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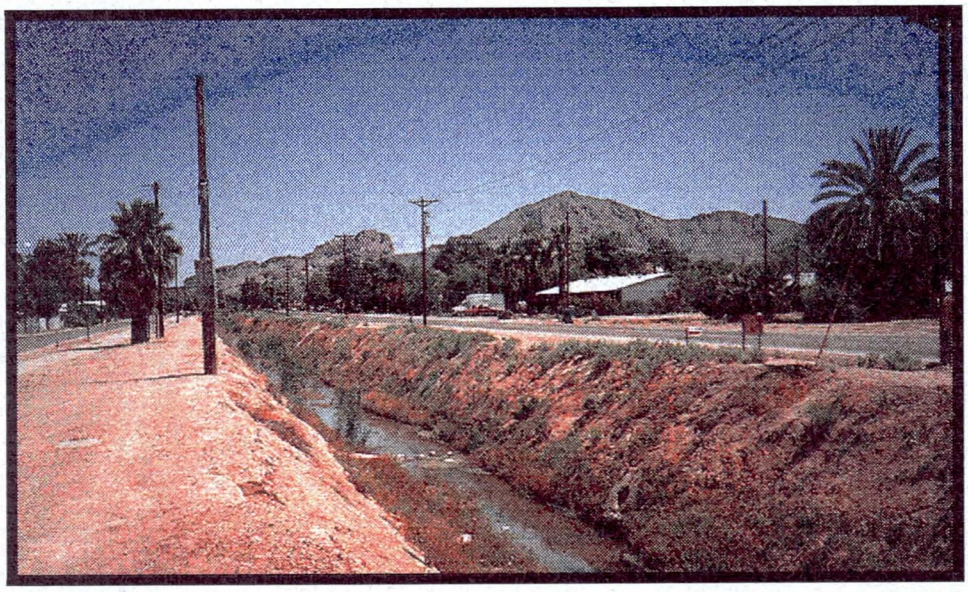
OLD CROSS CUT CANAL RECONSTRUCTION



BACKUP HYDRAULIC CALCULATIONS

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REMARKS

VOLUME 1

FCDMC PROJECT NO. 94-24
GREINER PROJECT NO. E002102

JULY 1994

BEGINNING EG & HG ELEVATIONS FOR THE STORM PROGRAM

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of ____

Subject Beginning Hydraulic Grade El. for the STORM Pgm.

By RHF Date 5-21-94 Checked By N.W. Date 7-22-94

Purpose : Determine the appropriate beginning hydraulic grade line elevation (HGL) to use in the STORM Pgm.

Procedure : Using the EG previously computed at Sta. 11+00 from the HEC-2 CHIMP Model, file name : ~~CMPMDL21.DAT~~ and El. ~~1197.82~~ 1197.89* minus the velocity head of the box culvert plus the feet of head loss due to the expansion loss from the proposed double 18'x10' CBC to the open channel reach from sta. 11+00 to Sta. 10+00.

Note Changes in red due to New HEC-2 CHIMP Run.

Calculations :

EG elevation at 11+00 = ~~1197.83~~ 1197.89

Box culvert velocity = 11.4 fps (Vu) HEC-2 File name is CMPMDL22.DAT.

Box culvert velocity head (hv) = 2.02'

HGL = EG - hv = ~~1195.81~~ 1195.87

Downstream Velocity = ~~1.08~~ 1.07 fps, (STA. 10+99) (V_D) Changed Weir elevation at McDowell to 1194.0 from 1193.0.

$$he = Ke \frac{(V_D - V_u)^2}{2g}$$

$$Ke = 3.50 \left(\tan \frac{\theta}{2}\right)^{1.22}$$ (Gibson Enlargers Standard of Hydraulic Institute)

$$\tan \frac{\theta}{2} = \frac{5.42'}{100'} = 0.0542$$ (see sheet 2)

$$Ke = 3.50 (0.0542)^{1.22} = 0.10$$

$$he = 0.10 \frac{(\cancel{1.08}\ 1.07 - 11.4)^2}{2g} = 0.17'$$ use 0.20'

Beginning HGL = [(EG + he) - hv] = [(~~1197.82~~ 1197.89 + 0.20) - 2.09] = ~~1196.01~~ 1196.01

Use 1196.01

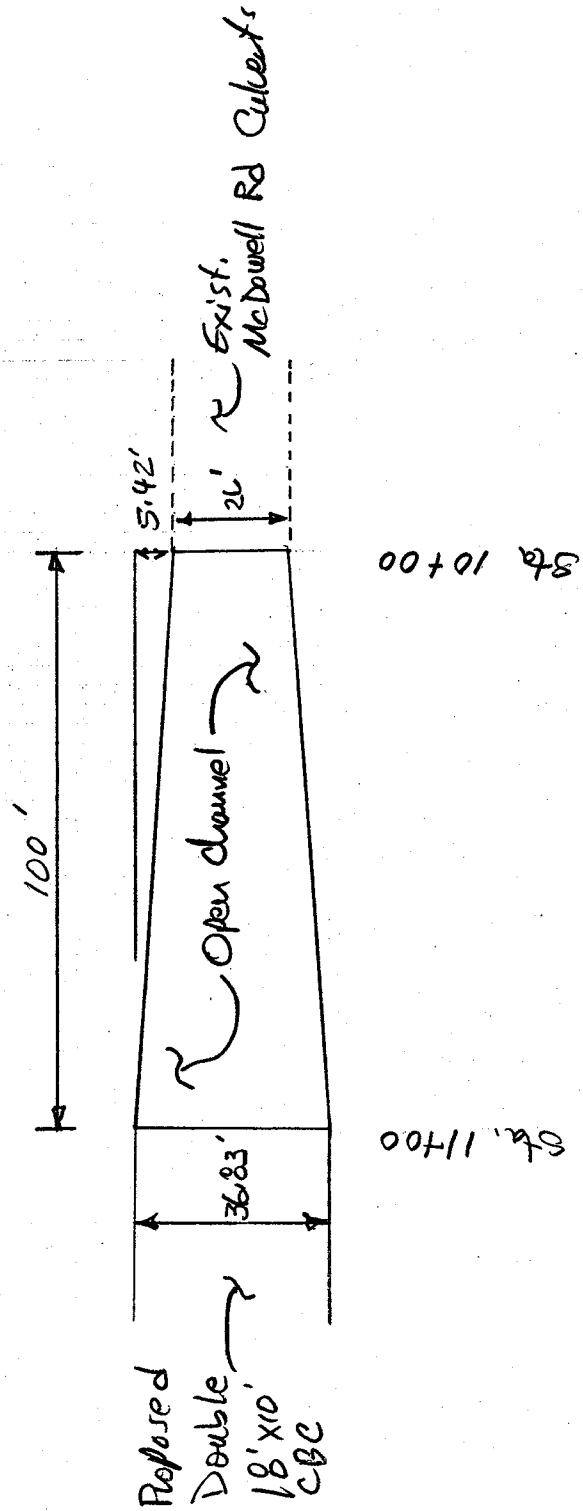
* Note this EG was agreeded upon by Greiner & FCDMC per letter dated 4-25-94.

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 2 of ____

Subject Beginning Hydraulic Grade El. for STORM Agm.

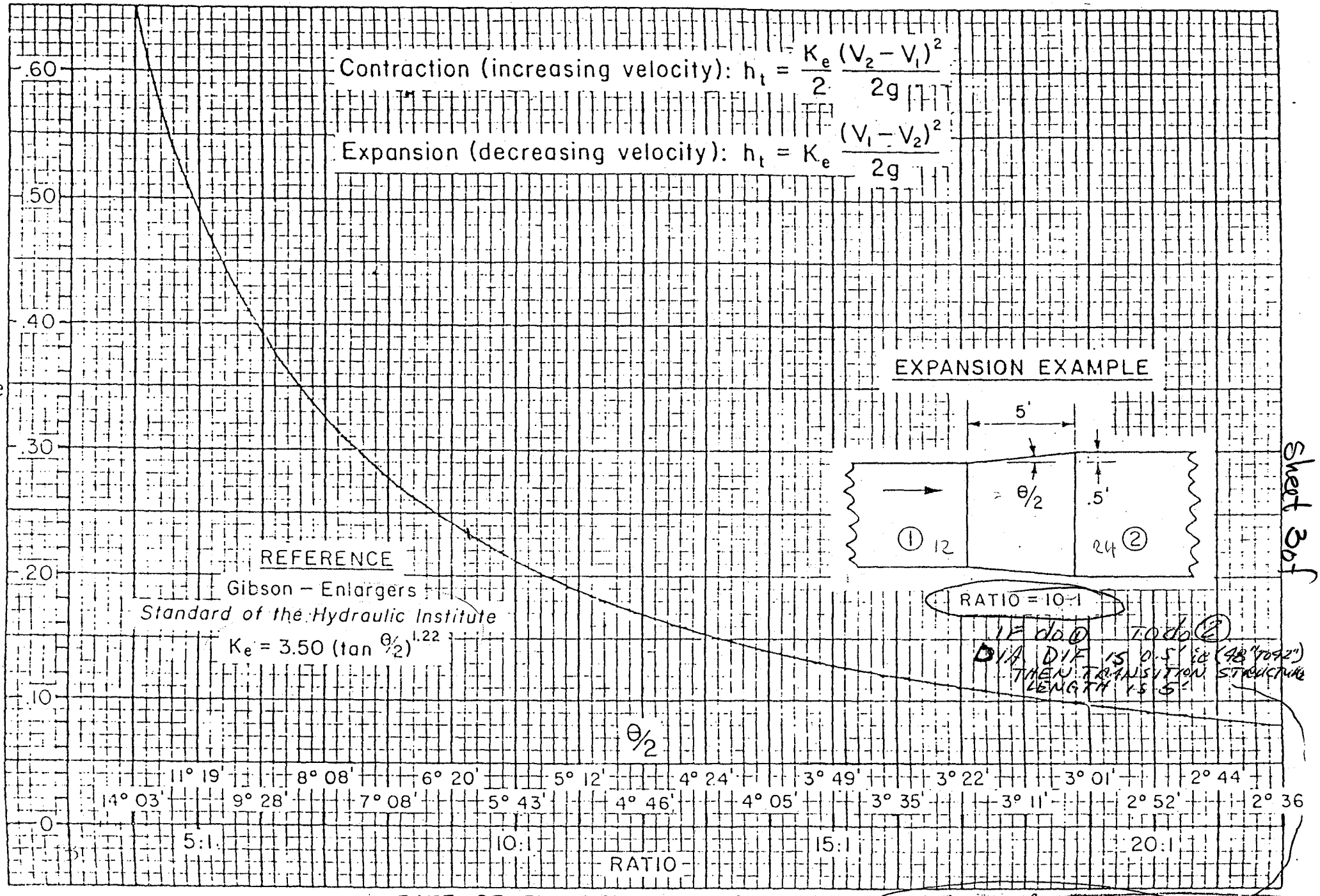
By RHF Date 5-21-94 Checked By N. W. Date 7-22-94



TRANSITION HEAD LOSS — Pressure Flow

Contraction (increasing velocity): $h_t = \frac{K_e}{2} \frac{(V_2 - V_1)^2}{2g}$

Expansion (decreasing velocity): $h_t = K_e \frac{(V_1 - V_2)^2}{2g}$



Sheet 3 of

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 4 of ____

Subject Beginning Hydraulic Grade El. for the STORM Pgm.

By RHF Date 5-21-94 Checked By N.W Date 7-22-94

Purpose: Determine the transition loss from the McDowell Road Culvert upstream through the open channel reach from sta. 10+00 to 11+00.

Procedure:

1.) The velocity in the McDowell Rd culvert = 11.58 fps.

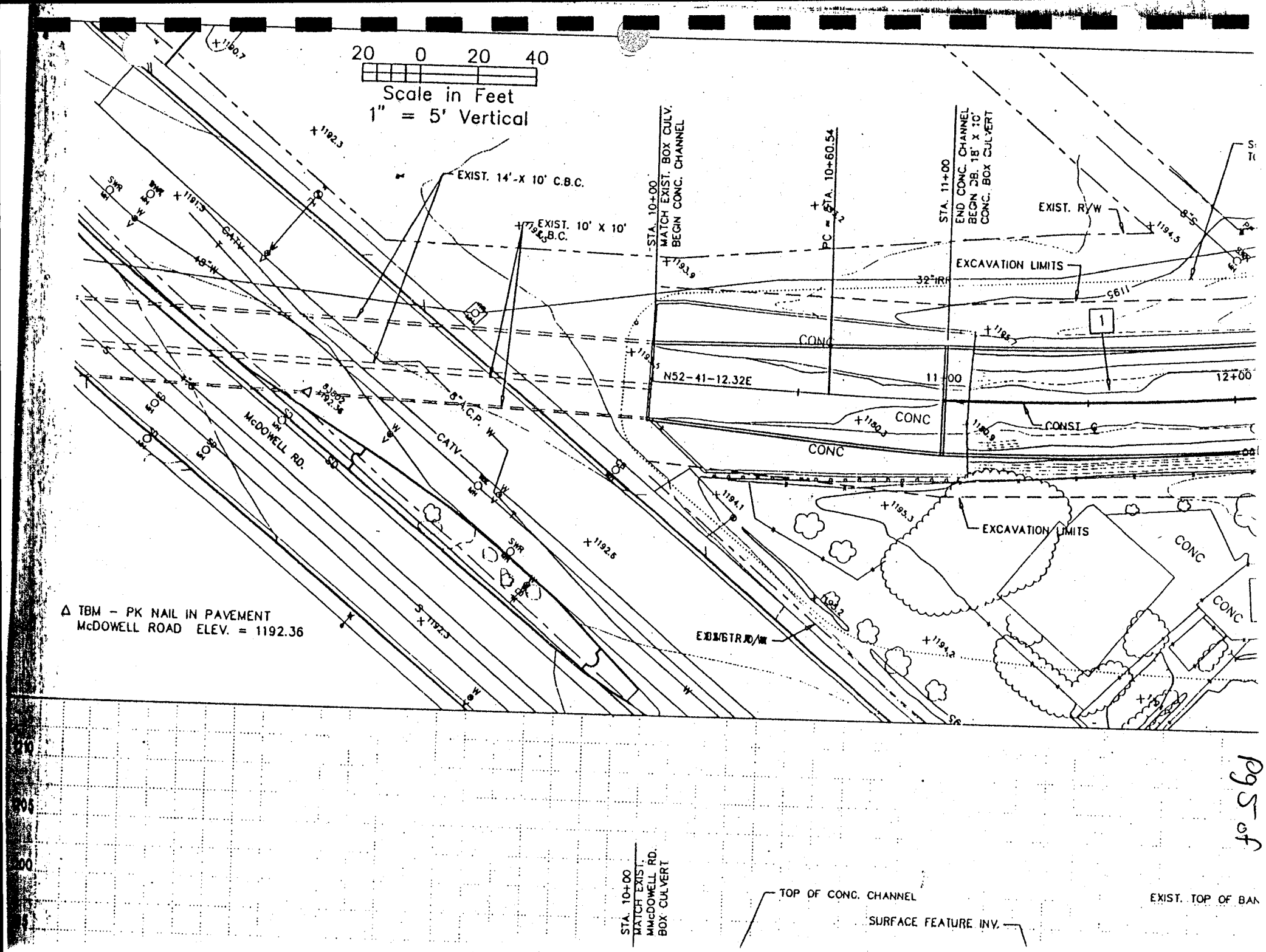
2.) Open channel Velocity = ~~1.08 fps.~~ 1.07 fps

3.) Culvert hv = 1.87 ft Sta 10+00 (Inlet Headwall McDowell)

4.) Channel hv = 0.02 ft. OK sta 10+99 (Outlet Headwall 2-15'x10' cpc)

5.) Δ hv = 1.85' OK since hv culvert > hv channel contraction.

6.) hc = Kc (|Δhv|) = 0.3 (1.85) = 0.56' OK


20 0 20 40
Scale in Feet
1" = 5' Vertical
EXIST. 14'-X 10' C.B.C.
EXIST. 10' X 10' C.B.C.
STA. 10+00
MATCH EXIST. BOX CULV.
BEGIN CONC. CHANNEL
PC = STA. 10+60.54
STA. 11+00
END CONC. CHANNEL
BEGIN DB. 18' X 10'
CONC. BOX CULVERT
EXIST. R/W
EXCAVATION LIMITS
32" IRF
N52-41-12.32E
11+00
12+00
CONC
CONST ℄
McDOWELL RD.
8" A.C.P.
CATV
EXIST. R/W
TBM - PK NAIL IN PAVEMENT
McDOWELL ROAD ELEV. = 1192.36
STA. 10+00
MATCH EXIST.
McDOWELL RD.
BOX CULVERT
TOP OF CONC. CHANNEL
SURFACE FEATURE INV.
EXIST. TOP OF BAN

pg 5 of

JUNCTION LOSSES

Revised 8-8-94

JUNCTION LOSS CALCULATIONS

PROJECT NAME: OLD CROSS CUT CANAL
PROJECT NO.: E002102

DESIGNER: RHT
DATE: 7-20-94

CKED BY: N.W.
DATE: 7-21-94

SECNO	STREET NAME	STORM PGM. MAINLINE NO.	DOWNSTREAM STRUCTURE Q (cfs)	NUMBER OF CULVERTS	WIDTH (ft.)	HEIGHT/ DIAMETER (ft.)	AREA (sq.ft.)	PERIMETER (ft.)	FRICTION SLOPE (ft/ft)	VELOCITY (fps)	VELOCITY HEAD (ft.)	
19+00	GRANADA	2	4170	2	18	10	360.00	112.00	0.0021647	11.58	2.08	ok
35+81	OAK	5	4170	2	18	10	360.00	112.00	0.0021647	11.58	2.08	ok
62+49	THOMAS S.	10	3900	2	18	10	360.00	112.00	0.0018935	10.83	1.82	
64+34	THOMAS N.	11	3900	2	18	10	360.00	112.00	0.0018935	10.83	1.82	
75+85	EARLL	14	3800	2	18	10	360.00	112.00	0.0017976	10.56	1.73	
83+00	RICHARDSON	17	3800	2	12	10	240.00	88.00	0.0050353	15.83	3.89	
88+61	OSBORN S.	18	2780	2	12	10	240.00	88.00	0.0026949	11.58	2.08	
90+50	OSBORN N.	19	2780	2	12	10	240.00	88.00	0.0026949	11.58	2.08	
96+64	WHITTON	20	2750	2	12	10	240.00	88.00	0.0026371	11.46	2.04	
100+22	WELDON	22	2720	2	12	10	240.00	88.00	0.0025798	11.33	1.99	

SECNO	STREET NAME	STORM PGM. MAINLINE NO.	UPSTREAM STRUCTURE Q (cfs)	NUMBER OF CULVERTS	WIDTH (ft.)	HEIGHT/ DIAMETER (ft.)	AREA (sq.ft.)	PERIMETER (ft.)	FRICTION SLOPE (ft/ft)	VELOCITY (fps)	VELOCITY HEAD (ft.)	
19+00	GRANADA	2	4113	2	18	10	360.00	112.00	0.0021060	11.43	2.03	ok
35+81	OAK	5	3721	2	18	10	360.00	112.00	0.0017237	10.34	1.66	ok
62+49	THOMAS S.	10	3894	2	18	10	360.00	112.00	0.0018877	10.82	1.82	
64+34	THOMAS N.	11	3850	2	18	10	360.00	112.00	0.0018452	10.69	1.78	
75+85	EARLL	14	2745	2	18	10	360.00	112.00	0.0009380	7.63	0.90	
83+00	RICHARDSON	17	3765	2	12	10	240.00	88.00	0.0049429	15.69	3.82	
88+61	OSBORN S.	18	2594	2	12	10	240.00	88.00	0.0023464	10.81	1.81	
90+50	OSBORN N.	19	2730	2	12	10	240.00	88.00	0.0025988	11.38	2.01	
96+64	WHITTON	20	2741	2	12	10	240.00	88.00	0.0026198	11.42	2.03	
100+22	WELDON	22	2646	2	12	10	240.00	88.00	0.0024414	11.03	1.89	

SECNO	STREET NAME	STORM PGM. MAINLINE NO.	LATERAL STRUCTURE Q (cfs)	NUMBER OF CULVERTS	WIDTH (ft.)	HEIGHT DIAMETER (ft.)	AREA (sq.ft.)	PERIMETER (ft.)	FRICTION SLOPE (ft/ft)	VELOCITY (fps)	VELOCITY HEAD (ft.)	
19+00	GRANADA	2	57	1		3.5	9.62	11.00	0.003210	5.92	0.55	ok
35+81	OAK	5	449	1	8	6	48.00	28.00	0.003264	9.35	1.36	ok
62+49	THOMAS S.	10	6	1		3	7.07	9.42	0.000081	0.85	0.01	
64+34	THOMAS N.	11	50	1		2.5	4.91	7.85	0.014860	10.19	1.61	
75+85	EARLL	14	1055	1	12	10	120.00	44.00	0.001552	8.79	1.20	
83+00	RICHARDSON	17	35	1		2.5	4.91	7.85	0.007281	7.13	0.79	
88+61	OSBORN S.	18	186	1	6	6	36.00	24.00	0.001190	5.17	0.41	
90+50	OSBORN N.	19	50	1		4	12.57	12.57	0.001212	3.98	0.25	
96+64	WHITTON	20	9	1		1.5	1.77	4.71	0.007341	5.09	0.40	
100+22	WELDON	22	74	1		4	12.57	12.57	0.002654	5.89	0.54	

810 610
4170
- 449
3721

SECNO	STREET NAME	STORM PGM. MAINLINE NO.	JUNC STR LGTH (ft.)	LAT ANGLE TO MAIN (deg.)	DELTA Y (ft.)	JUNC LOSS Hj (ft.)	Q IN STORM PROGRAM	V IN D.S. LINE	hv IN D.S. LINE	LOSS COEF. Kj	
19+00	GRANADA	2	4.04	60.0	0.107	0.0506	4100.00	11.39	2.01	0.03	ok
35+81	OAK	5	36.96	12.5	0.567	0.1426	4100.00	11.39	2.01	0.07	
62+49	THOMAS S.	10	3.46	60.0	0.018	0.0119	4100.00	10.83 ~~11.39~~	1.82 ~~2.01~~	0.01	
64+34	THOMAS N.	11	2.89	60.0	0.076 0.87	0.0299	4100.00	10.83 ~~11.39~~	1.82 ~~2.01~~	~~0.01~~ .02	
75+85	EARLL	14	55.44	12.5	0.949 ✓	0.1220	4100.00	10.56 11.39	1.73 ~~2.01~~	~~0.06~~ .07	
83+00	RICHARDSON	17	2.89	60.0	0.141	0.0696	4100.00	15.83 ~~17.08~~	3.89 ~~4.53~~	0.02	
88+61	OSBORN S.	18	27.72	12.5	0.487	0.2179	4100.00	11.58 ~~17.08~~	2.08 ~~4.53~~	~~0.05~~ 0.105	
90+50	OSBORN N.	19	5.66	45.0	0.145	0.0710	4100.00	11.58 ~~17.08~~	2.08 ~~4.53~~	~~0.02~~ 0.03	
96+64	WHITTON	20	1.50	90.0	0.031	0.0173	4100.00	11.46 ~~17.08~~	2.04 ~~4.53~~	~~0.00~~ 0.01	
100+22	WELDON	22	4.00	90.0	0.224	0.1171	4100.00	10.08	1.58 4.53	~~0.03~~ REV. 7-1-94 0.07	

4100
3900
3800
2780
2750
2720

why ? Not a true Velocity

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of

Subject Junction Loss Calculation at Granada example hand Calc.

By RHF Date 6-17-97 Checked By Neal Wanek Date 7-21-97

$h_j = \Delta y + hv_{US} - hv_{DS}$

where $\Delta y = \left[\dfrac{(Q_{DS}V_{DS}) - (Q_{US}V_{US}) - (Q_L V_L \cos\theta)}{[\frac{1}{2}(A_{UP} + A_{DS})]\, g}\right] + \left[\left[\frac{1}{2}(S_{f_{US}} + S_{f_{DS}})\right] L\right]$

	DS	U.S.	Lat.	Units
Q	4170	4113	57	cfs
V	11.5833	11.4250	5.9245	fps
A	360	360	9.6211	ft^2
Sf	0.0021647	.0021060	—	ft/ft
θ	—	—	60°	Degrees.

Length = 4.04'

$\Delta Y = 0.10718$ ft. ✓

$hv_{US} = 2.0269$ ft ✓

$hv_{DS} = 2.0834$ ft. ✓

$h_j = 0.10718 + 2.0269 - 2.0834 = 0.05068 = 0.0507$ ft.

$h_j = K_j\, hv_{DS}$

$K_j = (h_j / hv_{DS})$

$K_j = (.0507 / 2.0834) = 0.0243$ use 0.03

D.S. Structure = 2-18'x10' CBC

U.S. Structure = 2-18'x10' CBC

Lateral Structure = 1-42" RCP

Mannings: $Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$

Friction Slope: $S_f = \left[\dfrac{Qn}{1.486 A R^{2/3}}\right]^2$ ✓

TRANSITION LOSSES

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of 2

Subject Transition Loss Calculations Sta 77+00 to 77+25

By RHF Date 2-17-94 Checked By Ned Date 7-22-94

Revised 8/5/94

Why storm program has input of 1900 cfs

Down stream		Upstream	
1390 cfs/CBC	(2780 Total)	1390 cfs/CBC	(2780 Total)
1-18'x10'	(2-18'x10')	1-12'x10'	(2-12'x10')
A = 180 ft²	(360 ft²)	A = 120 ft²	(240 ft²)
V = 7.72 fps	(7.72 fps)	V = 7.72 fps	(7.72 fps)

BOX

Length = 25'

Transition Calculations:

Increasing velocity: $h_t = \frac{K_e}{2}(hv_D - hv_u)$

Decreasing Velocity $h_t = K_e(hv_u - hv_D)$ ✓

$K_e = 3.5\,(\tan \theta/2)^{1.22}$

$\tan \theta/2 = 6/12 = 0.24$ ✓ $\therefore K_e = 0.6137$ ✓

Decreasing Velocity.

