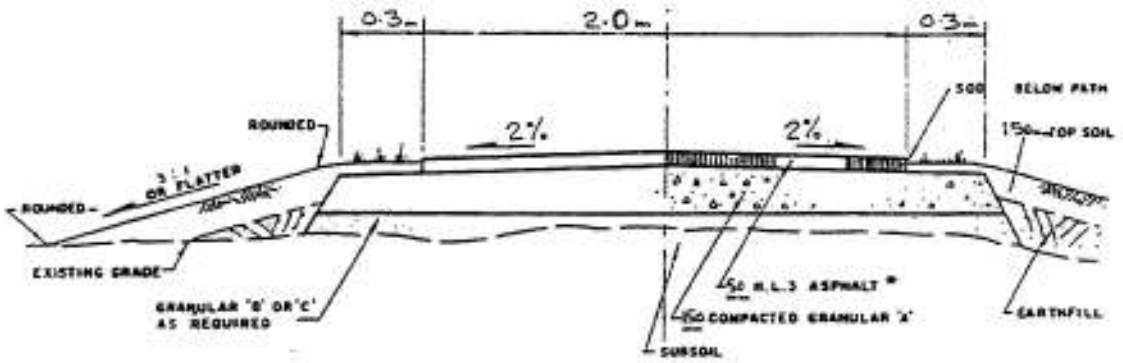
Rest areas should be provided, in scenic areas, at frequent intervals and wherever possible, set well back from the pathway. They should be equipped with benches, tables, etc., much the same as the roadside picnic areas which are found along the provincial highways. Wherever possible, restroom facilities and drinking water should also be provided.

3.7 References

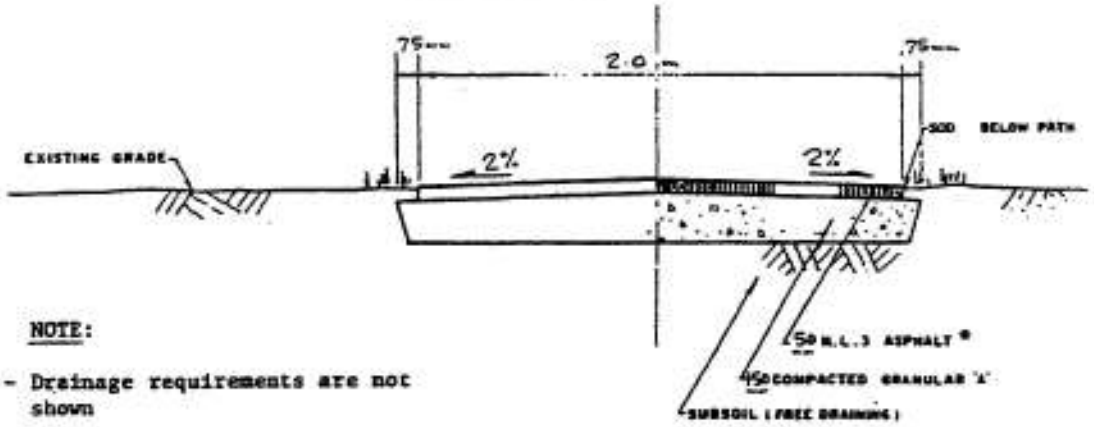
Subsequent to the preparation of this section the following publication has become available and is recommended for further reference:

"The Guide for the Design of Bikeways"
Roads and Transportation Association of Canada,
1765 St. Laurent Boulevard,
Ottawa, Ontario, Canada, K1G 3V4.

3.8 Typical Sections



FILL SECTION



CUT SECTION

NOTE:

- Drainage requirements are not shown
- Superelevation to be used on curvature of less than 450m radius
- Crown on tangents or curvature of more than 450m radius may be applied all to one side to facilitate drainage of surrounding area.

* Refer to 3.4, Standard Construction for further design proposals.

5.6 APPURTENANCES INDEX

1.0 Boulevard Treatment

2.0 Vegetation Consideration

3.0 Drinking Fountains

4.0 Outdoor Telephones

5.0 Trash Cans and Receptacles

6.0 Benches and Outdoor Seating

7.0 Sound Barrier Walls

5.6 APPURTENANCES

1.0 Boulevard Treatment

Providing there is an adequate boulevard width, and a reasonable expectation of satisfactory care without undue pedestrian traffic, grass provides the most pleasing boulevard treatment. The following rule of thumb guide is provided in this respect:

Boulevard Width Curb to Sidewalk	Area Zoning	Boulevard Treatment
Less than 1.0 m	All	Asphalt or equal
1.0 m and greater	Residential	Grass
Less than 1.8 m	Industrial & Commercial	Asphalt or equal
1.8 m and greater	Most areas	Grass

Too often the decision is made to place an asphalt surface rather than grass because of the effects of salt action. Lawn and garden gypsum is a natural soil conditioner which will efectively counteract the effects of de-icing salt and heavy snow pile-ups on roots of grass, shrubs and trees. Some varieties of grass are more resistant to the effects of salt, some are slower growing, and some have longer root systems and more resistant to wash-outs, dry weather, etc. Care should be taken in the selection of the grass variety to suit the particular conditions. Care should also be taken to ensure that a satisfactory quality and depth (min. 50 mm) of topsoil is placed prior to the laying of any sod.

Where an asphalt boulevard is placed, a minimum depth of 100 mm Granular 'A' and 50 mm HL3A asphalt is recommended. It is also suggested that a soil sterilant be used priot to placing the asphalt in an effort to reduce the weed growth in any cracks. Although coloured asphalt is available, it is not recommended except in unusual circumstances. It has a high initial cost and it is very difficult to make a satisfactory match in maintenance repairs.

As an attractive alternative to asphalt, particularly in commercial or high density residential areas where high pedestrian volumes demand a hard surface, pre-cast paving slabs or interlocking paving stones may be used. These can be obtained in a variety of colours. They are easily removed and replaced for any utility repairs needed in the boulevard area.

Paving stones are made of compressed concrete with a variety of interlocking patterns which ensure that the surface will not be dispalaced under the weight of traffic. This locking system prevents tipping of individual stones and ensures the entire paved area will remain smooth and level, a distinct advantage over the pre-cast paving slabs.

1.0 (continued)

The stones are available in various thicknesses to suit particular applications, for example, 70 mm thick stones are used for sidewalks, bicycle paths, driveways and parking lot areas, and 80 mm stones are used for areas involved with trucks. The stones are laid on a 50 mm to 300 mm depth of compacted granular base, the depth again subject to the loading conditions and the quality of the foundation materials.

In areas of cut or fill the boulevard slopes may be considerably greater than normal and often may require slope rights or additional road allowance to carry the slopes to the adjacent ground level. In some cases where the right-of-way is limited due to existing topography restrictions (e.g. buildings) hard surface slopes or retaining walk may be necessary.

Particular attention should be paid to ensure proper access to the sodded slope areas from the road so that routine maintenance may be carried out.

It is recommended that sodded slopes be not greater than 3:1, with attention made to the variety of grass used and to shrub plantings.

2.0 Vegetation Considerations



Overhead materials should be chosen for their particular characteristics. Avoid materials that have tendencies towards dropping excessive debris drooping or breaking under heavy snow loads, or winds, etc.

Maintain a minimum of 2.6 m vertical clearance over walks, bikeways, sitting areas, etc.

Improper location or poor maintenance can quickly lead to a reduction of the efficiency of lighting systems.

Avoid placing hazardous or nuisance materials adjacent to walkway or sitting areas.

Plant materials may affect the melting of ice and snow from walkways and stairs consider their mature shadow patterns during winter months before deciding on their final locations.

Many trees with shallow or surface root systems will heave or break up walkway surfaces use caution when choosing these varieties and their subsequent locations.

Avoid placing materials over or near underground utilities. If they have root systems that characteristically cause damage to pipelines, cables, sewers, etc.

Creeping ground covers, vines and other invasive materials can be troublesome if not contained - keep them off buildings, walkways, steps, ramps, signs, and lighting fixtures

2.0 (continued)

It is well known that high salt concentration can cause injury to vegetation. Salt in large quantities can interfere with the plant's absorption of moisture from the soil and by browning or burning leaves. Salt may indirectly affect plant growth by altering soil structure, permeability and aeration.

Road salt is just one of the many hostile factors affecting roadside vegetation. Plants must also adjust to compacted soil, injuries to root systems during road construction, changes in soil moisture levels due to paving, and auto exhaust emissions.

Stress for water increases the chance of salt damage to vegetation. The roadside environment robs plants of moisture; they are exposed to excessive wind flow, high soil and air temperature, and added exposure to sunlight. Drought may be the final straw.

After many years of salting, sodium buildup in roadside soils may indirectly affect plant growth. Soil can be chemically amended and salinity removed by adding gypsum or anhydrous ammonia. Gypsum treatments appear to be the most efficient and least expensive reclamation method.

Salt damage to trees can be minimized and often prevented by selection of the correct species and proper planting practices. Good street and highway design, including adequate drainage systems for highways, and curbs and gutters to carry street runoff into storm sewers, substantially reduces salt's effects.

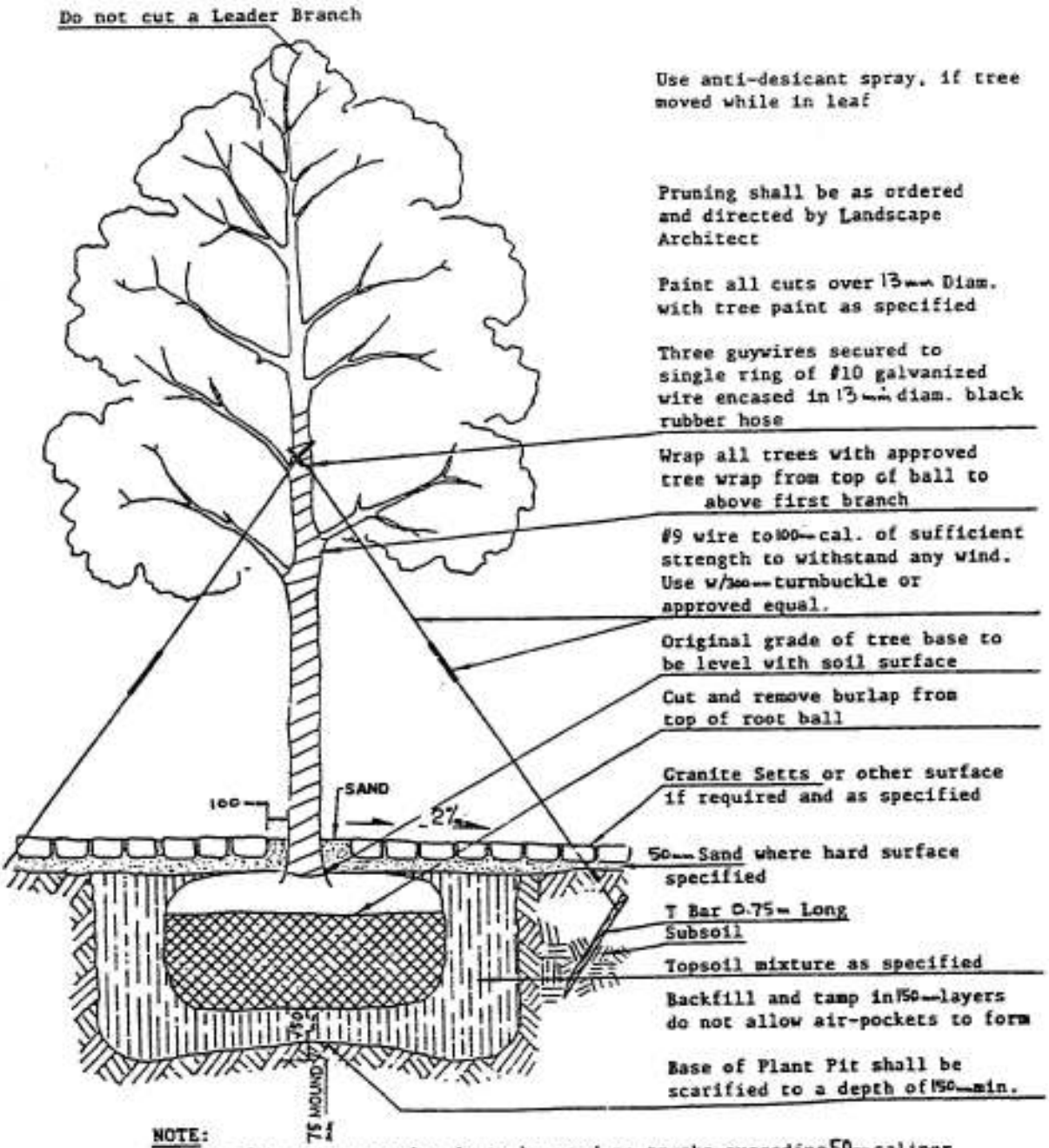
Many trees resist effects of salt very well. These include oaks, locusts, Scotch elm, Russian olive, hawthorne and silver and gray poplars.

Landscape planners should avoid use of sugar and red maples, Lombardy poplar, black walnut, and rose and spirea bushes in locations exposed to salt runoff and spray from de-icing operations.

Salt may be blamed for vegetation decline that is caused by other factors. Many trees die because their life cycle is naturally complete. Others die as a result of disease of infestation.

2.0 (continued)

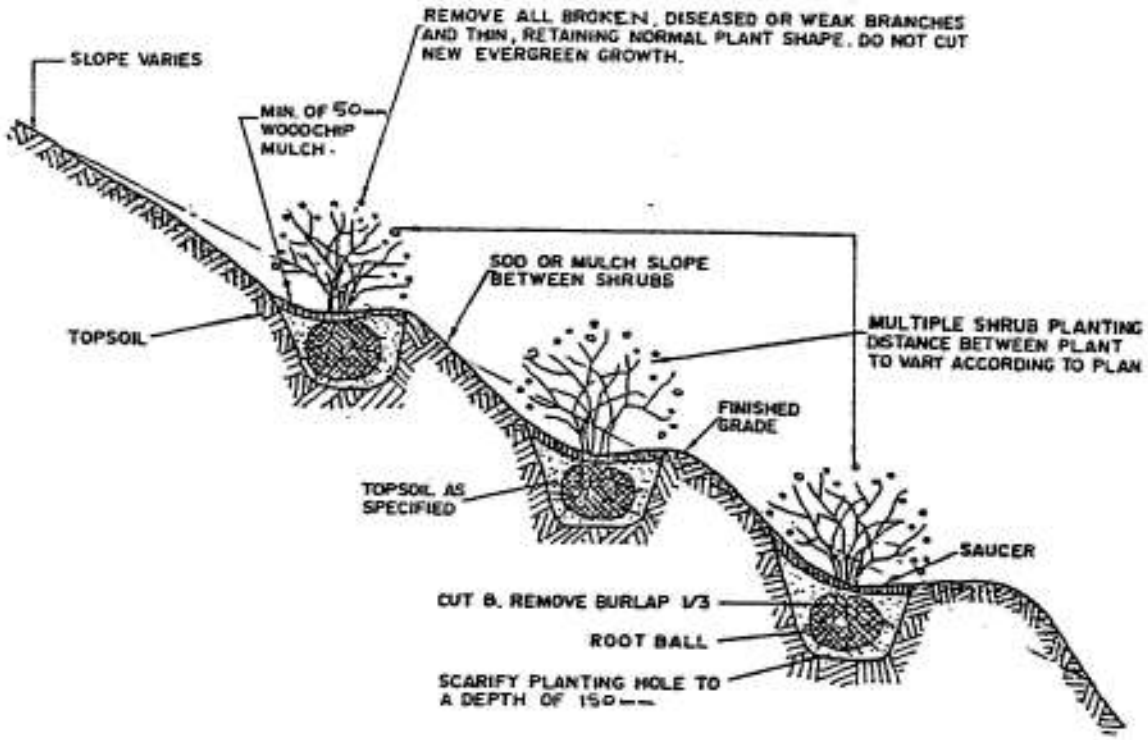
Typical 75 to 150 mm Tree Planting In Paved Boulevard



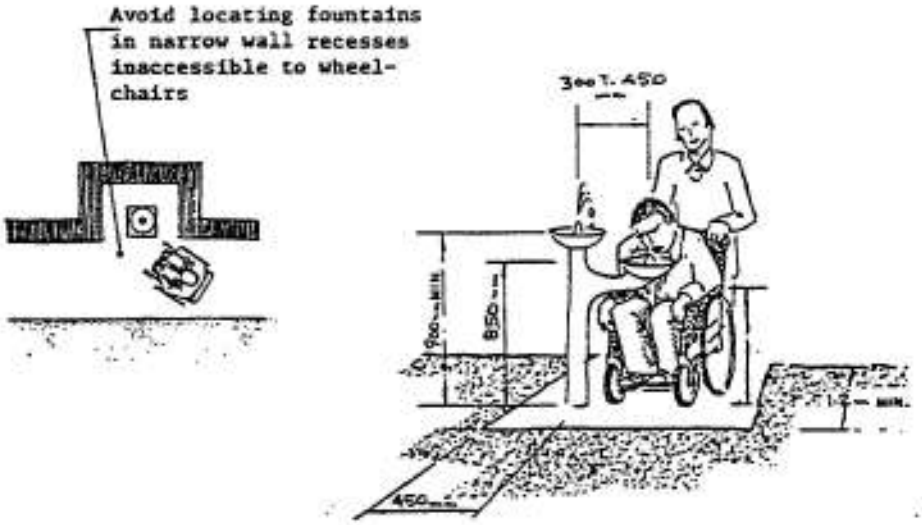
NOTE: Burlap tree wrapping is to be used on trunks exceeding 50 mm caliper and branches exceeding 75 mm caliper on those species designated by Landscape Architect.

2.0 (continued)

Multiple Shrub Planting On Slope



3.0 Drinking Foundations



Hand operated knobs or buttons and foot pedals are difficult for many handicapped people to operate. Hand levers are preferred.

Provide a minimum 750 — vertical clearance below fountain nozzle to allow wheelchairs leg room for access.

Nozzle heights should be approximately 850 — for wheelchairs and children and 900 to 1000 — for adults.

A minimum 450 — wide paved area around outdoor fountains avoids both mud and puddles.

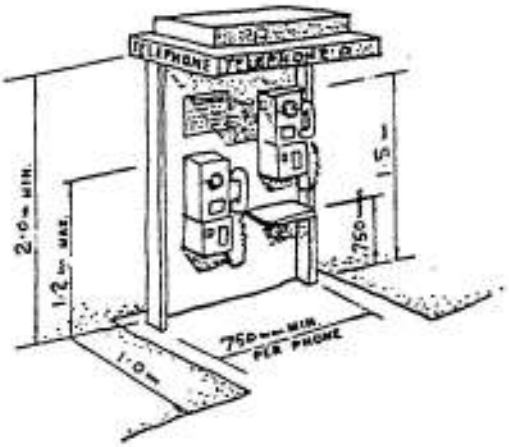
4.0 Outdoor Telephones

All groups of telephones should have at least one lower height telephone for use by the handicapped and children.

Phones for the handicapped should be no higher than 1·2m at the coin slots. Provisions for braille instructions, volume controls on headsets, and push button dials are helpful for many handicapped individuals.

Provide adequate lighting on the underside of overhangs for nighttime use.

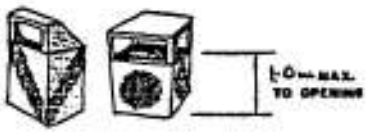
Package rests are a helpful convenience to all people.



5.0 Trash Cans And Receptacles



1. Open top easiest to discard trash; open to rain, wind, snow; needs weep holes for drainage; easy to empty; open to insects.



2. Semi-open top gives protection from elements; top hinged to allow removal of trash when unit is full; openings must be designed to accept size of anticipated trash; open to insects.



3. Hinged door openings are difficult for many handicapped people to operate; good protection from elements and insects; openings must be designed for anticipated trash sizes; spring loaded doors should be easily pushed open with one hand.

6.0 Benches And Outdoor Seating

Choose materials which do not retain heat or cold. Avoid rough materials or those that may splinter.

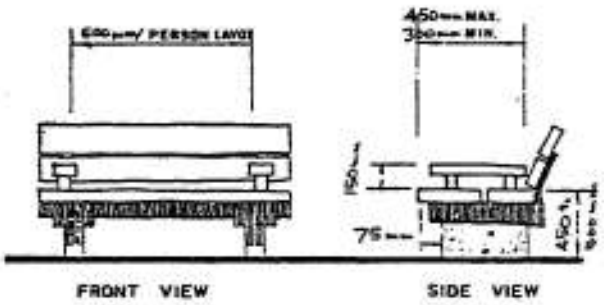
Sitting heights of 0.45 to 0.5 m are preferable.

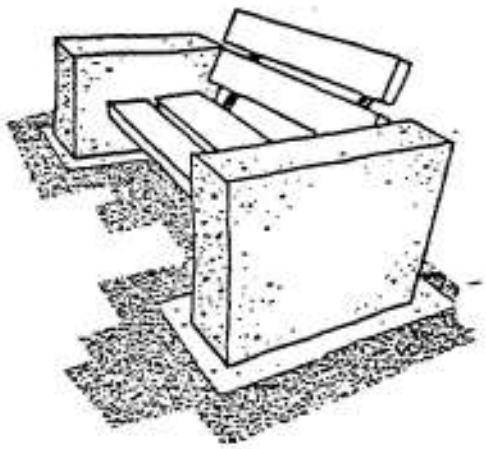
Sitting surfaces below 0.3 m width are uncomfortable for many adults, likewise, widths beyond 0.45 m become awkward for normal leg lengths.

Provisions for arm and back rests increase comfort. Arm rests are also helpful for getting into and out of seats and benches.

Provision for heel space of 75 mm makes rising from seated position easier.

Seat surfaces should be pitched to shed water.