$\therefore h_t = K_e(hv_u - hv_D)$

$h_t = 0.6137\left(\frac{15.8^2}{64.4} - \frac{10.6^2}{64.4}\right)$

$= .6137(3.88 - 1.74)$

$= 1.31$

$h_t = 0.6137\,(2.08 - 0.93) = 0.7058$ ✓

K_t in Storm Program $= \frac{0.7058}{\left(\frac{hv_u + hv_D}{2}\right)} = 0.47$

$K_e = 3.5 * (\tan \theta/2)^{1.22}$

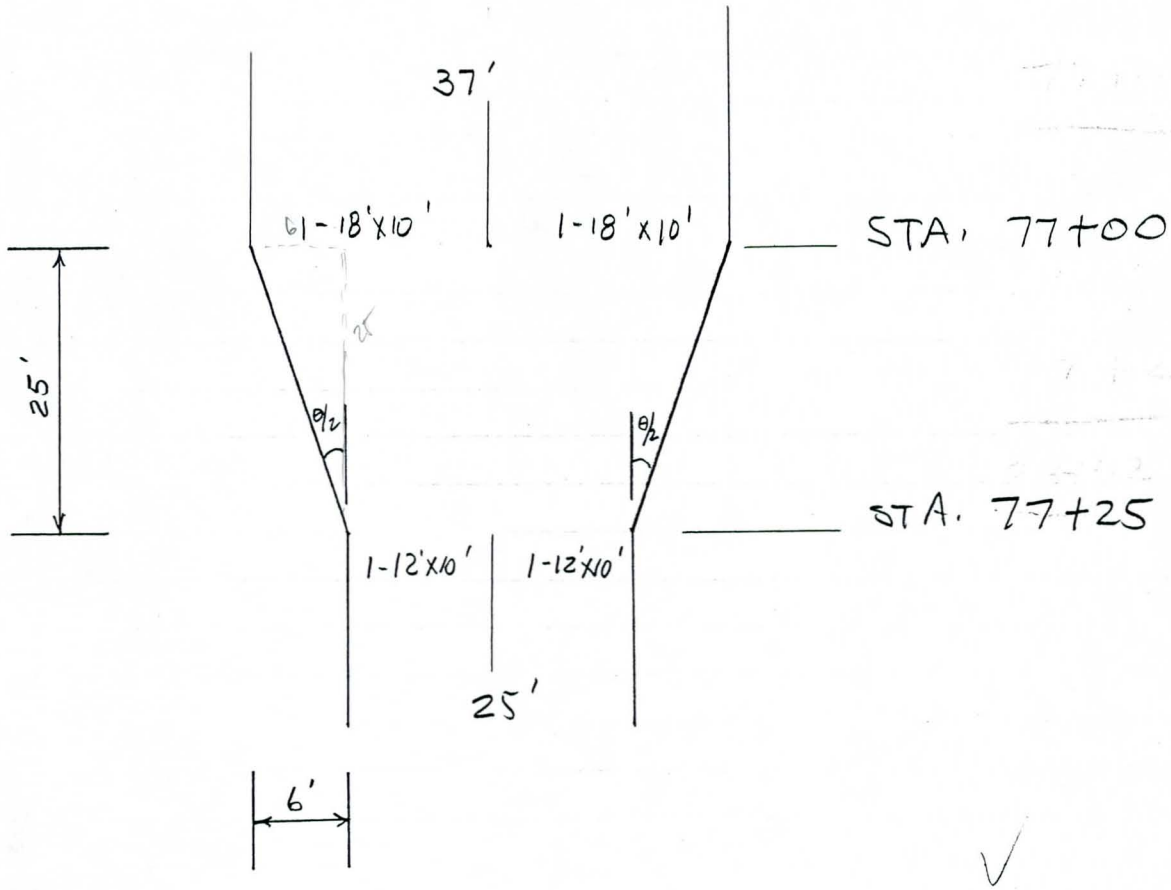
$= 3.5 * (0.24)^{1.22}$

$= 0.6137$

$K_t = \frac{1.31}{\left(\frac{3.88 + 1.74}{2}\right)} = 0.47$

Greiner

Job Old Cross Cut Canal | Project No. E002102 | Sheet 2 of 2
Description Transition Upstream of Ecell | Computed By RHF | Date 2-17-94
Checked By N. W. | Date 7-22-94



$\tan\frac{\theta}{2} = \frac{6}{25} = 0.24$

$\frac{\theta}{2} = \tan^{-1}(0.24)$

$= 13.5$

$\theta = 27^\circ$

Greiner

Job Old Cross Cut Canal | Project No. E002101 | Sheet 1 of 2
Description Transition Length | Computed By RHF | Date 2-17-94
Sta 101+00 | Checked By N.W. | Date 7-21-94

check storm program input

Downstream		Upstream	
Q = 1180 cfs/CBC	(2360 cfs)	? Q = 1180 cfs/barrel	(2360 cfs)
1 - 12'x10' CBC	(2-12'x10' CBC)	1 - 10' x 10' CBC	(2-10'x10')
A = 120 ft²	(240 ft²)	A = 100 ft²	(200 ft²)
V = 9.83 fps	(9.83 fps)	V = 11.80 fps	(11.80 fps)
hv_D = 1.50 ft.	(1.50 ft)	hv_u = 2.16 ft	(2.16 ft.)

Length = 10'

Increasing Velocity $h_t = K_e/2\,(hv_D - hv_u)$

Decreasing Velocity $h_t = K_e\,(hv_D - hv_u$

$K_e = 3.5\,(\tan \theta/2)^{1.22}$

$\tan \theta/2 = 2/10 = 0.2 \therefore K_e = 0.4913$ ✓

Decreasing Velocity

$\therefore h_t = K_e\,(hv_u - hv_D)$

$h_t = 0.4913\left(\frac{12.1^2}{64.4} - \frac{10.0^2}{64.4}\right)$

$= .4913\,(2.27 - 1.55)$

$= 0.35$ ft

$h_t = 0.4913\,(2.16 - 1.50) = 0.3243$

$K_t = \dfrac{h_t}{\left[\frac{(hv_u + hv_D)}{2}\right]} = 0.18$ ✓

$K_t = \dfrac{0.35 \times 2}{(2.27 + 1.55)}$

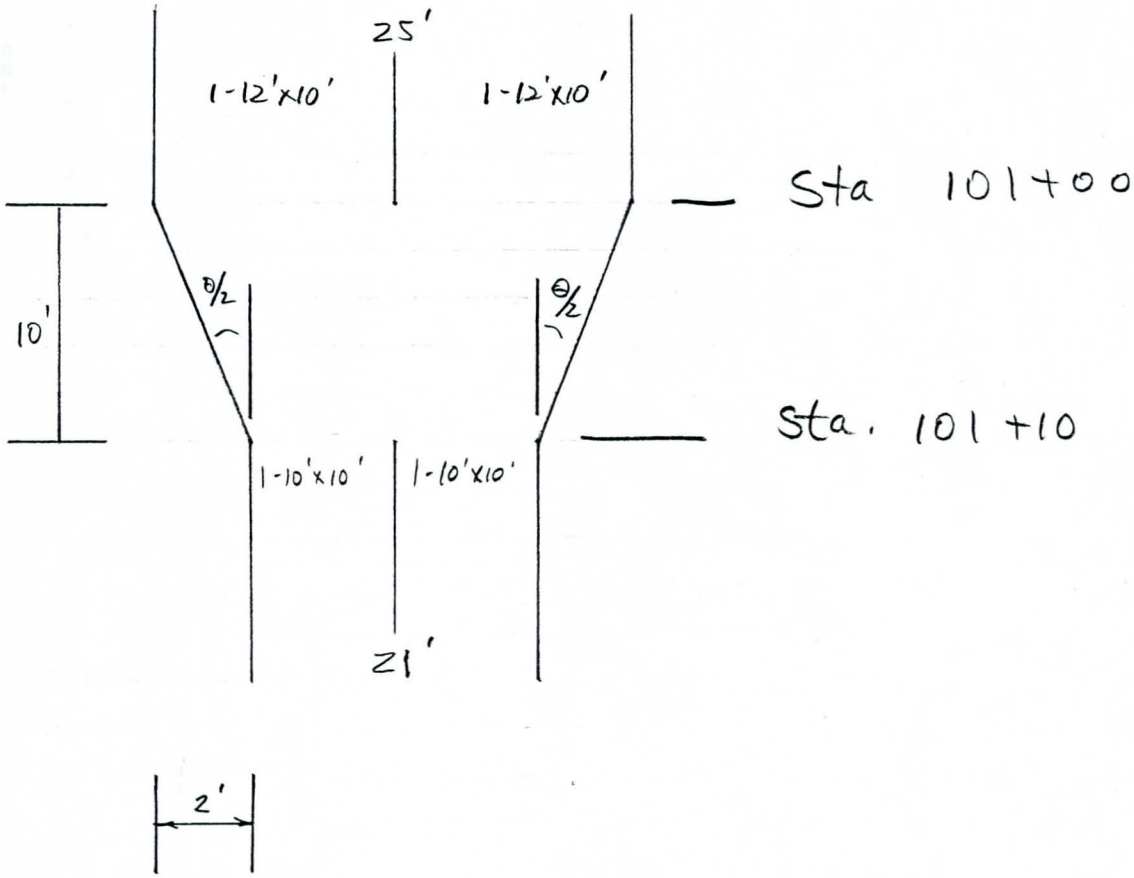
$= 0.185$

Greiner

Job Old Cross Cut Canal | Project No. E002102 | Sheet 2 of 2

Description Transition Length STA. 101+00 | Computed By RHF | Date 2-17-94

Checked By _____ | Date _____



HYDRAULICS FOR INLETS

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of 2

Subject Lateral Analysis for HG at Cat. Basin - General Storm

By RHF Date 7-19-94 Checked By N.W. Date 7-24-94

Roadway: Granada Connector Sta. = 19+00

South Side Inlet Q = 57 cfs Grate El: 1199.30

$A = \frac{\pi (\frac{42}{12})^2}{4} = 9.62 \text{ ft}^2$

EG at Mainline Sta 19+00 = 1199.92

Length of 42" RCP = 91'

$R = \frac{D}{4} = \frac{42/12}{4} = 0.875$

$\theta = 11°$ * (Mid-line bend)

$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$

$S_f = 0.003210$ ft/ft. OK

In plan sheets it's 109'!

$S_f = \left[\frac{Qn}{1.486 A R^{2/3}}\right]^2$

Eg in Lateral at Inlet:

$V = \frac{57}{9.62} = 5.93 \frac{ft}{sec}$

$S_f = \left[\frac{57 * .013}{1.486 * 9.62 * .875^{2/3}}\right]^2 = .00321$ ft/ft

$h_f = S_f * L = 91 (.003210) = 0.29'$

$h_b = K_b * h_v =$

$V = 5.92$ fps

$h_v = 0.54$

$K_b = 0.0335$

from where

$V_{lat} = Q_{lat} / A_{lat}$

$h_v = (V_{lat})^2 / 2g$

friction losses:

$h_f = S_f \times L = .00321 \times 109$

$= 0.35'$

$h_b = 0.0335 (0.54) = 0.02$

$h_b = K_b \frac{V^2}{2g} =$

$\Sigma h = h_f + h_b = 0.29 + 0.02 = 0.31$ ft.

Lateral EG at Inlet = D.S. Box EG + Σh = 1199.92 + 0.31 = 1200.23

Lateral HG at Inlet = EG at Inlet − h_v = 1200.23 − 0.54 = 1199.69

Note: Although HG > Grate El, by pressure analysis the inlet will intercept more than 57 cfs. This assumes 3 grates w/ 50% clogging. Note 4 grates provided

$\theta = \tan^{-1} (S_1 - S_2) = \tan^{-1} (.202 - .012) = 11.03°$

$$K_L = \frac{2g H_p}{(a/a_g)^2} =$$

PROJECT NAME:	OLD CROSS CUT CANAL	Grate Net Area:	4.25	No. OF GRATES:	3
JOB NUMBER:	E002102	Height of Catch Basin:	4	Ke:	0.50
		Top Slab Thickness:	1.75	Ko:	1.00
GRATE TYPE:	C.O.P. Standard Detail P1565 Type 1 Grate	Length:	5.75	Kf:	0.0611
CATCH BASIN LOCATION:	GRANADA RD.	Manning's n:	0.014		
TOP OF APRON ELEVATION	1201.80	Grate Total Area:	6.67		
PIPE HG AT INLET	1199.65 .65	Total Perimeter:	10.67		
GRATE ELEVATION:	1199.30	Hydraulic Radius:	0.6251		

Q Assumed Discharge	Hp Pressure Head	Ho Orifice Head	An Net Grate Open Area	Ag Total Grate Open Area	Kl Calculated Total Loss Coefficient	Kl−Kt Total Loss Coefficient Without Trashrack Loss	Ktg Calculated Trashrack Adjusted Coefficient	Vn Net Grate Area Velocity	Hvn Net Grate Area Velocity Head	Vg Total Grate Area Velocity	Hvg Total Grate Area Velocity Head	Ktn Calculated Trashrack Coefficient	Ktn "Small Dam's" Trashrack Coefficient	Qo Discharge Orifice Equation
60	2.11	2.50	12.75	20.01	15.1133	1.5611	13.5523	4.7059	0.3439	2.9985	0.1396	5.5022	0.7573	108.39
70	2.11	2.50	12.75	20.01	11.1037	1.5611	9.5426	5.4902	0.4680	3.4983	0.1900	3.8743	0.7573	108.39
80	2.11	2.50	12.75	20.01	8.5012	1.5611	6.9402	6.2745	0.6113	3.9980	0.2482	2.8177	0.7573	108.39
90	2.11	2.50	12.75	20.01	6.7170	1.5611	5.1560	7.0588	0.7737	4.4978	0.3141	2.0933	0.7573	108.39
100	2.11	2.50	12.75	20.01	5.4408	1.5611	3.8797	7.8431	0.9552	4.9975	0.3878	1.5752	0.7573	108.39
110	2.11	2.50	12.75	20.01	4.4965	1.5611	2.9355	8.6275	1.1558	5.4973	0.4693	1.1918	0.7573	108.39
120	2.11	2.50	12.75	20.01	3.7783	1.5611	2.2173	9.4118	1.3755	5.9970	0.5584	0.9002	0.7573	108.39
125	2.11	2.50	12.75	20.01	3.4821	1.5611	1.9211	9.8039	1.4925	6.2469	0.6060	0.7800	0.7573	108.39
126.015	2.11	2.50	12.75	20.01	3.4262	1.5611	1.8652	9.8835	1.5168	6.2976	0.6158	0.7573 =	0.7573	108.39
A	B	C	D	E	F	G	H	I	J	K	L	M	N	O

2.15
1201.8
1199.65
2.15

1201.8
−1199.3
2.5

4.25
×3

6.67
×3

NOTE: Qo Should Be > Q (Column 'A' < Column 'O'); Column 'M' Should = Column 'N'

$Q_o < Q$ Orifice Eq. Governs

$0.67*(3*4.25)\sqrt{2(32.2)(2.5)}$

$= 108.39$ cfs

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of 2

Subject Later Analysis for HG at Catch Basin - General Storm

By RHF Date 7-19-94 Checked By N.W. Date 7-25-94

Roadway: Oak Connector Sta. 35+81

Grate El. = 1200.68 Q = 449 cfs

EG at Mainline Sta. 35+81 = 1203.71

Length of 8'x6' CBC = 85'

There are 1 - angles of 12.5° use 13° each.

Sf = 0.003264 ft/ft

EG in Lateral at Inlet:

$h_f = S_f * L = 85 (0.003264) = 0.28'$

$h_b = K_b h_v = K_b h_v = (0.04)(1.36) = 0.05 ft.$

Is this from chart or eq.

$K_{b_1} = K_{b_2} = 0.040$

$V = 9.35$ fps

$h_v = 1.36$

$\Sigma h = h_f + h_b = 0.28 + 0.05 = 0.33$ ft.

EG at Inlet = 1203.71 + 0.33 = 1204.04

HG at Inlet = EG - h_v = 1204.04 - 1.36 = 1202.68

HGL 2' higher than ground!

Note: Although HG > Grate El, by pressure analysis the inlet will intercept more than 449 cfs. This assumes 19 grates w/ 50% clogging. note: 30 grates provided.

Even though we have provided 30 grates but since our HGL is 2' higher than the top of Grates, we will have ponding condition unless we increase the size of the box or somehow lower the HGL in mainline!

sta.	ele.
37+12 3700	205.33
3800	205.58

from HEC-II Run

1205.36'

PROJECT NAME:	OLD CROSS CUT CANAL	Grate Net Area:	4.25	No. OF GRATES:	19
JOB NUMBER:	E002102	Height of Catch Basin:	4	Ke:	0.50
		Top Slab Thickness:	1.75	Ko:	1.00
GRATE TYPE:	C.O.P. Standard Detail P1565 Type 1 Grate	Length:	5.75	Kf:	0.0611
CATCH BASIN LOCATION:	OAK STREET STA. 37+12.17	Manning's n:	0.014		
CHIMP HG ELEVATION:	1205.26	Grate Total Area:	6.67		
LATERAL HG AT INLET:	1202.68	Total Perimeter:	10.67		
GRATE ELEVATION:	1200.68 (CHIMP INV. EL. = 1202.04)	Hydraulic Radius:	0.6251		

Q Assumed Discharge	Hp Pressure Head	Ho Oriface Head	An Net Grate Open Area	Ag Total Grate Open Area	Kl Calculated Total Loss Coefficient	Kl−Kt Total Loss Coefficient Without Trashrack Loss	Ktg Calculated Trashrack Adjusted Coefficient	Vn Net Grate Area Velocity	Hvn Net Grate Area Velocity Head	Vg Total Grate Area Velocity	Hvg Total Grate Area Velocity Head	Ktn Calculated Trashrack Coefficient	Ktn "Small Dam's" Trashrack Coefficient	Qo Discharge Oriface Equation
400	2.58	4.58	80.75	126.73	16.6780	1.5611	15.1170	4.9536	0.3810	3.1563	0.1547	6.1375	0.7573	929.17
500	2.58	4.58	80.75	126.73	10.6739	1.5611	9.1129	6.1920	0.5953	3.9454	0.2417	3.6998	0.7573	929.17
600	2.58	4.58	80.75	126.73	7.4125	1.5611	5.8514	7.4303	0.8573	4.7345	0.3481	2.3757	0.7573	929.17
700	2.58	4.58	80.75	126.73	5.4459	1.5611	3.8848	8.6687	1.1669	5.5236	0.4738	1.5772	0.7573	929.17
800	2.58	4.58	80.75	126.73	4.1695	1.5611	2.6085	9.9071	1.5241	6.3126	0.6188	1.0590	0.7573	929.17
850	2.58	4.58	80.75	126.73	3.6934	1.5611	2.1324	10.5263	1.7205	6.7072	0.6985	0.8657	0.7573	929.17
860	2.58	4.58	80.75	126.73	3.6080	1.5611	2.0470	10.6502	1.7613	6.7861	0.7151	0.8311	0.7573	929.17
870	2.58	4.58	80.75	126.73	3.5255	1.5611	1.9645	10.7740	1.8025	6.8650	0.7318	0.7976	0.7573	929.17
880	2.58	4.58	80.75	126.73	3.4459	1.5611	1.8848	10.8978	1.8441	6.9439	0.7487	0.7652	0.7573	929.17
882.5	2.58	4.58	80.75	126.73	3.4264	1.5611	1.8653	10.9288	1.8546	6.9636	0.7530	0.7573	0.7573	929.17

4.25 × 19 = 80.75

6.67 × 19 = 126.73

Kl − (Kl − Kt)

NOTE: Qo Should Be > Q (Column 'A' < Column 'O'); Column 'M' Should = Column 'N'

Sheet 2 of 2

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of 2

Subject Drop Inlet Analysis - General Storm

By RHF Date 7-19-94 Checked By N.W Date 7-22-94

Inlet : N. Of Windsor. Sta. 57+90

Grate EL. = 1206.76 1206.73'

Mainline HG at Drop Inlet = 1206.77

Note: Although HG > Grate El., by pressure analysis the inlet will intercept more than the 60 cfs. This assumes 4 grates w/ 50% clogging. note 8 grates provided.

Sta	HGL EL
5700	208.25
5800	208.32

~~5700 206.80~~
~~5800 207.10~~ } ~~59+90 = 207.07'~~

57+90 = 1208.31

~~HG 1207.07~~
OK

PROJECT NAME:	OLD CROSS CUT CANAL	Grate Net Area:	4.25	No. OF GRATES:	4
JOB NUMBER:	E002102	Height of Catch Basin:	4	Ke:	0.50
		Top Slab Thickness:	1.75	Ko:	1.00
GRATE TYPE:	C.O.P. Standard Detail P1565 Type 1 Grate	Length:	5.75	Kf:	0.0611
CATCH BASIN LOCATION:	WINDSOR STA. 57+90	Manning's n:	0.014		
CHIMP HG AT INLET:	1208.31	Grate Total Area:	6.67		
MAINLINE HG AT DROP INLET:	1206.77	Total Perimeter:	10.67		
GRATE ELEVATION:	1206.76	Hydraulic Radius:	0.6251		

Q Assumed Discharge	Hp Pressure Head	Ho Oriface Head	An Net Grate Open Area	Ag Total Grate Open Area	Kl Calculated Total Loss Coefficient	Kl−Kt Total Loss Coefficient Without Trashrack Loss	Ktg Calculated Trashrack Adjusted Coefficient	Vn Net Grate Area Velocity	Hvn Net Grate Area Velocity Head	Vg Total Grate Area Velocity	Hvg Total Grate Area Velocity Head	Ktn Calculated Trashrack Coefficient	Ktn "Small Dam's" Trashrack Coefficient	Qo Discharge Oriface Equation
90	1.54	1.55	17	26.68	8.7155	1.5611	7.1545	5.2941	0.4352	3.3733	0.1767	2.9047	0.7573	113.80
100	1.54	1.55	17	26.68	7.0596	1.5611	5.4985	5.8824	0.5373	3.7481	0.2181	2.2324	0.7573	113.80
110	1.54	1.55	17	26.68	5.8344	1.5611	4.2733	6.4706	0.6501	4.1229	0.2640	1.7350	0.7573	113.80
120	1.54	1.55	17	26.68	4.9025	1.5611	3.3414	7.0588	0.7737	4.4978	0.3141	1.3566	0.7573	113.80
130	1.54	1.55	17	26.68	4.1773	1.5611	2.6162	7.6471	0.9080	4.8726	0.3687	1.0622	0.7573	113.80
140	1.54	1.55	17	26.68	3.6018	1.5611	2.0408	8.2353	1.0531	5.2474	0.4276	0.8286	0.7573	113.80
142	1.54	1.55	17	26.68	3.5011	1.5611	1.9400	8.3529	1.0834	5.3223	0.4399	0.7877	0.7573	113.80
143	1.54	1.55	17	26.68	3.4523	1.5611	1.8912	8.4118	1.0987	5.3598	0.4461	0.7678	0.7573	113.80
143.54	1.54	1.55	17	26.68	3.4264	1.5611	1.8653	8.4435	1.1070	5.3801	0.4495	0.7573	0.7573	113.80

NOTE: Qo Should Be > Q (Column 'A' < Column 'O'); Column 'M' Should = Column 'N'

~~We only have 0.34' of head to push the flow~~
OK!

Sheet 2 of 2

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of 2

Subject Lateral Analysis for HG at Catch Basin - General Storm

By RHF Date 7-19-94 Checked By N.W. Date 7-24-94

Roadway: Thomas South Connector Sta. 62+49

Grate El. = 1207.00 Q = 6 cfs

EG. at Main line Sta. 62+49 = 1209.50

Length of 36" RCP = 6'

$\theta = 60°$

$S_f = 0.000081$ ft/ft.

EG in Lateral at Inlet:

$h_f = S_f * L = 6(.000081 \text{ ft/ft}) = 0.01$ ft. (Min)

$h_b = K_b * V^2/2g =$

$V = 0.85$

$h_v = 0.01$

$h_f = 0.01$

$\Sigma h = h_f = 0.01$

EG in lateral at Inlet = 1209.50 + 0.01 = 1209.51

HG in lateral at Inlet = EG - h_v = 1209.51 - 0.01 = 1209.50

Note: Although HG > Grate El, by pressure analysis the inlet will intercept more than 6 cfs. This location assumes 2 grates w/ 50% clogging. Note - grates provided.

Sta HGLEle.
6200 1209.46
6300 1209.94 6214.9 = 1209.60 OK

PROJECT NAME:	OLD CROSS CUT CANAL	Grate Net Area:	4.25	No. OF GRATES:	2
JOB NUMBER:	E002102	Height of Catch Basin:	4	Ke:	0.50
		Top Slab Thickness:	1.75	Ko:	1.00
GRATE TYPE:	C.O.P. Standard Detail P1565 Type 1 Grate	Length:	5.75	Kf:	0.0611
CATCH BASIN LOCATION:	THOMAS RD. SOUTH SIDE	Manning's n:	0.014		
CHIMP HG AT INLET:	1209.61	Grate Total Area:	6.67		
HG IN LATERAL AT INLET:	1209.50	Total Perimeter:	10.67		
GRATE ELEVATION:	1207.00	Hydraulic Radius:	0.6251		

Q Assumed Discharge	Hp Pressure Head	Ho Oriface Head	An Net Grate Open Area	Ag Total Grate Open Area	Kl Calculated Total Loss Coefficient	Kl-Kt Total Loss Coefficient Without Trashrack Loss	Ktg Calculated Trashrack Adjusted Coefficient	Vn Net Grate Area Velocity	Hvn Net Grate Area Velocity Head	Vg Total Grate Area Velocity	Hvg Total Grate Area Velocity Head	Ktn Calculated Trashrack Coefficient	Ktn "Small Dam's" Trashrack Coefficient	Qo Discharge Oriface Equation
10	0.11	2.61	8.5	13.34	12.6064	1.5611	11.0453	1.1765	0.0215	0.7496	0.0087	4.4844	0.7573	73.83
15	0.11	2.61	8.5	13.34	5.6028	1.5611	4.0418	1.7647	0.0484	1.1244	0.0196	1.6410	0.7573	73.83
17	0.11	2.61	8.5	13.34	4.3621	1.5611	2.8010	2.0000	0.0621	1.2744	0.0252	1.1372	0.7573	73.83
18	0.11	2.61	8.5	13.34	3.8909	1.5611	2.3298	2.1176	0.0696	1.3493	0.0283	0.9459	0.7573	73.83
19	0.11	2.61	8.5	13.34	3.4921	1.5611	1.9310	2.2353	0.0776	1.4243	0.0315	0.7840	0.7573	73.83
19.182	0.11	2.61	8.5	13.34	3.4261	1.5611	1.8651	2.2567	0.0791	1.4379	0.0321	0.7572	0.7573	73.83

NOTE: Qo Should Be > Q (Column 'A' < Column 'O'); Column 'M' Should = Column 'N'

Sheet 2 of 2

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of 2

Subject Lateral Analysis for HG at Inlet - General Storm

By RHF Date 7-19-94 Checked By N.W Date 7-24-94

Roadway: Thomas North Connector Sta = 64+34

Grate El. = 1207.5 Q = 50 cfs

EG at Mainline Sta = 1209.87

Length of 30" RCP = 4'

$\theta = 60°$

$K_b = 0.20\sqrt{\frac{\Delta}{90}} = 0.20\sqrt{\frac{60}{90}}$

$K_b = \underline{0.163}$

$S_f = 0.01486$

EG in Lateral at Inlet:

$h_f = S_f * L = 4(.01486) = 0.06$ ft.

$V = 10.19$ fps

$h_v = 1.61$ ft.

$h_b = K_b * \frac{V^2}{2g} = 0.163 * \frac{(10.2)^2}{64.4}$

$= 0.26$

Since the V is too high the consultant shall cal. $h_b = K_b * \frac{V^2}{2g}$

already included w/ storm Input

$\Sigma h = h_f = 0.06$ ft. $+ 0.26' = 0.324'$

$EG = 1209.87 + 0.06 = 1210.03$ $EG = 1209.87 + 0.324 = 1210.19'$

$HG = EG - h_v = 1210.03 - 1.61 = 1208.42$

$1219.19 - 1.61 = 1208.58'$

Note: Although HG > Grate El, by pressure analysis the inlet will intercept more than 50 cfs.

Why clogging factor not applied to this Catch basins

6400 212.19'
6500 212.19' ⇒ HGL @ 64+34 = 212.19

~~Have to Include bend loss!~~ OK

PROJECT NAME:	OLD CROSS CUT CANAL	Grate Net Area:	4.25	No. OF GRATES:	2
JOB NUMBER:	E002102	Height of Catch Basin:	4	Ke:	0.50
		Top Slab Thickness:	1.75	Ko:	1.00
GRATE TYPE:	C.O.P. Standard Detail P1565 Type 1 Grate	Length:	5.75	Kf:	0.0611
CATCH BASIN LOCATION:	THOMAS RD. NORTH SIDE	Manning's n:	0.014		
CHIMP HG AT INLET:	1212.19	Grate Total Area:	6.67		
HG IN LATERAL AT INLET:	1208.42 — 1208.58	Total Perimeter:	10.67		
GRATE ELEVATION:	1207.50	Hydraulic Radius:	0.6251		

Q Assumed Discharge	Hp Pressure Head	Ho Oriface Head	An Net Grate Open Area	Ag Total Grate Open Area	Kl Calculated Total Loss Coefficient	Kl−Kt Total Loss Coefficient Without Trashrack Loss	Ktg Calculated Trashrack Adjusted Coefficient	Vn Net Grate Area Velocity	Hvn Net Grate Area Velocity Head	Vg Total Grate Area Velocity	Hvg Total Grate Area Velocity Head	Ktn Calculated Trashrack Coefficient	Ktn "Small Dam's" Trashrack Coefficient	Qo Discharge Oriface Equation
50	3.77	4.69	8.5	13.34	16.54 ~~17.2822~~	1.5611	14.987 ~~15.7211~~	5.8824	0.5373	3.7481	0.2181	6.3828	0.7573	98.97
60	3.77	4.69	8.5	13.34	12.0015	1.5611	10.4405	7.0588	0.7737	4.4978	0.3141	4.2388	0.7573	98.97
70	3.77	4.69	8.5	13.34	8.8174	1.5611	7.2564	8.2353	1.0531	5.2474	0.4276	2.9461	0.7573	98.97
80	3.77	4.69	8.5	13.34	6.7509	1.5611	5.1898	9.4118	1.3755	5.9970	0.5584	2.1071	0.7573	98.97
90	3.77	4.69	8.5	13.34	5.3340	1.5611	3.7730	10.5882	1.7408	6.7466	0.7068	1.5318	0.7573	98.97
100	3.77	4.69	8.5	13.34	4.137 ~~4.3205~~	1.5611	2.5761 ~~2.7595~~	11.7647	2.1492	7.4963	0.8726	1.19 1.1204	0.7573	98.97
110	3.77	4.69	8.5	13.34	3.5707	1.5611	2.0097	12.9412	2.6005	8.2459	1.0558	0.8159	0.7573	98.97
112	3.77	4.69	8.5	13.34	3.4443	1.5611	1.8833	13.1765	2.6960	8.3958	1.0946	0.7646	0.7573	98.97
112.295	3.77	4.69	8.5	13.34	3.4262	1.5611	1.8652	13.2112	2.7102	8.4179	1.1003	0.7573	0.7573	98.97
	3.61	ok												

NOTE: Qo Should Be > Q (Column 'A' < Column 'O'); Column 'M' Should = Column 'N'

$$\frac{2g\,H_p}{\left(\frac{Q_p}{a_g}\right)^2} = \frac{2(32.2)(3.61)}{\left(\frac{50}{13.34}\right)^2} = 16.5487$$

$$= \frac{2(32.2)(3.61)}{\left(100/13.34\right)^2} = 4.137$$

$$H_{vg} = H_{vt}$$

$$K_{tg} * h_{tg} = K_{tn} * h_{tn}$$

$$K_{tg} = \frac{K_{tg}\,h_{tg}}{h_{tn}} = \frac{14.987 * 0.2181}{0.5373} = 6.084$$

$$K_{tn} = 4.137 * 0.$$

Sheet 2 of 2

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of ___

Subject Lateral Analysis for HG at Catch Basin - General Storm

By RHF Date 7-19-94 Checked By N.W Date 7-24-94

Roadway : Earll Connection Sta = 75+85

Grate El. = 1209.75 Q = 1055 cfs

EG at Mainline sta. = 1212.11

Length of 12'x10' Lateral = 32'

$\theta = 12.5^\circ$ use 13°

$S_f = 0.001552$

$h_b = \cancel{K_b \frac{V^2}{2g}}$

$K_b = \cancel{0.2 * \sqrt{\frac{A}{90}}}$

$= 0.2 * \sqrt{\frac{13}{90}}$

$= 0.08$

EG in Lateral at Inlet:

$h_f = S_f * L = 32 (.001552) = 0.05$ ft.

$h_b = 0.08 * \frac{(8.79)^2}{64.4}$

$= 0.09$ ft.

$V = 8.79$ fps

$h_v = 1.20$

$h_t = h_f + h_b = 0.05 + 0.09 = 0.1412$ ft

$\Sigma h = h_f = 0.05$

EG in lateral at Inlet = 1212.11 + 0.05 = 1212.16 (0.1412) ~~1212.2512~~ OK

HG = 1212.16 - hv = 1212.16 - 1.20 = 1210.96 ~~1211.05~~ OK

Note: Although the HG > Grate El. by pressure analysis the inlet will intercept more that 1055 cfs

this assumes 42 grate w/ 50% clogging

60 provided.

~~See prev. page~~ OK

PROJECT NAME:	OLD CROSS CUT CANAL	Grate Net Area:	4.25	No. OF GRATES:	40
JOB NUMBER:	E002102	Height of Catch Basin:	4	Ke:	0.50
		Top Slab Thickness:	1.75	Ko:	1.00
GRATE TYPE:	C.O.P. Standard Detail P1565 Type 1 Grate	Length:	5.75	Kf:	0.0611
CATCH BASIN LOCATION:	EARLL DRIVE	Manning's n:	0.014		
MAXIMUM ALLOWABLE PONDING EL.:	1213.00	Grate Total Area:	6.67		
HG IN LATERAL AT INLET:	1210.96	Total Perimeter:	10.67		
GRATE ELEVATION:	1209.75	Hydraulic Radius:	0.6251		

1211.05

Q Assumed Discharge	Hp Pressure Head	Ho Oriface Head	An Net Grate Open Area	Ag Total Grate Open Area	Kl Calculated Total Loss Coefficient	Kl−Kt Total Loss Coefficient Without Trashrack Loss	Ktg Calculated Trashrack Adjusted Coefficient	Vn Net Grate Area Velocity	Hvn Net Grate Area Velocity Head	Vg Total Grate Area Velocity	Hvg Total Grate Area Velocity Head	Ktn Calculated Trashrack Coefficient	Ktn "Small Dam's" Trashrack Coefficient	Qo Discharge Oriface Equation
1000	2.04	3.25	170	266.8	9.3516	1.5611	7.7906	5.8824	0.5373	3.7481	0.2181	3.1630	0.7573	1647.81
1100	2.04	3.25	170	266.8	7.7286	1.5611	6.1676	6.4706	0.6501	4.1229	0.2640	2.5040	0.7573	1647.81
1200	2.04	3.25	170	266.8	6.4942	1.5611	4.9331	7.0588	0.7737	4.4978	0.3141	2.0029	0.7573	1647.81
1300	2.04	3.25	170	266.8	5.5335	1.5611	3.9725	7.6471	0.9080	4.8726	0.3687	1.6128	0.7573	1647.81
1400	2.04	3.25	170	266.8	4.7712	1.5611	3.2102	8.2353	1.0531	5.2474	0.4276	1.3033	0.7573	1647.81
1500	2.04	3.25	170	266.8	4.1563	1.5611	2.5952	8.8235	1.2089	5.6222	0.4908	1.0537	0.7573	1647.81
1600	2.04	3.25	170	266.8	3.6530	1.5611	2.0919	9.4118	1.3755	5.9970	0.5584	0.8493	0.7573	1647.81
1650	2.04	3.25	170	266.8	3.4349	1.5611	1.8739	9.7059	1.4628	6.1844	0.5939	0.7608	0.7573	1647.81
1652	2.04	3.25	170	266.8	3.4266	1.5611	1.8656	9.7176	1.4663	6.1919	0.5953	0.7574	0.7573	1647.81
1652.05	2.04	3.25	170	266.8	3.4264	1.5611	1.8654	9.7179	1.4664	6.1921	0.5954	0.7573	0.7573	1647.81

CHK. 1.95

NOTE: Qo Should Be > Q (Column 'A' < Column 'O'); Column 'M' Should = Column 'N'

Sheet 2 of 2

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of 2

Subject Lateral Analysis for HG at Inlet - General Storm

By RHF Date 7-19-94 Checked By N.W. Date 7-26-94

Roadway: Richardson Connector Sta. 83+00

Grate El. = 1214.15 Q = 35 cfs

EG at Mainline Sta. 83+00 = 1217.21

Length of 30" RCP = 6'

θ = 60°

Sf = 0.007281 ft/ft.

Why bend losses are ignored?

OK!

EG in Lateral at Inlet:

hf = Sf * L = 6(.007281) = 0.04'

V = 7.13 fps

hv = 0.79

Σh = hf = .04

Lateral EG at Inlet = 1217.21 + 0.04 = 1217.25

Lateral HG = 1217.25 - hv = 1216.48

PROJECT NAME:	OLD CROSS CUT CANAL	Grate Net Area:	4.25	No. OF GRATES:	2
JOB NUMBER:	E002102	Height of Catch Basin:	4	Ke:	0.50
		Top Slab Thickness:	1.75	Ko:	1.00
GRATE TYPE:	C.O.P. Standard Detail P1565 Type 1 Grate	Length:	5.75	Kf:	0.0611
CATCH BASIN LOCATION:	RICHARDSON	Manning's n:	0.014		
MAXIMUM ALLOWABLE PONDING EL.:	1217.31	Grate Total Area:	6.67		
HG IN LATERAL AT INLET:	1216.48	Total Perimeter:	10.67		
GRATE ELEVATION:	1214.15	Hydraulic Radius:	0.6251		

Q Assumed Discharge	Hp Pressure Head	Ho Oriface Head	An Net Grate Open Area	Ag Total Grate Open Area	Kl Calculated Total Loss Coefficient	Kl−Kt Total Loss Coefficient Without Trashrack Loss	Ktg Calculated Trashrack Adjusted Coefficient	Vn Net Grate Area Velocity	Hvn Net Grate Area Velocity Head	Vg Total Grate Area Velocity	Hvg Total Grate Area Velocity Head	Ktn Calculated Trashrack Coefficient	Ktn "Small Dam's" Trashrack Coefficient	Qo Discharge Oriface Equation
20	0.83	3.16	8.5	13.34	23.7802	1.5611	22.2192	2.3529	0.0860	1.4993	0.0349	9.0210	0.7573	81.24
40	2.04	3.16	8.5	13.34	14.6119	1.5611	13.0509	4.7059	0.3439	2.9985	0.1396	5.2987	0.7573	81.24
60	2.04	3.16	8.5	13.34	6.4942	1.5611	4.9331	7.0588	0.7737	4.4978	0.3141	2.0029	0.7573	81.24
80	2.04	3.16	8.5	13.34	3.6530	1.5611	2.0919	9.4118	1.3755	5.9970	0.5584	0.8493	0.7573	81.24
82.605	2.04	3.16	8.5	13.34	3.4262	1.5611	1.8652	9.7182	1.4665	6.1923	0.5954	0.7573	0.7573	81.24

NOTE: Qo Should Be > Q (Column 'A' < Column 'O'); Column 'M' Should = Column 'N'

Greiner

Job No. E002162 Project Old Cross Cut Canal Sheet 1 of 2

Subject Lateral Analysis for HG at Inlet - General Storm

By RHF Date 7-19-94 Checked By N.W. Date 7-24-94

$R = \frac{36}{2+1} = 1.5$

Roadway: Osborn South Connector Sta. 88+61

GRATE El. = 1218.17 Q = 186 cfs

EG at Mainline Sta. 88+61 = 1218.83

Length of 6'x6' CBC = 21'

$\theta = 12.5°$ use 13°

$S_f = 0.001190$

$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$

$S_f = \left[\frac{Q \times n}{1.486 \times A \times R^{2/3}}\right]^2$

$= \left[\frac{186 \times .013}{1.486 \times 36 \times 1.5^{2/3}}\right]^2$

$= .0012$ ft/ft

Eg in Lateral at Inlet:

$h_f = S_f * L = 21 (0.001190) = 0.025$

$h_v = k_b \times \frac{V^2}{2g}$

$= 1.2 \times \frac{13}{9.5} \times \frac{5.17^2}{2 \times 32.2}$

$= 0.1628 \times .42$

$= 0.07$

$V = 5.17$ fps

$h_v = 0.42$ ft.

$\Sigma h = h_f + h_v = 0.02 + 0.07$

$= 0.49$ ft/lge.

$\Sigma h = h_f = 0.02$ ft

Lateral EG at Inlet = D.S. Box Eg + Σh = 1218.83 + 0.02 = 1218.85

Lateral HG at Inlet = Lat. EG at Inlet - h_v = 1218.85 - 0.42 = 1218.43

Note: Although the HG > Grate El, by pressure analysis the inlet will intercept more than 57 cfs. This assumes 9 grates w/ 50 % clogging - Note 14 grates provided.

Refer to attached Lotus spreadsheet for pressure analysis.

PROJECT NAME:	OLD CROSS CUT CANAL	Grate Net Area:	4.25	No. OF GRATES:	9
JOB NUMBER:	E002102	Height of Catch Basin:	4	Ke:	0.50
		Top Slab Thickness:	1.75	Ko:	1.00
GRATE TYPE:	C.O.P. Standard Detail P1565 Type 1 Grate	Length:	5.75	Kf:	0.0611
CATCH BASIN LOCATION:	OSBORN SOUTH STA. 88+61	Manning's n:	0.014		
MAXIMUM ALLOWABLE PONDING EL.:	1220.50	Grate Total Area:	6.67		
HG IN LATERAL AT INLET:	1218.43	Total Perimeter:	10.67		
GRATE ELEVATION:	1218.17	Hydraulic Radius:	0.6251		

Q Assumed Discharge	Hp Pressure Head	Ho Oriface Head	An Net Grate Open Area	Ag Total Grate Open Area	Kl Calculated Total Loss Coefficient	Kl−Kt Total Loss Coefficient Without Trashrack Loss	Ktg Calculated Trashrack Adjusted Coefficient	Vn Net Grate Area Velocity	Hvn Net Grate Area Velocity Head	Vg Total Grate Area Velocity	Hvg Total Grate Area Velocity Head	Ktn Calculated Trashrack Coefficient	Ktn "Small Dam's" Trashrack Coefficient	Qo Discharge Oriface Equation
200	2.07	2.33	38.25	60.03	12.0097	1.5611	10.4487	5.2288	0.4245	3.3317	0.1724	4.2422	0.7573	313.93
300	2.07	2.33	38.25	60.03	5.3377	1.5611	3.7766	7.8431	0.9552	4.9975	0.3878	1.5333	0.7573	313.93
350	2.07	2.33	38.25	60.03	3.9215	1.5611	2.3605	9.1503	1.3001	5.8304	0.5279	0.9584	0.7573	313.93
360	2.07	2.33	38.25	60.03	3.7067	1.5611	2.1457	9.4118	1.3755	5.9970	0.5584	0.8711	0.7573	313.93
370	2.07	2.33	38.25	60.03	3.5090	1.5611	1.9480	9.6732	1.4530	6.1636	0.5899	0.7909	0.7573	313.93
374.44	2.07	2.33	38.25	60.03	3.4263	1.5611	1.8653	9.7893	1.4880	6.2375	0.6041	0.7573	0.7573	313.93

NOTE: Qo Should Be > Q (Column 'A' < Column 'O'); Column 'M' Should = Column 'N'

Sheet 2 of 2

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of 2

Subject Lateral Analysis for HG at Catch Basin - General Storm

By RHF Date 7-20-94 Checked By N.W Date 7-24-94

Roadway : Osborn North Connector Sta. 90+50

Grate El. = 1218.80 Q = 50 cfs

EG at Mainline Sta. 90+50 = 1219.38

Length of 48" RCP = 8'

$\theta = 45°$

$S_f = 0.001212$ ft/ft.

EG in Lateral at Inlet:

$h_f = S_f * L = (0.001212)(8') = 0.01$ ft.

V = 3.98 fps

$h_v = 0.25$ ft.

$\Sigma h = h_f = 0.01$ ft.

Lateral EG at Inlet = D.S. Box Eq + Σh = 1219.38 + 0.01 = 1219.39

Lateral HG at Inlet = Lat. EG at Inlet − h_v = 1219.39 − 0.25 = 1219.14

Note: Although the HG > Grate El., by pressure analysis the inlet will intercept more than 50 cfs. This assumes 3 grates w/ 50% Clogging. 6 grates provided.

Refer to attached spreadsheet for pressure analysis.

PROJECT NAME:	OLD CROSS CUT CANAL	Grate Net Area:	4.25	No. OF GRATES:	3
JOB NUMBER:	E002102	Height of Catch Basin:	4	Ke:	0.50
		Top Slab Thickness:	1.75	Ko:	1.00
GRATE TYPE:	C.O.P. Standard Detail P1565 Type 1 Grate	Length:	5.75	Kf:	0.0611
CATCH BASIN LOCATION:	OSBORN NORTH – STA. 90+50	Manning's n:	0.014		
MAXIMUM ALLOWABLE PONDING EL.:	1221.00	Grate Total Area:	6.67		
HG IN LATERAL AT INLET:	1219.14	Total Perimeter:	10.67		
GRATE ELEVATION:	1218.80	Hydraulic Radius:	0.6251		

Q	Hp	Ho	An	Ag	Kl	Kl–Kt	Ktg	Vn	Hvn	Vg	Hvg	Ktn	Ktn	Qo
Assumed Discharge	Pressure Head	Oriface Head	Net Grate Open Area	Total Grate Open Area	Calculated Total Loss Coefficient	Total Loss Coefficient Without Trashrack Loss	Calculated Trashrack Adjusted Coefficient	Net Grate Area Velocity	Net Grate Area Velocity Head	Total Grate Area Velocity	Total Grate Area Velocity Head	Calculated Trashrack Coefficient	"Small Dam's" Trashrack Coefficient	Discharge Oriface Equation
50	1.86	2.20	12.75	20.01	19.1846	1.5611	17.6236	3.9216	0.2388	2.4988	0.0970	7.1552	0.7573	101.68
100	1.86	2.20	12.75	20.01	4.7962	1.5611	3.2351	7.8431	0.9552	4.9975	0.3878	1.3135	0.7573	101.68
110	1.86	2.20	12.75	20.01	3.9638	1.5611	2.4027	8.6275	1.1558	5.4973	0.4693	0.9755	0.7573	101.68
115	1.86	2.20	12.75	20.01	3.6266	1.5611	2.0655	9.0196	1.2633	5.7471	0.5129	0.8386	0.7573	101.68
116	1.86	2.20	12.75	20.01	3.5643	1.5611	2.0033	9.0980	1.2853	5.7971	0.5218	0.8133	0.7573	101.68
117	1.86	2.20	12.75	20.01	3.5037	1.5611	1.9426	9.1765	1.3076	5.8471	0.5309	0.7887	0.7573	101.68
118	1.86	2.20	12.75	20.01	3.4445	1.5611	1.8835	9.2549	1.3300	5.8971	0.5400	0.7647	0.7573	101.68
118.315	1.86	2.20	12.75	20.01	3.4262	1.5611	1.8652	9.2796	1.3371	5.9128	0.5429	0.7573	0.7573	101.68

NOTE: Qo Should Be > Q (Column 'A' < Column 'O'); Column 'M' Should = Column 'N'

Sheet 2 of 2

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of 1

Subject Lateral Analysis for HG at Catch Basin - General Storm

By RHF Date 7-20-94 Checked By Ned Wang Date 7-21-94

Roadway: Weldon Ave. Connector Sta. 100+22

Grate El. = 1226.80 ✓ Q = 74 cfs

EG at Mainline Sta. 100+22 = 1221.79 ✓

Length of 48" RCP = 49'

$\theta = 90°$ (Mainline connection angle)

$S_f = 0.002654$ ✓

EG in Lateral at Inlet:

$h_f = S_f * L = (.002654)(49) = 0.13$ ft. ✓

$\Sigma h = h_f = 0.13$ ft. ✓

Lateral EG at Inlet = D.S. Box Eg + Σh = 1221.79 + 0.13' = 1221.92 ✓

No! The pipe is not in pressure flow. Analyze as open channel flow!

Partial Flow Conditions:

Q = 74 cfs

n = .013

S = 0.2128 ft/ft .1694

U.S. Inv. El. at Catch Basin = 1220.80

y_n = 0.90 ft

HG in Inlet = 1221.70 < Grate El.

HG over grate by orifice & weir analysis w/ 4 grates (6 total provided for clogging)

HG over Grate = Grate El. + 1.37' = 1228.17 OK

Note: No spread sheet provided.

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Job No. E002102 Project Old Cross Cut Canal Sheet 1 of 1
Subject Lateral Analysis for HG at Catch Basin - General Storm
By RHF Date 7-20-94 Checked By Ned Wang Date 7-21-94

Roadway: Whitton Connector Sta. 96+64

Grate El. = 1225.00 Q = 9 cfs
EG at Mainline Sta. 96+64 = 1221.02
Length of 18" RCP = 57'
$\theta = 90°$
Sf = 0.007341 ✓ V = 5.09 fps ✓ hv = 0.40 ✓

EG in Lateral at Inlet:
$h_f = S_f * L = (0.007341)(57) = 0.42'$ ✓
$\Sigma h = h_f = 0.42$ ✓

Lateral EG at Inlet = D.S. Box Eg + Σh = 1221.02 + 0.42 = 1221.44
Lateral HG at Inlet = Lat. Eg at Inlet - hv = 1221.44 - 0.40 = 1221.04

- Note the HG in the inlet is below the Grate El. The HG at the grate will equal the elevation from orifice flow. A grate has been provided which will intercept the flow w/ clogging considered.

- A spread sheet has not been provided at this location.

The total HG at the inlet is

Grate El + h over grate by weir or orifice analysis
h by weir ($Q = 3Lh^{3/2}$) = .68 h by orifice ($Q = .67A\sqrt{2gh}$) = .16

HG = 1225.00 + .68 = 1225.68 OK

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Job No. E002102 Project Old Cross Cut Canal Sheet 1 of 1

Subject Lateral Analysis for HG at Catch Basin - General Storm

By RHF Date 7-20-94 Checked By Ned Wang Date 7-21-94

Roadway: E. of 48th st. in R/W Sta. 114+25

Grate El. = 1239.50 Q < 1.0 cfs provided for nuisance flows.

Note the main line is not in pressure flow.

Inlet will function w/ 4" of ponding over the grate,

EG in Mainline Box = 1232.79 < Grate El. ✓

Inv. @ CB = 1237.20 S = 41.40 % 1240 adj GRADE

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Job No. E002102 Project Old Cross Cut Canal Sheet 1 of 1

Subject Drop Inlet Analysis for HG at Catch Basin - General Stm.

By RHF Date 7-20-94 Checked By Ned Wang Date 7-21-94

Roadway: N. of Indian School Inlet Sta. 116+83

Grate El. = 1241.0 Q = 13 cfs

2-grates provided

HG in Box at this location is 1235.00 < Grate El.

H over grate by orifice and weir analysis

H = 0.63' (weir control)

Grate El + H = 1241 + .63 = 1241.63 < 1243.5 (top of apron)

< 1242.9 (adj. Gutter El.)

✓

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Job No. E002102 Project Old Cross cut Canal Sheet 1 of 1

Subject Drop Inlet analysis for HG at Catch Basin - General Storm

By RHF Date Checked By Ned Wang Date 7-21-94

Roadway: W. of 48th st. in Park intercepting flow from S. Indian School. Q = 6cfs.

Sta. 111 +25

Grate El. = 1237.00

By weir analysis, 0.7' of head will intercept the flow.

EG in Box at this location is 1230.21 < Grate El.

PRESSURE CALCULATION PROCEDURE

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of 3

Subject Inlet Design with Pressure Flow

By RHF Date 6-20-94 Checked By Ned W. Date 7-24-94

Purpose: Use the pressure Eq. to determine the discharge intercepted by C.O.P. D1565 Type 1 Grates.

Equations: $Q_o = .67 A_n \sqrt{2 g H_o}$ EQ 1 (Orifice Eq.)

$K_t = 1.45 - \left(0.45 \left(\frac{a_n}{a_g}\right)\right) - \left(\frac{a_n}{a_g}\right)^2$ EQ 2 (Grate Loss Coefficient)

$K_f = \frac{29 n^2 L}{R^{4/3}} = 0.0611$ EQ 3 (Friction Loss Coefficient)

$K_L - K_t = K_e + K_f + K_v$ EQ 4 (Total Loss Coef. - Trash Rack)

$Q_p = A_g \sqrt{2g \frac{H_p}{K_L}}$ EQ 5 (Pressure Eq.)

$L = 4' + \text{Top Slab Thickness}$ EQ 6 (Height of Inlet)

$R = (a_g / P_g)$ EQ 7 (Hydraulic Radius of Inlet)

Variables: Q_o - Discharge by Orifice flow

K_t - Trash Rack Coefficient from Design of Small Dams.

a_n - Net Area of Grate (Area w/o Bars) = 4.25 ft²/grate

H_o - Orifice Head (El. difference CHIMP HG - Grate El.)

a_g - Total area of Grate (Area w/ Bars) = (6.67/Grate)

n - Manning's roughness coefficient = 0.013

K_L - Σ of All coefficients of Head Loss

$K_L - K_t$ - Σ of loss Coefficients - Trash Rack losses.

K_e - Entrance Loss Coefficient = 0.5 (square edge)

K_v - Outlet loss into a reservoir = 1.0

V_n - Net velocity Q/a_n

V_g - Total velocity Q/a_g

h_{vn} - Net Velocity Head $(V_n)^2/2g$

h_{vg} - Total Velocity Head $(V_g)^2/2g$

P_g - Total Grate Perimeter = 10.67 ft.

H_p - Pressure Head (El. difference CHIMP HG - Box HG)

L - Height of Catch Basin

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Job No. E002102 Project Old Cross Cut Canal Sheet 2 of 3

Subject Inlet Design with Pressure Flow

By RHF Date 6-20-94 Checked By N.W. Date 7-24-94

Note 1: K_t is to be multiplied by the net velocity head not the total velocity head. This requires that the trash rack losses be converted to total area coefficient for use in the pressure equation.

Note 2: Use orifice equation to compute discharge to compare to pressure discharge. $Q_p < Q_o$ otherwise use Q_o.

Example: Oak Street Sta. 37+12 35+81

Chk. latest HEC II Run.

$Q_o = 0.67(19 * \underset{A_n}{4.25})\sqrt{2g H_o}$ (Note: 19 grates)

CHIMP HG El. = 1205.26 1204.8'

GRATE El = CHIMP Inv. El. = 1200.68

$H_o = 1205.26 - 1200.68 = 4.58$ ft.

$Q_o = 929.17$ cfs ✓

$H_f = K * \frac{V^2}{2g}$

$\frac{H_p}{K_L} = V^2/2g$

$Q_p = a_g\sqrt{2g\frac{H_p}{K_L}}$ 1203.23' or 1204.20'?

Box Hg = 1202.74 ? (or lateral Hg at Inlet)

CHIMP Hg = 1205.26

1204.8'
−1204.2'
0.6'

$H_p = 1205.26 - 1202.74 = 2.52$ ft.

Assume → $K_L = 3.4263$

$a_g = (6.67 * 19) = 126.73$

$Q_p = 872.17$ cfs < 929.17 cfs → OK

Note 3: $Q_p = a_g\sqrt{2g\frac{H_p}{K_L}}$

$K_L = \frac{2g H_p}{\left(\frac{Q_p}{a_g}\right)^2}$

$H_p = 2.52$ ft.

$Q_p = 872.17$

$a_g = 126.73$

$\therefore K_L = 3.4263$

OK if is trial & Error Soln.

Does not make sense

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Job No. E002102 Project Old Cross Cut Canal Sheet 3 of 3

Subject Inlet Design with Pressure Flow

By RHF Date 6-20-94 Checked By N.W. Date 7-24-94

Note 3 (cont.): $K_L - K_{tg} = 0.5 + 0.0611 + 1.0 = 1.5611$

$K_{tg} = K_L - (K_L - K_{tg}) = 3.4263 - (1.5611) = 1.8653$

when $Q_p = 872.17$ cfs and $a_n = 80.75$ & $a_g = 126.73$

$V_n = 10.8009$ fps $h_{vn} = 1.8115$

$V_g = 6.8821$ fps $h_{vg} = 0.7355$

Convert to K_{tn} (Trashrack Loss coefficient for net area)

Know K_{tg} (Trashrack Loss Coefficient for total area)

$K_{tg} = 1.8653$

$h_{vg} = 0.7355$ (Total Velocity Head)

$h_{vn} = 1.8115$ (Net Velocity Head)

Note: $h_{tg} = h_{tn}$ $\therefore$ $K_{tg}(h_{vg}) = K_{tn}(h_{vn})$

and $K_{tn} = \dfrac{K_{tg}(h_{vg})}{h_{vn}}$

$K_{tn} = \dfrac{1.8653\,(0.7355)}{1.8115} = 0.7573$

This equals the K_{tn} from <u>Design of Small Dams</u> calculation. Therefore, calculated K_{tn} and Q_p are accurate

Note 4: CHIMP HG 1205.26; Lat. HG @ Inlet = 1202.74

$H_p = 2.52'$; $h_{vg} = 0.7355$; $h_{vn} = 1.8115$

$h_e = K_e(h_{vg}) = 0.5\,(0.7355) = 0.3678$

$h_t = K_{tn}(h_{vn}) = 0.7573\,(1.8115) = 1.3718$

$h_f = K_f(h_{vg}) = 0.0611\,(0.7355) = 0.0449$

$h_o = K_o(h_{vg}) = 1.0\,(0.7355) = 0.7355$

$H_p = 2.52 = 2.5200$ ft.