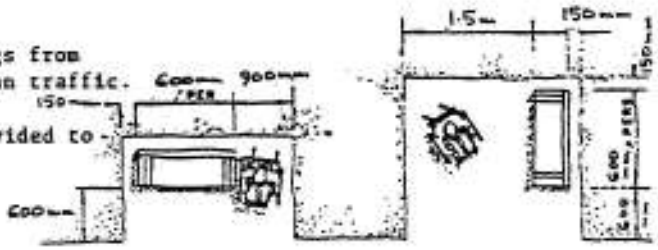




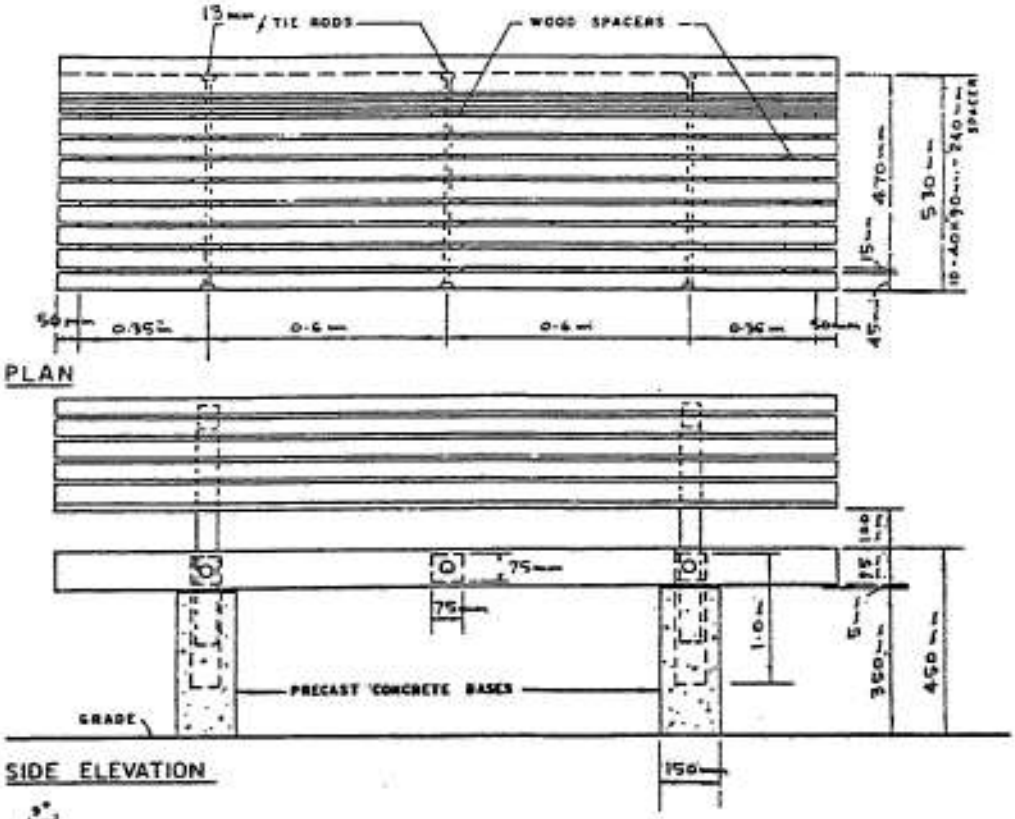
Space Around a Bench

Provide 600 mm setback to keep legs from interrupting adjacent pedestrian traffic.

A 900 mm min. space should be provided to allow room for wheelchairs.



6.0 Benches And Outdoor Seating (continued)



NOTES:

1. Seat and back - 40mm x 90mm D- 115mm select pine (outer pieces) remainder No. 1 white pine
2. Tie rods -38mm diameter - recess nuts washers and heads
3. Pine spacers - 38mm X75mm X75mm
4. Mild steel anchor plates - seat back 13mm X50mm X600mm
5. Wood finish -
   (a) ½ part linseed oil to ½ part turpentine rubbed on
   (b) ½ part "pentox to 1/2 part walnut paint rubbed on
   (c) 2 coats urethane varnish brilliant gloss brushed on

7.0 Sound Barrier Walls

Reference should be made to Ministry of Transportation and Communications Report No. RR180, September 1972, produced by the Research Division of the Ministry of Transportation and Communications.

Barrier walls have been constructed of earth embankment, normal density concrete panel wall, lightweight cellular concrete panel wall, combined cellular concrete wall and earth embankment, plywood, aluminum panel and gabions.

As outlined in the Ministry of Transportation and Communications' report, barriers are a relatively ineffective method of noise shielding. Therefore, greater attention and emphasis should be given to other noise control measures such as housing modifications, land use control and control of vehicular noise emissions at the source.

There have been indications from people living behind the barriers that they considered them beneficial and that they favoured their retention. The precise reasons for this are unknown, although they may be related to psychological and visual shielding and shielding against dirt and debris.

In most cases, barriers because of their high cost and their limited effectiveness and other adverse effects (aesthetics, shadow, effect on snow drifting), do not appear to be the most cost-effective solution to highway noise problems. Greater consideration should be given to alternative measures.

5.7 FINANCING

1.0 Methods of Financing

Although this chapter deals specifically with "Transportation", the methods of financing herein are as we generally know it under the Municipal Engineer's jurisdiction.

The council of the municipality dictates the policy that is followed in the paying for works. Management of the municipality assists council when requested to give them full knowledge of all facts. It is definitely in the Municipal Engineer's interest to have a good communication with the treasurer's department. By having this rapport there is a good appreciation between the two departments of the problems and the priorities of the day so that when decisions are made on financing all facts are reviewed in the decision making process.

There are three main ways that works can be financed, namely through debentures, through current revenue and/or a funding account procedure.

Capital financing through debentures is usually used when the cost of the work far exceeds the capability of the municipality to pay for them immediately. By the use of debentures we are able to finance through a pay-as-you-use program over a period of time by maintaining a constant rate of cost which avoids a high tax burden in any one year. The ability to finance through debentures is controlled by the Ontario Municipal Board whose approval is required. The Board reviews the ability of the municipality to pay and, as a guideline, the per capita debt and the debt to assessment that the municipality has accrued is assessed. The disadvantage of debenture financing is that it creates a certain extra expense to the work in interest payments of the cost of borrowing. The debenture charge can be assessed against the ratepayers as a whole in the municipality or a charge to users only. A charge to users only can be accomplished on an area basis or to abutting owners under the Local Improvement Act. The area basis is more commonly used in rural municipalities.

Where work is undertaken and does not create a high burden on the tax structure of the year the work is generally constructed from monies obtained out of current revenue. In other words, the work is estimated and the cost approved by council to be undertaken within the mill rate established for that particular year.

1.0 (continued)

Funding can be used where it is desirable to establish a reserve account to do certain works. This is done by making a levy against general ratepayers within the current budget expenses and these monies allocated for a certain cause or project. These monies can also be obtained from levies directed against developers under what is known as "Subdivision Agreements" or "Developer's Agreements". The monies are then set aside for the work and may be proceeded with when the cost is covered. Another example of funding would be in areas where a revenue is obtained by the municipality such as the revenue from parking meters. The profit is usually funded into an account for future off-street parking facilities. Reference can be made to the Manual of Municipal Accounting, published by the Ministry of Treasury Economics and Inter-Governmental Affairs.

2.0 Estimating

In the financing aspect of work the first thing the municipal engineer is requested to give advice on is a preliminary estimate. It is usual to have reference to a 'ballpark figure' of the cost of the services requested. This is obtained from costs of similar work within the municipality. The cost of work are influenced by the soil conditions within the municipality; the availability of contractors within the area; and amongst other things the weather conditions that prevail. Preliminary estimates should be readily available and when given out of the department there definitely should be an explanation that the estimate is only preliminary and could change when all details are known and calculated in a detailed estimate. Usually preliminary estimates are given as a guide only to see if the work should be investigated further. Once direction is given to proceed the engineer then prepares a detailed estimate. These estimates are completed only after a survey has been made of the work, plans prepared and all facts known. The format of the estimate should be made up in such a way keeping in mind the municipality's specifications. The detailed quantities in the estimates should be able to be used when preparing the bill of quantities within the Form of Tender of a contract.

Preliminary Estimate Cost Guide

The cost per metre indicated in the table is for the conditions as outlined below and will require adjustment in accordance with review of a particular project.

2.0 (continued)

- complete road reconstruction to provide curbs
- storm sewers and other municipal utilities existing
- catchbasins and connections reconstruction included
- normal urban driveway access reconstruction included
- nominal affected sidewalk reconstruction included
- nominal utility adjustment included
- collector and arterial roads include intersection channelization and bus bays as required
- 8.5, 9.7 & 13.8 m pavements have 140, 190 & 240 mm asphalt respectively on 260 mm crushed stone
- 15.0 & 16.8 m pavements have 75 mm asphalt on 200 mm concrete on 150 mm crushed stone
- does not include road allowance property widening costs

Width of Pavement (face to face of curb)		Year $/m 1980	1981	1982	1983	1984
Composite pavement with conc. base	16.8 m	800	870	990	1060	1110
	15.0 m	750	820	935	1000	1050
Deep str. asphalt pavement	13.8 m	680	740	845	905	950
	9.7 m	450	490	560	640	670
	8.5 m	365	400	455	485	510

2.0 (continued)

Date.....

Estimate Of Proposed Road Construction On

Street _____

From _____

To _____

Job No. _____ Length _____ m Width _____ m Plan No. _____

RELATED WORK

STORM SEWER Job No. _____ Dwg. No. _____

SIDEWALKS Job No. _____ Dwg. No. _____

EXTERNAL WORK

O.L.S. Survey.....	_____
Property Acquisition.....	_____
Ontario Hydro.....	_____
City Hydro.....	_____
Consumers' Gas.....	_____
Bell Telephone Company.....	_____
City Parks Department.....	_____
City Works Department.....	_____
City Traffic Department.....	_____
Materials to be supplied by City	_____

TOTAL OF EXTERNAL WORK $ _____

TENDER ITEMS	QUANTITY	UNIT	RATE	AMOUNT
Clearing the site	LUMP SUM PRICE			_____
Tree removal				
151 mm to 300 mm dia.....	_____	each	_____	_____
301 mm to 450 mm dia.....	_____	each	_____	_____
over 450 mm dia.....	_____	each	_____	_____
Stone for detour.....	_____	t	_____	_____
Asphalt for detour.....	_____	t	_____	_____
Excavation (calculated).....	_____	m^3	_____	_____
Additional excavation (estimated)	_____	m^3	_____	_____

2.0 (continued)

TENDER ITEMS	QUANTITY	UNIT	RATE	AMOUNT
Imported fill (calculated).....		m^3		
Remove existing sidewalks.....		m^2		
Remove existing curbs (incl. gutters).....		m		
Remove existing culverts (incl. headwalls).....		m		
Remove existing catchbasins....		each		
Adjust existing catchbasins....		each		
Adjust existing manholes.....		each		
Plug abandoned pipes.....		each		
Re-grade ditches.....		m		
Install 300 mm C.S.P.....		m		
Construct 600 mm catchbasin single.....		each		
Construct 600 mm catchbasin double (incl. sub-drain).....		each		
Install 150 mm dia. sub-drains.		m		
Install 200 mm catchbasin connection.....		m		
Install 250 mm catchbasin connection.....		m		
Install mm catchbasin connection.....		m		
Install 200 mm catchbasin risers.....		m		
Install 250 mm catchbasin risers.....		m		
Construct sidewalks 100mm thick		m^2		
150mm thick		m^2		
200mm thick		m^2		
Construct curb as per Std. 201		m		
Std. 203		m		
Std.		m		
Granular 'B' 150 mm thick.....		m		
Granular 'B'.....		t		
Granular 'A' 150 mm thick.....		m^2		
Granular 'A'.....		t		
Stone 150mm thick.....		m^2		
Stone.....		t		
Concrete base 200mm thick.....		m^2		
Concrete bus-bays 280 mm thick.		m^2		
Concrete median 230 mm thick...		m^2		
Concrete median 80 mm thick...		m^2		
Burn in asphalt joints.....		m		
Tack coat.....		m^2		
Saw cutting joints.....		m		
H.L. 3 asphalt 40 mm thick....		m^2		
H.L. 3 asphalt.....		t		

2.0 (continued)

TENDER ITEMS	QUANTITY	UNIT	RATE	AMOUNT
H.L. 5 asphalt 40 mm thick.....		m^2		
H.L. 5 asphalt.....		t		
H.L. 8 asphalt 50 mm thick.....		m^2		
H.L. 8 asphalt.....		t		
Carpet seal 25 mm thick.....		m^2		
Carpet seal.....		t		
Sidestreets.....		m^2		
Tapers 150 'B', 150 Stone, 100 H.L. 6 40 H.L. 3.....		m^2		
Shoulders 150 'B', 290 Stone...		m^2		
Paved Boulevards 100 'A', 50 H.L. 3A.....		m^2		
Residential driveway 150 St. 80 ASP.....		m^2		
Commercial driveway 150 'B', 150 St. 140 ASP.....		m^2		
Industrial driveway 150 'B', 150 St. 190 ASP.....		m^2		
Concrete driveway 150 mm thick.		m^2		
Stone in driveway.....		t		
Sod on 50 mm of topsoil.....		m^2		
Water (a) compaction.....		m^3		
(b) sod.....		m^3		
Traffic signal duct 75 mm.....		m		
Handwells.....		each		
Additional 2 bags (80 kg) cement		m^3		
Allowance for day work.....				
	Sub-Total			$
Add 5% for tender contingencies				$
	Total Tender			$
Add external work.....				$
				$
Add % Engineering and Contingencies				$
Total Estimated Cost				$
Anticipated M.T.C. Subsidy				$
Estimated City Share				$
Cost per metre $				

Prepared By_____ Checked By_____

5.8 DESIGN APPROVALS

1.0 Official Approvals

Approval for designs of transportation facilities are required from various departments of senior governments. Some of these are as follows: -

Road Projects

a) District Municipal Engineer - Ministry of Transportation and Communications.

b) Ministry of the Environment if sewers are involved.

c) C.T.C. if Railway Level Crossings are involved.

Bridge Projects

a) Municipal Structural Engineer - Ministry of Transportation and Communications.

b) Canadian Transport Commission, Ottawa and the Railway Company involved if the structure is over or under railway tracks.

c) Department of Transport - Marine Services, Ottawa if the structure is over a Navigable Water.

2.0 General Approval by the Public

Public informaton is fast becoming one of the critical planning areas for any proposed project. The type of program implemented will of course vary with the size and scope of the project, however at some point in the planning stage a decision must be made as to the degree of public participation, with particular reference to the requirements under the Environmental Assessment Protection Act.

Citizen participation can achieve better plans and increase the likelihood of implementation of those plans. Also, citizens can and do stop projects they perceive as undesirable. Engineers and planners will increasingly become involved in public participation situations. Increased costs are small especially in view of the magnitude of the decisions to be made or the possibility of costly stop work situations.

There is extensive literature describing various types of Public Information programs and Engineers or Planners designing a public participation program would benefit from study of this information. The following is a brief outline of some of the types and uses of Public Information programs.

2.0 (continued)

Minor Projects

- News Releases - to inform the public of impending projects. While resistance to small projects is usually minor and localized, advance publicity will enable affected residents to express concerns by letter or telephone.

- Direct Mailing - information about the nature and timing of a project is mailed directly to residents affected. Quite often unless local improvement changes are involved the first information is sent out just prior to construction. Information can be distributed by hand delivery along a particular route, however owners who reside elsewhere are missed unless notified by regular mails.

Major Projects

- Major Media Releases (Press, Television and Radio) - should be made early in the planning stage. The public reaction may influence the degree of public participation in later stages. New information about the progress of a project can also be disseminated this way. It may be prudent to provide elected representatives with new information prior to media releases.

- Public Meetings - also should be held early in the planning stages so citizen input does influence the project. The Engineers and Planners should listen to the ideas and preferences of citizen groups. Avoid trying to aggressively sell preconceived plans of transportation experts. The most aspect is two way flow of information.

- Separate Small Group Meetings - with community association leaders and local community groups. These can be held in the planning office and should be sit down, informal, direct discussion type of meetings without media representatives.

This avoids the sometimes hostile and emotional atmosphere which can occur at large public meetings.

- Mail-out Ballots - These are useful in determining public opinion on possible alternatives. It should be made known in advance that the ballot results will be made public to avoid citizen skepticism.

3.0 The Ontario Municipal Act

The Municipal Act Section 446 requires as follows:

446 - (1) Before passing a by-law for stopping up, altering, widening, diverting, selling or leasing a highway or for establishing or laying out a highway,

(a) notice of the proposed by-law shall be published at least once a week for four successive weeks, and in the case of a village or of a township with a population of less than 40,000 shall be posted up for at least one month in six of the most public places in the immediate neighbourhood of the highway or proposed highway; and

(b) the council or a committee of council shall hear in person or by his counsel, solicitor or agent any person who claims that his land will be prejudicially affected by the by-law and who applies to be heard.

(2) The Clerk shall give the notices upon payment by the applicant, if any, for the by-law, of the reasonable expenses to be incurred in so doing.
R.S.O. 1970, c.284, s. 446.

The Act does not specify the detail that must be provided in such notice and for the protection of the Municipality, presumably the minimum detail should be provided viz., - limits, width (E.G. - essentially 15.0 m wide with channelization at certain intersections).

Presumably in the event of a public hearing, the plans submitted at such a hearing would have to be adhered to and any major alterations to the plans would require additional notice.

Evidently many municipalities have not complied with the requirements of Section 446, and Section 447(a) is proposed to be added to protect such municipalities.

Section 447(a) provides that a judge may not order the removal of any work which was carried out, by reason only of failure of the Municipality to comply with Section 446, but does not affect the rights of persons to claim for damages against the Municipality in respect of the work carried out.

5.9 LEGAL ASPECTS

There are three basic types of legal documentation that are used.

The first priority is the statues that are passed by an act of provincial legislation. The statutes are given three readings in the Committee of the Legislature before becoming law. The most common acts that fall within the engineer's scope of work are the Municipal Act which basically provides the powers of the Municipal Council; The Local Improvement Act; the Environmental Protection Act; the Public Transportation and Highway Improvement Act; the Drainage Act; the Highway Traffic Act; the Construction Safety Act; and to some extent the Planning Act; The Public Service Works on Highways Act; Environmental Assessment Protection Act.

The second most important piece of legislation would be the regulations made pursuant to any specific act. Regulations can be thought of as a permissive legislation passed by the Lieutenant Governor in Council. This is the backup legislation on how the acts are to be reinforced or interpreted where the nature of the act requires legislation as to its use.

The third basic type of legislation is the Municipal By-Laws. Statutes of the provincial government gives the municipality certain powers which are then controlled by the municipal by-laws.

Ontario Municipal Board approval is required by a municipality on the debenture of monies for work proposed. If ratepayers object to the expenditure the Board usually calls a public hearing. The municipal engineer usually finds that he is required to be a witness at the O.M.B. hearing in support of the work. To this end it is necessary that the municipal engineer be knowledgeable as to what is expected of him.

Until recent years engineers from municipalities concentrated on engineering work, however, we are finding that this concept is rapidly changing. An engineer must not only be capable and competent in his professional qualities but he must also be capable of filling in different slots, particularly with respect to public relations, and most important must be prepared to go in the witness box on behalf of his municipality and be subject to the rigors of examination and cross-examination.

This is not always a pleasant duty to perform by being engaged in a frontal attack with a lawyer on matters of your work. It is however necessary if the municipality is prepared to advance in municipal standards and services.

5.9 (continued)

In the majority of O.M.B. hearings it is the duty of the municipality to put their case first. The lawyer for a municipality usually will put the municipal engineer in as his first witness and examine him in chief. This means that the municipal lawyer will be kind and careful and set an established pattern. He will not attack the engineer as a witness. It is most important that the municipal engineer and the solicitor for the municipality review the subject before being placed in the witness box. When the engineer's evidence is finished he is then subject to cross-examination by the other lawyer and this is when the municipal engineer must use all of his resources. Too often, as a witness, the engineer may seem to think that the examination by their own lawyer was sufficient. It is most necessary that the engineer be prepared to accept an offense against him.

If, during the examination, the engineer finds from the questions put to him that he has answered a previous question incorrectly, probably as a result of misunderstanding, then it is most important to turn to the chairman and correct the information. In other words the engineer readily sees his mistake and is prepared to admit it openly with apology.

There is one key note to giving municipal evidence -- that is "attitude". He must not develop an antagonistic attitude or an attitude of confrontation. His evidence may mean thousands of dollars to his employer. It is important that the engineer listen to the question, answer it directly and do not mislead the court. If the engineer is asked a question which is embarrasing to his department it must be answered directly and be honest about an error if it exists. There has never been a perfect witness, as of yet, and if the engineer thinks that he can accomplish this feat he will be mistaken. He must be prepared that he will make mistakes and when he does he has to admit them openly. When a witness 'digs in' and quarrels with a lawyer over a minor point it results not only in possibly losing the point but in that the chairman of the board decides that the witness's evidence may be of little value.

The following are certain ground rules that are recommended when giving evidence for a board hearing or a court, whatever the case may be:

Posture in the box is important - stand erect, do not slouch, do not slur your words and speak out. Be sure to talk in a direction where the chairman of the hearing can hear you. Quite often a lawyer questioning the engineer will walk around the courtroom and if the engineer speaks to him the chairman may not hear his comments. It is recommended that the engineer half face the chairman and half face the legal council table.

5.9 (continued)

Listen to the question. The witness usually gets into problems with his evidence because he has not listened to the question.

Make sure that the lawyer's question is understood. If it is not understood what the solicitor is getting at - and quite often lawyers mix up their own questions and don't know what they are looking for, then by all means, "ask the lawyer to repeat the question". It is surprising how many lawyers have not thought their question out carefully.

When a question can be answered in two ways then by all means phrase the answer in this method. The object is to help the court and not help to mislead them. The attitude should be maintained of co-operation and courtesy, both to the lawyer and to the chairman, no matter how angry the lawyer may become. Keep the composure and keep the courtesy. If the examiner becomes antagonistic it gives the engineer the opportunity to show courtesy and straightforwardness. Do not adopt the counter attitude of aggression. When the engineer requires a question to be repeated use the name of the examiner courteously and the question may then be answered by looking directly at the chairman. If the municipal engineer can stand up under an offensive attack, the court will usually have a greater respect for his opinion.

5.10 CONSTRUCTION, SURVEY AND INSPECTION

1.0 Introduction

Reference should be made to the following publication in particular

"Construction Manual"
Ministry of Transportation and Communications,
Ontario; Operations Division,
Construction Branch,
Downsview, Ontario.

2.0 PROJECT INSPECTION

The importance of good inspection on a construction project cannot be over emphasized. Good inspection cannot occur without a properly trained experienced inspector. He is the day to day link between the Municipality, the Contractor and the Public. This section will deal with the inspector, his duties, necessary education and training.

Dealing firstly with education and training it is the writers opinion that the Construction Inspector of today must be better educated and trained than ever before. The complexity of the work, the materials being used and the quality being demanded has never been greater. Only a well educated and trained inspector can provide the competence to perform his required duties. Basic minimum education should be at the Community College technical level. The education of the inspector cannot stop there. In our ever changing world of technology his education must be a continuing process. This can be accomplished formally through Continuing Education Programmes at the Community College Level, Courses sponsored by various Provincial Agencies or In-House training programmes. Irrespective of the formal education obtained by the Inspector it cannot and should not be considered adequate without considerable on-the-job training. The varied duties of todays Construction Inspector as outlined later demand this. There is no substitute for experience. The new Inspector should work with experienced personnel in the first instance and only with adequate experience gained and ability demonstrated should he be given a project of his own.

2.0 (continued)

As indicated above the duties of the inspector are varied. His main duty is to protect the interest of the Municipality but in carrying out this duty he must be fair to the Contractor. However, in being fair to the Contractor he must also be firm in his assistance that the works are constructed properly and in accordance with the Contract Documents. The majority of Contracts place the onus on the Contractor for complying with all applicable By-Laws, Statutes and Regulations, etc. and for carrying out the work in such a manner as will not unnecessarily or unreasonably inconvenience the public. While it is the Contractors responsibility to comply, etc., with the aforementioned it is the inspector's duty to make sure that he does and when necessary to remind him or order him. The inspector is often the last person who has the opportunity to spot mistakes or shoddy workmanship such as dimensional discrepancies, improper reinforcement placing or sizing or deterioration of soil conditions before the affected part of the work has been completed and any necessary corrections have become an expensive operation. The inspector must require that proper corrections be made in order that the work comply with requirements of the Contract. However, the inspector should be wary of making modifications himself to the original requirements and in general should refer such matters to his superivsor on those responsible for the design of the work. Most Contracts relegate the method of construction and construction equipment to the responsibility of the Contractor except in special circumstances involving complex design, particular soil conditions or restricted working area. There will be times however, when it becomes apparent to the experienced inspector that the method or equipment being used is unsuitable to the work. The inspector should advise the Contractor of this and if he is suitably experienced and confident he may suggest alternatives. The inspector should avoid running the Contractors job for him and sometimes he may be doing just that without realizing it. In extreme cases where the method or plant being used by the Contractor is dangerous to people, property or the work itself the inspector may have to order the work stopped or construction methods changed. This is a difficult decision to make and normally unless an immediately dangerous situation has arisen should not be taken without consultation with his supervisor. The inspector should try and spot substandard materials before they are incorporated in the work and make the Contractor aware of such.