Lat. HG @ Inlet + H_p = CHIMP HG = 1202.74 + 2.52 = 1205.26 OK

$$K_t = 1.45 - \left(0.45 * \frac{a_n}{a_g}\right) - \left(\frac{a_n}{a_g}\right)^2$$
$$= 1.45 - \left(0.45 * \frac{80.75}{126.73}\right) - \left(\frac{80.75}{126.73}\right)^2$$
$$= 1.45 - 0.29 - 0.41$$
$$= 0.754$$

$$K_L = K_t + K_e + K_o + K_f$$
$$K_L - K_t = 1.5611$$

K_L = 0.5
1.0
0.0611
1.5611

PROJECT NAME:	OLD CROSS CUT CANAL	Grate Net Area:	4.25	No. OF GRATES:	19
JOB NUMBER:	E002102	Height of Catch Basin:	4	Ke:	0.50
		Top Slab Thickness:	1.75	Ko:	1.00
GRATE TYPE:	C.O.P. Standard Detail P1565 Type 1 Grate	Length:	5.75	Kf:	0.0611
CATCH BASIN LOCATION:	OAK STREET STA. 37+12.17	Manning's n:	0.014		
CHIMP HG ELEVATION:	1205.26	Grate Total Area:	6.67		
LATERAL HG AT INLET:	1202.74	Total Perimeter:	10.67		
GRATE ELEVATION:	1200.68 (CHIMP INV. EL. = 1202.04)	Hydraulic Radius:	0.6251		

Q Assumed Discharge	Hp Pressure Head	Ho Oriface Head	An Net Grate Open Area	Ag Total Grate Open Area	Kl Calculated Total Loss Coefficient	Kl−Kt Total Loss Coefficient Without Trashrack Loss	Ktg Calculated Trashrack Adjusted Coefficient	Vn Net Grate Area Velocity	Hvn Net Grate Area Velocity Head	Vg Total Grate Area Velocity	Hvg Total Grate Area Velocity Head	Ktn Calculated Trashrack Coefficient	Ktn "Small Dam's" Trashrack Coefficient	Qo Discharge Oriface Equation
400	2.52	4.58	80.75	126.73	16.2902	1.5611	14.7291	4.9536	0.3810	3.1563	0.1547	5.9800	0.7573	929.17
500	2.52	4.58	80.75	126.73	10.4257	1.5611	8.8647	6.1920	0.5953	3.9454	0.2417	3.5990	0.7573	929.17
600	2.52	4.58	80.75	126.73	7.2401	1.5611	5.6790	7.4303	0.8573	4.7345	0.3481	2.3057	0.7573	929.17
700	2.52	4.58	80.75	126.73	5.3192	1.5611	3.7582	8.6687	1.1669	5.5236	0.4738	1.5258	0.7573	929.17
800	2.52	4.58	80.75	126.73	4.0725	1.5611	2.5115	9.9071	1.5241	6.3126	0.6188	1.0197	0.7573	929.17
850	2.52	4.58	80.75	126.73	3.6075	1.5611	2.0465	10.5263	1.7205	6.7072	0.6985	0.8309	0.7573	929.17
860	2.52	4.58	80.75	126.73	3.5241	1.5611	1.9631	10.6502	1.7613	6.7861	0.7151	0.7970	0.7573	929.17
870	2.52	4.58	80.75	126.73	3.4436	1.5611	1.8825	10.7740	1.8025	6.8650	0.7318	0.7643	0.7573	929.17
871	2.52	4.58	80.75	126.73	3.4357	1.5611	1.8746	10.7864	1.8066	6.8729	0.7335	0.7611	0.7573	929.17
872	2.52	4.58	80.75	126.73	3.4278	1.5611	1.8667	10.7988	1.8108	6.8808	0.7352	0.7579	0.7573	929.17
872.17	2.52	4.58	80.75	126.73	3.4264	1.5611	1.8654	10.8009	1.8115	6.8821	0.7355	0.7573	0.7573	929.17
					$\frac{2g(5.24-2.74)}{(A/B)^2}$		KL−(KL−Kt) F−6	$\frac{400}{80.75}$ A/D	$\frac{I^2}{2g}$	$\frac{A}{E}$	$\frac{K^2}{2g}$			
A	B	C	D	E	F	G	H	I	J	K	L	M	N	O

By trial & Error.

NOTE: Qo Should Be > Q (Column 'A' < Column 'O'); Column 'M' Should = Column 'N'

Note: If $H_p = 2.52$ $K_L = 3.4264$ $Q_p = A\sqrt{2g\,H/K_L} = [(6.67)(19)]\sqrt{2g\left(\frac{2.52}{3.4264}\right)} = 872.17$ cfs

$h_{tn} = K_{tn} * H_{tn}$

$K_{tn} = \frac{h_{tn}}{H_{tn}}$

$h_{tn} = K_{tn}(h_{vn}) = 0.7573(1.8115) = 1.3718$ ft.

$h_{eg} = K_e(h_{vg}) = 0.5(0.7355) = 0.3678$ ft.

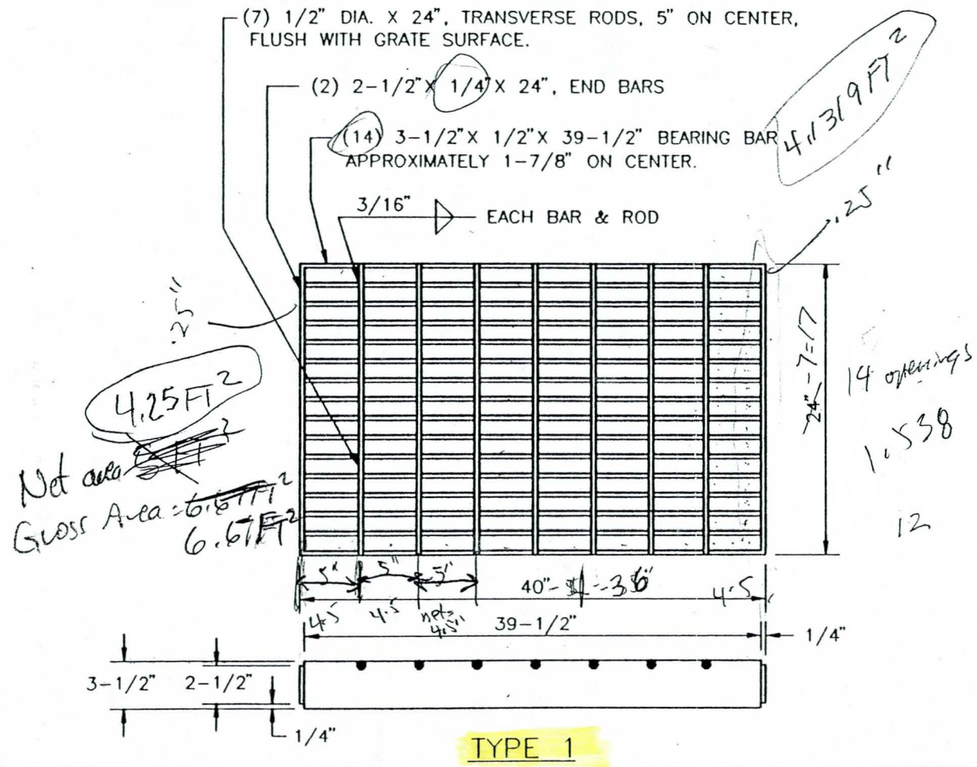
$h_f = K_f(h_{vg}) = 0.0611(0.7355) = 0.0449$ ft.

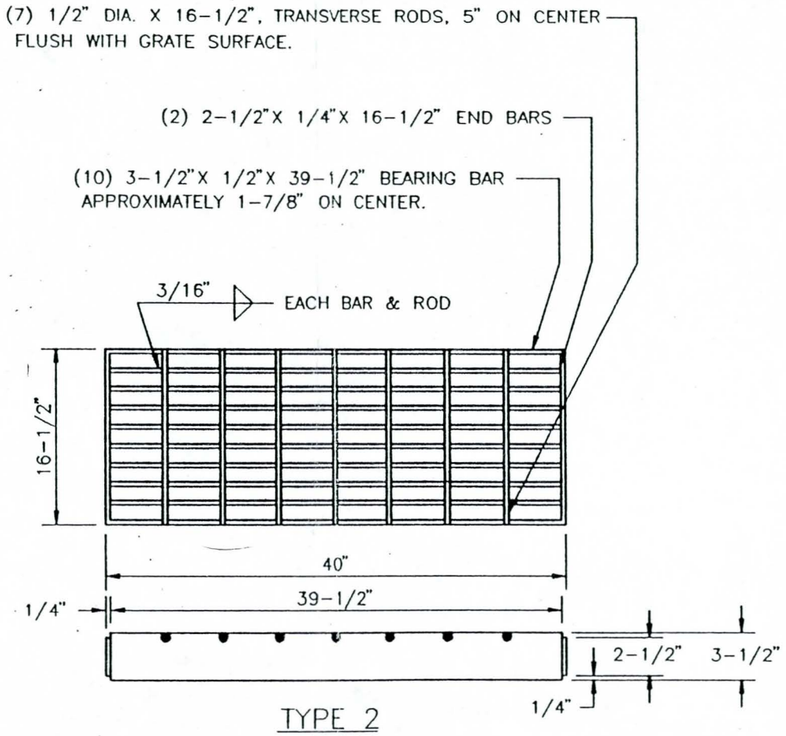
$h_o = K_o(h_{vg}) = 1.0(0.7355) = 0.7355$ ft.

$\Sigma h = h_L = H_p = 2.52$ ft. → OK

$\frac{1.4}{1.3}$ 2.4

$$\frac{K_{tg} * h_{vg}}{h_{vn}} = \frac{14.73 * 0.155}{0.3810} = 5.99$$





NOTES:

1. ALL STEEL SHALL BE IN ACCORDANCE WITH A.S.T.M. A-36.
2. WELDING SHALL BE IN ACCORDANCE WITH A.W.S. SPECIFICATIONS.
3. FRAME AND GRATE SHALL BE TESTED FOR ACCURRACY OF FIT AND SHALL BE MARKED IN SETS BEFORE DELIVERY.
4. THE COMPLETED ASSEMBLY SHALL BE GIVEN ONE SHOP COAT OF NO. 1 PAINT AND TWO FIELD COATS OF NO. 10 PAINT AS PER SECTION 790.
5. THE GRATE SHALL BE FABRICATED TO WITHIN 1/8" OF SPECIFIED DIMENSIONS.

DETAIL NO.	City of Phoenix STANDARD DETAIL	CATCH BASIN GRATES	APPROVED CITY ENGINEER	DATE	DETAIL NO.
P1565				7/1/92	P1565

① Ralph Goodall
② Joe Engeln } 495-2050.

③ Paul Driver - OCCC P.M.

Net Area = 4.25 ft²
Gross Area = 6.67 ft²

The first term on the right side of Equation 3.14 is the ratio of intercepted frontal flow to total gutter flow, and the second term is the ratio of intercepted side flow to total side flow. The second term is insignificant with high velocities and short grates.

The interception capacity of a grate catch basin on grade is equal to the efficiency of the grate multiplied by the total gutter flow:

$$Q_i = EQ = Q\,[R_f\,E_o + R_s\,(1 - E_o)] \tag{3.15}$$

Sumps: The efficiency of catch basins in passing debris is critical in sump locations because all runoff which enters the sump must be passed through the catch basin. Total or partial clogging of catch basins in these locations can result in hazardous ponding conditions. Grate catch basins alone are not recommended for use in sump locations because of the tendencies of grates to become clogged. Combination catch basins or curb-opening catch basins are recommended for use in these locations.

A grate catch basin in a sump location operates as a weir to depths dependent on the bar configuration and size of the grate and as an orifice at greater depths. Grates of larger dimension and grates with more open area, i.e., (with less space occupied by lateral and longitudinal bars), will operate as weirs to greater depths than smaller grates or grates with less open area.

The capacity of grate catch basins operating as weirs is:

$$Q_i = C_w\,P d^{\,1.5} \tag{3.16}$$

where $C_w = 3.0$.

The capacity of a grate catch basin operating as an orifice is:

$$Q_i = C_o\,A\,(2gd)^{0.5} \tag{3.17}$$

where $C_o = 0.67$.

Use of Equation 3.17 requires the clear opening area of the grate. Tests of three grates for the Federal Highway Administration showed that for flat bar grates, such as P–1–7/8–4 and P–1–1/8 grates, the clear opening is equal to the total area of the grate less the area occupied by longitudinal and lateral bars.

Figure 3.29 (page 3-44) is a plot of Equations 3.16 and 3.17 for various grate sizes. The effects of grate size on the depth at which a grate operates as an orifice is apparent from the chart. Transition from weir to orifice flow results in interception capacity less than that computed by either the weir or the orifice equation. This capacity can be approximated by drawing in a curve between the lines representing the perimeter and net area of the grate to be used.

GREINER
WATER RESOURCES

Drainage Design Manual for Maricopa County, Arizona

Volume II
Hydraulics

By
NBS Lowry
Engineers & Planners
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UNITED STATES DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

DESIGN OF SMALL DAMS

A Water Resources Technical Publication

suppression and tube geometry can be evaluated from figure 309, or from published data in various hydraulic handbooks [1, 2][2] and textbooks.

Flow in an open channel downstream from the headworks will be at either subcritical or supercritical stage, depending on the flow conditions through the control structure. In either case, flow depths and velocities throughout the channel can be determined from Bernoulli's equation, as discussed in section 203.

Flow in ungated outlet conduits will be similar to that in a culvert spillway, as discussed in section 213. Where the inlet geometry and the conduit slope are such that the control will remain at the inlet, part full flow will prevail and flow depths and velocities will be in accordance with the Bernoulli theorem for open channel flow. When flow from a pressure conduit discharges into a free-flow conduit, the flow in the latter most often will be at supercritical stage, with flow depths and velocities comparable to those which would prevail in an open channel. Computation procedures to determine the flow conditions according to the Bernoulli equation are presented in section 203.

Outlet conduits flowing partly full should be analyzed using maximum and minimum assumed values of the coefficient of roughness, n, when evaluating the required conduit size and the energy content of the flow as is done for spillway design (see sec. 203). To be assured of a sufficient conduit size to allow for air swell and surges, values of n of about 0.018 should be assumed in computing the depth or area of flow in a concrete-lined conduit. For computing the energy of flow at the end of the conduit to determine dissipator design, a value of n of about 0.008 should be assumed. To assure a free surface in the conduit for all stages of flow, and to guarantee against sealing of some portion from splashing or surging, the conduit should be designed to flow not more than 75 percent full at maximum capacity.

Terminal deflectors or energy dissipating devices placed at the downstream end of free-flow outlet conduits will be similar to those discussed in part E of chapter IX for spillways. Transitions to diverge the flow from the conduit portal to the stilling device and the allowable convex curvature of the floor entering the stilling device are determined as discussed in section 204.

234. *Pressure Flow in Outlet Conduits.*—If a control gate is placed at some point downstream from the conduit entrance, that portion above the control gate will flow under pressure. An ungated conduit may also flow full depending on the inlet geometry. The phenomena and the hydraulic equations for flow through an ungated conduit under pressure are discussed in section 213. The hydraulic design of a gated pressure conduit is similar to that for an ungated pressure conduit, discussed in section 213.

For flow in a closed pipe system, as shown on figure 310, Bernoulli's equation can be written as follows:

$$H_T = h_L + h_{v_2} \tag{2}$$

where:

H_T=the total head needed to overcome the various head losses to produce discharge, and

h_L=the cumulative losses of the system.

Equation (2) can be expanded to list each loss, as follows:

$$H_T = h_t + h_e + h_{b_5} + h_{f_5} + h_{ex_{(5-4)}} + h_{f_4} + h_{c_{(4-3)}} + h_{g_3} + h_{ex_{(3-1)}} + h_{f_1} + h_{b_1} + h_{c_{(1-2)}} + h_{g_2} + h_{v_2} \tag{3}$$

where:

h_t=trashrack losses,

h_e=entrance losses,

h_b=bend losses,

h_c=contraction losses,

h_{ex}=expansion losses,

h_g=gate or valve losses,

h_f=friction losses, and

h_v=velocity head exit loss at the outlet.

In the above equation the number subscripts refer to the various components, transitions, and reaches to which head losses apply.

For a free-discharging outlet, H_T is measured from the reservoir water surface to the center of the outlet gate or the outlet opening. If the

[2] Numbers in brackets refer to items in the bibliography, sec. 244.

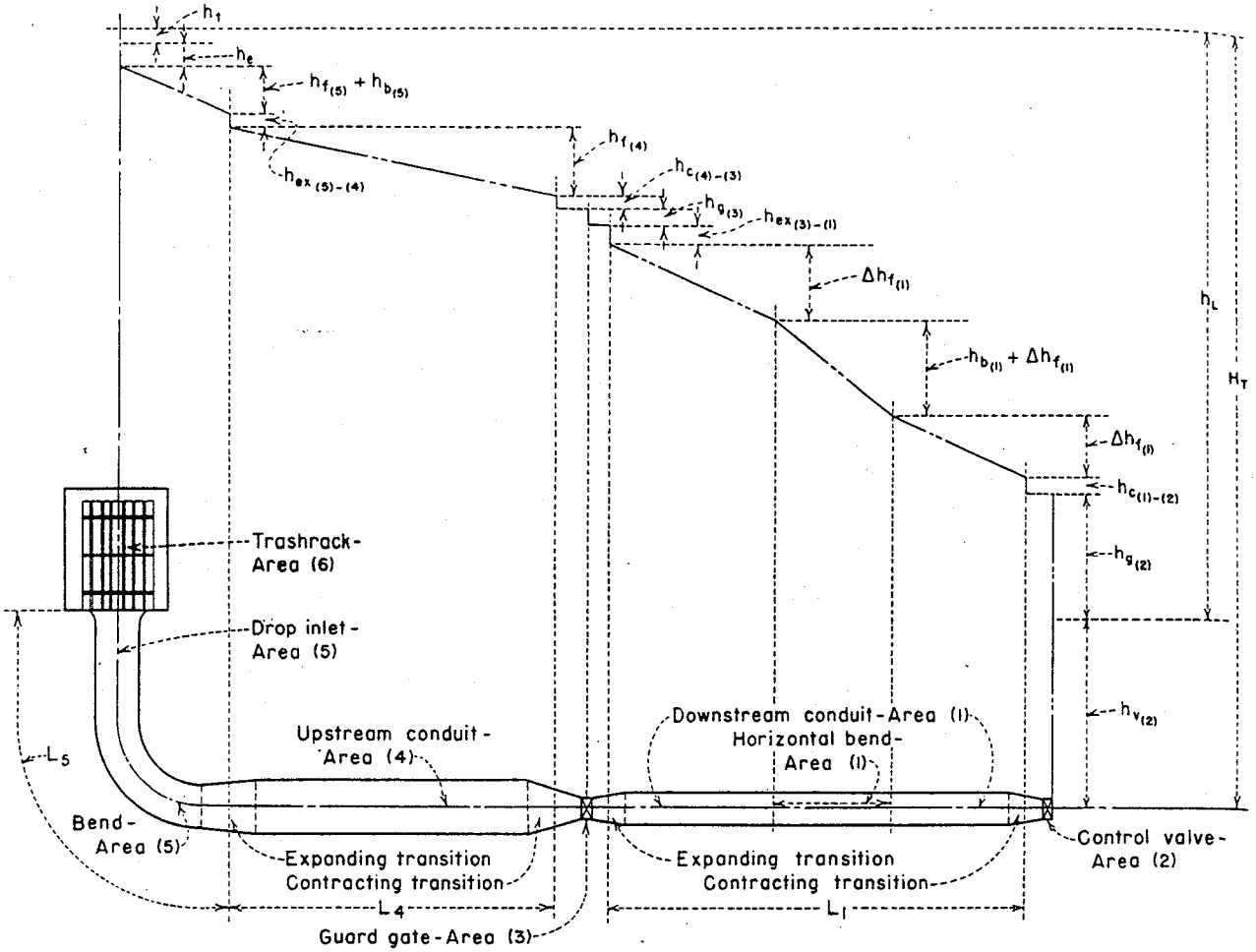


Figure 310. Pictorial representation of head losses in conduit flowing under pressure. 288—D—2532.

outflowing jet is supported on a downstream floor the head is measured to the top of the emerging jet at the point of greatest contraction; if the outlet portal is submerged the head is measured to the tailwater level.

Where the various losses are related to the individual components, equation (3) can be written:

$$H_T = K_t\left(\frac{v_6^2}{2g}\right) + K_e\left(\frac{v_5^2}{2g}\right) + K_{b_5}\left(\frac{v_5^2}{2g}\right) + \frac{fL_5}{D_5}\left(\frac{v_5^2}{2g}\right)$$
$$+ K_{ex}\left(\frac{v_5^2}{2g} - \frac{v_4^2}{2g}\right) + \frac{fL_4}{D_4}\left(\frac{v_4^2}{2g}\right) + K_c\left(\frac{v_3^2}{2g} - \frac{v_4^2}{2g}\right)$$
$$+ K_g\left(\frac{v_3^2}{2g}\right) + K_{ex}\left(\frac{v_3^2}{2g} - \frac{v_1^2}{2g}\right) + \frac{fL_1}{D_1}\left(\frac{v_1^2}{2g}\right) + K_{b_1}\left(\frac{v_1^2}{2g}\right)$$
$$+ K_c\left(\frac{v_2^2}{2g} - \frac{v_1^2}{2g}\right) + K_g\left(\frac{v_2^2}{2g}\right) + K_v\left(\frac{v_2^2}{2g}\right) \qquad (4)$$

where:

K_t=trashrack loss coefficient,
K_e=entrance loss coefficient,
K_b=bend loss coefficient,
f=friction factor in the Darcy-Weisbach equation for pipe flow (discussed in sec. 235(b)),
K_{ex}=expansion loss coefficient,
K_c=contraction loss coefficient,
K_g=gate loss coefficient, and
K_v=exit velocity head coefficient at the outlet.

Equation (4) can be simplified by expressing the individual losses in terms of an arbitrarily chosen velocity head. This velocity head is usually selected as that in a significant section of the system. If the various velocity heads

for the system shown on figure 310 are related to that in the downstream conduit, area (1), the conversion for "x" area is found as follows:

Since:

$$Q=a_1v_1=a_xv_x,\ a_1^2v_1^2=a_x^2v_x^2,\ \text{and}\ \frac{a_1^2v_1^2}{2g}=\frac{a_x^2v_x^2}{2g}$$

then:

$$\frac{v_x^2}{2g}=\left(\frac{a_1}{a_x}\right)^2\frac{v_1^2}{2g}$$

Equation (4) then can be written:

$$H_T=\frac{v_1^2}{2g}\left[\left(\frac{a_1}{a_6}\right)^2K_t+\left(\frac{a_1}{a_5}\right)^2\left(K_e+K_{b_5}+\frac{fL_5}{D_5}+K_{ex}\right)+\left(\frac{a_1}{a_4}\right)^2\left(\frac{fL_4}{D_4}-K_{ex}-K_c\right)+\left(\frac{a_1}{a_3}\right)^2(K_c+K_g+K_{ex})+\left(\frac{fL_1}{D_1}-K_{ex}+K_{b_1}-K_c\right)+\left(\frac{a_1}{a_2}\right)^2(K_c+K_g+K_v)\right] \tag{5}$$

If the bracketed part of the expression is represented by K_L, the equation can be written:

$$H_T=K_L\frac{v_1^2}{2g} \tag{6}$$

Then:

$$Q=a_1\sqrt{\frac{2gH_T}{K_L}} \tag{7}$$

235. *Pressure Flow Losses in Conduits.*—(a) *General.*—Head losses in outlet works conduits are caused primarily by the frictional resistance to flow along the conduit side walls. Additional losses result from trashrack interferences, entrance contractions, contractions and expansions at gate installations, bends, gate and valve constrictions, and other interferences in the conduit. As with free-flow conduits, greater than average loss coefficients should be assumed for computing required conduit and component sizes, and smaller loss coefficients should be used for computing energies of flow at the outlet. The major contributing losses of a conduit or pipe system are discussed in the remainder of this section.

(b) *Friction Losses.*—For flow in large pipes, the Darcy-Weisbach formula is most often employed to determine the energy losses due to frictional resistances of the conduit. The loss of head is stated by the equation:

$$h_f=\frac{fL}{D}\frac{v^2}{2g} \tag{8}$$

where f is the friction loss coefficient. This coefficient varies with the conduit surface roughness and with the Reynolds number. The latter is a function of the diameter of the pipe, and the velocity, viscosity, and density of the fluid flowing through it. Data and procedures for evaluating the loss coefficient are presented in a Bureau of Reclamation engineering monograph [3]. Since f is not a fixed value, many engineers are unfamiliar with its variations and would rather use Manning's coefficient of roughness, n, which has been more widely defined. If the effect of the Reynolds number influence is neglected, and if the roughness factor in relation to the pipe size is assumed constant, the relation of f in the Darcy-Weisbach equation to n in the Manning equation will be:

$$f=\frac{116.5n^2}{r^{1/3}}=\frac{185n^2}{D^{1/3}} \tag{9}$$

Relationships between the Darcy-Weisbach and Manning's coefficients can be determined from figure B–7 (appendix B).

Where the conduit cross section is horseshoe or rectangular in shape, the Darcy-Weisbach formula does not apply because it is for circular pipes, and the Manning equation may be used to compute the friction losses. Manning's equation as applied to closed conduit flow is:

$$h_f=29.1n^2\frac{L}{r^{4/3}}\frac{v^2}{2g} \tag{10}$$

Maximum and minimum values of n which may be used to determine the conduit size and the energy of flow are as follows:

	Maximum value	*Minimum value*
Concrete pipe or cast in place conduit	0.014	0.008
Steel pipe with welded joints	.012	.008
Unlined rock tunnel	.035	.020

(c) *Trashrack Losses.*—Trashrack structures which consist of widely spaced structural members without rack bars will cause very little head loss, and trashrack losses in such a case might be neglected in computing conduit losses. When the trash structure consists of racks of bars, the loss will depend on the bar thickness, depth, and spacing. An average approximation can be obtained [2] from the equation:

$$\text{Loss}=K_t\frac{v_n^2}{2g}$$

where:

$$K_t=1.45-0.45\frac{a_n}{a_g}-\left(\frac{a_n}{a_g}\right)^2 \qquad (11)$$

In the above:

K_t=the trashrack loss coefficient (empirical),
a_n=the net area through the rack bars,
a_g=the gross area of the racks and supports, and,
v_n=the velocity through the net trashrack area.

Where maximum loss values are desired, assume that 50 percent of the rack area is clogged. This will result in twice the velocity through the trashrack. For minimum trashrack losses, assume no clogging of the openings when computing the loss coefficient, or neglect the loss entirely.

(d) *Entrance Losses.*—The loss of head at the entrance of a conduit is comparable to the loss in a short tube or in a sluice. If H is the head producing the discharge, C is the coefficient of discharge, and a is the area, the discharge

$$Q \text{ is equal to } Ca\sqrt{2gH}$$

and the velocity

$$v \text{ is equal to } C\sqrt{2gH}.$$

Or,

$$H=\frac{1}{C^2}\frac{v^2}{2g} \qquad (12)$$

Since H is the sum of the velocity head h_v and the head lost at the entrance h_e, equation (12) may be written:

$$\frac{v^2}{2g}+h_e=\frac{1}{C^2}\frac{v^2}{2g} \text{ or } h_e=\left(\frac{1}{C^2}-1\right)\frac{v^2}{2g}$$

Then:

$$K_e=\left(\frac{1}{C^2}-1\right) \qquad (13)$$

Coefficients of discharge for square sluice entrances are shown on figure 309. Coefficients of discharge and loss coefficients for typical entrances for conduits, as given in various texts and technical papers, are listed in table 33.