2.0 (continued)

An important aspect of the inspectors work is the maintenance of good public relations on the project itself and as it relates to other affected parts of the Community. The inspector must make sure that adequate notification is given to affected local authorities and property owners and tenants of proposed interruption of services, temporary blocking of streets, detours, blasting operations, etc. He must be courteous to the public and willing to listen to their complaints and take appropriate action where necessary. By extending these courtesies he will find the people are generally more co-operative and more willing to put up with such inconveniences. His good public relations must also extend to the Contractor while still being firm with him and where necessary commanding authority. The period of inconvenience will be shortened and hence public relations improved if cleanup follows closely behind construction on streets and easements. Few Contractors will carry out cleanup and restoration promptly on their own accord as they tend to look on this work as a non-paying item and a nuisance. Good Contractors recognize the value of promptly completing this work however as stated it is a problem area. The Public on the other hand tends to judge a sewer or watermain job by what is visible above ground after the underground installations are complete. The inspector therefore must keep close watch on the timing of such work. When dealing with the Public and for that matter the Contractor the inspector must make sure that he does not make promises that cannot be carried out. He cannot commit the Contractor to work that is not required under the Contract nor can he commit the Municipality to additonal work without first obtaining prior approval.

Last but far from least the inspector must keep neat accurate records of daily progress and occurrences. His instructions to the Contractor must be well documented and explicit. Quite often he will be involved in the preparation of Payment Certificates and all such must be carefully substantiated. It can not be stressed enough the importance of good documentation of progress on a construction project. These documents will be retained and used not only at the time of construction but at completion and in the future should problems occur.

As stated at the outset of this section of the role of todays construction inspector is varied and complex. It is safe to say that properly educated and trained inspectors play one of the most important roles in any construction operation.

2.0 (continued)

CONSTRUCTION INSPECTOR'S DAILY REPORT

ROAD

DATE: April 16 1984
CONTRACTOR: Ballet Const.
STREET: First St.
JOB NO.: 9999
TIME CONTRACTOR STARTED: 7:00 AM
FROM: Second St.
CONTRACT NO.: C200-81-25
TIME CONTRACTOR FINISHED: 6:00 PM
TO: Third St.
DWG. NO.: PRB-1000
SOIL: CLAY
WEATHER & TEMPERATURE: Sunny & Warm 15°C

WORK VISITED BY

TITLE	NAME	TIME
Layout man	W. Warwick	8:** a.m.
"	R. Medley	10:00 am
	F.G. Marks	11:00 am
	A. Bernard	1:00 pm
	D. Brownlow	"
	A.L. Brownlow	"

MEN

NO.	DUTY
1	Foreman
4	Labourers

EQUIPMENT

TYPE	SIZE	OPER.
Asphalt Spread	B.G.	1
Grader	555	1
Vib. Roller	DA-50	1
Rub. Roller	BV210	1
Dozer	850	1
4 Trucks	Tan.	4

MATERIALS PAID BY THE TON

H.L.-6 Asphalt		20mm Crushed			
WEIGHT	DEL. SLIP NO.	WEIGHT	DEL. SLIP NO.	WEIGHT	DEL. SLIP NO.
40600	123456	10600	12345		
37000	654321	12300	54321		
		10000	67890		
		11500	05876		
		14700	10925		
		8700	52901		

DESCRIPTION & LOCATION OF DAY'S WORKING OPERATIONS

① Placing Curb & gutter on south side from Second St to sta. 5+75.

② Placing H.L.-6 Asphalt #3 lift from Second St. to 12+38.

DAY-WORK (INDICATE VARIATION ORDER)

V.O.-1000 - Adjust water valve @ sta 10+30 2 men @ 2 hrs each.

V.O.-1001 - Lower Existing M/H @ sta. 12+00

Foreman 2 hrs 2 lab. 2 hrs each

TESTING & TEST RESULTS

CONTRACT ITEMS COMPLETED TODAY

ITEM	DESCRIPTION	STATION	DAILY	TOTAL
1	Site clearing		0.2	0.5
2	General Excavation		1000 m³	1500 m³
6	Adjust Catchbasin	11+57	1	1 each
13	Curb & Gutter		200 m	275 m
14	50mm Crushed stone	12+38	650 m²	1250 m²
16	H.L.-6 Asphalt	12+38	300 m²	600 m²

REMARKS Etobicoke Hydro removing conflicting poles on the west side, station 10+00 to 12+50.

(use other side if additional space necessary)

TIME DETOUR SIGNS CHECKED: 9:00 a.m. 6:00 p.m.
TIME LIGHTS AND BARRICADES PLACED ON JOB: a.m. 5:30 p.m.
REPORT CHECKED BY: D Brownlow (Const. Section / Roads Section)

TIME IN 7:00 a.m. TIME OUT 6:00 p.m. TOTAL HOURS FOR DAY 10½ TOTAL HOURS ON THIS JOB 10½

Inspector's Signature

Foreman's Signature

(273) I. CONSTRUCTION SECTION FORM ENG No. 136 REVISION No 8 JAN. 73

2.0 (continued)

CONSTRUCTION INSPECTOR'S DAILY REPORT
SIDEWALK

DATE: APRIL 16 1981
CONTRACTOR: TRINETTI
STREET: FIRST St. (W/S)
JOB No: A-8300
TIME CONTRACTOR STARTED: 7:00 AM
FROM: Second St.
CONTRACT No: C200-80-7
TIME CONTRACTOR FINISHED: 6:00 PM
TO: Third St.
DWG. No: PWB-999
TOTAL LENGTH OF SIDEWALK:
LENGTH COMPLETED TO DATE:

WORK VISITED BY

Title	Name	Time
Inspector Main		
Chief Insp.	D. MacDonald	11:00 a.m.
Eng. Dept.	N. H. Orr	9:30 a.m.
	A. Bernard	2:15 p.m.
	D. Brownlow	"

MEN

No.	Duty
1	Foreman
4	Labourers

EQUIPMENT

Type	Size	Oper.
Loader	933F	1
Backhoe	580B	
Tandem Truck		1
Bomag	BW 100	

DATE JOB STARTED: APRIL 1 1981
DATE JOB COMPLETED:
WEATHER: Sunny & Warm 15°C

SUBGRADE PREPARED TODAY: From 0+00 To 1+00
SIDEWALK PLACED TODAY: From 0+00 To 1+00

CONTRACT ITEMS

Item	Description	Daily	Total
1.	Sq. M — Sidewalk 100mm Thick.	140.5	140.5
2.	Sq. M — Sidewalk 150mm Thick.	10.0	10.0
3.	Sq. M — Sidewalk 200mm Thick.	100.0	250.0
	PROVISIONAL ITEMS		
5	Supply & Place imported fill	7.5 m³	10.0 m³
7	Break out concrete	2 m²	2 m²
9	Remove curb & gutter	25 m	100 m

Item	Description	Daily	Total
	Note: Concrete supplier and ticket numbers and weights should be listed on back of report if more room is required.		

REMARKS

Concrete used: - 25 MPA - 32.5 kg/m³

1st load - 0+00 - 0+40 - 5.6% AIR - C.B.M # 123456
2nd load - 0+40 - 0+70 - 6% AIR - C.B.M. # 654321
3rd load - 0+70 - 1+00 - 6% AIR - C.B.M. # 222222

(Use other side if additional space necessary)

REPORT CHECKED BY
Chief Construction Inspector

TIME IN 7:00 A.M. TIME OUT 6:00 P.M. TOTAL HOURS FOR DAY 10½ TOTAL HOURS ON THIS JOB 10½

Inspector's Signature

Foreman's Signature

(E. 44)

FORM, ENG. No. 29 REV. No. 3 MARCH 69

1. CONSTRUCTION SECTION

2.0 (continued)

ENGINEERING DEPARTMENT
VARIATION ORDER

№ 3735

CONTRACT: 81-99
CONTRACTOR: Ace Const. Co.
JOB No. A-7500
STREET: First St.
UTILITY STORM
FROM: Second Ave. TO: Third Pl.
DATE Apr. 16 1981

Relay 300mm Storm sewer at Station 7+77 Due to confliction with existing Bell Telephone manhole and underground ducts.

Basis of Payment: Time and Materials.

ORDER CHECKED BY:
SIGNED:
TITLE:
INSPECTOR

FORM ENG. NO. 84 REVISION NO. 2 DEC. 74
2-95

2.0 (continued)

Performance and End of Maintenance Inspection - Roads Acceptance Criteria

After the construction is completed, A Performance Inspection of the road works is carried out.

Similarly, one month prior to the end of the maintenance period for roads constructed under Subdivision Agreement, or constructed by a contractor on behalf of the City, an End of Maintenance Inspection is carried out.

In each case, the inspection shall record particularly the following points:

(a) PAVEMENT

(1) Shall note size and location of any potholes.
(2) Shall note extent and location of settlement depressions and frost heaving.
(3) Shall note size and location of crazing and surface cracking of the asphalt, since this usually indicates sub-grade failure.
(4) Shall note extent and location of asphalt ravelling.
(5) Shall note location of opened joints in the asphalt course at construction joints.
(6) Shall note extent and location of surface ponding conditions.
(7) Shall note that pavement is constructed with a proper crossfall and an adequate crown.
(8) Shall note that construction is to correct dimensions.

(b) CURBS AND GUTTERS

(1) Shall note that concrete is true and smooth in surface texture.
(2) Shall note extent and location of scaling, spalling or pitting.
(3) Shall note line and level and shall record all instance of curbs with more than 13 mm settlement at joints.
(4) Shall note locations of extruding expansion joints.
(5) Shall note locations of structural failure and settlement cracks.
(6) Shall note non-standard driveway cut-outs.
(7) Shall note all damage to pavement, curbs, catchbasins, caused by any agency.

2.0 (continued)

(c) CATCHBASINS AND UTILITY APPURTENANCES

(1) Shall record all cases where catchbasins are wrongly located to intercept road drainage.
(2) Shall note all cases where valve-boxes, manholes, etc. are not to the finished pavement grade.
(3) Shall note all cases where frames and covers do not sit firm and true.
(4) Shall note all damaged covers, valve boxes, etc.
(5) Shall record any road materials noted in CB's and manholes.

Further to the above noted items, asphalt and concrete coring may be carried out as well as Benkelman Beam Testing, prior to the end of the maintenance period.

The extent and the responsibility for the required repairs or replacements shall be evaluated and the contractor advised accordingly.

Performance and End of Maintenance
Sidewalk Acceptance Criteria

After the sidewalk construction is completed, a Performance Inspection is carried out. Similarly, at the end of the maintenance period, an End of Maintenance Inspection shall be carried out.

In each case, the inspection shall note particularly the following points:-

(1) All bays containing one or more cracks in excess of 1.5 mm in width shall be reported.

(2) All bays having a displacement from the original grade in excess of 6 mm shall be reported.

(3) All locations where sidewalks obstruct drainage, shall be reported.

(4) All locations where expansion joint material project above the surface of the sidewalk, shall be reported.

(5) Spalling of the concrete surface shall be noted and the extent of the spalling shall be indicated.

2.0 (continued)

(6) Concrete cores may be taken (at the discretion of the City Engineer) every 150 m and the thickness of the concrete to the nearest 3 mm shall be recorded.

(7) Any other unusual construction or variations from the plans and specifications (e.g. broken corners) shall be recorded.

The extent and the responsibility for the required repairs or replacements shall be evaluated and the contractor advised accordingly.

3.0 PROJECT SUPERVISION

In the previous section the importance of good inspection was dealt with. Similarly this section will deal with good supervision which again cannot occur without properly trained, educated and experienced supervisors. The supervisor and supervision discussed here is primarily based in the construction project area. However, the qualities and requirements of good supervision are common irrespective of the area of concern and can be considered applicable in any area of supervision involvement.

What exactly is the function of the project supervisor? The answer to this is somewhat dependent on the type and size of the project being considered. However, the skills the supervisor must have and show are relatively common to all projects. For an example let us consider a Municipal Construction Contract. Once an award of this contract is made the supervisor assumes direction of the work as the authorized representative of the Municipality. It is the supervisor's function to inspect and approve all operations and finished products related to the Contract and to accept or reject the work on behalf of the Municipality. He usually has and is responsible for a number of technical staff to assist in his duties. They could include survey crews, inspectors, accounting and office staff, weighmen, checkers and quality control personnel. His duties are varied and his responsibility great.

Perhaps the best way to begin a section dealing with supervision is to define it. One of the simplist and perhaps the most meaningful definitions in the writers view is that a supervisor is anyone who has formal authority and responsibility over other workers. One of his primary concerns is to cause his individual workers to work co-operatively and collectively, not only as individuals to create the efficiency necessary to supply the required service. While doing this the supervisor must also have due regard for the personal interests of the employees.

Considering the above one often assumes rightly or wrongly that to be a good supervisor one must be jack-of-all-trades including a human behaviour scientist. The supervisor obviously must have a sound technical background and knowledge in his special area of concern. He is often expected to know as much or more about the technical details of his work than anyone else. It is not of prime importance that he be able to do the job of any of his subordinates better than the subordinate. However, it is of importance that he know the rudiments of their work, to be able to discuss with them their problems intelligently and to earn

3.0 (continued)

their confidence and respect. He must have knowledge of and skill in applying fundamental concepts in such areas as (1) human behaviour, (2) organization, (3) leadership and (4) the basic supervisory functions of planning, organizing, directing and controlling. He must also be skilled in public relations as much of his work involves public contact. He must be able to organize not only his subordinates but himself also to meet schedule changes, major breakdown, etc. He must be capable of solving technical problems which constantly confront him and his subordinates and which are often not only complex but also unpleasant. He must not waste his subordinates talents or resources nor must he forget the economic cost of his staff. Perhaps the definition of jack-of-all-trades is not too far off the mark.

The education of a supervisor must be a continuing process. He must not only be soundly educated and trained technically in his area of concern he must also know how to effectively communicate and deal with people and obtain from them their best efforts and performance. Everywhere there is evidence of increasing demands on the knowledge, skill and integrity of the supervisor. The relentlessly increasing knowledge regarding the technical, economic and social aspects of the supervisors work is a force of major consequence that the supervisor must deal with. Even as the supervisor goes about his job on a day to day basis his job is changing. New concepts and specialized techniques related to his work are constantly being developed and improved. He must continually supplement and reinforce his education and experience. He can not supervise effectively by following prevailing practice and his own past experience. The effective supervisor cannot remain static.

I briefly mentioned earlier the importance of communications. The importance of such cannot be overstressed with relation to the supervisors work. He is required to communicate effectively with his staff, contractors, the public and his superiors. Much has been written on the art of communication and it is not the writers intention to develop this area only to stress its importance, whether written or oral.

As has been pointed out the effective supervisor must be a many faceted being. Good supervisors are one of the most important cogs in any smooth running operation whether it be Municipal, or otherwise. They are normally in the "front lines" so to speak and can make or break an operation. They should be selected with care, properly trained and experienced on a continuing intelligent basis.

4.0 PAYMENTS AND COST ACCOUNTING

PAYMENTS

There are many different types of Contracts involving various methods of payment. To name a few of the most common - Lump Sum Contracts - Unit Price Contracts - Competitive Bid Contracts - Combined Lump Sum and Unit Price Contracts - Negotiated Contracts. A Municipality normally uses either the Competitively Bid Unit Price or Lump Sum Contracts. In either case as the work progresses, the Contractor is usually entitled to partial payments which are made on the Construction Supervision's estimates of the amount of work completed or as invoiced by the Contractor and substantiated by the Construction Supervisor. On Lump Sum Contracts the estimate will be expressed as a percentage of the total work to be done. For Unit Price Contracts the Construction Supervisor will measure the quantities of work completed under each item of the Contract and these will indicate the degree of completion. In arriving at the amount of partial payment, consideration may be given to the amount of material delivered to the site and work done by the Contractor at other assembly points as well as the completed work on the site.

Partial payments are usually made by the Municipality each month. The payment covers work completed the previous month and are normally due two weeks after month end. These payments generally cover 80-85 percent of the actual cost of the work completed. Fifteen percent of the monies due must be held as holdback by the Municipality in accordance with the Mechanics Lien Act. This fifteen percent is to provide protection to the Municipality against such claims or charges as failure of the Contractor to reimburse Sub-contractors, Material Dealers or Employees. It must be released to the Contractors 37 days following substantial completion of the work and proper certifications from the Contractor that there are no outstanding claims against the work. The above has been written in a very simplified manner and the reader is referred to the Mechanics Lien Act, Statutes of Ontario for detailed proceedings.

In addition to the 15 percent it is not uncommon for a Municipality to retain an additional 5 percent for a period of 12 months after certification of completion of the work as a guarantee against faulty workmanship.

The final payment to the Contractor will include all retained percentages. Before the final payment is made however, the Construction Supervisor will make a final inspection of the work as a condition of formal acceptance by the Municipality. The final inspection shall determine any omitted portions of the work, all defective work to be

4.0 (continued)

corrected and any deductions to be made in the Contract amount for defective work to be accepted in lieu of its removal and replacement. The Contractor should provide the Municipality certification that the final quantities, deductions, payment, etc. are acceptable and recognized as such and a release from any claims, liens, etc. against the work.

COST ACCOUNTING

The Accounting methods and procedure to be used on a construction project are of major importance. Rather than dealing with specific accounting methods and procedures which are readily available and varied this section will deal more generally with the ground rules of good accounting. The prime rule of any accounting procedure is that the records must be complete and accurate. A variety of practical alternatives exist in recording information. In addition to completeness and accuracy the quality of clarity and meaningfulness are important. A reader of the records should be able to discern quickly where items have been entered, what they are and their dollar effect. One important indication of the quality and effectiveness of a recording system is the ease with which updated cost data can be taken from such. It should be remembered that these records will normally be subjected to possibly both internal and external auditing.

Information to be recorded should be entered promptly so that the records are up to date at all times. Economy and efficiency in recording requires that a daily routine be established for entering information of all kinds and with no exceptions. The routines established should be as simple as possible and easily explained so the system will not be disrupted by the absence of the person normally doing the work and can readily be taken over by a replacement. Records should be designed for expansion so that increases in volumes of any item or transaction can be handled without disruption of the system. The daily records should provide a complete record of progress of the work.

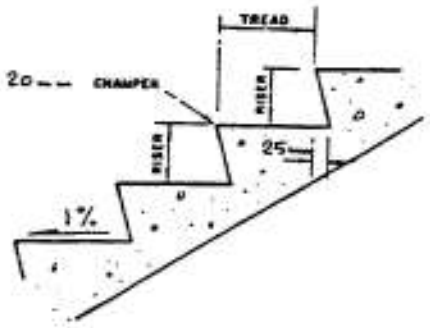
Dealing more specifically with the construction project the accounting and recording system to be used must be established before the project begins. It cannot be successfully put together after the work is underway. Also important is that a clear and systematic method of taking progress measurements and filing the progress calculations must be established at the commencement of the work in order to obtain correct progress quantities during the job and also

4.0 (continued)

to aid in compiling accurate final quantities. It should always be kept in mind that in the process of construction many essential measurements can be obtained only at a certain time. Close co-ordination between the Supervisory staff and the Contractor is a decisive factor in the timely acquisition of the necessary measurements. The Ministry of Transportation and Communications, Province of Ontario has developed excellent systems for recording and processing such data. The reader is referred to their Construction Manual for details of these systems.

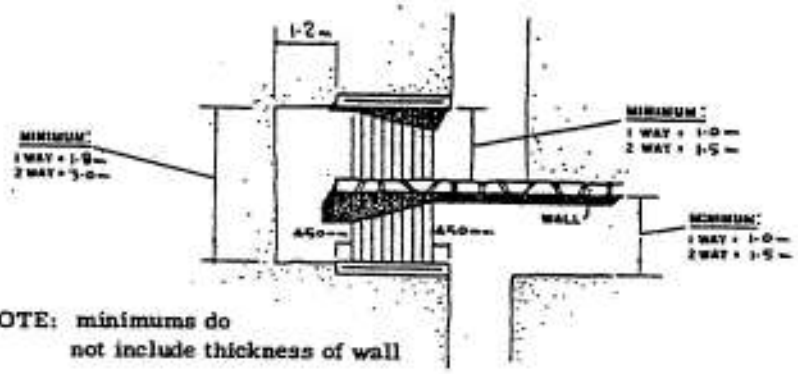
2.0 (continued)

OUTDOOR STEPS RULES-OF-THUMB



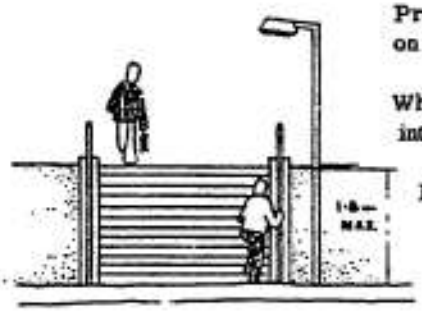
Acceptable Combinations in mm	
Risers	Treads
178	280 290
165	280, 290, 305, 320
152	280, 290, 305, 320, 330, 340, 355
140	280, 290, 205, 320, 330
127	280, 290, 305

OUTDOOR LANDINGS



*NOTE: minimums do not include thickness of wall

HEIGHT BETWEEN LANDINGS



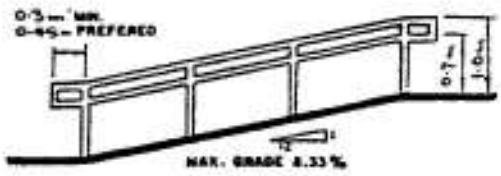
Provide 50 lux lighting on stair and landing areas.

Where total grade change exceeds 1-8 — intermediate landings are necessary.

Provide landing at 1-2 — intervals.

3.0 Handrails

HANDRAILS FOR RAMPS

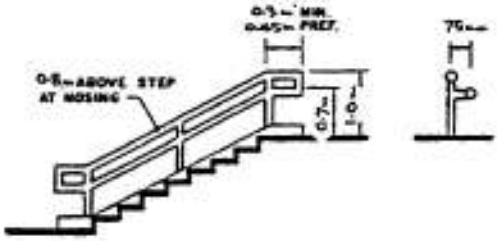


1·0m is the most comfortable height for handrails on ramps.

A second handrail, useful to people in wheelchairs and children should be placed at 0·7m.

Handrails should extend a minimum 0·3m beyond both ends of a ramp.

HANDRAILS FOR STAIRWAYS

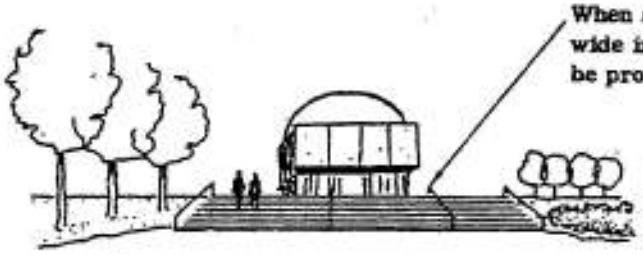


1·0m is the most comfortable height for railings at both ends of stairways. 0·8m is the accepted height on stairways.

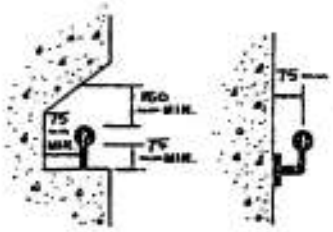
A second handrail, useful to children should be placed at 0·7m.

Handrails should extend a minimum of 0·3m beyond stairways.

HANDRAILS FOR EXTRA-WIDE STAIRWAYS



When steps become exceptionally wide intermediate handrails should be provided at 6.0 to 9.0m intervals.



All edges should be chamfered or rounded off, also keep wall surfaces smooth to limit cuts and scrapes.

HANDRAIL CROSS-SECTIONS

Do not allow hands to use natural opposing grip; usually because rail is too wide.



Allow hands to use natural, opposing grip.

5.4 PUBLIC TRANSIT FACILITIES INDEX

1.0 Buses

2.0 Bus Bays

3.0 Bus Loops

4.0 Bus Lanes

5.0 Bus Passenger Shelters

6.0 Subways and Commuter Trains

5.4 PUBLIC TRANSIT FACILITIES

1.0 Buses

The most common form of public transit facility which the urban engineer will become involved with is the bus.

Bus routes will be of prime concern both in the selection of of the pavement design and the road geometrics. The looping of a bus route on local residential streets will result in premature pavement failure and higher maintenance costs unless the original pavement design is upgraded to suit. Also road bends and corner radii may require increasing to avoid vehicle conflicts.

2.0 Bus Bays

On major collector and arterial streets it may be advisable to provide bus bays, particularly at locations where a bus makes frequent stops, or is normally delayed by the volume of passenger movement on and off the bus. Bus bays will assist the through traffic movement and reduce the accident prone weaving movement on four lane roads.

The location of the bus bay must be reviewed with the transit authority to ensure that it suits their operations. It will also require a careful site review to ensure that conflicts with driveways or other topographic features are minimized.

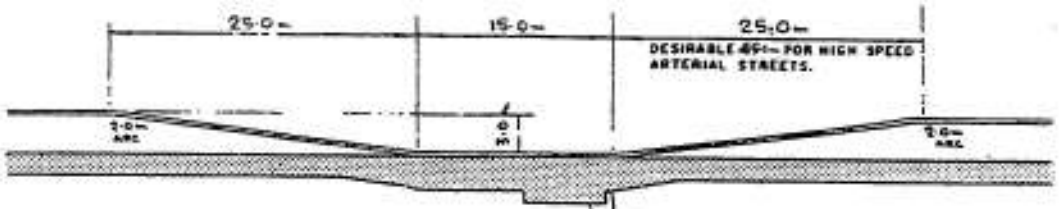
A near side bus bay, particularly where the bus usage is infrequent and of short duration, may be used to advantage as a right turn lane.

There are other minor advantages in the use of bus bays in that they may be used for deliveries to adjacent stores in the time period when transit vehicles are not in operation. They may also be used as pick up and drop off areas for taxis and cars.

Care in the design must be taken allowing long enough entrances and exists to discourage serious disruption of traffic movement in the adjacent lanes. Design characteristics of the bus bay are shown on the following page.

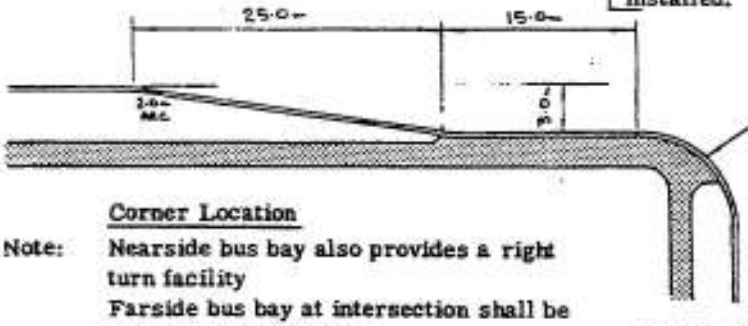
2.0 (continued)

BUS BAYS



Remote from Intersection.

It is recommended to widen the sidewalk to 24" at the bus stop. A further widening should be provided if a bus shelter will be installed.

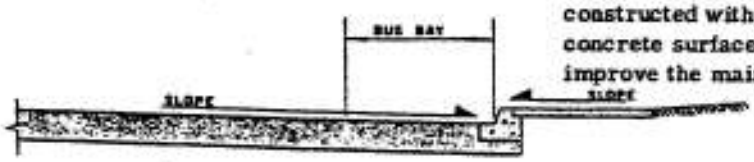


radius as per design plans (30" min.)

Corner Location

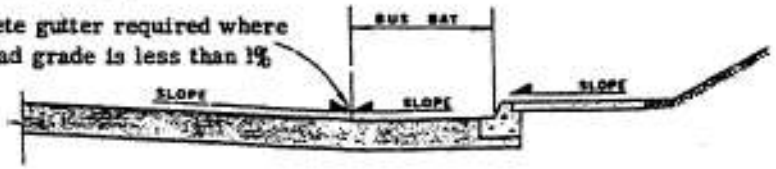
Note: Nearside bus bay also provides a right turn facility
Farside bus bay at intersection shall be opposite hand to above

Note: On concrete base pavements the bus bays may be constructed with an exposed concrete surface, which may improve the maintenance of bus stop area.



Recommended Section

concrete gutter required where the road grade is less than 1%



Alternate Section

used where existing topography cannot be altered practically.

3.0 Bus Loops

The Bus Loop in its simplest definition is a form of construction for the effcient and economical handling of passengers and buses at transfer and end of route locations. The construction and operation of Bus Loops is of prime importance and most beneficial at high use transfer points. There are a number of advantages to the use of Bus Loops besides the increased efficiency to the transit operation that can result. With their use passengers are able to complete their transfers in comparative safety not being subjected to the hazards associated with adjacent street traffic nor interfering with this traffic. Bus Loops free inportant curb space and operational lanes thereby reducing congestion which normally would occur with the buses standing at the curb to load and discharge passengers and maintain schedules.

The most appropriate location for the Bus Loop is dependent upon a detailed analysis of headways during peak hour operation on all the routes proposed to be serviced by the Loop. Normally the Loop is located on the route with the shortest headway.

In designing Bus Loops the most directly related consideration is the operating characteristics of the transit vehicles using the facility. The Loop exit drive should always be located on the shortest headway route. It should also be located as far as possible from an adjacent signalized intersection. Schematic examples of a Single and Double loop are shown on the following page.

The simplest form is the Single Loop with a width in the 4.5 to 6.0 m range. The minimum length of the Loop must be related to the turning radii of the transit vehicles being used and more importantly to the combined lengths of the maximum number of transit vehicles that will be using the Loop at any one time. To take full advantage of the additional length required because of designing for maximum usage the loading platform must be located at the exit end of the loop. In single loop design where required storage conflicts with scheduled operations due to multiplicity of route transfers it may be designed with an additional inner lane used for passing. Under this arrangement the loading platform must be designed to accommodate the number of transit vehicles required to use it at any one time. This length must also include maneuver space.

There are a number of variations of the double loop usually involving a centre loading platform. The physical design again giving consideration to the basics such as characteristics of user vehicles, storage requirements and maneuverability. As shown in the schematic drawing given certain conditions the concentric Loops can operate in a counterclockwise fashion on each Loop adjacent to the central loading platform.

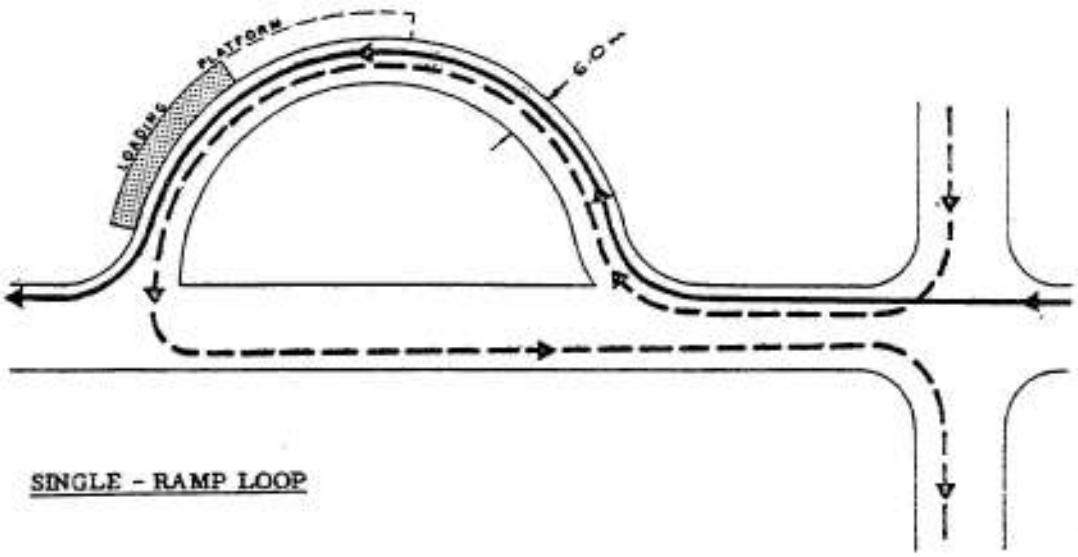
3.0 (continued)

Loops are initially expensive due to property and construction costs and as in all engineering endeavours their use should be economically justified. The location of the Loop must avoid conflicts with the area residence, and to reduce the property requirements. The Loops are often designed to minimum standards.

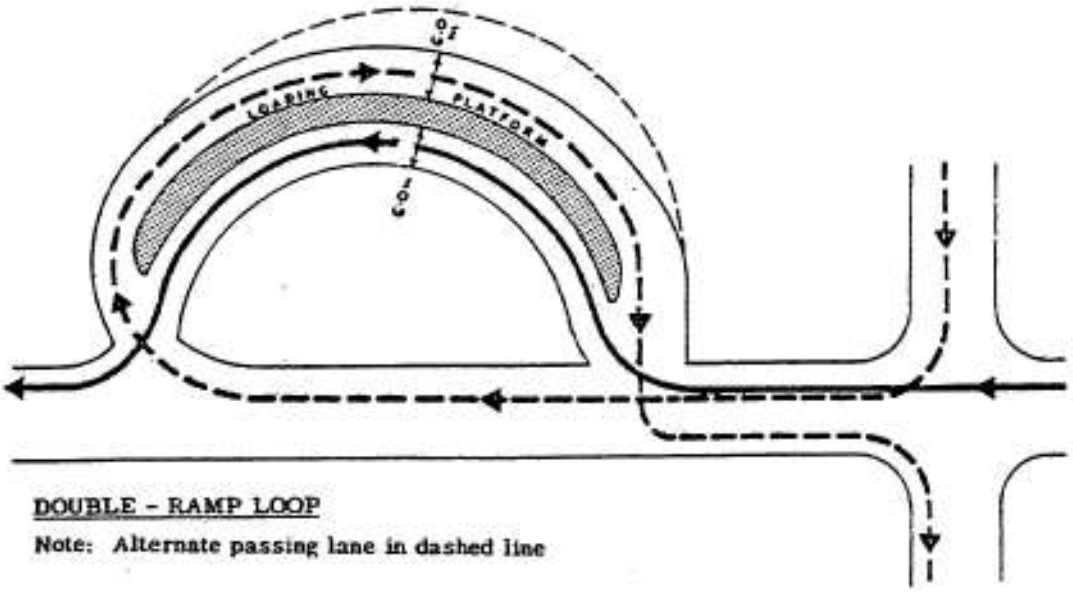
Examples outlining typical minimum requirements for a 12.2 m bus are provided.

3.0 (continued)

BUS LOOPS



SINGLE - RAMP LOOP



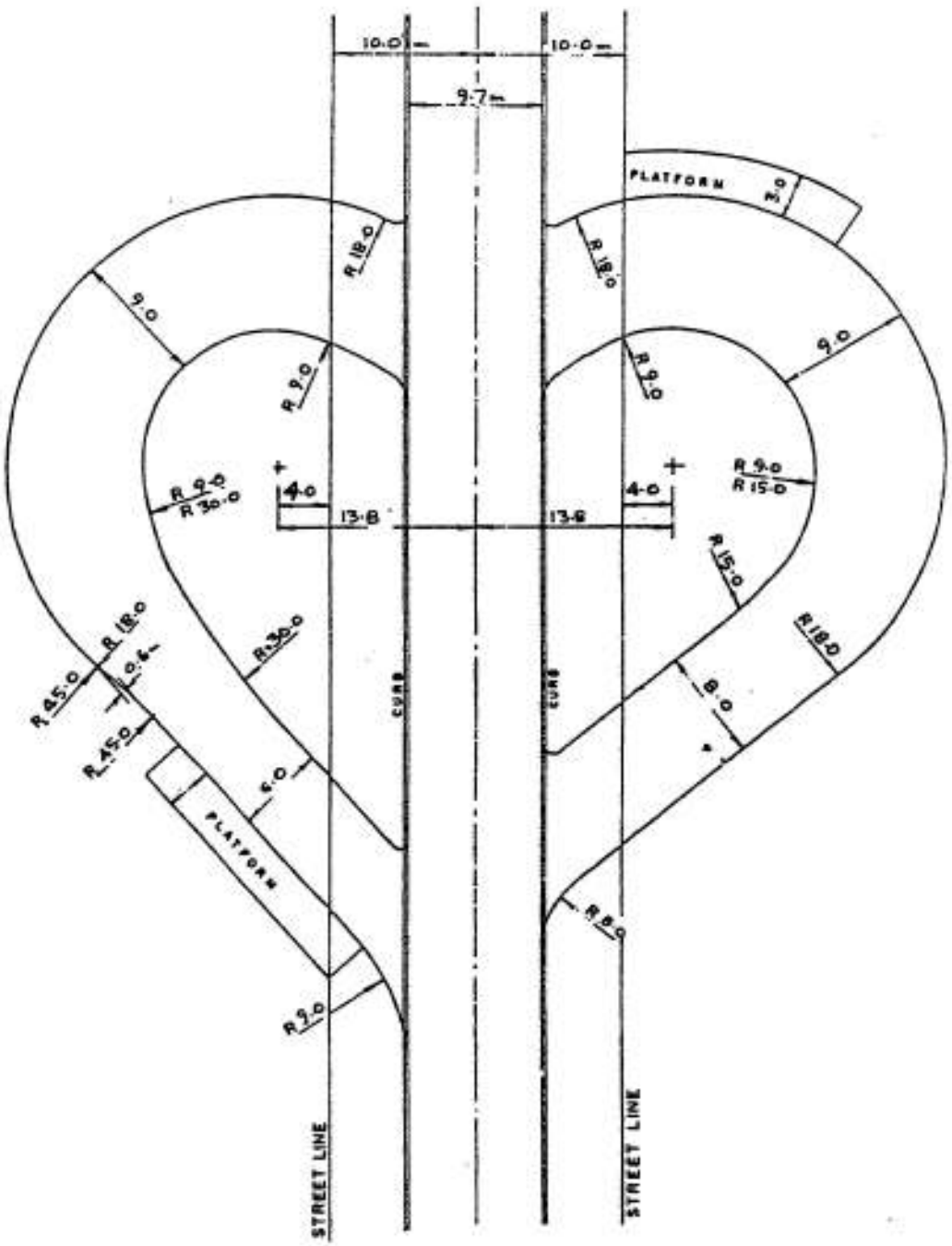
DOUBLE - RAMP LOOP

Note: Alternate passing lane in dashed line

3.0 (continued)

BUS LOOPS

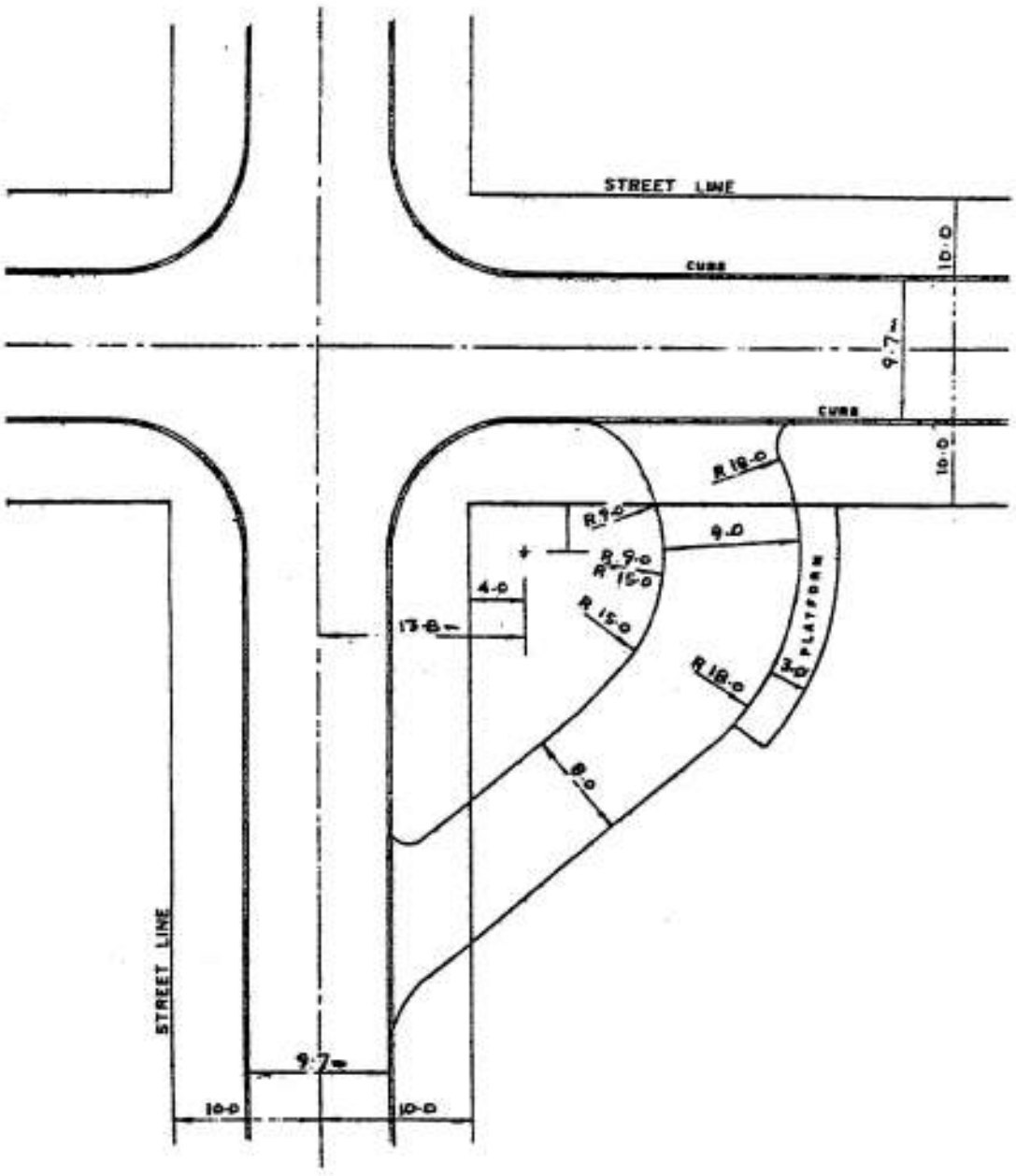
12 m Bus



3.0 (continued)

BUS LOOPS

12 m Bus



4.0 Bus Lanes

A Bus Lane in its most simple form is one or possibly two lanes of a municipal street reserved for buses only. The use of a Bus Lane is a recognized method of providing additional capacity in the peak direction on streets with heavy bus volumes. There are a number of variations in the actual use of the bus lane. In some instances, taxis, carpools and emergency vehicles or some combination thereof are also allowed to use the Bus Lane. Whereas some are exclusively used for buses. The lanes may be exclusively used for such traffic or only at certain times of the day depending on the circumstances. Their use is dependent on their ability to move at least as many people as a normal traffic lane would. They should not interfere with normal traffic movements nor access to street frontage where existant. They are generally considered a low cost technique of providing additional street capacity. They are usually created by the passing of appropriate By-Laws, proper signing and barriers where necessary and most important proper and consistant enforcement.

The prime objective of the Bus Lane is to provide a more efficient bus service by improving bus speeds through congested areas. Hopefully this improved service will attract increased ridership thereby reducing the number of cars using the road. Although it does not always provide increased capacity for other road users it can help reduce congestion by separating from the rest of traffic the slower moving and frequently stopping buses. The obvious disadvantage of the Bus Lane is it reduces or usually decreases the number of moving lanes available to the rest of traffic. If the Bus Lane is immediately adjacent to the curb or street frontage it makes right hand turns difficult and street access loading zones virtually impossible during the hours of the Bus Lane's operation. If a centre lane is used for the Bus Lane it can create a hazardous situation for the pedestrian in the loading zones. While use of the centre lane facilitates right turn movements if used on a two way street will cause problems or make difficult the left turn movement.

Bus Lanes are usually carefully delineated by overhead signs or signs or utility poles immediately adjacent to the lane and with special pavement markings including printed messages. If Bus Lanes are located in the centre of the street pedestrian safety islands are usually constructed carefully delineated to provide some safety to the waiting passengers.

4.0 (continued)

Another variation of the Bus Lane is the Contraflow Bus Lane. As the same implies it is the use of an exclusive lane for Buses in the opposite direction to normal flow. In many cases a typical divided street will have a peak traffic periods a congested traffic condition in one direction with limited flow in the opposite direction. The Contraflow Lane is temporarily placed on the low density side opposing normal traffic flow. Effective temporary signing, barriers and signalization must be used along with adequate enforcement during the hours of operation to ensure non interference between the bus lane and normal traffic flow.

Whatever variation of the bus lane is used it will only be successful with constant and consistent police enforcement and a good program to acquaint the motoring public with its use.

5.0 Bus Passenger Shelters

Bus shelters are fast becoming an integral part of the urban bus transit picture. A combination of the inflationary aspects of the economy; the increased crowding of our streets and highways and the dwindling accessible parking and cost thereof have caused more people to look to the urban bus system as a major means of their daily transportation. With the swing to bus transportation more public demands, particularly in areas of severe climate, are being made for some type of shelter at bus stops. Particularly at the more heavily used transfer points and in locations of high bus user generators such as hospitals and shopping plazas. Also no longer is the public satisfied with makeshift shelters. Attractive, functional and well maintained facilities are required.

There are a number of factors used in determining the need for such shelters including the most obvious the number of users benefited; also the average duration of wait between buses; severity of climate; availability of other nearby shelter and by no means the least important public relations value.

The siting or location for each shelter should be carefully studied including the geometrics of the bus stopping locations; the distances from the shelter to waiting buses and possible conflicts with other activities in the immediate area. Even the prevailing winds should be considered in site location if possible. Some diligent engineering at this particular phase can save a multitude of problems at a later date which can be expensive and sometimes impossible to resolve.