TABLE 33.—*Coefficients of discharge and loss coefficients for conduit entrances*

	Coefficient C			Loss coefficient K_e		
	Maximum	Minimum	Average	Maximum	Minimum	Average
(a) Gate in thin wall—unsuppressed contraction	0.70	0.60	0.63	1.80	1.00	1.50
(b) Gate in thin wall—bottom and sides suppressed.	.81	.68	.70	1.20	0.50	1.00
(c) Gate in thin wall—corners rounded	.95	.71	.82	1.00	.10	0.50
(d) Square-cornered entrances	.85	.77	.82	.70	.40	.50
(e) Slightly rounded entrances	.92	.79	.90	.60	.18	.23
(f) Fully rounded entrances $\frac{r}{D}\geq 0.15$	.96	.88	.95	.27	.08	.10
(g) Circular bellmouth entrances	.98	.95	.98	.10	.04	.05
(h) Square bellmouth entrances	.97	.91	.93	.20	.07	.16
(i) Inward projecting entrances	.80	.72	.75	.93	.56	.80

(e) *Bend Losses.*—Bend losses in closed conduits in excess of those due to friction loss through the length of the bend are a function of the bend radius, pipe diameter, and the angle through which the bend turns. Although experimental data on bend losses in large pipes are meager, the loss can be related to those determined for smaller pipe. Figure 311(A) shows the coefficients found by various investigators for 90° bends for various ratios of radius of bend to diameter of pipe, and an adjusted curve assumed to be suitable for large pipes.

Figure 311(B) indicates the correction factor to be applied to the values indicated in figure 311(A) for other than 90° bends. The value of the loss coefficient, K_b for various values of $\frac{R_b}{D}$ can be applied directly for circular conduits; for rectangular conduits D is taken as the height of the section in the plane of the bend.

(f) *Transition Losses.*—Head losses in gradual contractions or expansions in a conduit can be considered in relation to the increase or decrease in velocity head, and will vary according to the rate of change of the area and the length of the transition. For contractions the loss of head, h_c, will be approximately equal to $K_c\left(\frac{v_2^2}{2g}-\frac{v_1^2}{2g}\right)$, where K_c varies from 0.1 for gradual contractions to 0.5 for abrupt contractions. Where the flare angle does not exceed that indicated in section 236(b), the loss coefficient can be assumed as 0.1. For greater flare angles, the loss coefficient can be assumed to vary in a straight-line relationship to a maximum of 0.5 for a right angle contraction.

For expansions, the loss of head, h_{ex}, will be approximately equal to $K_{ex}\left(\frac{v_1^2}{2g}-\frac{v_2^2}{2g}\right)$, where K_{ex} is as follows:

Flare angle α	2°	5°	10°	12°	15°	20°	25°	30°	40°	50°	60°
K_{ex} [1]	0.03	0.04	0.08	0.10	0.16	0.31	0.40	0.49	0.60	0.67	0.72
K_{ex} [4]	.02	.12	.16	-----	.27	.40	.55	.66	.90	1.00	----

(g) *Gate and Valve Losses.*—No gate loss need be assumed where a gate is mounted at the entrance to a conduit so that when wide open it does not interfere with the entrance flow conditions. Where a gate is mounted at either the upstream or downstream side of a thin headwall such that the sides and bottom of the jet are suppressed but the top is contracted, loss coefficients shown as item b in table 33 will apply. Where a gate is mounted in a conduit so that the floor, sides, and roof both upstream and downstream are continuous with the gate opening, only the losses due to the slot will need to be considered, for which a value of K_g not exceeding 0.1 might be assumed. For partly open gates, the coefficient of loss will depend on the top contraction; for smaller openings it will approach the value of 1.0 as shown for item b in table 33.

For wide open gate valves K_g will approximate 0.19. Similar to partly open gates, values of the loss coefficient will increase for smaller valve openings. Indicated loss coefficients for partly open gate valves are 1.15 for three-fourths open, 5.6 for one-half open, and 24.0 for one-fourth open. Average values of K_g for butterfly valves in the wide open position are about 0.15; values vary between 0.1 and 0.5, depending on the thickness of the gate leaf in relation to the gross area. Losses in spherical valves are negligible.

(h) *Exit Losses.*—No recovery of velocity head will occur where the release from a pressure conduit freely discharges, or is submerged or supported on a downstream floor. The velocity head loss coefficient, K_v, in these instances is equal to 1.0. When a diverging tube is provided at the end of a conduit, recovery of a portion of the velocity head will be obtained if the tube expands gradually and if the end of the tube is submerged. The velocity head loss coefficient will then be reduced from the value of 1.0 by the degree of velocity head recovery. If a_1 is the area at the beginning of the diverging tube and a_2 is the area at the end of the tube, K_v is equal to $\left(\frac{a_1}{a_2}\right)^2$.

236. *Transition Shapes.*—(a) *Entrances.*—To minimize head losses and to avoid zones where cavitation pressures can develop, the entrance to a pressure conduit should be streamlined to provide smooth, gradual changes in the flow. To obtain the best inlet efficiency, the shape of the entrance should simulate that of a jet discharging into air. As with the nappe-shaped weir, the entrance shape should guide and support the jet with minimum interference until it is contracted to the dimensions of the conduit. If the entrance curve is too sharp or too short, subatmospheric pressure areas which may induce cavitation will develop. A bellmouth entrance which conforms to or slightly encroaches upon the free-jet profile will provide the best entrance shape. For a circular entrance, this shape can be approximated by

an elliptical entrance curve represented by the equation:

$$\frac{x^2}{(0.5D)^2}+\frac{y^2}{(0.15D)^2}=1 \qquad (14)$$

where x and y are coordinates whose x-x axis is parallel to and $0.65D$ from the conduit centerline and whose y-y axis is normal to the conduit-centerline and $0.5D$ downstream from the entrance face. The factor D is the diameter of the conduit at the end of the entrance transition.

The jet issuing from a square or rectangular opening is not as easily defined as one issuing from a circular opening; the top and bottom curves may differ from the side curves both in length and curvature. Consequently, it is more difficult to determine a transition which will eliminate subatmospheric pressures. An elliptical curved entrance which will tend to minimize the negative pressure effects is defined by the equation:

$$\frac{x^2}{D^2}+\frac{y^2}{(0.33D)^2}=1 \qquad (15)$$

where D is the vertical height of the conduit for defining the top and bottom curves, and is the horizontal width of the conduit for defining the side curves. The major and minor axes are positioned similarly to those indicated for the circular bellmouth.

For a rectangular entrance with the bottom placed even with the upstream floor and with curved guide piers at each side of the entrance opening, both the bottom and side contractions will be suppressed and a sharper contraction will take place at the top of the opening. For this condition the top contraction curve is defined by the equation:

$$\frac{x^2}{D^2}+\frac{y^2}{(0.67D)^2}=1 \qquad (16)$$

where D is the vertical height of the conduit downstream from the entrance shape.

(b) *Contractions and Expansions.*—To minimize head losses and to avoid cavitation tendencies along the conduit surfaces, contraction and expansion transitions to and from gate control sections in a pressure conduit should be gradual. For contractions, the maximum convergent angle should not exceed that indicated by the relationship:

$$\tan\alpha=\frac{1}{U} \qquad (17)$$

where:

α=an angle of the conduit wall surfaces with respect to its centerline, and

U=an arbitrary parameter $\frac{v}{\sqrt{gD}}$.

The values of v and D are the average of the velocities and diameters at the beginning and end of the transition.

Expansions should be more gradual than contractions because of the danger of cavitation where sharp changes in the side walls occur. Furthermore, as has been indicated in section 235(f), loss coefficients for expansions increase rapidly after the flare angle exceeds about 10°. Expansions should be based on the relationship:

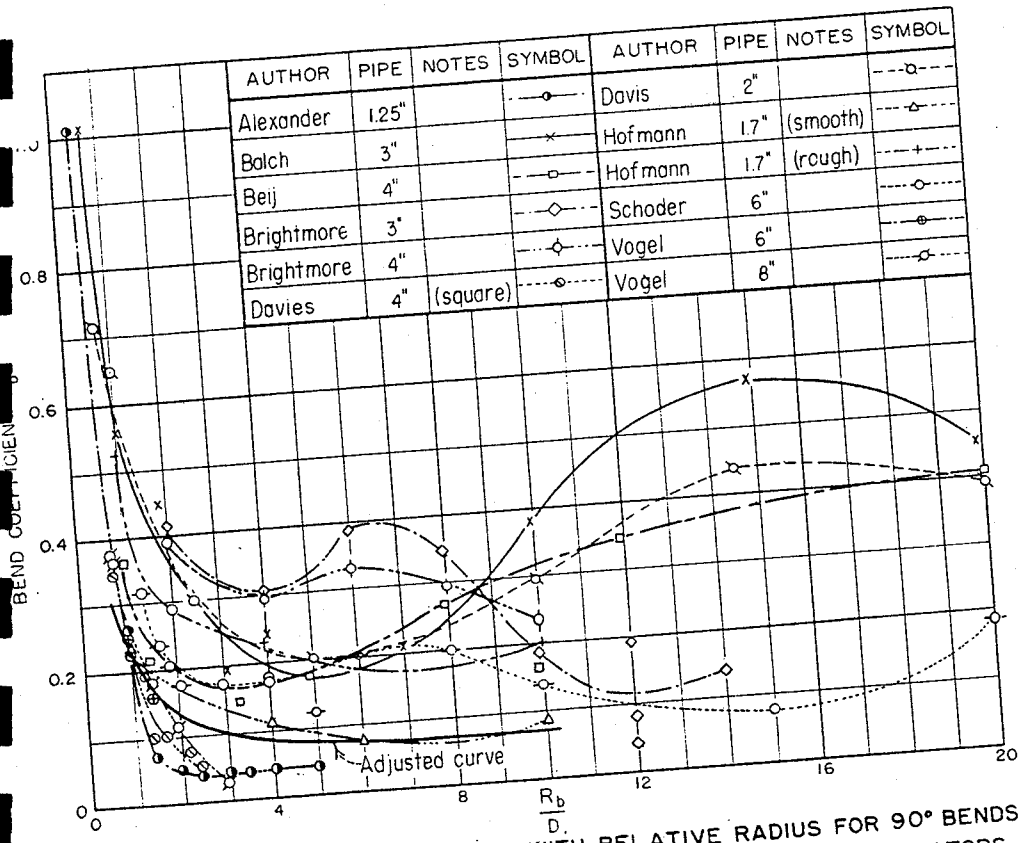
$$\tan\alpha=\frac{1}{2U} \qquad (18)$$

The notations are the same as for equation (17). For usual installations, the flare angle should not exceed about 10°.

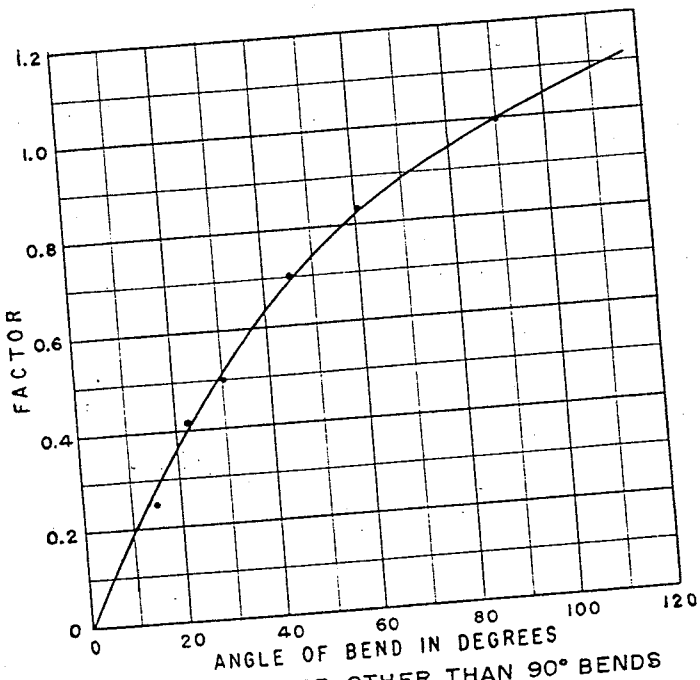
The criteria for establishing maximum contraction and expansion angles for conduits flowing partly full are the same as those for open channel flow, as given in section 204(b).

(c) *Exit Transitions.*—When a circular conduit flowing partly full empties into a chute, the transition from the circular section to one with a flat bottom can be made in the open channel downstream from the conduit portal, or it can be made within the conduit so that the bottom will be flat at the portal section.

Two types of transition are commonly used. The first type is made by constructing a straight line of intersection between the 45° points on each side of the invert of the upstream circular section to the corners of the flat bottom at the downstream end of the transition. The radius of curvature between the



(A) VARIATION OF BEND COEFFICIENT WITH RELATIVE RADIUS FOR 90° BENDS OF CIRCULAR CROSS SECTION, AS MEASURED BY VARIOUS INVESTIGATORS



(B) FACTORS FOR OTHER THAN 90° BENDS

Figure 311. Bend loss coefficients. 288–D–2533.

springline and the intersection line increases from the radius of the conduit at the upstream end to a radius of infinity along the vertical wall at the downstream end. The radius of curvature of the bottom of the transition also increases from the radius of the conduit at the upstream end to infinity along the flat bottom of the downstream end. Figure 312 shows a typical example of this type of transition, including the conduit transition table. The distance "X" in figure 312 denotes the flair from the upstream end of the transition to the wider section downstream as in the case of a stilling basin.

The second type of transition is made by gradually decreasing the radius of the circular quadrants in the lower half of the pipe from their initial radius at the upstream end to a radius of zero at the corners of the flat bottom at the downstream end. In both transitions the roof of the conduit remains curved. These transitions must satisfy the expansion criteria established in section 236 (b), above.

For usual installations the length of the transition can be related to the exit velocity. An empirical rule which will give a satisfactory transition is:

$$L \text{ (in feet)} = \frac{vD}{5} \qquad (19)$$

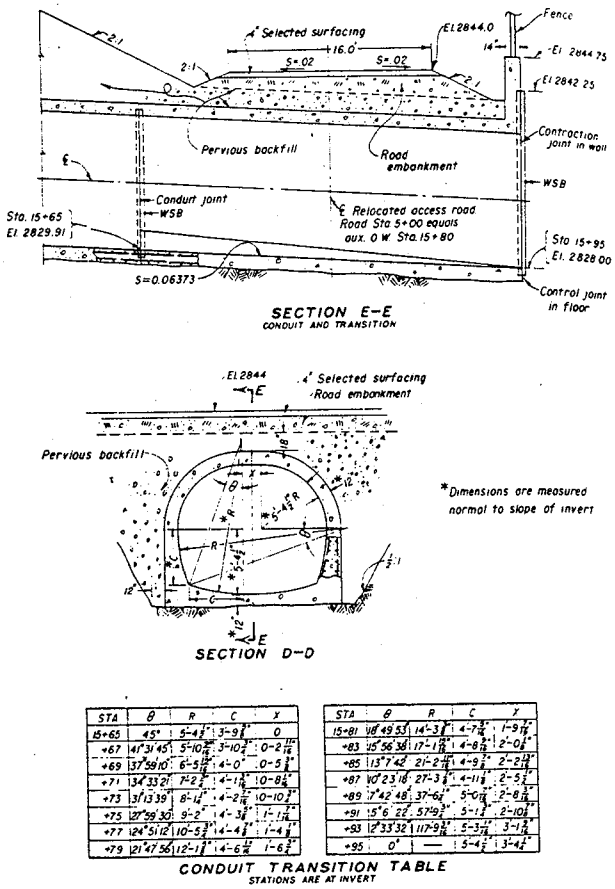
where:

D=the exit velocity in feet per second, and

v=the conduit diameter in feet.

Downstream from a free-flow conduit the chute sections, including the transition into a stilling basin, will be governed by open channel flow criteria. Floor curvatures and maximum flare angles should be determined by equations (19) and (21), respectively, of chapter IX. To reduce the length of the open channel portion from the conduit portal to the stilling basin, the beginning of the flare and of the convex curve may be located inside the conduit. This transition may be combined with the transition of the bottom shape.

In certain instances, as illustrated on figure 301, Crane Prairie Dam, and figure 303, Scofield and Newton Dams, an adverse slope and a hump have been employed immediately downstream from the portal to permit more rapid widening of the channel before it enters the basin. No firm criteria have been established for the design of these devices; the details were determined by model tests. Certain inherent disadvantages to this type of design are: (1) care must be taken to avoid a hump of such height that back pressure will cause a hydraulic jump to occur inside the conduit, (2) the floor section at the hump must be made structurally sufficient to withstand the large dynamic forces resulting from impingement of the flow on the rising floor, (3) during periods of no flow a pond which can freeze during the winter is formed in the conduit unless provision is made to drain the sump, and (4) access into the downstream conduit is difficult unless drainage is provided. Depending on tailwater conditions, pumping may be required to provide drainage.



STA	θ	R	C	X
15+65	45°	5'-4 3/8"	3'-9 3/8"	0
+67	41°31'45"	5'-10 25/32"	3'-10 3/8"	0'-2 11/16"
+69	37°59'10"	6'-5 15/16"	4'-0"	0'-5 1/8"
+71	34°33'21"	7'-2 3/8"	4'-1 1/8"	0'-8 5/16"
+73	31°13'39"	8'-1 1/4"	4'-2 1/8"	0'-10 3/4"
+75	27°59'30"	9'-2"	4'-3 1/8"	1'-1 1/16"
+77	24°51'12"	10'-5 3/4"	4'-4 3/8"	1'-4 1/8"
+79	21°47'56"	12'-1 3/8"	4'-6 13/16"	1'-6 3/4"

STA	θ	R	C	X
15+81	18°49'53"	14'-3 3/8"	4'-7 5/8"	1'-9 7/16"
+83	15°56'38"	17'-1 13/16"	4'-8 5/8"	2'-0 3/4"
+85	13°7'42"	21'-2 11/16"	4'-9 3/8"	2'-2 13/16"
+87	10°23'18"	27'-3 3/4"	4'-11 1/8"	2'-5 1/2"
+89	7°42'48"	37'-6 1/4"	5'-0 1/8"	2'-8 11/16"
+91	5°6'22"	57'-9 3/4"	5'-1 1/8"	2'-10 3/4"
+93	2°33'32"	117'-9 13/16"	5'-3 1/16"	3'-1 3/16"
+95	0°	—	5'-4 1/2"	3'-4 1/2"

CONDUIT TRANSITION TABLE
STATIONS ARE AT INVERT

Figure 312. A typical conduit transition. 288—D—2809.

237. *Terminal Structures.*—(a) *General.*—Deflector buckets, hydraulic jump basins, and impact type stilling basins are suitable terminal structures for free-flow conduits, when appropriately used. These structures are commonly used in conjunction with spillways, and their hydraulic designs are discussed in part E of chapter IX. Other types of stilling devices employed more often with outlet works than with spillways are plunge basins and stilling wells. The hydraulic designs of these structures are discussed in this section.

The hydraulic jump stilling basin is most often used for energy dissipation of outlet works discharges. However, where flow emerges from the outlet in the form of a free jet, as will be the case with valve-controlled outlets of pressure conduits, it must be directed onto the transition floor approaching the basin so it will become uniformly distributed before entering the basin. Otherwise proper dissipation of energy will not be obtained.

To evaluate the energy which must be dissipated by the stilling device, the losses through the outlet system should be minimized, as discussed in sections 233 and 235(b). The specific energy immediately downstream from a gate or valve control will equal the exit velocity head based on minimum losses through the pressure system, as measured above the outflowing water surface. If specific energies have not been computed, approximate basin depths can be obtained from figure 268, as discussed in section 206(d).

(b) *Plunge Basins.*—Where the outlet conduit ends with a flip bucket or where flows issue from a downstream control valve or freely discharging pipe, a riprap- or concrete-lined trapezoidal plunge basin might be utilized. Such a basin should be employed only where the jet discharges into the air and then plunges downward into the basin. Tests have shown that if the angle of impingement is too flat the jet will ride and skip across the surface at high velocity. This will cause waves and eddies in the basin sufficient to erode the side slopes, and there will be high exit velocities.

As indicated in section 210, no fixed criteria have yet been established for plunge basins which will provide satisfactory dissipation for all heads, discharges, and incoming jet conditions. However, criteria that were established for several small outlet works plunge basins which have operated reasonably satisfactorily are herewith presented for use only as a preliminary guide to determine approximate basin geometry. The general arrangement of this basin is represented on figure 313. The basin depths were made about one-fifth of the difference in elevation between maximum reservoir water surfaces and maximum tailwater levels. The minimum bottom widths were made the width of the incoming jet, or the width required to limit the average velocity at the end of the basin to about 3 feet per second, whichever was greater.

(c) *Stilling Wells.*—Stilling well designs as described in section 230 are illustrated on figures 314 and 315. The well dimensions and performance criteria for these designs were established from model tests, and general criteria for such designs applicable to various conditions were not determined. The hydraulic stilling action in these devices results from turbulence and diffusion of the incoming high-energy flow into the water bulk in the well, and successful stilling is aided materially by special fillets and diffuser blocks incorporated along the sides and in the corners of the well. The net area of the well is generally selected by limiting the average rising velocity to between 1 and 3 feet per second. The total depth of the well will be dictated by the energy of the incoming flow which must be dissipated, and by the effectiveness of the diffuser blocks and fillets in diffusing the rising flow. Basins with similar criteria can be patterned after those illustrated in the figures. Basins for considerably different conditions should be model tested.

238. *Chart for Estimating Pressure Conduit Sizes.*—Figure 316 is a nomograph for the solution of the equation for pipe flow. By use of this figure, the required conduit diameter can be determined for a given length and gross head from reservoir water surface to the end of the pipe. As will be noted from the equation, an average total head loss coefficient of 0.5 has been assumed for all contributing losses except

EQUALIZER OPENINGS

PROJECT NAME: OLD CROSS CUT CANAL

DESIGN BY: RHF
DATE: 7-5-94

CHECKED BY: N.W.
DATE: 7-22-94

PROJECT NUMBER: E002102

EQUILIZING OPENING SIZING

STREET NAME: GRANADA ROAD STA.=

Q DS.=	4100 2050 2085	DS. SIZE=	2–18'x10'	DS. AREA=	180	Hj=	0.0678
Q US.=	2010 2045	US. SIZE=	2–18'x10'	US. AREA=	180	LAT. WDTH.=	4
Q LAT.=	54 80	LAT. SIZE=	1–48"	OP. AREA=	17.00	LAT. ANG.=	60
MANNING'S 'n'=	0.014						

ANGLE	Vd	Vu	Hvd	Hvu	Vo	ACTUAL DELTA y	Q OPENING	CALCULATED DELTA y
45.540819313	11.58	11.36	2.08	2.00	2.352941	0.1470	40	0.146973376

NOTE THAT THE ANGLE IS THE CONFLUENCE ANGLE AND THE AREA IS AT A SCEW TO THE WALL. THE TOTAL AREA OF WALL OPENING NEEDED OF EQUILIZATION OF FLOW IS OP. AREA DIVIDED BY THE SINE OF THE ANGLE. USE STANDARD OPENING OF 4'x8'.

TOTAL AREA OF OPENING NEEDED=	32
TOTAL NUMBER OF 4'x8' OPENINGS=	1

$.0678 = \Delta y - 2.08 + 2.00$

$\Delta y = .0678 + 2.08 - 2.00 = 0.1478$

* PROVIDE MINIMUM WALL OPENING OF 1–4'x8' OPENING

$$h_j = \Delta y - hv_d + hv_u$$

$$\Delta y = h_j + hv_d - hv_u$$

$$\Delta y = \frac{(Q_d V_d) - (Q_u V_u) - [(Q_o V_o \cos\theta)]}{0.5\,[g\,(A_u + A_d)]} + \cancel{0.5\,[L\,(S_{f_1} + S_{f_2})]}$$

neglect minor

Solve for θ as $\cos^{-1}$ approaches 1.0.

Use h_j from the junction loss calculations for this location and solve for Δy, then compute Δy so that they are equal by varying the θ.

PROJECT NAME: OLD CROSS CUT CANAL

DESIGN BY: RHF
DATE: 7-5-94

CHECKED BY: N.W.
DATE: 7-22-94

PROJECT NUMBER: E002102

EQUILIZING OPENING SIZING

STREET NAME: OAK STREET STA.=

Q DS.=	2085 (2050)	DS. SIZE=	2–18'x10'	DS. AREA=	180	Hj=	0.1852
Q US.=	1861	US. SIZE=	2–18'x10'	US. AREA=	180	LAT. WDTH.=	8
Q LAT.=	449	LAT. SIZE=	1–8'x6'	OP. AREA=	36.20	LAT. ANG.=	12.5
MANNING'S 'n'=	0.014						

ANGLE	Vd	Vu	Hvd	Hvu	Vo	ACTUAL DELTA y	Q OPENING	CALCULATED DELTA y
4.9695205176	11.58	10.34	2.08	1.66	6.201657	0.6097	225	0.6097096

NOTE THAT THE ANGLE IS THE CONFLUENCE ANGLE AND THE AREA IS AT A SCEW TO THE WALL. THE TOTAL AREA OF WALL OPENING NEEDED OF EQUILIZATION OF FLOW IS OP. AREA DIVIDED BY THE SINE OF THE ANGLE. USE STANDARD OPENING OF 8'x4'.

TOTAL AREA OF OPENING NEEDED=	418
TOTAL NUMBER OF 8'x4' OPENINGS=	13

PROVIDE 14 ON THE PLANS

$$h_j = \Delta y - h_{vd} + h_{vu}$$

$$\Delta y = h_j + h_{vd} - h_{vu}$$

$$\Delta y = \frac{(Q_d V_d) - (Q_u V_u) - [(Q_o V_o \cos\theta)]}{0.5\,[g\,(A_u + A_D)]} + \cancel{0.5\,[L\,(S_{f1} + S_{f2})]}$$

neglect minor

Solve for θ as $\cos^{-1}$ approaches 1

PROJECT NAME: OLD CROSS CUT CANAL

PROJECT NUMBER: E002102

DESIGN BY: RHF
DATE: 7-21-94

CHECKED BY: N.W.
DATE: 7-22-94

EQUILIZING OPENING SIZING

STREET NAME: THOMAS ROAD SOUTH STA.=

Q DS.=	1800	DS. SIZE=	2–18'x10'	DS. AREA=	180	Hj=	0.01
Q US.=	1797	US. SIZE=	2–18'x10'	US. AREA=	180	LAT. WDTH.=	3
Q LAT.=	6	LAT. SIZE=	1–36"	OP. AREA=	0.001	LAT. ANG.=	60
MANNING'S 'n'=	0.014						

ANGLE	Vd	Vu	Hvd	Hvu	Vo	ACTUAL DELTA y	Q OPENING	CALCULATED DELTA y
90.178158333	10.00	9.98	1.55	1.55	3000.000	0.0152	3	0.01517167

NOTE THAT THE ANGLE IS THE CONFLUENCE ANGLE AND THE AREA IS AT A SCEW TO THE WALL. THE TOTAL AREA OF WALL OPENING NEEDED OF EQUILIZATION OF FLOW IS OP. AREA DIVIDED BY THE SINE OF THE ANGLE. USE STANDARD OPENING OF 8'x4'.