5.0 (continued)

The shelters themselves must be acceptable to the public. They are an important source of public relations or advertising to the transit company. If they are readily accepted by the public and do provide comfortable waiting they can attract increased ridership. They should be well constructed and preferably portable in order that they can be readily dismantled and relocated if for example a major route change occurs eliminating the necessity of a shelter at a specific location. One of the most functional types are those made out of a light metal framework and metal panelling incorporating sufficient area of unbreakable glass or plexiglass in their design.

There are a number of small yet important points to be considered in the overall functional design of bus shelters. One of the most important is good visibility. The two end walls and front should contain large window areas to provide ample view of route approaches and as a form of security for the user. Adequate lighting should be provided both interior and exterior. This lighting should be protected either through height or security meshing. Access to the shelter should be adequate to provide easy exist and entry, light and visibility. If seats are to be provided they should be located on the side and near walls and in most cases should be minimized as they are often subject of vandalism and other uses than the intended purposes. Similarly washrooms should be avoided unless at terminal or main transfer stations where constant supervision is available. Easily accessible refuse containers should be located in close proximity to the shelter. The colour of the shelters should be closely considered. There are two divergent schools of thought on this. Some felt the shelters should be colour coded the same as the transit system's buses; while others feel they should blend into the surrounding neighbourhood.

Policing and maintenance of bus shelters are important factors to take into consideration. Bus shelters tend to be targets of vandalism and as such their design should be as simple and functional as possible leaving little to attract vandals. Periodic checks by Police on their normal rounds is also of assistance. The shelters cannot simply be constructed and forgotten. They must be systematically cleaned and maintained. This, in itself, tends to lessen attention by vandals. As stated earlier, they are a form of advertisement for the transit operation and their condition can influence people's thoughts towards using the system.

5.0 (continued)

BUS PASSENGER SHELTER EVALUATION FACTORS

FACTOR	RELATIVE IMPORTANCE IN PERCENTAGE OF TOTAL POINTS	
A. PHYSICAL ASPECTS		
Exposure to weather conditions	20%	
Lighting	5%	
Obstruction to visibility	5%	
All physical aspects		30%
B. PASSENGER USAGE (Determine by T.T.C. on-passenger count)		
Low on-passenger count	10%	
Medium on-passenger count	20%	
High on-passenger count	30%	
Total usage		30%
C. LENGTH OF WAIT		
During rush hours	5%	
During off-peak	15%	
Total length of wait		20%
D. AREA BEING SERVED		20%
ALL FACTORS		100%

5.0 (continued)

WARRANTS FOR
BUS PASSENGER SHELTER EVALUATION

LOCATION _____ and _____

CORNER _____ ROUTE _____ DIRECTION _____

A. PHYSICAL ASPECTS	
Exposure to weather conditions	Minimum 5 / Average 10 / Full 20
Lighting	Poor 0 / Good 5
Obstacle to visibility	Yes 0 / No 5
B. PASSENGER USAGE (Determined by T.T.C. on-passenger count)	Low 10 / Average 20 / High 30 — TOTAL ON COUNT
C. PASSENGER WAITING TIME (Half of headway)	

Rush Hour		Off-peak	
Less than 2.0	0	Less than 4.0	0
2.1 - 4.0	1	4.1 - 8.0	3
4.1 - 6.0	2	8.1 -12.0	6
6.1 - 8.0	3	12.1 -16.0	09
8.1 -10.0	4	16.1 -20.0	12
10.1 - UP	5	20.1- UP	15

D. AREA BEING SERVICED	
Undeveloped	0
Commercial-Industrial	5
Residential-High density	10
Transfer point	20
TOTAL OF A to D	ORDER OF PRIORITY ON PROGRAMME

5.0 (continued)

SPECIFICATIONS FOR BUS PASSENGER SHELTERS

Size 1.2 m x 3.65 m x 2.1 m (small)
Size 2.4 m x 3.65 m x 2.1 m (large)

(1) Shelters to be constructed to the size and design illustrated.

(2) Shelters to be constructed of anodized aluminum and glass except for the use of 13 mm plywood in accordance with item 3.

(3) All lower panels to consist of two sheets of 0.032 gauge minimum aluminum with reinforcing sheet of 13 mm plywood between the panels.

(4) All upper panels to consist of a single sheet of .032 gauge minimum aluminum.

(5) All frame members and base channels to be .050 gauge aluminum and corners to be .062 gauge aluminum.

(6) All upper and lower panels to be anodized and finished in blue enamel.

(7) All glass used must be tempered and have a minimum thickness of 5.5 mm.

(8) Glazing - aluminum angle glazing with self tapping stainless steel screws, for easy accessibility - plastic glazing stops not acceptable.

(9) Roof - 60 mm 0.17 kg corrugated white fibre glass fastened with tapping screws at one foot centres to aluminum under-structure, with sufficient supports in centre to give minimum load capacity of 195 kg/m^2 or 0.17 kg pen type white fibre glass with rafters @ 400 mm centres. Aluminum rafter ties are to be installed along the length of the shelter - two sections above the wall on each small shelter with added sections in centre of each large shelter, riveted to each rafter and to the outside frame. In addition, the rafter ties and rafters are to be bolted at the four corners of each shelter with 63. mm x 6.4 mm cadium plated steel bolts. The roof must be manufactured with a full perimeter flange to attach to the wall panels, brackets not acceptable. Roof pitch must be 50 mm minimum and have overhang of 150 mm on all sides.

5.0 (continued)

(10) At the base to securely hold the shelter in place a 25 mm x 50 mm base channel to be supplied for the full perimeter of the shelter, excepting only at the entrance. Brackets or outside angles not acceptable.

(11) To securely hold the panels together, aluminium "I" shaped extrusions to be supplied.

(12) Tenders must include drawings illustrating all construction detail including dimensions and gauge of aluminium used.

(13) Delivery date _____ weeks.

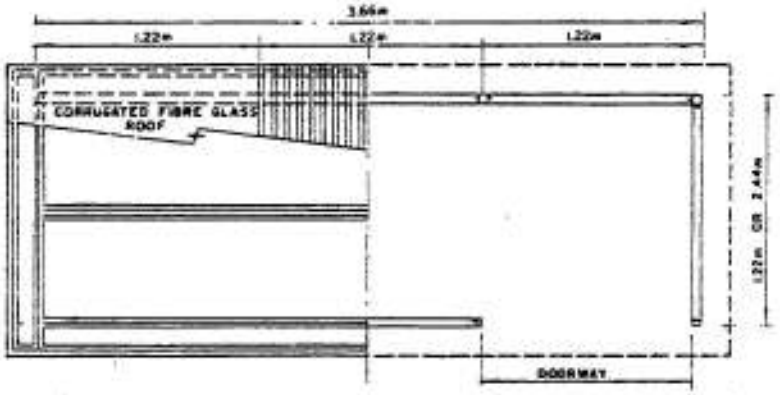
(14) The concrete pads for shelters will be constructed by the City of Etobicoke, Works Department, Roads Division.

Warranty:

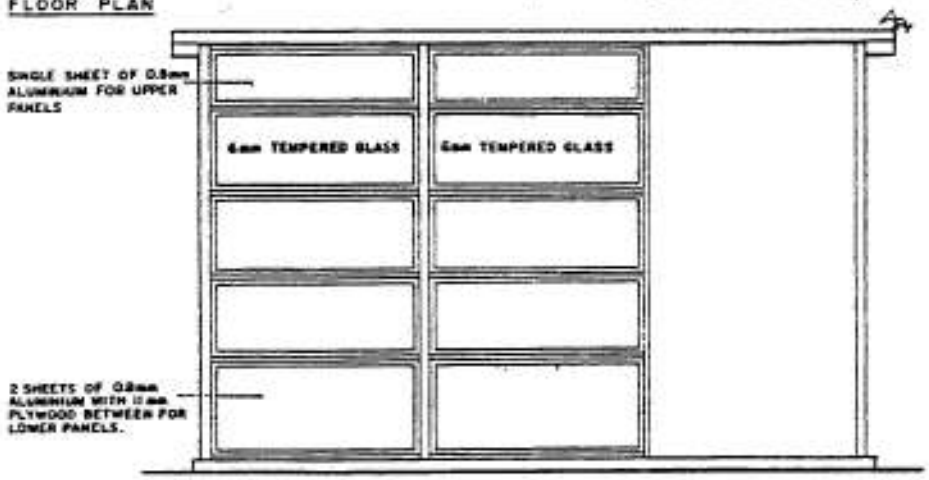
The Manufacturer is required to repair or replace, as necessary, free of charge any items found to be defective due to faulty manufacture or workmanship, within two years of the supply of any individual shelter.

5.0 (continued)

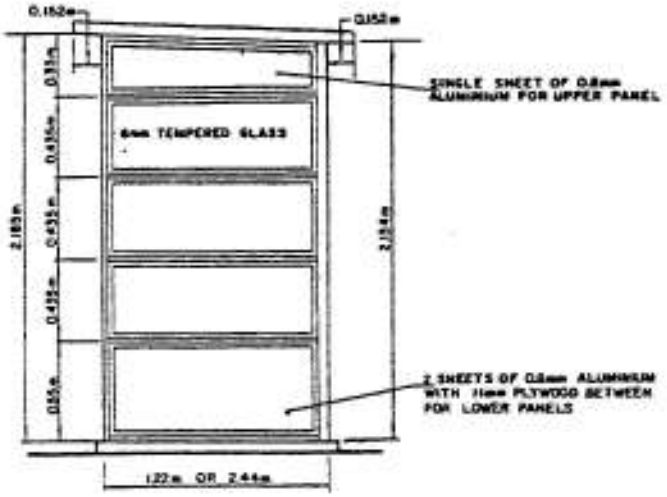
BUS PASSENGER SHELTER



FLOOR PLAN



FRONT ELEVATION



SIDE ELEVATION

6.0 Subways and Commutor Trains

The subway or commutor trains station will increase the vehicular movement in its area. Consideration should be given to the road pattern, the road widths, and particular attention to vehicle turning movements and affected intersections.

Parking should be provided close to the station with long term provisions. Also "Kiss and Ride" facilities (area for pick up and drop off for car passengers) should be considered particularly in suburban areas.

When the station acts as a transfer point for buses, adequate looping facilities should be provided.

Particularly careful consideration should be given to all these facilities at the terminal point of subway or commutor train service, since these often develop as the most congested points for both other transit facilities and for cars.

5.5 BICYCLE FACILITIES INDEX

1.0 On Public Road Allowances

1.1 General Guidelines

2.0 In Recreation Areas

3.0 Bicycle Path Design Standard

3.1 Alignment

3.2 Grades

3.3 Width and Clearance

3.4 Surfaces and Construction

3.5 Drainage

3.6 Other Services

3.7 References

3.8 Typical Sections

BICYCLE FACILITIES

1.0 On Public Road Allowances

There are three basic methods of dealing with bicycle traffic on public road allowances:

(a) Outright banishment of bicycle traffic;

(b) The part-time use of roads exclusively for bicycle and other non-motorized traffic;

(c) Sharing of facilities between all types of traffic.

Alternative (a), Banishment, is the logical step to take in certain instances such as urban controlled access freeways and other high-speed, high-volume roadways where the danger to cyclists is high.

Alternative (b), the Part-Time Use of Roadways exclusively for Bicycles and other Non-Motorized Traffic, is preferable from a safety viewpoint. There may, however, be vigorous opposition from those people who reside on these streets and from businesses that would be affected by the closure of these roadways to automotive traffic. Even businesses in districts served adequately by back-lanes for motor vehicle access may depend upon automobile traffic for drive-in trade. However, precedents have been set for the banning of motor traffic from public streets on a fulltime basis; i.e. pedestrian and shopping malls, and some municipalities have banned motor vehicles from certain streets on specified days for cyclists.

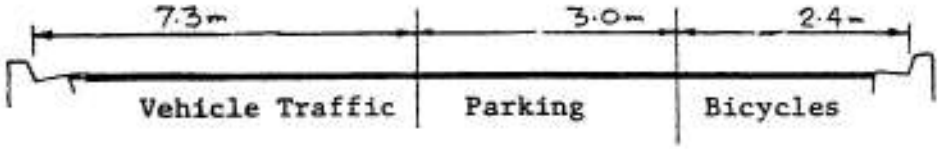
Alternative (c), Shared Facilities, would seem to be the most widely used and therefore the most acceptable method. Since bicycles are not entirely compatible with either motor vehicles or pedestrians, there are safety problems in the sharing of road facilities. As the number of bicycles is increasing rapidly, these safety problems arising out of their operation in close proximity to other traffic can also be expected to increase. The provision for "bike lanes" or "bike paths" is the most feasible way of giving an acceptable degree of protection to the cyclist sharing the roadway with vehicular traffic. Similarly, bike lanes and bike paths eliminate the conflicts with pedestrian traffic.

1.1 General Guidelines

As sharing of the road allowance is the most common method employed, it is worth noting some of the guidelines and techniques now being used. The usual method of providing bike lanes on existing streets has been to mark out an area about 6 to 8 feet (2 metres) wide adjacent to the curb and identify it accordingly.

1.1 (continued)

No barriers are erected to prevent vehicles from encroaching on the bike lanes, and on local streets where speed and traffic volumes are low, this method is functional. If bike lanes are to be provided on existing streets where traffic volumes are high or where the speed limit is higher than 35 m.p.h. (60 Kilo. per hour), some consideration should be given to the provision of a barrier to prevent motor vehicles from straying into the bike lanes and visa versa. One method which has been used in some American cities has been the placing of concrete bumper curbs along the edge of the bike lanes with the area being utilized for snow storage where bicycle traffic is seasonal. A solution such as this would require a minimum pavement width of 42 feet (approximately 13 metres) allowing for two lanes of traffic with parking on one side plus the bike lane.



Although this method provides adequate safety protection for the bike rider, it would require either widening of the roadway or reducing the number of lanes available for traffic, or both, on most roads where this degree of protection is required. On new roads and existing roads where major reconstruction is contemplated, serious consideration should be given to the construction of a separate bike path as part of the roadway. The decision on a separate path would consider financial limitations and space availability on that particular road.

A pathway exclusively for bicycles, physically separated from both vehicular and pedestrian traffic by a grassed or planted boulevard is, without a doubt, the best method of providing for cyclists on busy, high-speed roads. The separation should, of course, be as great as possible. In urban areas adequate provision must be made for them at intersections (dropped curbs, signing, pavement markings, etc.) to enable the cyclist to use the bike paths safely. Where bike paths must cross freeways, rivers, wide ravines, etc., roadway structures should be designed to accommodate this need.

In rural areas, some effort should be made to avoid long, straight sections. This can be accomplished by changing sides occasionally at culverts or bridges, by closely following the natural topography rather than the highway grades, or by other innovative ideas depending on the natural features available. Adequate rest areas, with at least minimal facilities, must be provided.

1.1 (continued)

BIKEWAY CLASSIFICATIONS

CLASS I



TOTAL SEPARATION / DIVIDING STRIP BETWEEN RIGHT OF WAYS ON SEPARATE SURFACES.

CLASS II



TOTAL OR PARTIAL SEPARATION / ADJACENT, BUT SEPARATED RIGHT OF WAYS ON SAME SURFACE.

CLASS III



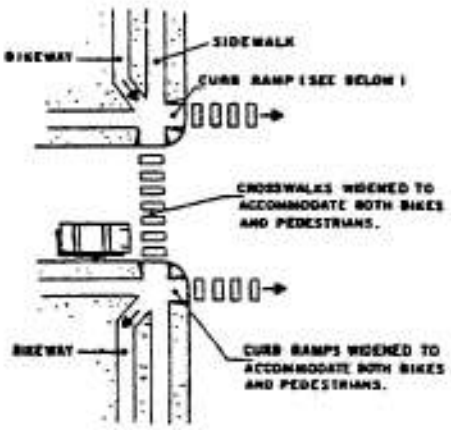
PARTIAL OR NO SEPARATION / SHARED RIGHT OF WAY ON SAME SURFACE

POSSIBLE LOCATIONS FOR BIKEWAYS

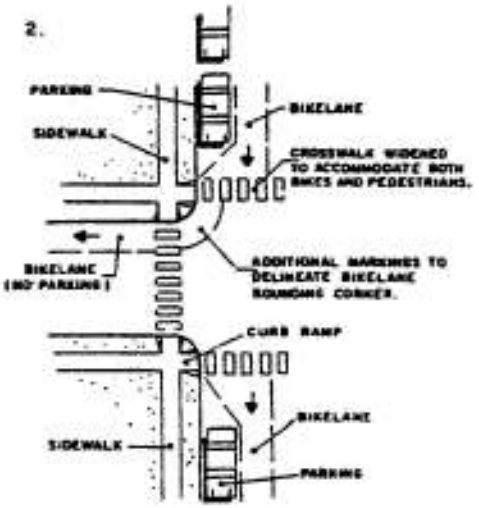
Abandoned R.R. right-of-ways;
Electric and pipeline right-of-ways;
River banks;
Dry washes
Beach fronts, lake fronts;
Flood control dikes and levees;
Irrigation canal banks and dikes;
Fire breaks

INTERSECTIONS FOR BIKEWAYS

1.



2.



2.0 In Recreational Areas

Bicycle paths in public recreational areas should be:

(a) Kept separate from pedestrian and vehicle traffic whenever possible.

(b) Constructed wide enough to permit two-way traffic and the access of maintenance and emergency vehicles.

(c) Surfaced with a material that is compatible with the thin tires popular on present-day bicycles.

(d) Blend in with the natural features of the ground to be traversed; and

(e) Of sufficient length and in locations which encourage their use. A comprehensive survey should be carried out prior to the development of any bicycle path to ensure that the proposed bicycle path will get the intended usage. Unless it is of a sufficient length and is properly located, usage will likely not justify the development costs.

3.0 Bike Path Design Standards

3.1 Alignment

In general, the use of long tangents should be avoided. It is generally acknowledged that a bicyclist can become discouraged by visible stretches of bike path in excess of 400 m. The alignment, therefore, should be carefully chosen in the field to take optimum advantage of the natural topography and surface features. A curvilinear alignment is preferable to one with short tangents connected with curves. The curves can be quite sharp, 10 m radius, as long as superelevation is provided. On steep uphill sections, the curves can be sharper, 3 m radius, and superelevation is not required.

3.1 (continued)

Table I Bicycle Path Superelevation

Design Speed	Radius (m)	Superelevation (m/m)
20 km/h	11 m (min.)	0.10
	12 m	0.08
(Level Sections)	14 m	0.05
	15 m	0.03
	17 m	0.02
	18 m	0.02
	20 m	0.02
	21 m	-0.02
	23 m	-0.02
30 km/h	24 m (min.)	0.10
	27 m	0.08
(Downhill Sections)	30 m	0.06
	34 m	0.04
	37 m	0.02
	40 m	0.02
	43 m	0.02
	46 m	-0.02

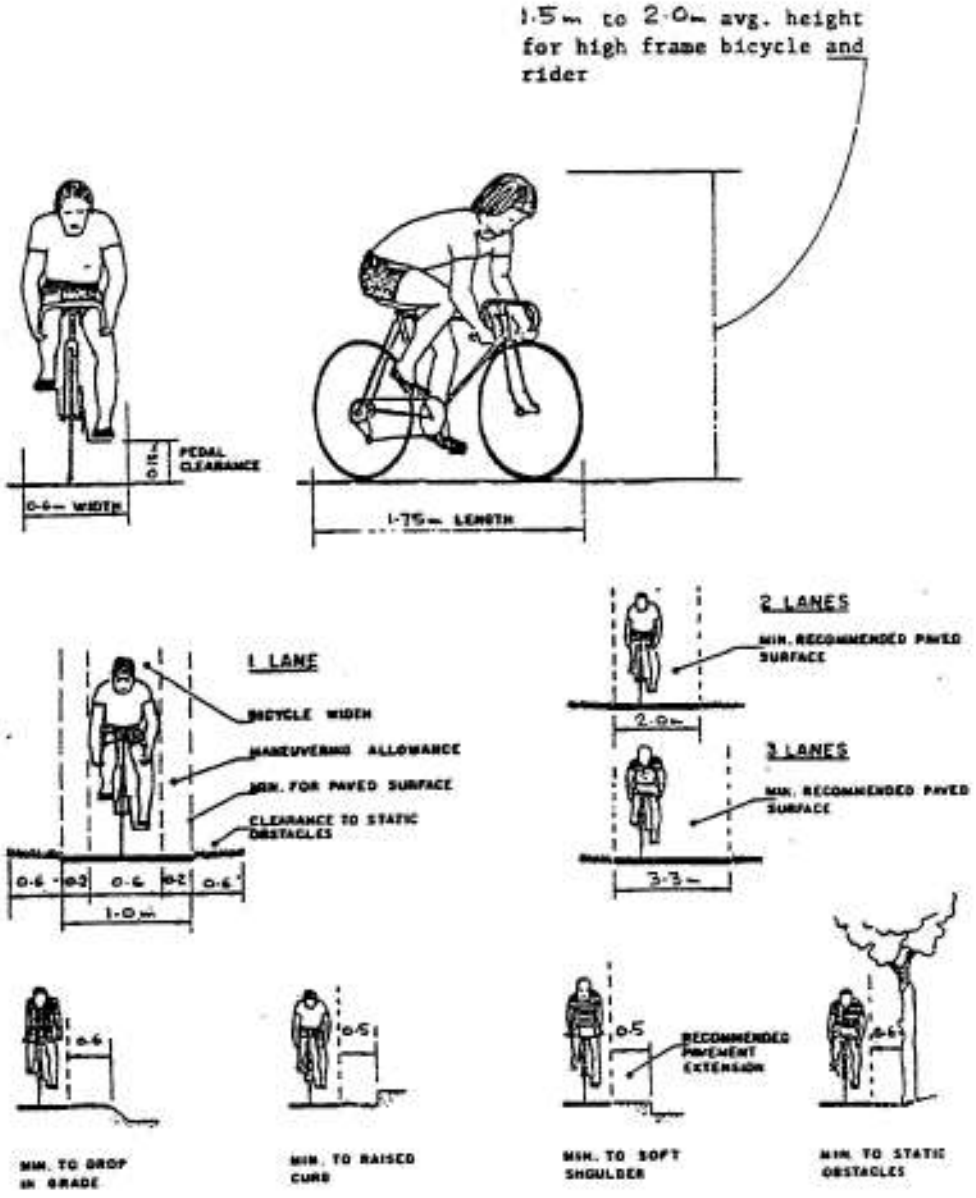
Negative Superelevation indicates that advere banking for drainage purposes may take precedence over banking to counteract centrifugal forces.

3.2 Grades

Long grades should not exceed 3%. Short lengths (up to 75 m) with grades as high as 8% can be tolerated. Rolling grades are preferable to steady uphill sections, as they give the cyclist a time to coast and catch his breath. Grades steeper than noted will force riders to dismount and push their bikes, and if these sections are numerous, will likely result in the pathway not being used to its capacity. If the topography requires the construction of paths with grades steeper than indicated, separate one-way paths should be considered. The downhill path should not be steeper than about 6% with the curves superelevated for a design speed of 30 km/h. The uphill portion can be considerable steeper (20%) and shorter, reducing the distance to push the bike proportionately.

Gradient	Length	
	Norm.	Max.
1.5%	490 m	
3 %	120 m	240 m
4.5%	45 m	90 m
10 %	9 m	18 m

3.3 Width and Clearance



3.3 (continued)

One-way paths, when constructed, should have a minimum width of 1 m with a slightly wider width 1.2 m preferred. Two-way paths should have a width of 2.1 m minimum and preferably 2.5 m. Where traffic is heavy, consideration should be given to providing for a minimum of 3 lanes by widening the pathway to 3 m or more. One-way paths should be avoided wherever possible, as the narrow width does not allow for the passing of slower riders.

Fixed objects should be kept clear of the edge of the path by 0.5 m. A minimum overhead clearance of 3 m is desirable and in no case should the overhead clearance be less than 2.1 m. Bridge, rails, guard rails, fence posts, etc., should have a horizontal clearance of at least 0.3 m.

3.4 Surfaces and Construction

A survey conducted by the American Institute of Park Executives showed that over 75% of people preferred the use of asphalt as a surfacing material for bicycle paths. Second choice was either dirt, gravel or soil cement with approximately 5% each. Other types of surfaces gained negligible support. The type of bicycle currently in vogue with its thin, high-pressure tires is very difficult to operate on anything but a smooth hard surface. The National Capital Commission standard for bicycle paths is 50 mm HL3 asphalt surface over 150 mm of compacted granular "A" base, and this would seem to be suitable for most areas of the province. A full depth asphalt surface 75 to 100 mm thick placed in one lift, on a compacted natural earth subgrade may be more economical and would probably be suitable in most areas also.

3.4 (continued)

Recommended Surfaces



Bituminous Conc. mixes



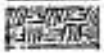
Concrete



Stone Chips



Soil Cement



Stablized Earth



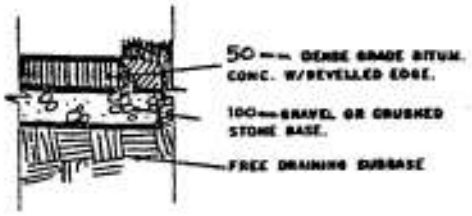
Wood Planking

All weather, permanent surfaces:
(most widely used; highest installation costs, longest wearing life; especially good for heavy use in urban areas.)

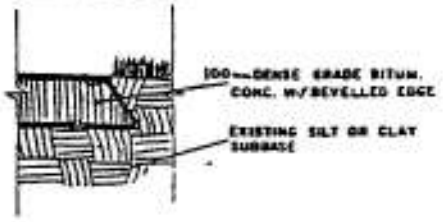
Loose aggregate, compositional, natural surfaces:
(lower installation costs; high maintenance requirements, susceptible to poor natural drainage; will not stand up to continual heavy use.)

Wood Surfaces:
(use only when laid perpendicular to direction of travel, usually on light bridges, boardwalks, etc.)

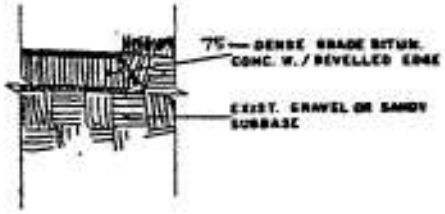
Standard Construction



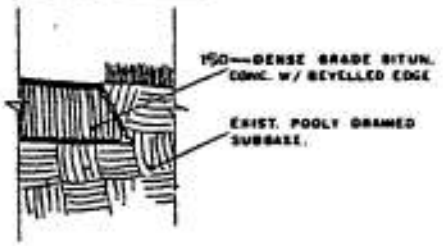
Silts & Clays



Gravels & Sands



Poorly Drained



3.5 Drainage

Surface run-off from small areas can be allowed to drain directly across the pathway and the pathway should be sloped to either side with a crossfall of 2% to 3% to facilitate this drainage. Where large amounts of water must be accommodated, culverts should be installed. Side ditches, where required, should have flat side slopes or be constructed at distances of 1 m or more from the edge of the path to the top of the ditch slope.

3.6 Other Services

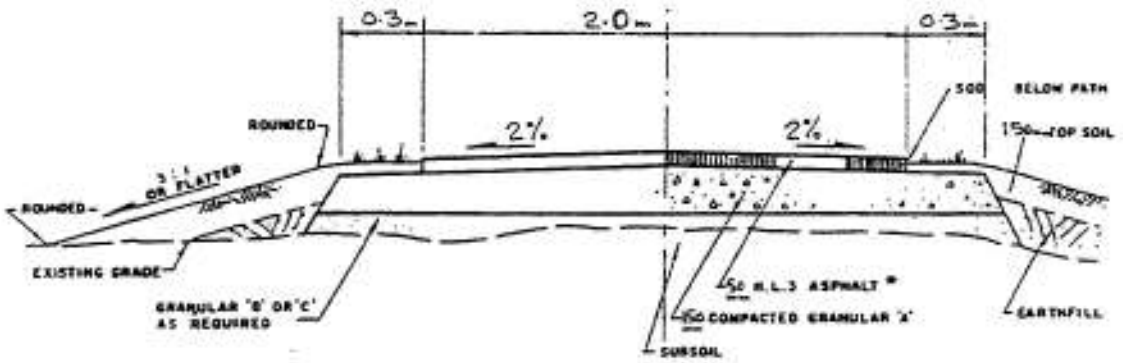
Rest areas should be provided, in scenic areas, at frequent intervals and wherever possible, set well back from the pathway. They should be equipped with benches, tables, etc., much the same as the roadside picnic areas which are found along the provincial highways. Wherever possible, restroom facilities and drinking water should also be provided.

3.7 References

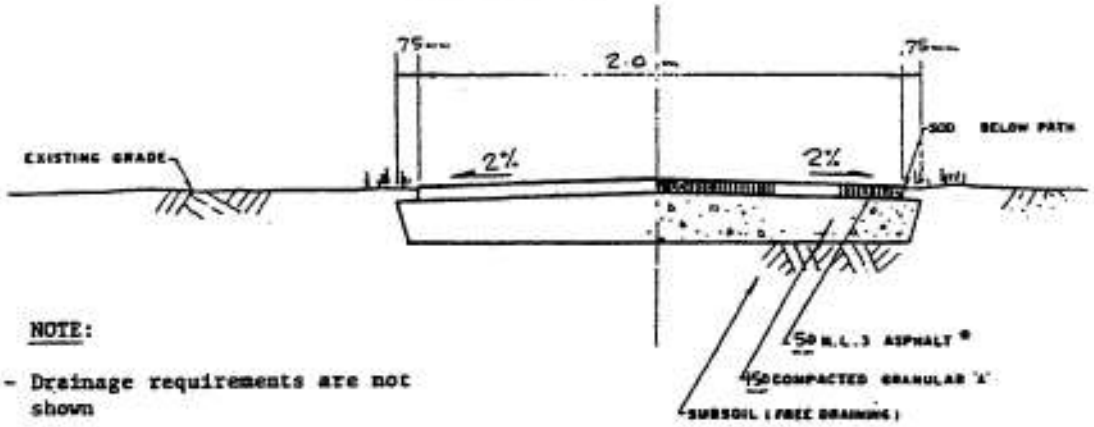
Subsequent to the preparation of this section the following publication has become available and is recommended for further reference:

"The Guide for the Design of Bikeways"
Roads and Transportation Association of Canada,
1765 St. Laurent Boulevard,
Ottawa, Ontario, Canada, K1G 3V4.

3.8 Typical Sections



FILL SECTION



CUT SECTION

NOTE:

- Drainage requirements are not shown
- Superelevation to be used on curvature of less than 450m radius
- Crown on tangents or curvature of more than 450m radius may be applied all to one side to facilitate drainage of surrounding area.

* Refer to 3.4, Standard Construction for further design proposals.

5.6 APPURTENANCES INDEX

1.0 Boulevard Treatment

2.0 Vegetation Consideration

3.0 Drinking Fountains

4.0 Outdoor Telephones

5.0 Trash Cans and Receptacles

6.0 Benches and Outdoor Seating

7.0 Sound Barrier Walls

5.6 APPURTENANCES

1.0 Boulevard Treatment

Providing there is an adequate boulevard width, and a reasonable expectation of satisfactory care without undue pedestrian traffic, grass provides the most pleasing boulevard treatment. The following rule of thumb guide is provided in this respect:

Boulevard Width Curb to Sidewalk	Area Zoning	Boulevard Treatment
Less than 1.0 m	All	Asphalt or equal
1.0 m and greater	Residential	Grass
Less than 1.8 m	Industrial & Commercial	Asphalt or equal
1.8 m and greater	Most areas	Grass

Too often the decision is made to place an asphalt surface rather than grass because of the effects of salt action. Lawn and garden gypsum is a natural soil conditioner which will efectively counteract the effects of de-icing salt and heavy snow pile-ups on roots of grass, shrubs and trees. Some varieties of grass are more resistant to the effects of salt, some are slower growing, and some have longer root systems and more resistant to wash-outs, dry weather, etc. Care should be taken in the selection of the grass variety to suit the particular conditions. Care should also be taken to ensure that a satisfactory quality and depth (min. 50 mm) of topsoil is placed prior to the laying of any sod.

Where an asphalt boulevard is placed, a minimum depth of 100 mm Granular 'A' and 50 mm HL3A asphalt is recommended. It is also suggested that a soil sterilant be used priot to placing the asphalt in an effort to reduce the weed growth in any cracks. Although coloured asphalt is available, it is not recommended except in unusual circumstances. It has a high initial cost and it is very difficult to make a satisfactory match in maintenance repairs.

As an attractive alternative to asphalt, particularly in commercial or high density residential areas where high pedestrian volumes demand a hard surface, pre-cast paving slabs or interlocking paving stones may be used. These can be obtained in a variety of colours. They are easily removed and replaced for any utility repairs needed in the boulevard area.

Paving stones are made of compressed concrete with a variety of interlocking patterns which ensure that the surface will not be dispalaced under the weight of traffic. This locking system prevents tipping of individual stones and ensures the entire paved area will remain smooth and level, a distinct advantage over the pre-cast paving slabs.

1.0 (continued)

The stones are available in various thicknesses to suit particular applications, for example, 70 mm thick stones are used for sidewalks, bicycle paths, driveways and parking lot areas, and 80 mm stones are used for areas involved with trucks. The stones are laid on a 50 mm to 300 mm depth of compacted granular base, the depth again subject to the loading conditions and the quality of the foundation materials.

In areas of cut or fill the boulevard slopes may be considerably greater than normal and often may require slope rights or additional road allowance to carry the slopes to the adjacent ground level. In some cases where the right-of-way is limited due to existing topography restrictions (e.g. buildings) hard surface slopes or retaining walk may be necessary.

Particular attention should be paid to ensure proper access to the sodded slope areas from the road so that routine maintenance may be carried out.

It is recommended that sodded slopes be not greater than 3:1, with attention made to the variety of grass used and to shrub plantings.

2.0 Vegetation Considerations



Overhead materials should be chosen for their particular characteristics. Avoid materials that have tendencies towards dropping excessive debris drooping or breaking under heavy snow loads, or winds, etc.

Maintain a minimum of 2.6 m vertical clearance over walks, bikeways, sitting areas, etc.

Improper location or poor maintenance can quickly lead to a reduction of the efficiency of lighting systems.

Avoid placing hazardous or nuisance materials adjacent to walkway or sitting areas.

Plant materials may affect the melting of ice and snow from walkways and stairs consider their mature shadow patterns during winter months before deciding on their final locations.

Many trees with shallow or surface root systems will heave or break up walkway surfaces use caution when choosing these varieties and their subsequent locations.

Avoid placing materials over or near underground utilities. If they have root systems that characteristically cause damage to pipelines, cables, sewers, etc.

Creeping ground covers, vines and other invasive materials can be troublesome if not contained - keep them off buildings, walkways, steps, ramps, signs, and lighting fixtures

2.0 (continued)

It is well known that high salt concentration can cause injury to vegetation. Salt in large quantities can interfere with the plant's absorption of moisture from the soil and by browning or burning leaves. Salt may indirectly affect plant growth by altering soil structure, permeability and aeration.

Road salt is just one of the many hostile factors affecting roadside vegetation. Plants must also adjust to compacted soil, injuries to root systems during road construction, changes in soil moisture levels due to paving, and auto exhaust emissions.

Stress for water increases the chance of salt damage to vegetation. The roadside environment robs plants of moisture; they are exposed to excessive wind flow, high soil and air temperature, and added exposure to sunlight. Drought may be the final straw.

After many years of salting, sodium buildup in roadside soils may indirectly affect plant growth. Soil can be chemically amended and salinity removed by adding gypsum or anhydrous ammonia. Gypsum treatments appear to be the most efficient and least expensive reclamation method.

Salt damage to trees can be minimized and often prevented by selection of the correct species and proper planting practices. Good street and highway design, including adequate drainage systems for highways, and curbs and gutters to carry street runoff into storm sewers, substantially reduces salt's effects.

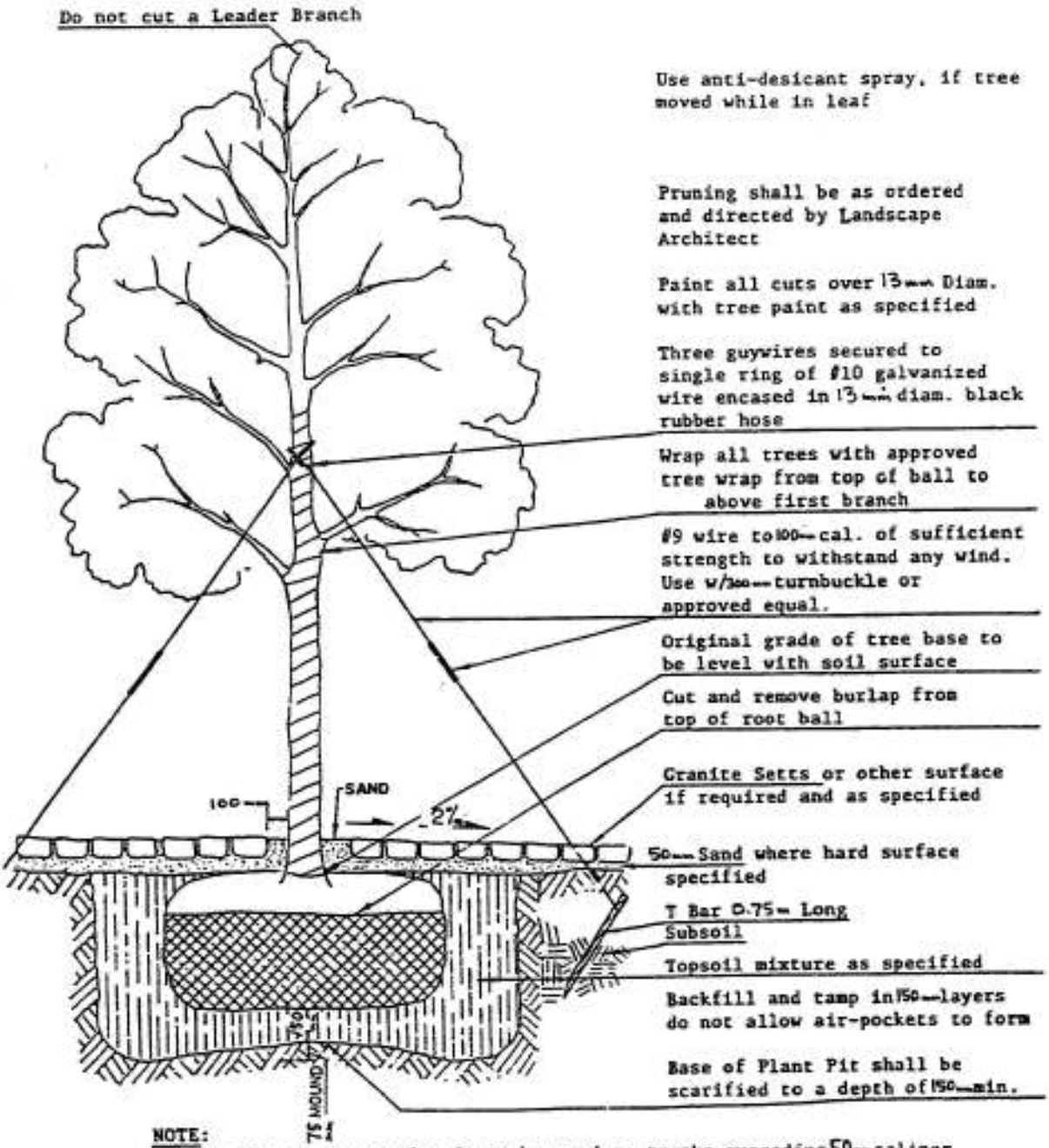
Many trees resist effects of salt very well. These include oaks, locusts, Scotch elm, Russian olive, hawthorne and silver and gray poplars.

Landscape planners should avoid use of sugar and red maples, Lombardy poplar, black walnut, and rose and spirea bushes in locations exposed to salt runoff and spray from de-icing operations.

Salt may be blamed for vegetation decline that is caused by other factors. Many trees die because their life cycle is naturally complete. Others die as a result of disease of infestation.

2.0 (continued)

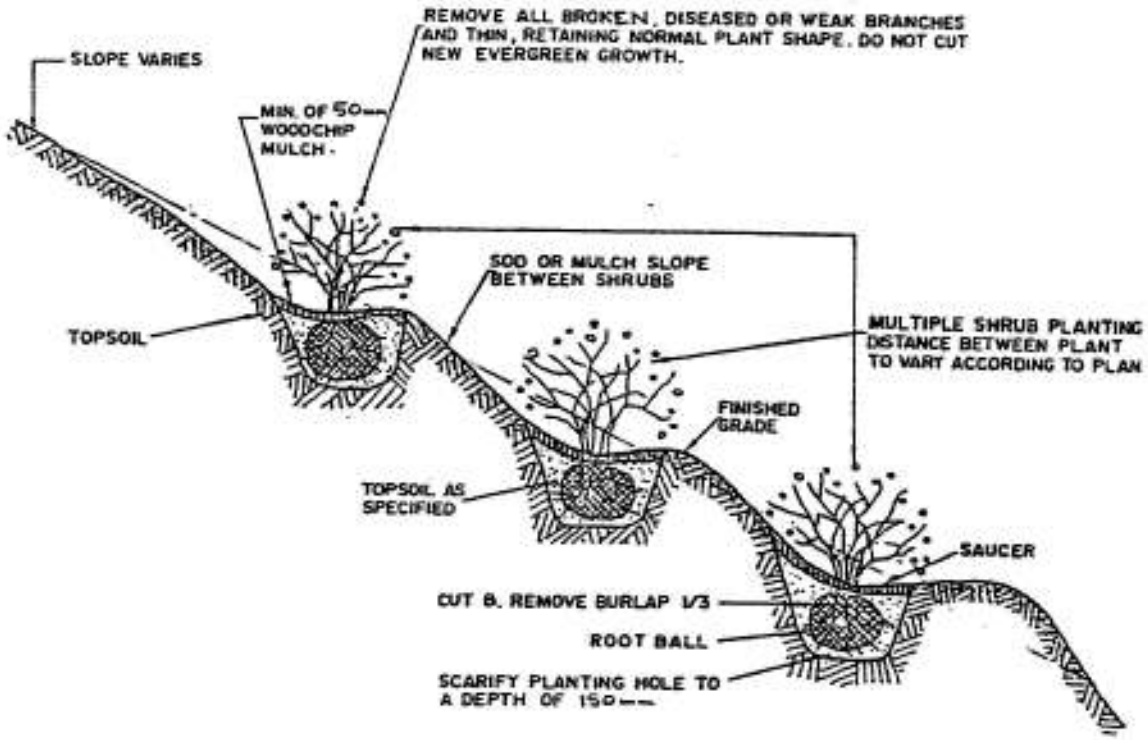
Typical 75 to 150 mm Tree Planting In Paved Boulevard



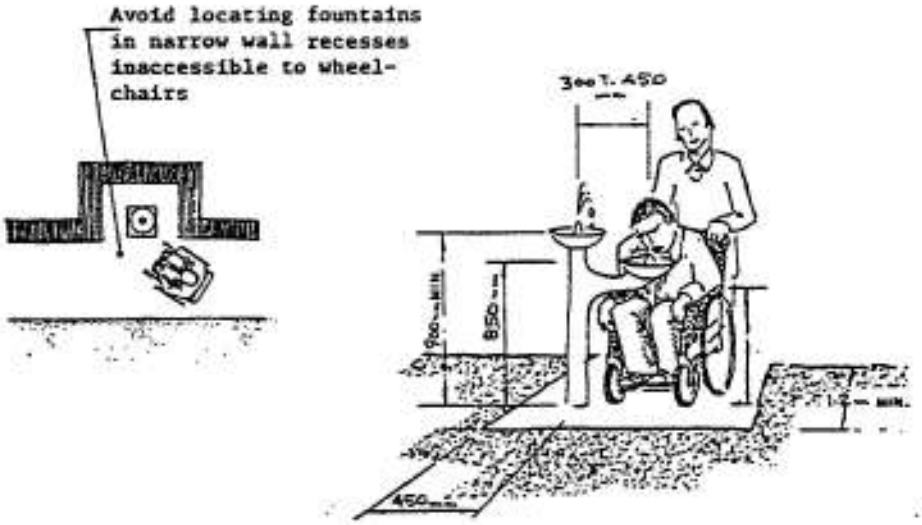
NOTE: Burlap tree wrapping is to be used on trunks exceeding 50 mm caliper and branches exceeding 75 mm caliper on those species designated by Landscape Architect.

2.0 (continued)

Multiple Shrub Planting On Slope



3.0 Drinking Foundations



Hand operated knobs or buttons and foot pedals are difficult for many handicapped people to operate. Hand levers are preferred.

Provide a minimum 750 — vertical clearance below fountain nozzle to allow wheelchairs leg room for access.

Nozzle heights should be approximately 850 — for wheelchairs and children and 900 to 1000 — for adults.

A minimum 450 — wide paved area around outdoor fountains avoids both mud and puddles.

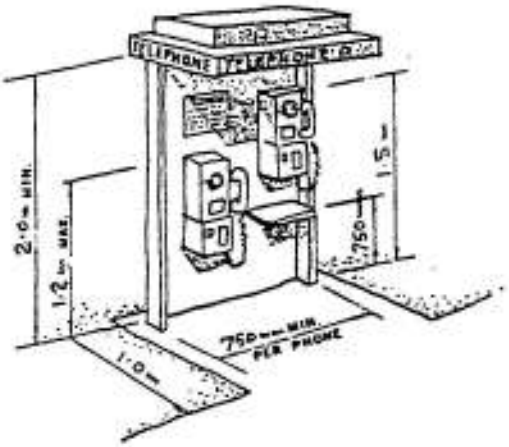
4.0 Outdoor Telephones

All groups of telephones should have at least one lower height telephone for use by the handicapped and children.

Phones for the handicapped should be no higher than 1·2m at the coin slots. Provisions for braille instructions, volume controls on headsets, and push button dials are helpful for many handicapped individuals.

Provide adequate lighting on the underside of overhangs for nighttime use.

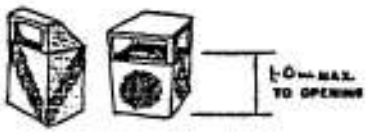
Package rests are a helpful convenience to all people.