TOTAL AREA OF OPENING NEEDED=	32
TOTAL NUMBER OF 8'x4' OPENINGS=	1

PROJECT NAME: OLD CROSS CUT CANAL

DESIGN BY: RHF
DATE: 7-5-94

CHECKED BY: N. W
DATE: 7-22-94

PROJECT NUMBER: E002102

EQUILIZING OPENING SIZING

should be 50 cfs

STREET NAME: THOMAS ROAD NORTH STA.=

Q DS.=	1950	DS. SIZE=	2–18'x10'	DS. AREA=	180	Hj=	0.0331
Q US.=	1935	US. SIZE=	2–18'x10'	US. AREA=	180	LAT. WDTH.=	3
Q LAT.=	30	LAT. SIZE=	1–36"	OP. AREA=	1.00	LAT. ANG.=	60
MANNING'S 'n'=	0.014						

ANGLE	Vd	Vu	Hvd	Hvu	Vo	ACTUAL DELTA y	Q OPENING	CALCULATED DELTA y
97.655215506	10.83	10.75	1.82	1.79	15.000000	0.0610	15	0.061028744

✓

NOTE THAT THE ANGLE IS THE CONFLUENCE ANGLE AND THE AREA IS AT A SCEW TO THE WALL. THE TOTAL AREA OF WALL OPENING NEEDED FOR EQUILIZATION OF FLOW IS OP. AREA DIVIDED BY THE SINE OF THE ANGLE. USE STANDARD OPENING OF 4'x8'.

TOTAL AREA OF OPENING NEEDED=	32
TOTAL NUMBER OF 4'x8' OPENINGS=	1

PROJECT NAME: OLD CROSS CUT CANAL

DESIGN BY: RHF
DATE: 7-5-94

CHECKED BY: N.W
DATE: 7-22-94

PROJECT NUMBER: E002102

EQUILIZING OPENING SIZING

1055 cfs

STREET NAME: EARLL DRIVE STA.=

Q DS.=	1900	DS. SIZE=	2-18'x10'	DS. AREA=	180	Hj=	0.122
Q US.=	1351	US. SIZE=	2-18'x10'	US. AREA=	180	LAT. WDTH.=	12
Q LAT.=	1099	LAT. SIZE=	1-12'x10'	OP. AREA=	50.00	LAT. ANG.=	12.5
MANNING'S 'n'=	0.014						

ANGLE	Vd	Vu	Hvd	Hvu	Vo	ACTUAL DELTA y	Q OPENING	CALCULATED DELTA y
45.211690807	10.56	7.50	1.73	0.87	10.990000	0.9780	550	0.978026067

NOTE THAT THE ANGLE IS THE CONFLUENCE ANGLE AND THE AREA IS AT A SCEW TO THE WALL. THE TOTAL AREA OF WALL OPENING NEEDED OF EQUILIZATION OF FLOW IS OP. AREA DIVIDED BY THE SINE OF THE ANGLE. USE STANDARD OPENING OF 8'x4'.

TOTAL AREA OF OPENING NEEDED=	70
TOTAL NUMBER OF 8'x4' OPENINGS=	2

PROJECT NAME: OLD CROSS CUT CANAL

PROJECT NUMBER: E002102

DESIGN BY: RHF
DATE: 7-5-94

CHECKED BY: N.W
DATE: 7-22-94

EQUILIZING OPENING SIZING

STREET NAME: OSBORN ROAD SOUTH STA.=

Q DS.=	1390	DS. SIZE=	2-12'x10'	DS. AREA=	120	Hj=	0.2179
Q US.=	1297	US. SIZE=	2-12'x10'	US. AREA=	120	LAT. WDTH.=	6
Q LAT.=	186	LAT. SIZE=	1-6'x6'	OP. AREA=	35.00	LAT. ANG.=	12.5
MANNING'S 'n'=	0.014						

ANGLE	Vd	Vu	Hvd	Hvu	Vo	ACTUAL DELTA y	Q OPENING	CALCULATED DELTA y
36.264537909	11.58	10.81	2.08	1.81	2.657143	0.4874	93	0.487364933

NOTE THAT THE ANGLE IS THE CONFLUENCE ANGLE AND THE AREA IS AT A SCEW TO THE WALL. THE TOTAL AREA OF WALL OPENING NEEDED OF EQUILIZATION OF FLOW IS OP. AREA DIVIDED BY THE SINE OF THE ANGLE. USE STANDARD OPENING OF 8'x4'.

TOTAL AREA OF OPENING NEEDED=	59
TOTAL NUMBER OF 8'x4' OPENINGS=	2

PROJECT NAME: OLD CROSS CUT CANAL

DESIGN BY: RHF
DATE: 7-5-94

CHECKED BY: N.W
DATE: 7-22-94

PROJECT NUMBER: E002102

EQUILIZING OPENING SIZING

STREET NAME: OSBORN ROAD NORTH STA.=

Q DS.=	1390	DS. SIZE=	2–12'x10'	DS. AREA=	120	Hj=	0.071
Q US.=	1365	US. SIZE=	2–12'x10'	US. AREA=	120	LAT. WDTH.=	4
Q LAT.=	50	LAT. SIZE=	1–48"	OP. AREA=	40.00	LAT. ANG.=	45
MANNING'S 'n'=	0.014						

ANGLE	Vd	Vu	Hvd	Hvu	Vo	ACTUAL DELTA y	Q OPENING	CALCULATED DELTA y
36.035693843	11.58	11.38	2.08	2.01	0.625000	0.1453	25	0.145269971

NOTE THAT THE ANGLE IS THE CONFLUENCE ANGLE AND THE AREA IS AT A SCEW TO THE WALL. THE TOTAL AREA OF WALL OPENING NEEDED OF EQUILIZATION OF FLOW IS OP. AREA DIVIDED BY THE SINE OF THE ANGLE. USE STANDARD OPENING OF 8'x4'.

TOTAL AREA OF OPENING NEEDED=	68
TOTAL NUMBER OF 8'x4' OPENINGS=	2

PROJECT NAME: OLD CROSS CUT CANAL

DESIGN BY: RHF
DATE: 7-5-94

CHECKED BY: N.W
DATE: 7-22-94

PROJECT NUMBER: E002102

EQUILIZING OPENING SIZING

STREET NAME: WHITTON AVENUE STA.=

Q DS.=	1375	DS. SIZE=	2−12'x10'	DS. AREA=	120	Hj=	0.0173
Q US.=	1371	US. SIZE=	2−12'x10'	US. AREA=	120	LAT. WDTH.=	1.5
Q LAT.=	9	LAT. SIZE=	1−18"	OP. AREA=	0.10	LAT. ANG.=	90
MANNING'S 'n'=	0.014						

ANGLE	Vd	Vu	Hvd	Hvu	Vo	ACTUAL DELTA y	Q OPENING	CALCULATED DELTA y
94.352744375	11.46	11.42	2.04	2.03	45.000000	0.0306	5	0.030622496

NOTE THAT THE ANGLE IS THE CONFLUENCE ANGLE AND THE AREA IS AT A SCEW TO THE WALL. THE TOTAL AREA OF WALL OPENING NEEDED OF EQUILIZATION OF FLOW IS OP. AREA DIVIDED BY THE SINE OF THE ANGLE. USE STANDARD OPENING OF 8'x4'.

TOTAL AREA OF OPENING NEEDED=	32
TOTAL NUMBER OF 8'x4' OPENINGS=	1

PROJECT NAME: OLD CROSS CUT CANAL

PROJECT NUMBER: E002102

DESIGN BY: RHF
DATE: 7-5-94

CHECKED BY: N.W
DATE: 7-22-94

EQUILIZING OPENING SIZING

STREET NAME: WELDON AVENUE STA.=

Q DS.=	1210	DS. SIZE=	2-12'x10'	DS. AREA=	120	Hj=	0.0173
Q US.=	1173	US. SIZE=	2-12'x10'	US. AREA=	120	LAT. WDTH.=	4
Q LAT.=	74	LAT. SIZE=	1-48"	OP. AREA=	0.10	LAT. ANG.=	90
MANNING'S 'n'=	0.014						

ANGLE	Vd	Vu	Hvd	Hvu	Vo	ACTUAL DELTA y	Q OPENING	CALCULATED DELTA y
88.74210397	10.08	9.78	1.58	1.48	370.000000	0.1124	37	0.112377424

NOTE THAT THE ANGLE IS THE CONFLUENCE ANGLE AND THE AREA IS AT A SCEW TO THE WALL. THE TOTAL AREA OF WALL OPENING NEEDED OF EQUILIZATION OF FLOW IS OP. AREA DIVIDED BY THE SINE OF THE ANGLE. USE STANDARD OPENING OF 8'x4'.

TOTAL AREA OF OPENING NEEDED=	32
TOTAL NUMBER OF 8'x4' OPENINGS=	1

CATCH BASIN GRATE DESIGN

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of ____

Subject Local Small Catch Basins to Intercept Nuisance Flows

By RHF Date 6-22-94 Checked By JVG Date 7/22/94

Granada: 1 - C.O.P. P1568 Type 'L' North Curb
C.O.P. P1570 Type 'N' w/ 4-C.O.P. P1565 TYPE 1 Grates in S. R/W

Holly - Save exist. inlets - No new CB for nuisance

Oak ; Retain Existing CB - No new CB required for nuisance

Virginia: 2 - C.O.P. P1567 Type 'L' modified & Type L, 1 located in each curb North & South.

Cambridge: 1 - C.O.P. P1570 Type 'N' CB in N. R/W
1 - C.O.P. P1568 Type 'L' CB in 48th south of Cambridge at curb return.

Thomas: Retain existing.

Pinchot: Nuisance flow crosses 48th & into CHIMP - South Reti
Save Exist North of Pinchot.

Earll : 2 C.O.P. P-1569, Type M. — 1 - Located in Earll at the curb return

Richardson: None needed

Osborn : Use existing

Whitton : No additional needed

Weldon : No additional needed

Indian School: 1 - C.O.P. P1570 Type 'N' Located in R/W S. of Indian School at sta. 114+25 27' Rt.

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of

Subject Granada Inlets: Catch Basin Grate Design Local Storm

By RHF Date 6-2-94 Checked By N.W. Date 7-14-94

Purpose: Determine the discharge to be intercepted by the large inlet south of Granada Rd. E. of 48th ST.

Procedure: Calculate the discharge intercepted by the proposed C.O.P. Catch Basin Type L proposed in the north gutter of Granada Rd. when the depth of flow is 0.88'. Use the C.O.P. nomograph for a catch basin in a sump condition, pg 29 Storm Drain Design Manual - Storm Drains with Paving of Major Streets. Subtract this flow from the 100-yr. flow to determine the design discharge.

Granada Rd. 100-YR $Q = 145$ cfs

$Y_o = 0.88'$ (depth of flow)

$L = 8$ ft. (Length of inlet)

$h = 5''$ (height of curb opening)

$Y_o/h = 2.1$ ✓

$Q/L = 1.89$

$Q_i = 15$ cfs (Q intercepted by CB)

Total Q to intercept is 122 cfs

Q Intercepted by N. CB in Granada = 23 cfs

Design Q = 122 cfs − 23 cfs = 99 cfs

Procedure: Calculate South Inlet: With orifice flow 4-grates will intercept 99 cfs with a head of 2.5'. The Grate El is 1199.30. Outlet Pipe is a 42" RCP connecting to the double 18'X10' CBC at a 60° angle. Box Sta. for connection is 19+09.5. Provide an ADOT Modified Detale C-15.70 around the inlet.

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of 1

Subject Granada: Catch Basin Design for Inlets @ Granada - General Stm.

By RHF Date 7-10-94 Checked By N.W Date 7-24-94

Purpose: Determine number of Grates to intercept 57 cfs, the flow in excess of the road conveyance capacity. General storm = 80 cfs.

Procedure: Use orifice flow conditions with a total head of 2.5'.

$$Q = .67 A \sqrt{2gH}$$

$H = 2.5'$

$Q = 57 cfs$

$$A = \frac{Q}{.67\sqrt{2gH}} = 6.7048 ft^2 \rightarrow \text{Use 2 C.O.P. Type 1 Grates}^*$$

Clogging: 50%

Use 3 grates.

* Note 1-C.O.P. Type 1 P1565 grates as a net open area of 4.25 ft^2.

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet ___ of ___

Subject Catch Basin Design in a Sump - FCDMC Procedure

By RHF Date 6-27-94 Checked By N.W. Date 7-24-94

Granada North Curb Return:

Purpose: Determine the discharge intercepted at this inlet. Use the Depth of Flow over the inlet from the Roadway flow analysis. Use a C.O.P. Type 'L' inlet. However, neglect the flow intercepted on the back side.

Procedure: Use Eq. 3.9 in the FCDMC Hydraulics Manual on page 3-18. (Note $d \leq h$; $h = 0.88'$)

$$Q_i = C_o h L (2gd)^{0.5} \qquad C_o = 0.67$$

$C_o = 0.67$

$h = (5''/12) + (2''/12)' = 0.58'$

$L = 6.4$ (Curb opening Clogging = 0.8 ∴ $6.4/0.8 = 8.0'$)

$d = 0.88'$

$Q_i = 19$ cfs

From this analysis it can be determined that of the 122 cfs to be intercepted, the Large inlet South of Granada in the R/W must intercept 103 cfs (122 - 19)

PROJECT: OCCC
PROJECT NO.: E002102

GROSS AREA = 6.67 ft^2
NET AREA = 4.25 ft^2
N/G = 0.64

Street Name: Granada Rd Q= 103 cfs Local Storm conditions
CLOG FACTOR= 2

WIER:
Q=3*L*H^1.5

H	LENGTH	GRATES	WITH CLOGGING
1	34.3	10	15
1.5	18.7	6	8
2	12.1	4	5
2.5	8.7	3	4
3	6.6	2	3
3.5	5.2	2	2

ORFICE:
Q=0.67A*(SQRT(2GH))

H	AREA	GRATES	WITH CLOGGING
1	19.2	5	7
1.5	15.6	4	6
2	13.5	3	5
2.5	12.1	3	4
3	11.1	3	4
3.5	10.2	2	4

Same in Pressure flow Calc.

FRONTAL FLOW INTERCEPTION:
L=(V/2)*((D+Db)^.5)

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of

Subject Oak Inlets: Catch Basin Grate Design

By RHF Date 6-20-94 Checked By N.W Date 7-26-94

Oak: Sta. 37+12, 39' Rt.

- Local: 1540 cfs from the east.
  376 cfs weirs North into the N. Cemetary.
  614 cfs weirs South into the S. Cemetary
- 550 cfs continues West in Oak.
- Of the 376 cfs weiring north into the N. Cemetary, 137 cfs Turns South toward Oak.
- Total flow to the Oak inlet = 550 + 137 = 687 cfs
- General: 806 cfs from the east.
  146 cfs weirs north into N. Cemetary
  281 cfs weirs south into S. Cemetary
  380 cfs continues west in Oak
- Of the 146 cfs weiring north into the N. Cemetary, 69 cfs Turns South toward Oak.
- Total flow to the Oak Inlet = 380 cfs + 69 = 449 cfs

Note: Design Inlet to intercept Local Storm

30 grates w/ 6'x8' CBC 12.5° junction Angle.
h = 2.5' Orifice flow conditions.
Grate El. = 1200.68

PROJECT: OCCC
PROJECT NO.: E002102

GROSS AREA = 6.67 ft^2
NET AREA = 4.25 ft^2
N/G = 0.64

Street Name: Oak Street Q= 687 cfs Local storm
CLOG FACTOR= 2

WIER:
Q=3*L*H^1.5

H	LENGTH	GRATES	WITH CLOGGING
1	229.0	69	103
1.5	124.7	37	56
2	81.0	24	36
2.5	57.9	17	26
3	44.1	13	20
3.5	35.0	10	16

ORFICE:
Q=0.67A*(SQRT(2GH))

H	AREA	GRATES	WITH CLOGGING
1	127.8	30	45
1.5	104.3	25	37
2	90.3	21	32
2.5	80.8	19	29 → use 30
3	73.8	17	26
3.5	68.3	16	24

FRONTAL FLOW INTERCEPTION:
L=(V/2)*((D+Db)^.5)

Spillway Width Total (from 4:1 expansion) = 96'
30 grates provided and oriented as shown in the plans

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of 2

Subject Design Width of opening to intercept the flow

By RHF Date 6-22-94 Checked By N.W. Date 7-20-94

Purpose: Determine the width of opening that will intercept the flow coming down the spillway and not allow to over shoot the inlet.

Procedure: Use the procedure discussed in Open Channel Hydraulics by V. CHOW pg 423, 1969.

General Storm

$Q = 449$ cfs

b = Spillway Width = 96'

$q = Q/b = (449/96) = 4.677$

$D = \frac{q^2}{g h^3}$

$h = 1204.64 - 1200.89 = 3.80'$ (Hinge El. - Gate El.)

$D = 0.012381$

$L_d/h = 4.30 D^{0.27} = 1.3137$

$L_d = 5$ ft. → OK 6.5 provided.

Local Storm

$Q = 687$ cfs

$b = 96'$

$q = Q/b = (687/96) = 7.1563$

$D = \frac{q^2}{g h^3} = 0.028984$

$L_d/h = 4.30 D^{0.27} = 1.65$

$L_d = 6.28$ → OK < 6.5'

Sheet 2 of 2

15-15. The Straight Drop Spillway. The aerated free-falling nappe in a straight drop spillway (Fig. 15-18) will reverse its curvature and turn smoothly into supercritical flow on the apron. Consequently, a hydraulic jump may be formed downstream. Based on his own experimental data

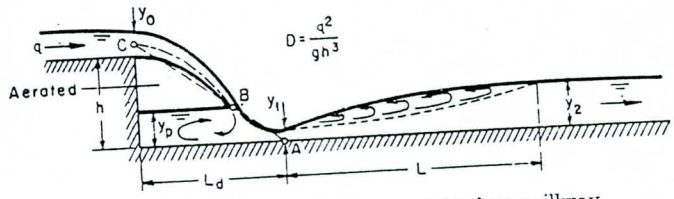


FIG. 15-18. Flow geometry of a straight drop spillway.

and those of Moore [40] and Bakhmeteff and Feodoroff [65], Rand [66] found that the flow geometry at straight drop spillways can be described by functions of the *drop number*, which is defined as

$$D = \frac{q^2}{gh^3} \tag{15-9}$$

where q is the discharge per unit width of the crest of overfall, g is the acceleration of gravity, and h is the height of the drop. The functions are

$$\frac{L_d}{h} = 4.30D^{0.27} \tag{15-10}$$

$$\frac{y_p}{h} = 1.00D^{0.22} \tag{15-11}$$

$$\frac{y_1}{h} = 0.54D^{0.425} \tag{15-12}$$

$$\frac{y_2}{h} = 1.66D^{0.27} \tag{15-13}$$

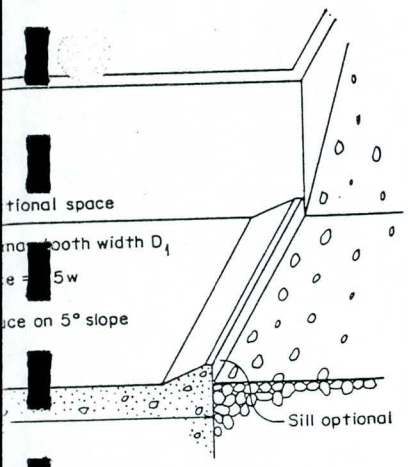
where L_d is the drop length, that is, the distance from the drop wall to the position of the depth y_1; y_p is the pool depth under the nappe; y_1 is the depth at the toe of the nappe or the beginning of the hydraulic jump; and y_2 is the tailwater depth sequent to y_1. The position of the depth y_1 can be approximately determined by the straight line ABC which joins the point A on the apron at the position of y_1, the point B on the axis of the nappe at the height of pool depth, and the point C on the axis of the nappe at the crest of the fall. The fact that these three points lie on a straight line was also verified by experiment.

For a given height h and discharge q per unit width of the fall crest, the sequent depth y_2 and the drop length L_d can be computed by Eqs. (15-10) and (15-13). On the one hand, if the tailwater depth is less than

FLOW

tering the curve in Fig. 15-16c with
16 × 36.9 = 154 ft.
ks as recommended are D_1; thus the
ntated sill is $0.2D_2$, or 7 ft 5 in., and
$.15D_2$, or 5 ft 6 in.

F_1 = 2.5 to 4.5, an oscillating
in, generating a wave that is
basin IV (Fig. 15-17) is designed
e wave at its source.[1] This is



IV (*U.S. Bureau of Reclamation* [34].)

hich appears in the upper portion of
jets deflected from large chute
wn in Fig. 15-17 is the minimum
better hydraulic performance, it is
ower than indicated, preferably
to 10% greater than the sequent
he basin is made equal to the length
basin without appurtenances and,
e in Fig. 15-4 (i.e., also equal to
in IV is applicable to rectangular

designs to substitute for basin IV, such
ave suppressors, and the impact-type

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Job No. E002102 Project Old Cross Cut Canal Sheet 1 of

Subject Flow split Analysis & Catch Basin Grate Design

By RHF Date 6-20-94 Checked By N.W Date 7-26-94

Thomas Rd. South:

Local: 267 cfs from the east.
57 cfs weirs North
210 cfs weirs South
90 cfs Intercepted in South Inlet
57 cfs Intercepted in North Inlet

General: 156 cfs from the east.
30 cfs Weirs North
126 cfs Weirs South.
6 cfs Intercepted in Inlet.
30 + 20 (from north) = 50 cfs in north

Design both north & south inlets for the Local Storm.

Refer to pressure analysis for the hydraulics for inlets to confirm that the Grate size works for the general storm.

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Job No. E002102 Project Old Cross Cut Canal Sheet ____ of ____

Subject Thomas Rd Inlets : Catch Basin Gr Inlets

By RHF Date 6-22-94 Checked By N.W. Date 7-24-94

Thomas Road South Side Inlet : Sta. 62+55, 24' Rt.

Q = 90 cfs (Local) 6 cfs (general) use Local

Requires 4 Grates w/ 2.5' of head - Orifice flow

Grate El. = 1207.00 36" RCP w/ ~~30°~~ 60° skew

CB height = 6'

Thomas Road North Side Inlet : Sta. 64+37.5, 22.5' Rt.

Q = 57 cfs (Local) 50 cfs (General) use Local

Requires 2 Grates w/ 2.5' of head - Orifice flow.

Grate El. = 1207.5 1-30" RCP w/ ~~30°~~ 60° skew

CB height = 6'

PROJECT: OCCC
PROJECT NO.: E002102

GROSS AREA = 6.67 ft^2
NET AREA = 4.25 ft^2
N/G = 0.64

Street Name: Thomas N. Q= 57 cfs Local Storm
CLOG FACTOR= 2

WIER:
Q=3*L*H^1.5

H	LENGTH	GRATES	WITH CLOGGING
1	19.0	6	9
1.5	10.3	3	5
2	6.7	2	3
2.5	4.8	1	2
3	3.7	1	2
3.5	2.9	1	1

ORFICE:
Q=0.67A*(SQRT(2GH))

H	AREA	GRATES	WITH CLOGGING
1	10.6	2	4
1.5	8.7	2	3
2	7.5	2	3
2.5	6.7	2	2
3	6.1	1	2
3.5	5.7	1	2

FRONTAL FLOW INTERCEPTION:
L=(V/2)*((D+Db)^.5)

PROJECT: OCCC
PROJECT NO.: E002102

GROSS AREA = 6.67 ft^2
NET AREA = 4.25 ft^2
N/G = 0.64

Street Name: Thomas S. Q= 90 cfs Local storm
CLOG FACTOR= 2

WIER:
Q=3*L*H^1.5

H	LENGTH	GRATES	WITH CLOGGING
1	30.0	9	14
1.5	16.3	5	7
2	10.6	3	5
2.5	7.6	2	3
3	5.8	2	3
3.5	4.6	1	2

ORFICE:
Q=0.67A*(SQRT(2GH))

H	AREA	GRATES	WITH CLOGGING
1	16.7	4	6
1.5	13.7	3	5
2	11.8	3	4
2.5	10.6	2	4
3	9.7	2	3
3.5	8.9	2	3

FRONTAL FLOW INTERCEPTION:
L=(V/2)*((D+Db)^.5)

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Job No. E002102 Project Old Cross Cut Canal Sheet 1 of

Subject Ewell Drive: Catch Basin Grate Design

By RHF Date 6-15-94 Checked By N.W. Date 7-24-94

Ewell Sta. 76+31.5, 29' Rt.

Grading: The inlet shall be placed in a local sump offset to the south to miss the existing 54" storm drain which will tie into the proposed double 18'x10' CBC. Flow will need to be directed to the inlet.

Street Geometry: 48± st. will have a 1.0% cross slope. The east edge of pavement (EOP) elevation at the ℄ of Ewell is 1213.71. The west EOP el. is 1213.30.
The grate elevation is 1210.50. This causes the watersurface to be 0.3 below the EOP.

Local Storm = 1509 cfs use 63 grates.

General Storm = 1055 cfs - refer to the Hydraulics of Inlets for pressure analysis.

V

PROJECT: OCCC
PROJECT NO.: E002102

GROSS AREA = 6.67 ft^2
NET AREA = 4.25 ft^2
N/G = 0.64

Street Name: Earll Drive
Q= 1509 cfs Local Storm
CLOG FACTOR= 2

WIER:
Q=3*L*H^1.5

H	LENGTH	GRATES	WITH CLOGGING
1	503.0	151	226
1.5	273.8	82	123
2	177.8	53	80
2.5	127.3	38	57
3	96.8	29	44
3.5	76.8	23	35

why the grates designed for 1509 but the box is designed to carry 1055 cfs?

ok because of pressure flow the pipe or box can carry 1652 cfs which is greater than 1055 cfs

ORFICE:
Q=0.67A*(SQRT(2GH))

H	AREA	GRATES	WITH CLOGGING
1	280.7	66	99
1.5	229.2	54	81
2	198.5	47	70
2.5	177.5	42	63
3	162.0	38	57
3.5	150.0	35	53

FRONTAL FLOW INTERCEPTION:
L=(V/2)*((D+Db)^.5)

V = 18.43 fps
D = Y2 = 0.7651'
Db = 0.2917
L = 9.5' < 4

Note: use 2.5' of Depth & 63 Grates (Orifice w/ Clogging is 63 grates). 4'-wide 107'-Long

Note: 60 grates provides 43% clogging for the local storm.

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Job No. E002102 Project Old Cross Cut Canal Sheet 1 of

Subject Richardson : Catch Basin Grate Design.

By RHF Date 6-11-94 Checked By N. W. Date 7-24-94

Richardson. Sta. 82+63, 18' Rt.

Grading: The inlet shall be placed at the bottom of the spillway and require a local depression of 2.5'.

Street Geometry: 48th St. will be graded with a 2% cross slope. The east EOP elevation at alley ℄ = 1217.36. Proposed roadway width = 40'. West EOP elevation is 1216.56.
Box ℄ Sta. = 83+23 ; Box Inv. El. = 1199.19
Box outside top of slab El. = 1210.31.
Total Cover over box = 1216.56 − 1210.31 = 6.25
Grate El. w/ 2.5' depression = 1216.56 − 2.5' = 1214.06
El. difference Grate to Top of Slab = 1214.06 − 1210.31 = 3.75'

No
Not 4' of Cover over box.

* However, if Sx = 0.0138 ft/ft.
West EOP = 1217.36 − (40 * 0.0138) = 1216.81
Grate Elevation = 1216.81 − 2.5' = 1214.31 use 1214.10

Local Storm: Q = 66 cfs OK

General storm: Q = 35 cfs Refer to hydraulics for inlets to be sure the inlet works w/ pressure flow.

Use 4-Inlets connected to the 2-12'x10' by 1-30" RCP with a junction angle of 60°

PROJECT: OCCC
PROJECT NO.: E002102

GROSS AREA = 6.67 ft^2
NET AREA = 4.25 ft^2
N/G = 0.64

Street Name: Richardson Q= 66 cfs Local Storm
CLOG FACTOR= 2

WIER:
Q=3*L*H^1.5

H	LENGTH	GRATES	WITH CLOGGING
1	22.0	7	10
1.5	12.0	4	5
2	7.8	2	4
2.5	5.6	2	3
3	4.2	1	2
3.5	3.4	1	2

ORFICE:
Q=0.67A*(SQRT(2GH))

H	AREA	GRATES	WITH CLOGGING
1	12.3	3	4
1.5	10.0	2	4
2	8.7	2	3
2.5	7.8	2	3 ←
3	7.1	2	3
3.5	6.6	2	2

FRONTAL FLOW INTERCEPTION:
L=(V/2)*((D+Db)^.5)

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Job No. E002102 Project Old Cross Cut Canal Sheet 1 of

Subject Osborn Rd: Catch Basin Grate Design.