5.0 Trash Cans And Receptacles



1. Open top easiest to discard trash; open to rain, wind, snow; needs weep holes for drainage; easy to empty; open to insects.



2. Semi-open top gives protection from elements; top hinged to allow removal of trash when unit is full; openings must be designed to accept size of anticipated trash; open to insects.



3. Hinged door openings are difficult for many handicapped people to operate; good protection from elements and insects; openings must be designed for anticipated trash sizes; spring loaded doors should be easily pushed open with one hand.

6.0 Benches And Outdoor Seating

Choose materials which do not retain heat or cold. Avoid rough materials or those that may splinter.

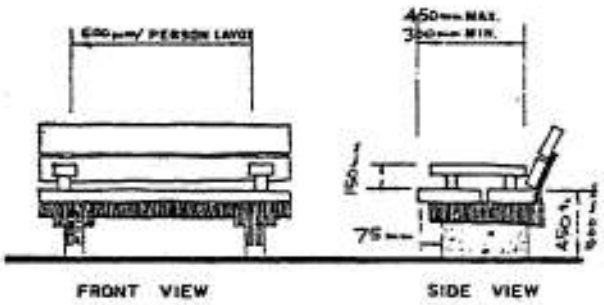
Sitting heights of 0.45 to 0.5 m are preferable.

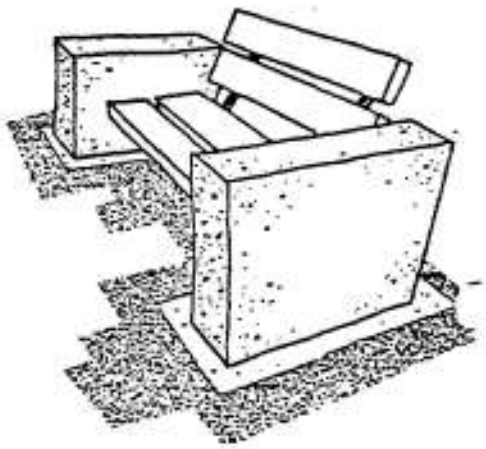
Sitting surfaces below 0.3 m width are uncomfortable for many adults, likewise, widths beyond 0.45 m become awkward for normal leg lengths.

Provisions for arm and back rests increase comfort. Arm rests are also helpful for getting into and out of seats and benches.

Provision for heel space of 75 mm makes rising from seated position easier.

Seat surfaces should be pitched to shed water.

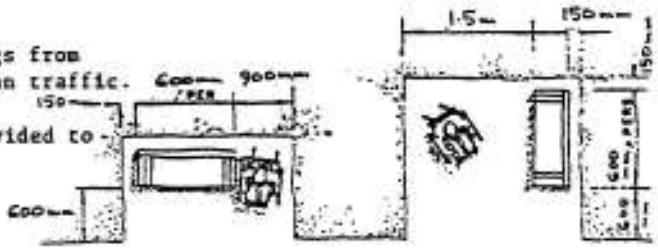




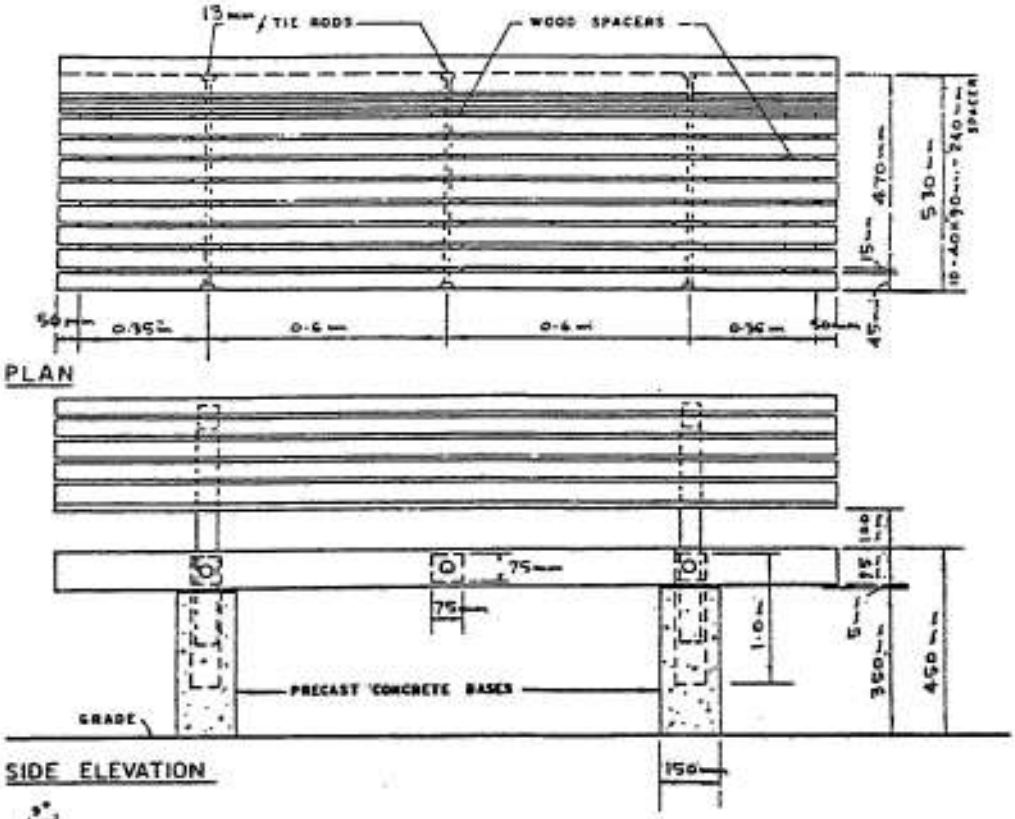
Space Around a Bench

Provide 600 mm setback to keep legs from interrupting adjacent pedestrian traffic.

A 900 mm min. space should be provided to allow room for wheelchairs.



6.0 Benches And Outdoor Seating (continued)



NOTES:

1. Seat and back - 40mm x 90mm D- 115mm select pine (outer pieces) remainder No. 1 white pine
2. Tie rods -38mm diameter - recess nuts washers and heads
3. Pine spacers - 38mm X75mm X75mm
4. Mild steel anchor plates - seat back 13mm X50mm X600mm
5. Wood finish -
   (a) ½ part linseed oil to ½ part turpentine rubbed on
   (b) ½ part "pentox to 1/2 part walnut paint rubbed on
   (c) 2 coats urethane varnish brilliant gloss brushed on

7.0 Sound Barrier Walls

Reference should be made to Ministry of Transportation and Communications Report No. RR180, September 1972, produced by the Research Division of the Ministry of Transportation and Communications.

Barrier walls have been constructed of earth embankment, normal density concrete panel wall, lightweight cellular concrete panel wall, combined cellular concrete wall and earth embankment, plywood, aluminum panel and gabions.

As outlined in the Ministry of Transportation and Communications' report, barriers are a relatively ineffective method of noise shielding. Therefore, greater attention and emphasis should be given to other noise control measures such as housing modifications, land use control and control of vehicular noise emissions at the source.

There have been indications from people living behind the barriers that they considered them beneficial and that they favoured their retention. The precise reasons for this are unknown, although they may be related to psychological and visual shielding and shielding against dirt and debris.

In most cases, barriers because of their high cost and their limited effectiveness and other adverse effects (aesthetics, shadow, effect on snow drifting), do not appear to be the most cost-effective solution to highway noise problems. Greater consideration should be given to alternative measures.

5.7 FINANCING

1.0 Methods of Financing

Although this chapter deals specifically with "Transportation", the methods of financing herein are as we generally know it under the Municipal Engineer's jurisdiction.

The council of the municipality dictates the policy that is followed in the paying for works. Management of the municipality assists council when requested to give them full knowledge of all facts. It is definitely in the Municipal Engineer's interest to have a good communication with the treasurer's department. By having this rapport there is a good appreciation between the two departments of the problems and the priorities of the day so that when decisions are made on financing all facts are reviewed in the decision making process.

There are three main ways that works can be financed, namely through debentures, through current revenue and/or a funding account procedure.

Capital financing through debentures is usually used when the cost of the work far exceeds the capability of the municipality to pay for them immediately. By the use of debentures we are able to finance through a pay-as-you-use program over a period of time by maintaining a constant rate of cost which avoids a high tax burden in any one year. The ability to finance through debentures is controlled by the Ontario Municipal Board whose approval is required. The Board reviews the ability of the municipality to pay and, as a guideline, the per capita debt and the debt to assessment that the municipality has accrued is assessed. The disadvantage of debenture financing is that it creates a certain extra expense to the work in interest payments of the cost of borrowing. The debenture charge can be assessed against the ratepayers as a whole in the municipality or a charge to users only. A charge to users only can be accomplished on an area basis or to abutting owners under the Local Improvement Act. The area basis is more commonly used in rural municipalities.

Where work is undertaken and does not create a high burden on the tax structure of the year the work is generally constructed from monies obtained out of current revenue. In other words, the work is estimated and the cost approved by council to be undertaken within the mill rate established for that particular year.

1.0 (continued)

Funding can be used where it is desirable to establish a reserve account to do certain works. This is done by making a levy against general ratepayers within the current budget expenses and these monies allocated for a certain cause or project. These monies can also be obtained from levies directed against developers under what is known as "Subdivision Agreements" or "Developer's Agreements". The monies are then set aside for the work and may be proceeded with when the cost is covered. Another example of funding would be in areas where a revenue is obtained by the municipality such as the revenue from parking meters. The profit is usually funded into an account for future off-street parking facilities. Reference can be made to the Manual of Municipal Accounting, published by the Ministry of Treasury Economics and Inter-Governmental Affairs.

2.0 Estimating

In the financing aspect of work the first thing the municipal engineer is requested to give advice on is a preliminary estimate. It is usual to have reference to a 'ballpark figure' of the cost of the services requested. This is obtained from costs of similar work within the municipality. The cost of work are influenced by the soil conditions within the municipality; the availability of contractors within the area; and amongst other things the weather conditions that prevail. Preliminary estimates should be readily available and when given out of the department there definitely should be an explanation that the estimate is only preliminary and could change when all details are known and calculated in a detailed estimate. Usually preliminary estimates are given as a guide only to see if the work should be investigated further. Once direction is given to proceed the engineer then prepares a detailed estimate. These estimates are completed only after a survey has been made of the work, plans prepared and all facts known. The format of the estimate should be made up in such a way keeping in mind the municipality's specifications. The detailed quantities in the estimates should be able to be used when preparing the bill of quantities within the Form of Tender of a contract.

Preliminary Estimate Cost Guide

The cost per metre indicated in the table is for the conditions as outlined below and will require adjustment in accordance with review of a particular project.

2.0 (continued)

- complete road reconstruction to provide curbs
- storm sewers and other municipal utilities existing
- catchbasins and connections reconstruction included
- normal urban driveway access reconstruction included
- nominal affected sidewalk reconstruction included
- nominal utility adjustment included
- collector and arterial roads include intersection channelization and bus bays as required
- 8.5, 9.7 & 13.8 m pavements have 140, 190 & 240 mm asphalt respectively on 260 mm crushed stone
- 15.0 & 16.8 m pavements have 75 mm asphalt on 200 mm concrete on 150 mm crushed stone
- does not include road allowance property widening costs

Width of Pavement (face to face of curb)		Year $/m 1980	1981	1982	1983	1984
Composite pavement with conc. base	16.8 m	800	870	990	1060	1110
	15.0 m	750	820	935	1000	1050
Deep str. asphalt pavement	13.8 m	680	740	845	905	950
	9.7 m	450	490	560	640	670
	8.5 m	365	400	455	485	510

2.0 (continued)

Date.....

Estimate Of Proposed Road Construction On

Street _____

From _____

To _____

Job No. _____ Length _____ m Width _____ m Plan No. _____

RELATED WORK

STORM SEWER Job No. _____ Dwg. No. _____

SIDEWALKS Job No. _____ Dwg. No. _____

EXTERNAL WORK

O.L.S. Survey.....	_____
Property Acquisition.....	_____
Ontario Hydro.....	_____
City Hydro.....	_____
Consumers' Gas.....	_____
Bell Telephone Company.....	_____
City Parks Department.....	_____
City Works Department.....	_____
City Traffic Department.....	_____
Materials to be supplied by City	_____

TOTAL OF EXTERNAL WORK $ _____

TENDER ITEMS	QUANTITY	UNIT	RATE	AMOUNT
Clearing the site	LUMP SUM PRICE			_____
Tree removal				
151 mm to 300 mm dia.....	_____	each	_____	_____
301 mm to 450 mm dia.....	_____	each	_____	_____
over 450 mm dia.....	_____	each	_____	_____
Stone for detour.....	_____	t	_____	_____
Asphalt for detour.....	_____	t	_____	_____
Excavation (calculated).....	_____	m^3	_____	_____
Additional excavation (estimated)	_____	m^3	_____	_____

2.0 (continued)

TENDER ITEMS	QUANTITY	UNIT	RATE	AMOUNT
Imported fill (calculated).....		m^3		
Remove existing sidewalks.....		m^2		
Remove existing curbs (incl. gutters).....		m		
Remove existing culverts (incl. headwalls).....		m		
Remove existing catchbasins....		each		
Adjust existing catchbasins....		each		
Adjust existing manholes.....		each		
Plug abandoned pipes.....		each		
Re-grade ditches.....		m		
Install 300 mm C.S.P.....		m		
Construct 600 mm catchbasin single.....		each		
Construct 600 mm catchbasin double (incl. sub-drain).....		each		
Install 150 mm dia. sub-drains.		m		
Install 200 mm catchbasin connection.....		m		
Install 250 mm catchbasin connection.....		m		
Install mm catchbasin connection.....		m		
Install 200 mm catchbasin risers.....		m		
Install 250 mm catchbasin risers.....		m		
Construct sidewalks 100mm thick		m^2		
150mm thick		m^2		
200mm thick		m^2		
Construct curb as per Std. 201		m		
Std. 203		m		
Std.		m		
Granular 'B' 150 mm thick.....		m		
Granular 'B'.....		t		
Granular 'A' 150 mm thick.....		m^2		
Granular 'A'.....		t		
Stone 150mm thick.....		m^2		
Stone.....		t		
Concrete base 200mm thick.....		m^2		
Concrete bus-bays 280 mm thick.		m^2		
Concrete median 230 mm thick...		m^2		
Concrete median 80 mm thick...		m^2		
Burn in asphalt joints.....		m		
Tack coat.....		m^2		
Saw cutting joints.....		m		
H.L. 3 asphalt 40 mm thick....		m^2		
H.L. 3 asphalt.....		t		

2.0 (continued)

TENDER ITEMS	QUANTITY	UNIT	RATE	AMOUNT
H.L. 5 asphalt 40 mm thick.....		m^2		
H.L. 5 asphalt.....		t		
H.L. 8 asphalt 50 mm thick.....		m^2		
H.L. 8 asphalt.....		t		
Carpet seal 25 mm thick.....		m^2		
Carpet seal.....		t		
Sidestreets.....		m^2		
Tapers 150 'B', 150 Stone, 100 H.L. 6 40 H.L. 3.....		m^2		
Shoulders 150 'B', 290 Stone...		m^2		
Paved Boulevards 100 'A', 50 H.L. 3A.....		m^2		
Residential driveway 150 St. 80 ASP.....		m^2		
Commercial driveway 150 'B', 150 St. 140 ASP.....		m^2		
Industrial driveway 150 'B', 150 St. 190 ASP.....		m^2		
Concrete driveway 150 mm thick.		m^2		
Stone in driveway.....		t		
Sod on 50 mm of topsoil.....		m^2		
Water (a) compaction.....		m^3		
(b) sod.....		m^3		
Traffic signal duct 75 mm.....		m		
Handwells.....		each		
Additional 2 bags (80 kg) cement		m^3		
Allowance for day work.....				
	Sub-Total			$
Add 5% for tender contingencies				$
	Total Tender			$
Add external work.....				$
				$
Add % Engineering and Contingencies				$
Total Estimated Cost				$
Anticipated M.T.C. Subsidy				$
Estimated City Share				$
Cost per metre $				

Prepared By_____ Checked By_____

5.8 DESIGN APPROVALS

1.0 Official Approvals

Approval for designs of transportation facilities are required from various departments of senior governments. Some of these are as follows: -

Road Projects

a) District Municipal Engineer - Ministry of Transportation and Communications.

b) Ministry of the Environment if sewers are involved.

c) C.T.C. if Railway Level Crossings are involved.

Bridge Projects

a) Municipal Structural Engineer - Ministry of Transportation and Communications.

b) Canadian Transport Commission, Ottawa and the Railway Company involved if the structure is over or under railway tracks.

c) Department of Transport - Marine Services, Ottawa if the structure is over a Navigable Water.

2.0 General Approval by the Public

Public informaton is fast becoming one of the critical planning areas for any proposed project. The type of program implemented will of course vary with the size and scope of the project, however at some point in the planning stage a decision must be made as to the degree of public participation, with particular reference to the requirements under the Environmental Assessment Protection Act.

Citizen participation can achieve better plans and increase the likelihood of implementation of those plans. Also, citizens can and do stop projects they perceive as undesirable. Engineers and planners will increasingly become involved in public participation situations. Increased costs are small especially in view of the magnitude of the decisions to be made or the possibility of costly stop work situations.

There is extensive literature describing various types of Public Information programs and Engineers or Planners designing a public participation program would benefit from study of this information. The following is a brief outline of some of the types and uses of Public Information programs.

2.0 (continued)

Minor Projects

- News Releases - to inform the public of impending projects. While resistance to small projects is usually minor and localized, advance publicity will enable affected residents to express concerns by letter or telephone.

- Direct Mailing - information about the nature and timing of a project is mailed directly to residents affected. Quite often unless local improvement changes are involved the first information is sent out just prior to construction. Information can be distributed by hand delivery along a particular route, however owners who reside elsewhere are missed unless notified by regular mails.

Major Projects

- Major Media Releases (Press, Television and Radio) - should be made early in the planning stage. The public reaction may influence the degree of public participation in later stages. New information about the progress of a project can also be disseminated this way. It may be prudent to provide elected representatives with new information prior to media releases.

- Public Meetings - also should be held early in the planning stages so citizen input does influence the project. The Engineers and Planners should listen to the ideas and preferences of citizen groups. Avoid trying to aggressively sell preconceived plans of transportation experts. The most aspect is two way flow of information.

- Separate Small Group Meetings - with community association leaders and local community groups. These can be held in the planning office and should be sit down, informal, direct discussion type of meetings without media representatives.

This avoids the sometimes hostile and emotional atmosphere which can occur at large public meetings.

- Mail-out Ballots - These are useful in determining public opinion on possible alternatives. It should be made known in advance that the ballot results will be made public to avoid citizen skepticism.

3.0 The Ontario Municipal Act

The Municipal Act Section 446 requires as follows:

446 - (1) Before passing a by-law for stopping up, altering, widening, diverting, selling or leasing a highway or for establishing or laying out a highway,

(a) notice of the proposed by-law shall be published at least once a week for four successive weeks, and in the case of a village or of a township with a population of less than 40,000 shall be posted up for at least one month in six of the most public places in the immediate neighbourhood of the highway or proposed highway; and

(b) the council or a committee of council shall hear in person or by his counsel, solicitor or agent any person who claims that his land will be prejudicially affected by the by-law and who applies to be heard.

(2) The Clerk shall give the notices upon payment by the applicant, if any, for the by-law, of the reasonable expenses to be incurred in so doing.
R.S.O. 1970, c.284, s. 446.

The Act does not specify the detail that must be provided in such notice and for the protection of the Municipality, presumably the minimum detail should be provided viz., - limits, width (E.G. - essentially 15.0 m wide with channelization at certain intersections).

Presumably in the event of a public hearing, the plans submitted at such a hearing would have to be adhered to and any major alterations to the plans would require additional notice.

Evidently many municipalities have not complied with the requirements of Section 446, and Section 447(a) is proposed to be added to protect such municipalities.

Section 447(a) provides that a judge may not order the removal of any work which was carried out, by reason only of failure of the Municipality to comply with Section 446, but does not affect the rights of persons to claim for damages against the Municipality in respect of the work carried out.

5.9 LEGAL ASPECTS

There are three basic types of legal documentation that are used.

The first priority is the statues that are passed by an act of provincial legislation. The statutes are given three readings in the Committee of the Legislature before becoming law. The most common acts that fall within the engineer's scope of work are the Municipal Act which basically provides the powers of the Municipal Council; The Local Improvement Act; the Environmental Protection Act; the Public Transportation and Highway Improvement Act; the Drainage Act; the Highway Traffic Act; the Construction Safety Act; and to some extent the Planning Act; The Public Service Works on Highways Act; Environmental Assessment Protection Act.

The second most important piece of legislation would be the regulations made pursuant to any specific act. Regulations can be thought of as a permissive legislation passed by the Lieutenant Governor in Council. This is the backup legislation on how the acts are to be reinforced or interpreted where the nature of the act requires legislation as to its use.

The third basic type of legislation is the Municipal By-Laws. Statutes of the provincial government gives the municipality certain powers which are then controlled by the municipal by-laws.

Ontario Municipal Board approval is required by a municipality on the debenture of monies for work proposed. If ratepayers object to the expenditure the Board usually calls a public hearing. The municipal engineer usually finds that he is required to be a witness at the O.M.B. hearing in support of the work. To this end it is necessary that the municipal engineer be knowledgeable as to what is expected of him.

Until recent years engineers from municipalities concentrated on engineering work, however, we are finding that this concept is rapidly changing. An engineer must not only be capable and competent in his professional qualities but he must also be capable of filling in different slots, particularly with respect to public relations, and most important must be prepared to go in the witness box on behalf of his municipality and be subject to the rigors of examination and cross-examination.

This is not always a pleasant duty to perform by being engaged in a frontal attack with a lawyer on matters of your work. It is however necessary if the municipality is prepared to advance in municipal standards and services.

5.9 (continued)

In the majority of O.M.B. hearings it is the duty of the municipality to put their case first. The lawyer for a municipality usually will put the municipal engineer in as his first witness and examine him in chief. This means that the municipal lawyer will be kind and careful and set an established pattern. He will not attack the engineer as a witness. It is most important that the municipal engineer and the solicitor for the municipality review the subject before being placed in the witness box. When the engineer's evidence is finished he is then subject to cross-examination by the other lawyer and this is when the municipal engineer must use all of his resources. Too often, as a witness, the engineer may seem to think that the examination by their own lawyer was sufficient. It is most necessary that the engineer be prepared to accept an offense against him.

If, during the examination, the engineer finds from the questions put to him that he has answered a previous question incorrectly, probably as a result of misunderstanding, then it is most important to turn to the chairman and correct the information. In other words the engineer readily sees his mistake and is prepared to admit it openly with apology.

There is one key note to giving municipal evidence -- that is "attitude". He must not develop an antagonistic attitude or an attitude of confrontation. His evidence may mean thousands of dollars to his employer. It is important that the engineer listen to the question, answer it directly and do not mislead the court. If the engineer is asked a question which is embarrasing to his department it must be answered directly and be honest about an error if it exists. There has never been a perfect witness, as of yet, and if the engineer thinks that he can accomplish this feat he will be mistaken. He must be prepared that he will make mistakes and when he does he has to admit them openly. When a witness 'digs in' and quarrels with a lawyer over a minor point it results not only in possibly losing the point but in that the chairman of the board decides that the witness's evidence may be of little value.

The following are certain ground rules that are recommended when giving evidence for a board hearing or a court, whatever the case may be:

Posture in the box is important - stand erect, do not slouch, do not slur your words and speak out. Be sure to talk in a direction where the chairman of the hearing can hear you. Quite often a lawyer questioning the engineer will walk around the courtroom and if the engineer speaks to him the chairman may not hear his comments. It is recommended that the engineer half face the chairman and half face the legal council table.

5.9 (continued)

Listen to the question. The witness usually gets into problems with his evidence because he has not listened to the question.

Make sure that the lawyer's question is understood. If it is not understood what the solicitor is getting at - and quite often lawyers mix up their own questions and don't know what they are looking for, then by all means, "ask the lawyer to repeat the question". It is surprising how many lawyers have not thought their question out carefully.

When a question can be answered in two ways then by all means phrase the answer in this method. The object is to help the court and not help to mislead them. The attitude should be maintained of co-operation and courtesy, both to the lawyer and to the chairman, no matter how angry the lawyer may become. Keep the composure and keep the courtesy. If the examiner becomes antagonistic it gives the engineer the opportunity to show courtesy and straightforwardness. Do not adopt the counter attitude of aggression. When the engineer requires a question to be repeated use the name of the examiner courteously and the question may then be answered by looking directly at the chairman. If the municipal engineer can stand up under an offensive attack, the court will usually have a greater respect for his opinion.

5.10 CONSTRUCTION, SURVEY AND INSPECTION

1.0 Introduction

Reference should be made to the following publication in particular

"Construction Manual"
Ministry of Transportation and Communications,
Ontario; Operations Division,
Construction Branch,
Downsview, Ontario.

2.0 PROJECT INSPECTION

The importance of good inspection on a construction project cannot be over emphasized. Good inspection cannot occur without a properly trained experienced inspector. He is the day to day link between the Municipality, the Contractor and the Public. This section will deal with the inspector, his duties, necessary education and training.

Dealing firstly with education and training it is the writers opinion that the Construction Inspector of today must be better educated and trained than ever before. The complexity of the work, the materials being used and the quality being demanded has never been greater. Only a well educated and trained inspector can provide the competence to perform his required duties. Basic minimum education should be at the Community College technical level. The education of the inspector cannot stop there. In our ever changing world of technology his education must be a continuing process. This can be accomplished formally through Continuing Education Programmes at the Community College Level, Courses sponsored by various Provincial Agencies or In-House training programmes. Irrespective of the formal education obtained by the Inspector it cannot and should not be considered adequate without considerable on-the-job training. The varied duties of todays Construction Inspector as outlined later demand this. There is no substitute for experience. The new Inspector should work with experienced personnel in the first instance and only with adequate experience gained and ability demonstrated should he be given a project of his own.

2.0 (continued)

As indicated above the duties of the inspector are varied. His main duty is to protect the interest of the Municipality but in carrying out this duty he must be fair to the Contractor. However, in being fair to the Contractor he must also be firm in his assistance that the works are constructed properly and in accordance with the Contract Documents. The majority of Contracts place the onus on the Contractor for complying with all applicable By-Laws, Statutes and Regulations, etc. and for carrying out the work in such a manner as will not unnecessarily or unreasonably inconvenience the public. While it is the Contractors responsibility to comply, etc., with the aforementioned it is the inspector's duty to make sure that he does and when necessary to remind him or order him. The inspector is often the last person who has the opportunity to spot mistakes or shoddy workmanship such as dimensional discrepancies, improper reinforcement placing or sizing or deterioration of soil conditions before the affected part of the work has been completed and any necessary corrections have become an expensive operation. The inspector must require that proper corrections be made in order that the work comply with requirements of the Contract. However, the inspector should be wary of making modifications himself to the original requirements and in general should refer such matters to his superivsor on those responsible for the design of the work. Most Contracts relegate the method of construction and construction equipment to the responsibility of the Contractor except in special circumstances involving complex design, particular soil conditions or restricted working area. There will be times however, when it becomes apparent to the experienced inspector that the method or equipment being used is unsuitable to the work. The inspector should advise the Contractor of this and if he is suitably experienced and confident he may suggest alternatives. The inspector should avoid running the Contractors job for him and sometimes he may be doing just that without realizing it. In extreme cases where the method or plant being used by the Contractor is dangerous to people, property or the work itself the inspector may have to order the work stopped or construction methods changed. This is a difficult decision to make and normally unless an immediately dangerous situation has arisen should not be taken without consultation with his supervisor. The inspector should try and spot substandard materials before they are incorporated in the work and make the Contractor aware of such.

2.0 (continued)

An important aspect of the inspectors work is the maintenance of good public relations on the project itself and as it relates to other affected parts of the Community. The inspector must make sure that adequate notification is given to affected local authorities and property owners and tenants of proposed interruption of services, temporary blocking of streets, detours, blasting operations, etc. He must be courteous to the public and willing to listen to their complaints and take appropriate action where necessary. By extending these courtesies he will find the people are generally more co-operative and more willing to put up with such inconveniences. His good public relations must also extend to the Contractor while still being firm with him and where necessary commanding authority. The period of inconvenience will be shortened and hence public relations improved if cleanup follows closely behind construction on streets and easements. Few Contractors will carry out cleanup and restoration promptly on their own accord as they tend to look on this work as a non-paying item and a nuisance. Good Contractors recognize the value of promptly completing this work however as stated it is a problem area. The Public on the other hand tends to judge a sewer or watermain job by what is visible above ground after the underground installations are complete. The inspector therefore must keep close watch on the timing of such work. When dealing with the Public and for that matter the Contractor the inspector must make sure that he does not make promises that cannot be carried out. He cannot commit the Contractor to work that is not required under the Contract nor can he commit the Municipality to additonal work without first obtaining prior approval.

Last but far from least the inspector must keep neat accurate records of daily progress and occurrences. His instructions to the Contractor must be well documented and explicit. Quite often he will be involved in the preparation of Payment Certificates and all such must be carefully substantiated. It can not be stressed enough the importance of good documentation of progress on a construction project. These documents will be retained and used not only at the time of construction but at completion and in the future should problems occur.

As stated at the outset of this section of the role of todays construction inspector is varied and complex. It is safe to say that properly educated and trained inspectors play one of the most important roles in any construction operation.

2.0 (continued)

CONSTRUCTION INSPECTOR'S DAILY REPORT

ROAD

DATE: April 16 1984
CONTRACTOR: Ballet Const.
STREET: First St.
JOB NO.: 9999
TIME CONTRACTOR STARTED: 7:00 AM
FROM: Second St.
CONTRACT NO.: C200-81-25
TIME CONTRACTOR FINISHED: 6:00 PM
TO: Third St.
DWG. NO.: PRB-1000
SOIL: CLAY
WEATHER & TEMPERATURE: Sunny & Warm 15°C

WORK VISITED BY

TITLE	NAME	TIME
Layout man	W. Warwick	8:** a.m.
"	R. Medley	10:00 am
	F.G. Marks	11:00 am
	A. Bernard	1:00 pm
	D. Brownlow	"
	A.L. Brownlow	"

MEN

NO.	DUTY
1	Foreman
4	Labourers

EQUIPMENT

TYPE	SIZE	OPER.
Asphalt Spread	B.G.	1
Grader	555	1
Vib. Roller	DA-50	1
Rub. Roller	BV210	1
Dozer	850	1
4 Trucks	Tan.	4

MATERIALS PAID BY THE TON

H.L.-6 Asphalt		20mm Crushed			
WEIGHT	DEL. SLIP NO.	WEIGHT	DEL. SLIP NO.	WEIGHT	DEL. SLIP NO.
40600	123456	10600	12345		
37000	654321	12300	54321		
		10000	67890		
		11500	05876		
		14700	10925		
		8700	52901		

DESCRIPTION & LOCATION OF DAY'S WORKING OPERATIONS

① Placing Curb & gutter on south side from Second St to sta. 5+75.

② Placing H.L.-6 Asphalt #3 lift from Second St. to 12+38.

DAY-WORK (INDICATE VARIATION ORDER)

V.O.-1000 - Adjust water valve @ sta 10+30 2 men @ 2 hrs each.

V.O.-1001 - Lower Existing M/H @ sta. 12+00

Foreman 2 hrs 2 lab. 2 hrs each

TESTING & TEST RESULTS

CONTRACT ITEMS COMPLETED TODAY

ITEM	DESCRIPTION	STATION	DAILY	TOTAL
1	Site clearing		0.2	0.5
2	General Excavation		1000 m³	1500 m³
6	Adjust Catchbasin	11+57	1	1 each
13	Curb & Gutter		200 m	275 m
14	50mm Crushed stone	12+38	650 m²	1250 m²
16	H.L.-6 Asphalt	12+38	300 m²	600 m²

REMARKS Etobicoke Hydro removing conflicting poles on the west side, station 10+00 to 12+50.

(use other side if additional space necessary)

TIME DETOUR SIGNS CHECKED: 9:00 a.m. 6:00 p.m.
TIME LIGHTS AND BARRICADES PLACED ON JOB: a.m. 5:30 p.m.
REPORT CHECKED BY: D Brownlow (Const. Section / Roads Section)

TIME IN 7:00 a.m. TIME OUT 6:00 p.m. TOTAL HOURS FOR DAY 10½ TOTAL HOURS ON THIS JOB 10½

Inspector's Signature

Foreman's Signature

(273) I. CONSTRUCTION SECTION FORM ENG No. 136 REVISION No 8 JAN. 73

2.0 (continued)

CONSTRUCTION INSPECTOR'S DAILY REPORT
SIDEWALK

DATE: APRIL 16 1981
CONTRACTOR: TRINETTI
STREET: FIRST St. (W/S)
JOB No: A-8300
TIME CONTRACTOR STARTED: 7:00 AM
FROM: Second St.
CONTRACT No: C200-80-7
TIME CONTRACTOR FINISHED: 6:00 PM
TO: Third St.
DWG. No: PWB-999
TOTAL LENGTH OF SIDEWALK:
LENGTH COMPLETED TO DATE:

WORK VISITED BY

Title	Name	Time
Inspector Main		
Chief Insp.	D. MacDonald	11:00 a.m.
Eng. Dept.	N. H. Orr	9:30 a.m.
	A. Bernard	2:15 p.m.
	D. Brownlow	"

MEN

No.	Duty
1	Foreman
4	Labourers

EQUIPMENT

Type	Size	Oper.
Loader	933F	1
Backhoe	580B	
Tandem Truck		1
Bomag	BW 100	

DATE JOB STARTED: APRIL 1 1981
DATE JOB COMPLETED:
WEATHER: Sunny & Warm 15°C

SUBGRADE PREPARED TODAY: From 0+00 To 1+00
SIDEWALK PLACED TODAY: From 0+00 To 1+00

CONTRACT ITEMS

Item	Description	Daily	Total
1.	Sq. M — Sidewalk 100mm Thick.	140.5	140.5
2.	Sq. M — Sidewalk 150mm Thick.	10.0	10.0
3.	Sq. M — Sidewalk 200mm Thick.	100.0	250.0
	PROVISIONAL ITEMS		
5	Supply & Place imported fill	7.5 m³	10.0 m³
7	Break out concrete	2 m²	2 m²
9	Remove curb & gutter	25 m	100 m

Item	Description	Daily	Total
	Note: Concrete supplier and ticket numbers and weights should be listed on back of report if more room is required.		

REMARKS

Concrete used: - 25 MPA - 32.5 kg/m³

1st load - 0+00 - 0+40 - 5.6% AIR - C.B.M # 123456
2nd load - 0+40 - 0+70 - 6% AIR - C.B.M. # 654321
3rd load - 0+70 - 1+00 - 6% AIR - C.B.M. # 222222

(Use other side if additional space necessary)

REPORT CHECKED BY
Chief Construction Inspector

TIME IN 7:00 A.M. TIME OUT 6:00 P.M. TOTAL HOURS FOR DAY 10½ TOTAL HOURS ON THIS JOB 10½

Inspector's Signature

Foreman's Signature

(E. 44)

FORM, ENG. No. 29 REV. No. 3 MARCH 69

1. CONSTRUCTION SECTION

2.0 (continued)

ENGINEERING DEPARTMENT
VARIATION ORDER

№ 3735

CONTRACT: 81-99
CONTRACTOR: Ace Const. Co.
JOB No. A-7500
STREET: First St.
UTILITY STORM
FROM: Second Ave. TO: Third Pl.
DATE Apr. 16 1981

Relay 300mm Storm sewer at Station 7+77 Due to confliction with existing Bell Telephone manhole and underground ducts.

Basis of Payment: Time and Materials.

ORDER CHECKED BY:
SIGNED:
TITLE:
INSPECTOR

FORM ENG. NO. 84 REVISION NO. 2 DEC. 74
2-95

2.0 (continued)

Performance and End of Maintenance Inspection - Roads Acceptance Criteria

After the construction is completed, A Performance Inspection of the road works is carried out.

Similarly, one month prior to the end of the maintenance period for roads constructed under Subdivision Agreement, or constructed by a contractor on behalf of the City, an End of Maintenance Inspection is carried out.

In each case, the inspection shall record particularly the following points:

(a) PAVEMENT

(1) Shall note size and location of any potholes.
(2) Shall note extent and location of settlement depressions and frost heaving.
(3) Shall note size and location of crazing and surface cracking of the asphalt, since this usually indicates sub-grade failure.
(4) Shall note extent and location of asphalt ravelling.
(5) Shall note location of opened joints in the asphalt course at construction joints.
(6) Shall note extent and location of surface ponding conditions.
(7) Shall note that pavement is constructed with a proper crossfall and an adequate crown.
(8) Shall note that construction is to correct dimensions.

(b) CURBS AND GUTTERS

(1) Shall note that concrete is true and smooth in surface texture.
(2) Shall note extent and location of scaling, spalling or pitting.
(3) Shall note line and level and shall record all instance of curbs with more than 13 mm settlement at joints.
(4) Shall note locations of extruding expansion joints.
(5) Shall note locations of structural failure and settlement cracks.
(6) Shall note non-standard driveway cut-outs.
(7) Shall note all damage to pavement, curbs, catchbasins, caused by any agency.

2.0 (continued)

(c) CATCHBASINS AND UTILITY APPURTENANCES

(1) Shall record all cases where catchbasins are wrongly located to intercept road drainage.
(2) Shall note all cases where valve-boxes, manholes, etc. are not to the finished pavement grade.
(3) Shall note all cases where frames and covers do not sit firm and true.
(4) Shall note all damaged covers, valve boxes, etc.
(5) Shall record any road materials noted in CB's and manholes.

Further to the above noted items, asphalt and concrete coring may be carried out as well as Benkelman Beam Testing, prior to the end of the maintenance period.

The extent and the responsibility for the required repairs or replacements shall be evaluated and the contractor advised accordingly.

Performance and End of Maintenance
Sidewalk Acceptance Criteria

After the sidewalk construction is completed, a Performance Inspection is carried out. Similarly, at the end of the maintenance period, an End of Maintenance Inspection shall be carried out.

In each case, the inspection shall note particularly the following points:-

(1) All bays containing one or more cracks in excess of 1.5 mm in width shall be reported.

(2) All bays having a displacement from the original grade in excess of 6 mm shall be reported.

(3) All locations where sidewalks obstruct drainage, shall be reported.

(4) All locations where expansion joint material project above the surface of the sidewalk, shall be reported.

(5) Spalling of the concrete surface shall be noted and the extent of the spalling shall be indicated.

2.0 (continued)

(6) Concrete cores may be taken (at the discretion of the City Engineer) every 150 m and the thickness of the concrete to the nearest 3 mm shall be recorded.

(7) Any other unusual construction or variations from the plans and specifications (e.g. broken corners) shall be recorded.

The extent and the responsibility for the required repairs or replacements shall be evaluated and the contractor advised accordingly.

3.0 PROJECT SUPERVISION

In the previous section the importance of good inspection was dealt with. Similarly this section will deal with good supervision which again cannot occur without properly trained, educated and experienced supervisors. The supervisor and supervision discussed here is primarily based in the construction project area. However, the qualities and requirements of good supervision are common irrespective of the area of concern and can be considered applicable in any area of supervision involvement.

What exactly is the function of the project supervisor? The answer to this is somewhat dependent on the type and size of the project being considered. However, the skills the supervisor must have and show are relatively common to all projects. For an example let us consider a Municipal Construction Contract. Once an award of this contract is made the supervisor assumes direction of the work as the authorized representative of the Municipality. It is the supervisor's function to inspect and approve all operations and finished products related to the Contract and to accept or reject the work on behalf of the Municipality. He usually has and is responsible for a number of technical staff to assist in his duties. They could include survey crews, inspectors, accounting and office staff, weighmen, checkers and quality control personnel. His duties are varied and his responsibility great.

Perhaps the best way to begin a section dealing with supervision is to define it. One of the simplist and perhaps the most meaningful definitions in the writers view is that a supervisor is anyone who has formal authority and responsibility over other workers. One of his primary concerns is to cause his individual workers to work co-operatively and collectively, not only as individuals to create the efficiency necessary to supply the required service. While doing this the supervisor must also have due regard for the personal interests of the employees.

Considering the above one often assumes rightly or wrongly that to be a good supervisor one must be jack-of-all-trades including a human behaviour scientist. The supervisor obviously must have a sound technical background and knowledge in his special area of concern. He is often expected to know as much or more about the technical details of his work than anyone else. It is not of prime importance that he be able to do the job of any of his subordinates better than the subordinate. However, it is of importance that he know the rudiments of their work, to be able to discuss with them their problems intelligently and to earn

3.0 (continued)

their confidence and respect. He must have knowledge of and skill in applying fundamental concepts in such areas as (1) human behaviour, (2) organization, (3) leadership and (4) the basic supervisory functions of planning, organizing, directing and controlling. He must also be skilled in public relations as much of his work involves public contact. He must be able to organize not only his subordinates but himself also to meet schedule changes, major breakdown, etc. He must be capable of solving technical problems which constantly confront him and his subordinates and which are often not only complex but also unpleasant. He must not waste his subordinates talents or resources nor must he forget the economic cost of his staff. Perhaps the definition of jack-of-all-trades is not too far off the mark.

The education of a supervisor must be a continuing process. He must not only be soundly educated and trained technically in his area of concern he must also know how to effectively communicate and deal with people and obtain from them their best efforts and performance. Everywhere there is evidence of increasing demands on the knowledge, skill and integrity of the supervisor. The relentlessly increasing knowledge regarding the technical, economic and social aspects of the supervisors work is a force of major consequence that the supervisor must deal with. Even as the supervisor goes about his job on a day to day basis his job is changing. New concepts and specialized techniques related to his work are constantly being developed and improved. He must continually supplement and reinforce his education and experience. He can not supervise effectively by following prevailing practice and his own past experience. The effective supervisor cannot remain static.

I briefly mentioned earlier the importance of communications. The importance of such cannot be overstressed with relation to the supervisors work. He is required to communicate effectively with his staff, contractors, the public and his superiors. Much has been written on the art of communication and it is not the writers intention to develop this area only to stress its importance, whether written or oral.

As has been pointed out the effective supervisor must be a many faceted being. Good supervisors are one of the most important cogs in any smooth running operation whether it be Municipal, or otherwise. They are normally in the "front lines" so to speak and can make or break an operation. They should be selected with care, properly trained and experienced on a continuing intelligent basis.

4.0 PAYMENTS AND COST ACCOUNTING

PAYMENTS

There are many different types of Contracts involving various methods of payment. To name a few of the most common - Lump Sum Contracts - Unit Price Contracts - Competitive Bid Contracts - Combined Lump Sum and Unit Price Contracts - Negotiated Contracts. A Municipality normally uses either the Competitively Bid Unit Price or Lump Sum Contracts. In either case as the work progresses, the Contractor is usually entitled to partial payments which are made on the Construction Supervision's estimates of the amount of work completed or as invoiced by the Contractor and substantiated by the Construction Supervisor. On Lump Sum Contracts the estimate will be expressed as a percentage of the total work to be done. For Unit Price Contracts the Construction Supervisor will measure the quantities of work completed under each item of the Contract and these will indicate the degree of completion. In arriving at the amount of partial payment, consideration may be given to the amount of material delivered to the site and work done by the Contractor at other assembly points as well as the completed work on the site.

Partial payments are usually made by the Municipality each month. The payment covers work completed the previous month and are normally due two weeks after month end. These payments generally cover 80-85 percent of the actual cost of the work completed. Fifteen percent of the monies due must be held as holdback by the Municipality in accordance with the Mechanics Lien Act. This fifteen percent is to provide protection to the Municipality against such claims or charges as failure of the Contractor to reimburse Sub-contractors, Material Dealers or Employees. It must be released to the Contractors 37 days following substantial completion of the work and proper certifications from the Contractor that there are no outstanding claims against the work. The above has been written in a very simplified manner and the reader is referred to the Mechanics Lien Act, Statutes of Ontario for detailed proceedings.

In addition to the 15 percent it is not uncommon for a Municipality to retain an additional 5 percent for a period of 12 months after certification of completion of the work as a guarantee against faulty workmanship.

The final payment to the Contractor will include all retained percentages. Before the final payment is made however, the Construction Supervisor will make a final inspection of the work as a condition of formal acceptance by the Municipality. The final inspection shall determine any omitted portions of the work, all defective work to be

4.0 (continued)

corrected and any deductions to be made in the Contract amount for defective work to be accepted in lieu of its removal and replacement. The Contractor should provide the Municipality certification that the final quantities, deductions, payment, etc. are acceptable and recognized as such and a release from any claims, liens, etc. against the work.

COST ACCOUNTING

The Accounting methods and procedure to be used on a construction project are of major importance. Rather than dealing with specific accounting methods and procedures which are readily available and varied this section will deal more generally with the ground rules of good accounting. The prime rule of any accounting procedure is that the records must be complete and accurate. A variety of practical alternatives exist in recording information. In addition to completeness and accuracy the quality of clarity and meaningfulness are important. A reader of the records should be able to discern quickly where items have been entered, what they are and their dollar effect. One important indication of the quality and effectiveness of a recording system is the ease with which updated cost data can be taken from such. It should be remembered that these records will normally be subjected to possibly both internal and external auditing.

Information to be recorded should be entered promptly so that the records are up to date at all times. Economy and efficiency in recording requires that a daily routine be established for entering information of all kinds and with no exceptions. The routines established should be as simple as possible and easily explained so the system will not be disrupted by the absence of the person normally doing the work and can readily be taken over by a replacement. Records should be designed for expansion so that increases in volumes of any item or transaction can be handled without disruption of the system. The daily records should provide a complete record of progress of the work.

Dealing more specifically with the construction project the accounting and recording system to be used must be established before the project begins. It cannot be successfully put together after the work is underway. Also important is that a clear and systematic method of taking progress measurements and filing the progress calculations must be established at the commencement of the work in order to obtain correct progress quantities during the job and also

4.0 (continued)

to aid in compiling accurate final quantities. It should always be kept in mind that in the process of construction many essential measurements can be obtained only at a certain time. Close co-ordination between the Supervisory staff and the Contractor is a decisive factor in the timely acquisition of the necessary measurements. The Ministry of Transportation and Communications, Province of Ontario has developed excellent systems for recording and processing such data. The reader is referred to their Construction Manual for details of these systems.