By RHF Date 6-22-94 Checked By N.W. Date 7-14-94

Osborn Road South: Sta. 88+61.4, 21' Lt.

Q local = 318 cfs
Use 14 Grates w/ a 2.5' sump - Orifice flow
1-6'x6' CBC @ 12.5° Junction angle.
Grate El. = 1218.20

Osborn Road North: Sta. 90+50, 19' Lt.

Q local = 113 cfs
Use 6 Grates w/ a 2.5' sump - Orifice Flow
1-48" RCP @ 45° Junction angle.
GRATE El. = 1218.80

✓

Refer to Hydraulics of Inlets section for general storm pressure flow analysis. ✓

PROJECT: Old Cross Cut Canal
JOB NO.: E002102

FLOW SPLIT ANALYSIS FOR SPILLWAY AND INLET DESIGN

LOCAL STORM DISCHARGE: 431
ROADWAY RATING: Osborn Road GENERAL STORM DISCHARGE: 235

Elevation	North Roadway Capacity	South Roadway Capacity	Total Roadway Capacity
1223.00	45.36	175.43	220.79
1223.10	59.94	209.69	269.63
1223.21	76.77	250.05	326.82
1223.32	98.29	293.09	391.38
1223.44	126.00	342.91	468.91
1223.56	156.33	395.85	552.18
1223.68	189.13	451.82	640.95
1223.80	224.34	510.43	734.77

Storm	Elevation	North Roadway Discharge	South Roadway Discharge	Total Discharge
General	1223.03	49.5	185.5	235.0
Local	1223.38	113.0	318.0	431.0

PROJECT: OCCC
PROJECT NO.: E002102

GROSS AREA = 6.67 ft^2
NET AREA = 4.25 ft^2
N/G = 0.64

Street Name: Osborn S.

Q= 318 cfs Local Storm

CLOG FACTOR= 2

WIER:
Q=3*L*H^1.5

H	LENGTH	GRATES	WITH CLOGGING
1	106.0	32	48
1.5	57.7	17	26
2	37.5	11	17
2.5	26.8	8	12
3	20.4	6	9
3.5	16.2	5	7

ORFICE:
Q=0.67A*(SQRT(2GH))

H	AREA	GRATES	WITH CLOGGING
1	59.1	14	21
1.5	48.3	11	17
2	41.8	10	15
2.5	37.4	9	13
3	34.1	8	12
3.5	31.6	7	11

FRONTAL FLOW INTERCEPTION:
L=(V/2)*((D+Db)^.5)

PROJECT: OCCC
PROJECT NO.: E002102

GROSS AREA = 6.67 ft^2
NET AREA = 4.25 ft^2
N/G = 0.64

Street Name: Osborn N.

Q= 113 cfs Local Storm
CLOG FACTOR= 2

WIER:
Q=3*L*H^1.5

H	LENGTH	GRATES	WITH CLOGGING
1	37.7	11	17
1.5	20.5	6	9
2	13.3	4	6
2.5	9.5	3	4
3	7.2	2	3
3.5	5.8	2	3

ORFICE:
Q=0.67A*(SQRT(2GH))

H	AREA	GRATES	WITH CLOGGING
1	21.0	5	7
1.5	17.2	4	6
2	14.9	3	5
2.5	13.3	3	5
3	12.1	3	4
3.5	11.2	3	4

FRONTAL FLOW INTERCEPTION:
L=(V/2)*((D+Db)^.5)

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of ____

Subject Whitton Ave: Catch Basin Grate Design

By RMF Date 6-25-94 Checked By N.W. Date 7-24-94

Whitton Ave: Box Sta. 96+64, 69.5' Rt = Road Sta. 285+89.5, 32.5' Rt.

Q local = 18 cfs

USE 1 Grate w/ 1.5' of head - Orifice flow.

Grate El. = 1225.00

1-18" RCP @ 90°

Q general = 9 cfs Refer to Hydraulics for Inlets for general storm Design.

✓

PROJECT: OCCC
PROJECT NO.: E002102

GROSS AREA = 6.67 ft^2
NET AREA = 4.25 ft^2
N/G = 0.64

Street Name: Whitton Ave. Q= 18 cfs Local Storm
CLOG FACTOR= 2

WIER:
Q=3*L*H^1.5

H	LENGTH	GRATES	WITH CLOGGING
1	6.0	2	3
1.5	3.3	1	1
2	2.1	1	1
2.5	1.5	0	1
3	1.2	0	1
3.5	0.9	0	0

ORFICE:
Q=0.67A*(SQRT(2GH))

H	AREA	GRATES	WITH CLOGGING
1	3.3	1	1
1.5	2.7	1	1
2	2.4	1	1
2.5	2.1	0	1
3	1.9	0	1
3.5	1.8	0	1

FRONTAL FLOW INTERCEPTION:
L=(V/2)*((D+Db)^.5)

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Job No. E002102 Project Old Cross Cut Canal Sheet 1 of

Subject Weldon Avenue: Catch Basin Grate Design

By RHF Date 6-25-94 Checked By N.W. Date 7-24-94

Weldon Ave.: Box Sta. 100+22, 68' Rt. = Road Sta. 289+57, 33' Rt.

Q_{local} = 149 cfs

Use 6-P1565 TYPE 1 Grates. 2.5' head - Orifice Flow

Grate El. = 1226.80

1-48" w/ 90° RCP.

Refer to Hydraulics for Inlets section for general storm pressure calculations.

✓

PROJECT: OCCC
PROJECT NO.: E002102

GROSS AREA = 6.67 ft^2
NET AREA = 4.25 ft^2
N/G = 0.64

Street Name: Weldon Ave. Q= 149 cfs Local Storm
CLOG FACTOR= 2

WIER:
Q=3*L*H^1.5

H	LENGTH	GRATES	WITH CLOGGING
1	49.7	15	22
1.5	27.0	8	12
2	17.6	5	8
2.5	12.6	4	6
3	9.6	3	4
3.5	7.6	2	3

ORFICE:
Q=0.67A*(SQRT(2GH))

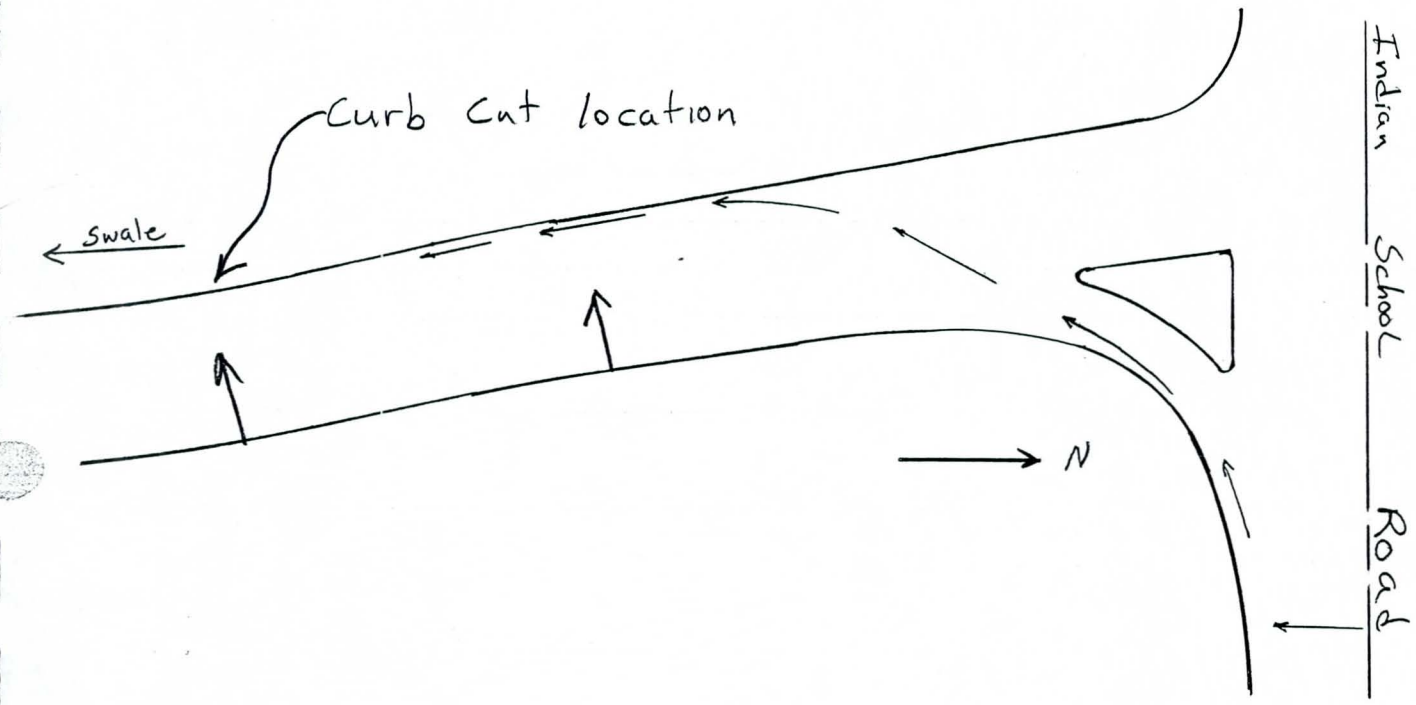
H	AREA	GRATES	WITH CLOGGING
1	27.7	7	10
1.5	22.6	5	8
2	19.6	5	7
2.5	17.5	4	6 ←
3	16.0	4	6
3.5	14.8	3	5

FRONTAL FLOW INTERCEPTION:
L=(V/2)*((D+Db)^.5)

Greiner

Job OCCC | Project No. E002102 | Sheet 1 of ____
Description ISR Inlet for 100 yr storm - South side | Computed By JUG | Date 6/24/94
Checked By N.W | Date 7-26-94

Determine Curb opening length required to convey 13 CFS (100YR "LOCAL" from ISR) into the graded swale on West side of 48th Street.



Curb opening Req. to intercept 100% of flow

RDWY STA. 300+40 TO STA. 302+70

Ref: Drainage Design Manual for Maricopa County. Pg 3-18 Hydraulics Vol. II

$S_{long.} = 0.0033 \, ^1/_1$

$S_{cross} = \frac{.02 + .015}{2} = .0175 \, ^1/_1$

$Q_i = C_w \, L \, d^{1.5}$ $\qquad C_w = 3.0$

$L = \frac{Q_i}{C_w \, d^{1.5}}$

$L = \frac{13}{3.0 \, (0.488)^{1.5}} = 12.71$ feet ✓

Determine # of 3' openings needed to avoid using bollards.

① $Q_i = (3.0)(3.0')(0.488)^{1.5} = 3.07$ CFS ✓

② $Q_i = (3.0)(3.0')(0.446)^{1.5} = 2.68$ CFS ✓

Greiner

Job _____ Project No. _____ Sheet 2 of ____

Description _____ Computed By VG Date 6/24/94

Checked By N. W. Date 7-24-94

(3.) $Q_i = (3.0)(3.0)(0.402)^{1.5} = 2.29$ CFS ✓

(4.) $Q_i = (3.0)(3.0)(0.355)^{1.5} = 1.90$ CFS ✓

(5.) $Q_i = (3.0)(3.0)(0.305)^{1.5} = 1.52$ CFS ✓

(6.) $Q_i = (3.0)(3.0)(0.246)^{1.5} = 1.10$ CFS ✓

(7.) $Q_i = (3.0)(3.0)(0.169)^{1.5} = 0.62$ CFS ✓

Total = 13.18 CFS ✓

Note: for d values see sheets 3-6

Requires 7 - 3' curb openings

Use 1 13 foot curb opening to allow water into swale.

Check depth of flow in swale → (See sheet 7)

$d_n = 0.62'$ V = 3.3 FPS OK ✓

```
     C U R B   A N D   G U T T E R   F L O W   C A L C U L A T I O N S
DDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDD
Location:  48TH STREET JUST SOUTH OF ISR 300+40 - 302+70
Compound section geometry data:
  Manning's n = .015                    Slope of Curb face = 0 Horiz : 1 Vert
  Gutter width = 1.416667 feet          Gutter drop =  1 inch absolute
  Cross slope = .0194 ft/ft             Roadway Slope = .0052 ft/ft
Calculate depth of flow in section:
  Flow = 13 cfs                         depth of flow = .4649544 feet
  Pvmt Spread = 21.08822 feet           Average Velocity = 2.986259 fps

Inlet Calculation Menu
  1.  New location
  2.  Revise flow calculations
  3.  City of Phoenix curb opening inlet on grade
  4.  Curb opening inlet in sump
  5.  Slotted drain on grade
  6.  Grate inlet on grade
  7.  Go to DOS
  8.  Quit
DDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDD
Enter your selection:
```

```
     C U R B   A N D   G U T T E R   F L O W   C A L C U L A T I O N S
DDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDD
Location:  48TH STREET JUST SOUTH OF ISR 300+40 - 302+70
Compound section geometry data:
  Manning's n = .015                    Slope of Curb face = 0 Horiz : 1 Vert
  Gutter width = 1.416667 feet          Gutter drop =  1 inch absolute
  Cross slope = .0194 ft/ft             Roadway Slope = .0052 ft/ft
Calculate depth of flow in section:
  Flow = 9.93 cfs                       depth of flow = .4250005 feet
  Pvmt Spread = 19.02838 feet           Average Velocity = 2.795823 fps

Inlet Calculation Menu
  1.  New location
  2.  Revise flow calculations
  3.  City of Phoenix curb opening inlet on grade
  4.  Curb opening inlet in sump
  5.  Slotted drain on grade
  6.  Grate inlet on grade
  7.  Go to DOS
  8.  Quit
DDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDD
Enter your selection:
```

```
     C U R B    A N D    G U T T E R    F L O W    C A L C U L A T I O N S
DDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDD
Location:  48TH STREET JUST SOUTH OF ISR 300+40 - 302+70
Compound section geometry data:
  Manning's n = .015                     Slope of Curb face = 0 Horiz : 1 Vert
  Gutter width = 1.416667 feet           Gutter drop =  1 inch absolute
  Cross slope = .0194 ft/ft              Roadway Slope = .0052 ft/ft
Calculate depth of flow in section:
  Flow = 7.25 cfs                        depth of flow = .3830895 feet
  Pvmt Spread = 16.86801 feet            Average Velocity = 2.589749 fps

Inlet Calculation Menu
  1.   New location
  2.   Revise flow calculations
  3.   City of Phoenix curb opening inlet on grade
  4.   Curb opening inlet in sump
  5.   Slotted drain on grade
  6.   Grate inlet on grade
  7.   Go to DOS
  8.   Quit
DDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDD
Enter your selection:
```

```
     C U R B    A N D    G U T T E R    F L O W    C A L C U L A T I O N S
DDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDD
Location:  48TH STREET JUST SOUTH OF ISR 300+40 - 302+70
Compound section geometry data:
  Manning's n = .015                     Slope of Curb face = 0 Horiz : 1 Vert
  Gutter width = 1.416667 feet           Gutter drop =  1 inch absolute
  Cross slope = .0194 ft/ft              Roadway Slope = .0052 ft/ft
Calculate depth of flow in section:
  Flow = 4.96 cfs                        depth of flow = .3385288 feet
  Pvmt Spread = 14.57102 feet            Average Velocity = 2.363014 fps

Inlet Calculation Menu
  1.   New location
  2.   Revise flow calculations
  3.   City of Phoenix curb opening inlet on grade
  4.   Curb opening inlet in sump
  5.   Slotted drain on grade
  6.   Grate inlet on grade
  7.   Go to DOS
  8.   Quit
DDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDD
Enter your selection:
```

C U R B   A N D   G U T T E R   F L O W   C A L C U L A T I O N S

DDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDD

Location: 48TH STREET JUST SOUTH OF ISR 300+40 - 302+70

Compound section geometry data:

Manning's n = .015
Slope of Curb face = 0 Horiz : 1 Vert
Gutter width = 1.416667 feet
Gutter drop = 1 inch absolute
Cross slope = .0194 ft/ft
Roadway Slope = .0052 ft/ft

Calculate depth of flow in section:

Flow = 3.06 cfs
depth of flow = .2900073 feet
Pvmt Spread = 12.06996 feet
Average Velocity = 2.10643 fps

Inlet Calculation Menu

1. New location
2. Revise flow calculations
3. City of Phoenix curb opening inlet on grade
4. Curb opening inlet in sump
5. Slotted drain on grade
6. Grate inlet on grade
7. Go to DOS
8. Quit

DDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDD

Enter your selection:

C U R B   A N D   G U T T E R   F L O W   C A L C U L A T I O N S

DDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDD

Location: 48TH STREET JUST SOUTH OF ISR 300+40 - 302+70

Compound section geometry data:

Manning's n = .015
Slope of Curb face = 0 Horiz : 1 Vert
Gutter width = 1.416667 feet
Gutter drop = 1 inch absolute
Cross slope = .0194 ft/ft
Roadway Slope = .0052 ft/ft

Calculate depth of flow in section:

Flow = 1.54 cfs
depth of flow = .2339164 feet
Pvmt Spread = 9.17868 feet
Average Velocity = 1.797453 fps

Inlet Calculation Menu

1. New location
2. Revise flow calculations
3. City of Phoenix curb opening inlet on grade
4. Curb opening inlet in sump
5. Slotted drain on grade
6. Grate inlet on grade
7. Go to DOS
8. Quit

DDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDD

Enter your selection:

```
     C U R B   A N D   G U T T E R   F L O W   C A L C U L A T I O N S
DDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDD
Location:  48TH STREET JUST SOUTH OF ISR 300+40 - 302+70
Compound section geometry data:
  Manning's n = .015                    Slope of Curb face = 0 Horiz : 1 Vert
  Gutter width = 1.416667 feet          Gutter drop =  1 inch absolute
  Cross slope = .0194 ft/ft             Roadway Slope = .0052 ft/ft
Calculate depth of flow in section:
  Flow = .44 cfs                        depth of flow = .1598052 feet
  Pvmt Spread = 5.358518 feet           Average Velocity = 1.383285 fps

Inlet Calculation Menu
  1.  New location
  2.  Revise flow calculations
  3.  City of Phoenix curb opening inlet on grade
  4.  Curb opening inlet in sump
  5.  Slotted drain on grade
  6.  Grate inlet on grade
  7.  Go to DOS
  8.  Quit
DDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDD
Enter your selection:
```

DRAINAGE SWALE - SOUTH OF INDIAN SCHOOL RD NEAR STA. 301+00
CHANNEL HYDRAULICS CHARACTERISTICS

```
**                          **
 **-------------------------**
  **          0.62        **
   **       FT DEEP     **
    **                  **
     ****************
```

Q=	13.00	cfs	
S=	0.009	ft/ft	
n=	0.025		
b=	4	ft	
z=	4	:1	
TRIAL DEPTH=	0.62	ft	

CALCULATED

FLOW=	13.00	CFS	CRITICAL DEPTH=	0.57	FT
NORMAL DEPTH=	0.62	FT	VELOCITY=	3.7	FPS
VELOCITY=	3.3	FPS	SLOPE=	0.0125	FT/FT
FROUDE #=	0.7				
TOP WIDTH WS=	8.9	FT			

VALUES = FOR CRITICAL DEPTH

44.79	44.79
	NUM

-Jun-94 12:11 PM

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of

Subject Flow Split Analysis

By RHF Date 6-24-94 Checked By N.W. Date 7-24-94

Indian School Rd.

Purpose: Determine Discharge flowing in the north and South ½ roadway sections for catch basin design.

Procedure: Use the flow split used in the 30% submittal for catch basin design to determine a flow split percentage and apply it to the local storm discharge. Once the north and south discharges have been computed, determine that the section that should convey the larger Q has the capacity for that Q.

From 30% Submittal:

$Q_{Total} = 19 cfs$

$Q_{north} = 13 cfs$

% of Q in North Section = 68.42%.

For 60% Submittal:

$Q_{Local} = 41 cfs$

$Q_{north} = 0.6842 (Q_{Local}) = 28 cfs$

$Q_{south} = Q_{Local} - Q_{north} = 41 cfs - 28 cfs = 13 cfs.$

Note: North ½ Roadway Can Convey 25 cfs. However use 28 as OK.

PROJECT: OCCC
PROJECT NO.: E002102

GROSS AREA = 6.67 ft^2
NET AREA = 4.25 ft^2
N/G = 0.64

Street Name: Indian Sch.S
Q= 13 cfs
CLOG FACTOR= 2

WIER:
Q=3*L*H^1.5

H	LENGTH	GRATES	WITH CLOGGING
1	4.3	1	2
1.5	2.4	1	1
2	1.5	0	1
2.5	1.1	0	0
3	0.8	0	0
3.5	0.7	0	0

ORFICE:
Q=0.67A*(SQRT(2GH))

H	AREA	GRATES	WITH CLOGGING
1	2.4	1	1
1.5	2.0	0	1
2	1.7	0	1
2.5	1.5	0	1
3	1.4	0	0
3.5	1.3	0	0

FRONTAL FLOW INTERCEPTION:
L=(V/2)*((D+Db)^.5)

Inlet in Orifice Flow w/ 1 grate and 1.5' of head over the grate.

Inlet at sta ~~112 +40~~ 111 +25 ±

PROJECT NAME:	OLD CROSS CUT CANAL	Grate Net Area:	4.25	No. OF GRATES:	1
JOB NUMBER:	E002102	Height of Catch Basin:	4	Ke:	0.50
		Top Slab Thickness:	1.75	Ko:	1.00
GRATE TYPE:	C.O.P. Standard Detail P1565 Type 1 Grate	Length:	5.75	Kf:	0.0611
CATCH BASIN LOCATION:	SOUTH OF INDIAN SCHOOL RD.	Manning's n:	0.014		
CHIMP ENERGY ELEVATION:	1240.12	Grate Total Area:	6.67		
BOX CULVERT ENERGY ELEVATION:	1229.93	Total Perimeter:	10.67		
GRATE ELEVATION:	1237.00	Hydraulic Radius:	0.6251		

Q	Hp	Ho	An	Ag	Kl	Kl−Kt	Ktg	Vn	Hvn	Vg	Hvg	Ktn	Ktn	Qo
Assumed Discharge	Pressure Head	Oriface Head	Net Grate Open Area	Total Grate Open Area	Calculated Total Loss Coefficient	Total Loss Coefficient Without Trashrack Loss	Calculated Trashrack Adjusted Coefficient	Net Grate Area Velocity	Net Grate Area Velocity Head	Total Grate Area Velocity	Total Grate Area Velocity Head	Calculated Trashrack Coefficient	"Small Dam's" Trashrack Coefficient	Oriface Equation
20	1.86	3.28	4.25	6.67	13.3226	1.5611	11.7616	4.7059	0.3439	2.9985	0.1396	4.7752	0.7573	41.39
25	1.86	3.28	4.25	6.67	8.5265	1.5611	6.9654	5.8824	0.5373	3.7481	0.2181	2.8280	0.7573	41.39
30	1.86	3.28	4.25	6.67	5.9212	1.5611	4.3601	7.0588	0.7737	4.4978	0.3141	1.7702	0.7573	41.39
35	1.86	3.28	4.25	6.67	4.3503	1.5611	2.7892	8.2353	1.0531	5.2474	0.4276	1.1324	0.7573	41.39
36	1.86	3.28	4.25	6.67	4.1119	1.5611	2.5509	8.4706	1.1141	5.3973	0.4523	1.0357	0.7573	41.39
37	1.86	3.28	4.25	6.67	3.8927	1.5611	2.3316	8.7059	1.1769	5.5472	0.4778	0.9466	0.7573	41.39
38	1.86	3.28	4.25	6.67	3.6905	1.5611	2.1294	8.9412	1.2414	5.6972	0.5040	0.8645	0.7573	41.39
39	1.86	3.28	4.25	6.67	3.5037	1.5611	1.9426	9.1765	1.3076	5.8471	0.5309	0.7887	0.7573	41.39
39.437	1.86	3.28	4.25	6.67	3.4264	1.5611	1.8654	9.2793	1.3370	5.9126	0.5428	0.7573	0.7573	41.39

NOTE: Qo Should Be > Q (Column 'A' < Column 'O'); Column 'M' Should = Column 'N'

Design Discharge is 13 cfs. There are no adverse effects from the energy in the box culvert at this location.

PROJECT: Old Cross Cut Canal JOB NUMBER: E002102 DATE: 7-1-94

DISCRIPTION: Drop Inlet Design at Sta 111+25 (S. of Indian School Rd.)

COMPUTED BY: RHF CHECKED BY: _____ DATE CHECKED: _____ SHT 1 OF ____

FCDMC – FIGURE 4–9

INLET STATION: 111 +25 DOWNSTREAM HG: 1214.83 (@ 101+10)

1. DESIGN CONDITION:
   - A. INLET LOCATED OVER THE BOX CULVERT.
   - B. USE FCDMC VOLUME II – HYDRAULICS PP 77 &78; SECTION 4.2.4.3; FIGURE 4.9.
   - C. Qo= 2360 ; Qu= 2347 ; Qg= 13 ; (Qu/Qo)= 0.9945 ; (Qg/Qo)= 0.0055 ;
   - D. Do= 10 ; Du= 10 ; B= 21 ; (Du/Do)= 1.0 ; (B/Do)= 2.1 ;
   - E. MAINLINE OUTLET STRUCTURE: 2 - 10'X10'
   - F. MAINLINE UPSTREAM STRUCTURE: 2 - 10'X10'

2. GENERAL INSTRUCTION No. 1:
   - A. DOWNSTREAM HG= 1214.83
   - B. BOX REACH LENGTH= 1015.0
   - C. BOX Sf= 0.00314
   - D. Hf = Sf * L = .00314 (1015.0) = 3.19
   - E. PRESSURE LINE EL. AT D.S. SIDE OF JUNCTION STRUCTURE = 1214.83 + 3.19 = 1218.02

3. GENERAL INSTRUCTION No. 2:
   - A. Vo= 20.40 ; Ho= 6.46 ; [Ho=(Vo^2)/2g]

4. GENERAL INSTRUCTION No. 4:
   - A. STRUCTURE INVERT ELEVATION = 1217.78
   - B. PRESSURE DEPTH = 1218.02 - 1217.78 = 0.24 ; ((2.E) – (4.A)) Open Channel Flow
   - C. K(Vo^2/2g) WHERE K=3(Qg/Qo)= _____ ; (PER SECTION 4.2.4.3 No. 4 – pg 4–23)
   - C. d = _____ ; ((4.B) + (4.C))
   - D. d/Do = _____ = _____ ; USE _____

NOTE:

FOR Qg/Qo = _____ Du/Do = _____

Ku = _____

FOR d/Do = _____ ; INCREMENT OF Ku = _____

THEREFORE Ku = Ku + Ku INCREMENT = _____

Hu = Ku(Vo^2/2g) = _____

TRIAL UPSTREAM OF JUNCTION STRUCTURE PRESSURE LINE EL = _____ ((2.E) + (Hu))

CHECK d:

- A. (UPSTREAM PRESSURE LINE EL.) – (STRUCTURE INVERT EL.) = _____ ; USE = _____
- B. d/Do = _____ = _____ ; USE _____

DOES d/Do = THE ASSUMED d/Do _____

- A. IF YES STOP
- B. IF NO COMPUTE THE CORRECT d/Do AND READ INCREMENT OF Ku AND PROCEED:
  1. d/Do = _____ ; (FROM CHECK d ABOVE)
  2. INCREMENT OF Ku = _____ ; (FROM UPPER GRAPH OF FIGURE 4.9)
  3. CORRECT Ku = Ku + INCREMENT OF Ku = _____

Hu = Ku(Vo^2/2g) = _____

HGu = (UPSTREAM PRESSURE LINE) + (Hu) = _____

FILE: DF-INLET.WK3

GREINER, INC.

To use Figure 4.8:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate the velocity head in the outfall (Gen. Instr. 2).
3. Calculate the size ratios D_U/D_O and A/D_U (Gen. Instr. 3).
4. Note whether the outfall pipe entrance is to be square-edged or rounded smooth (note Gen. Instr. 6).
5. Enter Figure 4.8 at the pipe size ratio D_U/D_O and read K_U at the curve for the proper value of A/D_U for a square-edged entrance condition, or at the dashed curve for a rounded entrance.
6. Calculate h_U (positive or negative) as indicated on the diagrams on Figure 4.8 and by Gen. Instr. 7.
7. Add a positive h_U to (or subtract the absolute value of a negative h_U from) the elevation of the outfall pressure at the junction center to obtain the elevation of the upstream pipe pressure line at the same location.
8. The water surface elevation in the junction corresponds to that of the upstream pipe, whether above or below the outfall pressure line.
9. Check to be sure the water surface elevation in the junction is at least 6 inches below the top of the junction box so that overflow may not occur.

Comments: For a square-edged entrance to the outfall pipe, values of A/D_U less than 1 do not appreciably reduce the values of K_U shown for $A/D_U = 1$. K_U increases for distances A/D_U greater than 3, but such values are not usual in storm drain construction. For rounded entrances, the curve shown will apply with sufficient accuracy for all values of A/D_U up to 3.

4.2.4.3 Rectangular Manhole—Through Pipeline—with Inlet Flow: Coefficients for pressure changes are presented in Figure 4.9 for use in determining the common elevation of the upstream in-line pipe pressure line and the water surface in the manhole. The in-line pipes connect at the manhole sides (long dimension) and must meet the alignment requirement stated for Figure 4.9. As much as half the total flow may enter through a top inlet. The main graph of Figure 4.9 includes effects of various portions of grate flow for a relative water depth d/D_O of 2.5. Increments of K_U for other relative water depths are shown in the supplemental graphs; positive increments for d/D_O less than 2.5 and negative for greater depths.

To use Figure 4.9:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate velocity head in the outfall (Gen. Instr. 2).
3. Calculate the ratios of D_U/D_O and Q_U/Q_O (Gen. Instr. 3). (The inlet flow ratio $Q_G/Q_O = 1 - Q_U/Q_O$).
4. Estimate a value for the water depth, d.
   - » Follow Gen. Instr. 4.
   - » Estimate $K = 3\ Q_G/Q_O$.
5. Calculate the corresponding relative water depth d/D_O.

6. If the estimated d/D_O is approximately 2.5, enter the lower graph on Figure 4.9 at the pipe size ratio D_U/D_O and read K_U at the curve of interpolated curve for Q_U/Q_O: then proceed as in Step 9.
7. If the estimated d/D_O is other than 2.5, follow Step 6, then enter the upper graph on Figure 4.9 at the given D_U/D_O and determine the increment of K_U required to account for the effects of the estimated relative water depth d/D_U.
8. Add K_U from Step 6 and the increment from Step 7 to determine the total value of K_U. Note that negative values of K_U may occur.
9. For a rounded outfall pipe entrance or one consisting of a pipe socket, reduce K_U according to Gen. Instr. 6.
10. Calculate h_U as indicated on the diagram on the Figure and by Gen. Instr. 7.
11. Add h_U to the elevation of the outfall pressure line at the inlet center to obtain a more precise value for the water depth, d.
12. Repeat the above procedure with the improved value of d from Step 11 if necessary. Such repetition may not be necessary if the original estimated d/D_O of Step 5 was reasonably accurate.
13. Check to be sure the water elevation is at least 6 inches below the gutter elevation at the inlet so that inflow may be admitted.

4.2.4.4 Rectangular Manhole—Upstream Main and 90 Degree Lateral Pipe—with or without Inlet Flow: Pressure change coefficients are presented in Figure 4.10 for use in determining the common elevation of the two upstream pipe pressure lines and the water surface in the manhole. Flow into the combination inlet and junction box is supplied by an upstream main, in-line with the outfall and flowing through the short dimension of the manhole, and a 90 degree lateral pipe connected at one end of the box, supplemented by flow through a top inlet. The main graph of Figure 4.10 applies directly for no flow into the manhole through the inlet. Increments of K_U and K_L for inlet flow conditions are shown in the supplementary graphs of the upper portion of the figure.

To use Figure 4.10:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate the velocity head in the outfall (Gen. Instr. 2).
3. Calculate the ratios D_U/D_O, Q_U/Q_O (Gen. Instr. 3).
4. If no inlet flow is involved, enter the lower graph on Figure 4.10 at the pipe size ratio D_U/D_O and read K_U (or K_L) at the curve or interpolated curve for Q_U/Q_O, then proceed as in Step 10.
5. With inlet flow, estimate a value for the water depth, d.
   - » Follow Gen. Instr. 4
   - » Estimate K = 1.5
6. Calculate the corresponding relative water depth d/D_O.
7. Enter the lower graph and obtain K_U (or K_L) as in Step 4, this value applying for $Q_G/Q_O = 0$.

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Job No. E002102 Project Old Cross Cut Canal Sheet 1 of

Subject Indian School Rd. N. ; Catch Basin Grate Design

By RHF Date 6-25-94 Checked By N.W. Date 7-24-94

Indian School Rd. : Box Sta 116+83, 14' Rt = Road Sta. 306+11.5, 37.5' Rt.

Purpose: Design an inlet to intercept the flow in the north 1/2 of Indian School Road.

Procedure: Install a MAG Std. Dtl. 206 Scupper to intercept 28 cfs. Collect the flow in a channel on the back side of the scupper and direct it to 2 - C.O.P P1565 Type-1 grates with a modified Adot Apron. The inlet discharges into the top of the box.

Grate El. = 1241.00

2 - Grates w/ 2.5' of head Orifice Flow

- Refer to the Hydraulics for Inlets section for general storm pressure analysis.

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of

Subject Flow Split Analysis

By RHF Date 6-24-94 Checked By N.W. Date 7-26-94

Indian School Rd.

Purpose: Determine Discharge flowing in the north and South ½ roadway sections for catch basin design.

Procedure: Use the flow split used in the 30% submittal for catch basin design to determine a flow split percentage and apply it to the local storm discharge. Once the north and south discharges have been computed, determine that the section that should convey the larger Q has the capacity for that Q.

From 30% Submittal:

$Q_{Total} = 19 cfs$

$Q_{north} = 13 cfs$

% of Q in North Section = 68.42%.

For 60% Submittal:

$Q_{Local} = 41 cfs$

$Q_{north} = 0.6842 (Q_{Local}) = 28 cfs$

$Q_{south} = Q_{Local} - Q_{north} = 41 cfs - 28 cfs = 13 cfs.$

Note: North ½ Roadway Can Convey 25 cfs. However use 28 as OK.

PROJECT: OCCC
PROJECT NO.: E002102

GROSS AREA = 6.67 ft^2
NET AREA = 4.25 ft^2
N/G = 0.64

Street Name: Indian Sch. N. Q= 28 cfs Local Storm
CLOG FACTOR= 2

WIER:
Q=3*L*H^1.5

H	LENGTH	GRATES	WITH CLOGGING
1	9.3	3	4
1.5	5.1	2	2
2	3.3	1	1
2.5	2.4	1	1
3	1.8	1	1
3.5	1.4	0	1

ORFICE:
Q=0.67A*(SQRT(2GH))

H	AREA	GRATES	WITH CLOGGING
1	5.2	1	2
1.5	4.3	1	2
2	3.7	1	1
2.5	3.3	1	1
3	3.0	1	1
3.5	2.8	1	1

FRONTAL FLOW INTERCEPTION:
L=(V/2)*((D+Db)^.5)

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Job No. E002102 Project Old Cross Cut Canal Sheet 1 of

Subject Single Grate Design in the Invert of the CHIMP

By RHF Date 7-21-94 Checked By N.W Date 7-22-94

Local Storm Conditions:

Purpose: • Determine the amount of flow for the local storm in excess of the CHIMP Capacity. Using Orifice and Weir equations determine the number of grates required to intercept this flow.

• Determine the amount of flow for the general storm in excess of the CHIMP Capacity, Using Pressure Flow procedures, determine grate number and coefficient of flow for the STORM Program.

*: See See" 78D file.

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Job No. E002102 Project Old Cross Cut Canal Sheet 1 of ____

Subject Single Inlet Design: General vs Local Qs & grates/location

By RHF Date 7-10-94 Checked By N. W. Date 7-22-94

Street	General Q	General No. of grates	Local Q	Local No. of Grates	Provide
Granada	~~0~~	~~0~~	~~0~~	~~0~~	~~0~~
Holly	~~0~~	~~0~~	72	3	4
S. Cemetery	~~0~~	~~0~~	159	6	6
Oak	~~0~~	~~0~~	~~0~~	~~0~~	~~0~~
N. Cemetery	~~0~~	~~0~~	129	5	6
Virginia	~~0~~	~~0~~	~~0~~	~~0~~	~~0~~
Cambridge	~~0~~	~~0~~	~~0~~	~~0~~	~~0~~
Windsor	60	5 *	108	8	8

* Pressure flow analysis using FCDMC Procedure for the inlet design & verified 5 grates will intercept the flow.

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Job No. E002102 Project Old Cross Cut Canal Sheet 1 of

Subject Small Inlets: Local STORM

By RHF Date 7-10-94 Checked By N. W Date 7-22-94

Sta 28+17 ~~not shown on plans~~ OK

STREET: Holly Sta. 27+17 Local Q = 142 cfs

Purpose: Determine the number of Grates required to intercept 72 cfs which is the flow in excess of the CHIMP Capacity for Local Storm.

Procedure: Box not flowing full. CHIMP full. use difference in elevation for CHIMP CWSEL to grate elevation from HEC-2 to determine Head over grate. use weir & orifice equations to determine total number of C.O.P. Type-1 P1565 grates to intercept the flow. Use a 50% clogging factor.

Revised 8/24

CHIMP CWSEL = ~~1203.26~~ 3.36 CHIMP Inv. El. = 1199.43

Grate El. = 1199.10 99.60

Height over grate: ~~4.16~~ 3.76 ft. ← (1203.26 − 1199.10)

Orifice Flow

$Q = .67 A \sqrt{2gH}$

solve for A

$A = \frac{Q}{.67\sqrt{2gH}}$

$A = 6.6\ ft^2$ = 6.9

use 2 grates

Weir Flow

$Q = 3.0 L h^{3/2}$

Solve for L

$L = \frac{Q}{3.0(h)^{3/2}}$

L = ~~2.83'~~ 3.3'

use 1 grate

Orifice Flow controls.

With Clogging use 3 grates.

Note: 1 - C.O.P. Type-1 Grate has net area = 4.25 ft² & 40" long.

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Job No. E002102 Project Old Cross Cut Canal Sheet 1 of

Subject Small Inlets: Local Storm

By RHF Date 7-10-94 Checked By N.W Date 7-22-94

34+00

Street: S. Cemetery Drive Sta. 33+57 Local Q = 229 cfs

Purpose: Determine the number of Grates required to intercept 159 cfs which is the flow in excess of the CHIMP Capacity for the local storm.

Procedure: Box not flowing full. CHIMP Full. Use difference in elevation for CHIMP CWSEL to Grate elevation from HEC-2 to determine head over Grate. Use Weir & orifice equations to determine total number of C.O.P. Type-1 P1565 grates to intercept the flow. Use a 50% Clogging Factor.

Revised 8/24/94

1204.65

CHIMP CWSEL = 1204.54 CHIMP Inv. El. = 1201.21

Grate El = CHIMP Inv. El. - 4" = 1200.88 1201.08

Height over grate = 3.66 ft. 3.57'

Orifice Flow

$Q = .67 A \sqrt{2gH}$

Solve for area

$A = \frac{Q}{.67\sqrt{2gH}}$

A = 15.455 15.65

Use 4 grates

Weir Flow

$Q = 3.0 L h^{3/2}$

Solve for Length of weir.

$L = \frac{Q}{3 h^{3/2}}$

L = 7.57 ft. 7.86

Use 3 grates

Orifice flow controls.

With Clogging use 6 grates

Note: 1 - C.O.P. Type-1 P1565 grate has 4.25 ft² and is 40" long.

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Job No. E002102 Project Old Cross Cut Canal Sheet 1 of

Subject Small Inlets : Local Storm

By RHF Date 7-10-94 Checked By N.W. Date 7-22-94

41+10

Street : N. Cemetary Dr. Sta. 40+57 Local Q = 239 cfs

Purpose : Determine the number of grates required to intercept 129 cfs which is the flow in excess of the CHIMP capacity for the local storm.

Procedure: Box not flowing full. CHIMP full. Use difference in elevation for CHIMP CWSEL to grate elevation from HEC-2 to determine head over grate. Use weir and orifice equations to determine total number of C.O.P. Type-1 P1565 grates to intercept the flow. Use a 50% clogging factor.

Revised 8/24

CHIMP CWSEL = 1206.06 1206.14 CHIMP Inv. El. = 1202.91

Grate El. = CHIMP Inv. El. - 4" = 1202.58 1202.78'

Height over Grate = 3.48 ft. 3.36'

Orifice Flow

$Q = .67 A \sqrt{2gH}$

$A = \frac{Q}{.67\sqrt{2gH}}$

A = 12.8612 ft² 13.09 ft²

Use 3 grates

Weir Flow

$Q = 3Lh^{3/2}$

$L = \frac{Q}{3h^{3/2}}$

L = 6.6237 ft 6.98

Use 2 grates

Orifice flow controls

With clogging use 5 grates

Note: 1 - C.O.P. Type-1 P1565 grate has net area of 4.25 ft² and is 40" long.

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Job No. E002102 Project Old Cross Cut Canal Sheet 1 of

Subject Small Inlets : Local Storm

By RHF Date 7-10-94 Checked By N.W. Date 7-22-94

57+50

Street : Windsor Sta. 56+75.32 Local Q = 108 cfs

Purpose : Determine the number of grates required to intercept 108 cfs which is the flow in excess of the CHIMP capacity for the local storm.

Procedure : Using the orifice and weir equations to determine the number of C.O.P. Type 1 P1565 grates to intercept the flow - CHIMP full.

57+00 208.25
58+00 208.82

1208.29'

CHIMP CWSEL = 1208.24 CHIMP Inv. El. = 1206.75

Grate El. = CHIMP Inv. El. - 4" = 1206.42 1206.72

Height over grate = 1.82 ft.

1.57'

Revised 8/24/94

Orifice Flow	Weir Flow
$Q = .67 A \sqrt{2gH}$	$Q = 3Lh^{3/2}$
$A = \frac{Q}{.67\sqrt{2gH}}$	$L = \frac{Q}{3h^{3/2}}$
A = 14.8892 ft.² 16.03 ft²	L = 14.6621 ft. 18.3
Use 4 grates	Use 5 grates 5.5 Use 6 grates

Weir Controls

With clogging use 8 grates.

OK

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of 9

Subject Small Inlets: General Storm Grate Design

By RHF Date 7-10-94 Checked By N. W. Date 7-22-94

Street: Windsor Sta. 56+75.32 General Q = 60 cfs

Purpose: Determine the number of Grates required to Intercept 60 cfs which is the flow in excess of the CHIMP capacity for the local storm.

Procedure: Using the orifice and weir equations to determine the number of C.O.P. Type-1 P1565 grates to intercept the flow - CHIMP Full.

Revised 8/24/94

CHIMP CWSEL = 1208.24 CHIMP Inv. El. = 1206.75

Grate El = CHIMP Inv. El. - 4" = 1206.42

Height over grate = 1.82 ft.

Orifice Flow

$Q = .67 A \sqrt{2gH}$

$A = \frac{Q}{.67\sqrt{2gH}}$

$A = 8.27\ ft^2$

use 2 grates

Weir Flow

$Q = 3Lh^{3/2}$

$L = 40''$ (one grate)

Q/grate = 24 cfs

Use 3 grates

Weir Controls

With Clogging use 5 grates

Refer to the Hydraulics for Inlets section for pressure analysis.

OK We have 8 grates

PROJECT: Old Cross Cut Canal JOB NUMBER: E002102 DATE: 7-18-94

DISCRIPTION: Individual Drop Inlets at Windsor

COMPUTED BY: RHF CHECKED BY: N.W. DATE CHECKED: 7-12-94 SHT 2 OF 9

FCDMC – FIGURE 4–9

INLET STATION: 57+90 DOWNSTREAM HG: 1206.18 (sta. 55+17)

1. DESIGN CONDITION:
   - A. INLET LOCATED OVER THE BOX CULVERT.
   - B. USE FCDMC VOLUME II – HYDRAULICS PP 77 &78; SECTION 4.2.4.3; FIGURE 4.9.
   - C. Qo= 3910 ; Qu= 3850 ; Qg= 60 ; (Qu/Qo)= 0.9847 ; (Qg/Qo)= 0.0153 ;
   - D. Do= 10' ; Du= 10' ; B= 37 ; (Du/Do)= 1.0 ; (B/Do)= 3.7 ;
   - E. MAINLINE OUTLET STRUCTURE: 2-18x10
   - F. MAINLINE UPSTREAM STRUCTURE: 2-18x10

2. GENERAL INSTRUCTION No. 1:
   - A. DOWNSTREAM HG= 1206.18
   - B. BOX REACH LENGTH= 158
   - C. BOX Sf= 0.001903
   - D. Hf = Sf * L = (158) * (.001903) = 0.3
   - E. PRESSURE LINE EL. AT D.S. SIDE OF JUNCTION STRUCTURE = 1206.18 + 0.3 = 1206.48

3. GENERAL INSTRUCTION No. 2:
   - A. Vo= 10.86 ; Ho= 1.83 ; [Ho=(Vo^2)/2g]

4. GENERAL INSTRUCTION No. 4:
   - A. STRUCTURE INVERT ELEVATION = 1191.02
   - B. PRESSURE DEPTH = 1206.48 - 1191.02 = 15.46 ; ((2.E) – (4.A))
   - C. K(Vo^2/2g) WHERE K=3(Qg/Qo)= .08 ; (PER SECTION 4.2.4.3 No. 4 – pg 4–23)
   - C. d = 15.46 + 0.08 = 15.54 ; ((4.B) + (4.C))
   - D. d/Do = 15.54/10 = 1.55 ; USE 1.5

NOTE:

FOR Qg/Qo = 0.02 Du/Do = 1.0
Ku = 0.12
FOR d/Do = 1.5 ; INCREMENT OF Ku = 0.06
THEREFORE Ku = Ku + Ku INCREMENT = 0.12 + 0.06 = 0.18
Hu = Ku(Vo^2/2g) = 0.18 (1.83) = 0.33
TRIAL UPSTREAM OF JUNCTION STRUCTURE PRESSURE LINE EL. = 1206.48 + 0.33 = 1206.81 ((2.E) + (Hu))

CHECK d:

- A. (UPSTREAM PRESSURE LINE EL.) – (STRUCTURE INVERT EL.) = 1206.81 - 1191.02 = 15.79; USE = 15.8
- B. d/Do = 15.8 /10 = 1.58 ; USE 1.5

DOES d/Do = THE ASSUMED d/Do Yes

- A. IF YES STOP
- B. IF NO COMPUTE THE CORRECT d/Do AND READ INCREMENT OF Ku AND PROCEED:
  1. d/Do = _____ ; (FROM CHECK d ABOVE)
  2. INCREMENT OF Ku = _____ ; (FROM UPPER GRAPH OF FIGURE 4.9)
  3. CORRECT Ku = Ku + INCREMENT OF Ku = _____

Hu = Ku(Vo^2/2g) = 0.18(1.83) = 0.33

HGu = (UPSTREAM PRESSURE LINE) + (Hu) = 1206.48 + 0.33 = 1206.81

FILE: DF-INLET.WK3

Ku = 0.18 put into STORM Pgm for minor Loss.

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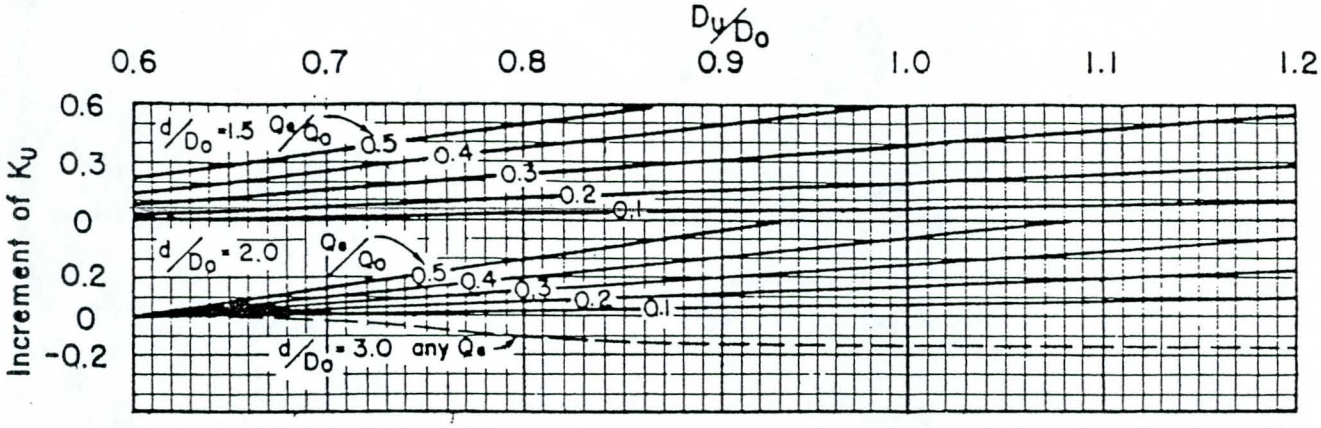
sht. 3 of 9



Supplementary Chart for Modification of K_u for Depth in Inlet other than 2.5 D_o

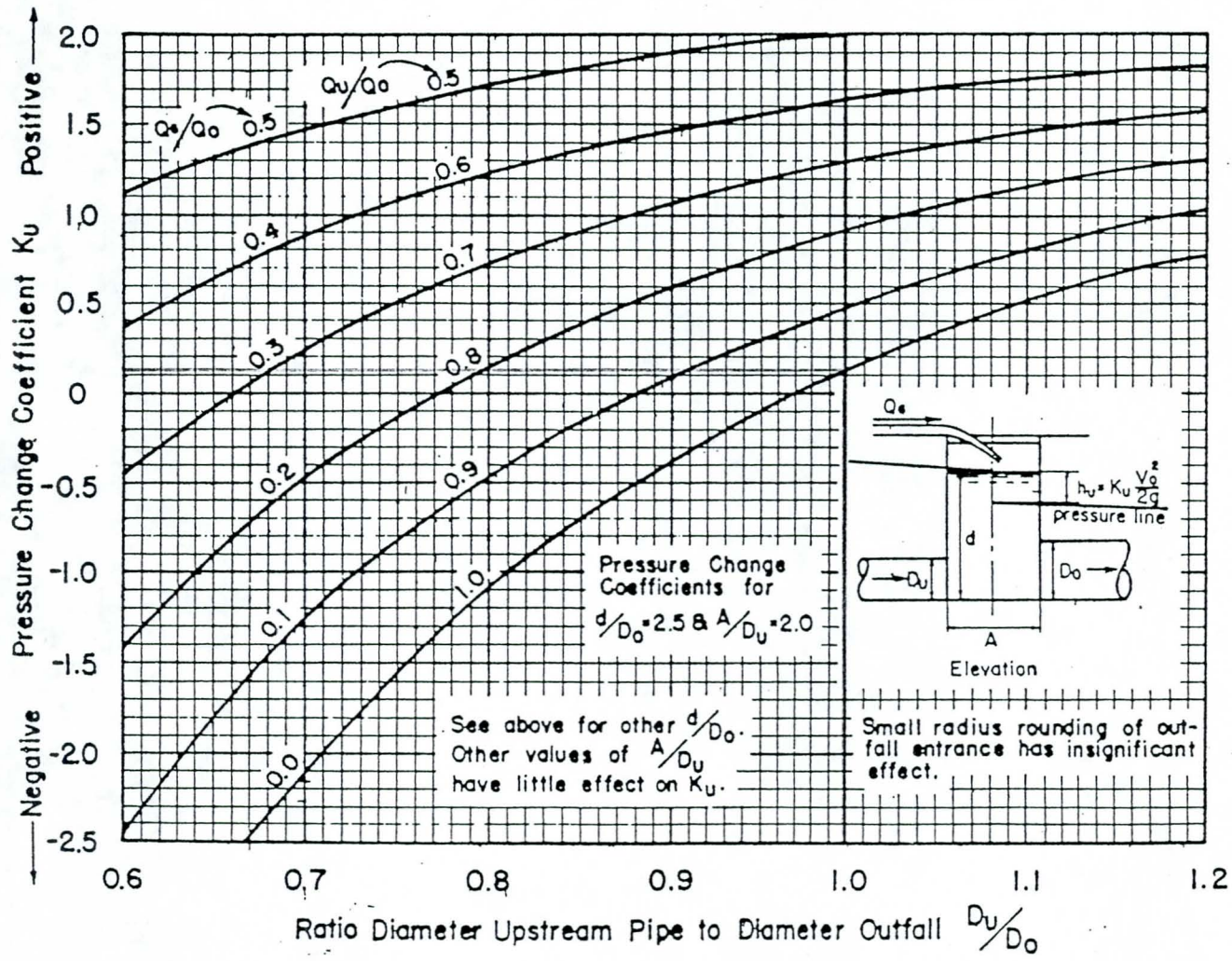


Figure 4.9
Rectangular Manhole with Through Pipeline and Inlet Flow

To use Figure 4.8:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate the velocity head in the outfall (Gen. Instr. 2).
3. Calculate the size ratios D_U/D_O and A/D_U (Gen. Instr. 3).
4. Note whether the outfall pipe entrance is to be square-edged or rounded smooth (note Gen. Instr. 6).
5. Enter Figure 4.8 at the pipe size ratio D_U/D_O and read K_U at the curve for the proper value of A/D_U for a square-edged entrance condition, or at the dashed curve for a rounded entrance.
6. Calculate h_U (positive or negative) as indicated on the diagrams on Figure 4.8 and by Gen. Instr. 7.
7. Add a positive h_U to (or subtract the absolute value of a negative h_U from) the elevation of the outfall pressure at the junction center to obtain the elevation of the upstream pipe pressure line at the same location.
8. The water surface elevation in the junction corresponds to that of the upstream pipe, whether above or below the outfall pressure line.
9. Check to be sure the water surface elevation in the junction is at least 6 inches below the top of the junction box so that overflow may not occur.

Comments: For a square-edged entrance to the outfall pipe, values of A/D_U less than 1 do not appreciably reduce the values of K_U shown for $A/D_U = 1$. K_U increases for distances A/D_U greater than 3, but such values are not usual in storm drain construction. For rounded entrances, the curve shown will apply with sufficient accuracy for all values of A/D_U up to 3.

4.2.4.3 Rectangular Manhole—Through Pipeline—with Inlet Flow: Coefficients for pressure changes are presented in Figure 4.9 for use in determining the common elevation of the upstream in-line pipe pressure line and the water surface in the manhole. The in-line pipes connect at the manhole sides (long dimension) and must meet the alignment requirement stated for Figure 4.9. As much as half the total flow may enter through a top inlet. The main graph of Figure 4.9 includes effects of various portions of grate flow for a relative water depth d/D_O of 2.5. Increments of K_U for other relative water depths are shown in the supplemental graphs; positive increments for d/D_O less than 2.5 and negative for greater depths.

To use Figure 4.9:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate velocity head in the outfall (Gen. Instr. 2).
3. Calculate the ratios of D_U/D_O and Q_U/Q_O (Gen. Instr. 3). (The inlet flow ratio $Q_G/Q_O = 1 - Q_U/Q_O$).
4. Estimate a value for the water depth, d.
   - » Follow Gen. Instr. 4.
   - » Estimate $K = 3\ Q_G/Q_O$.
5. Calculate the corresponding relative water depth d/D_O.

6. If the estimated d/D_O is approximately 2.5, enter the lower graph on Figure 4.9 at the pipe size ratio D_U/D_O and read K_U at the curve of interpolated curve for Q_U/Q_O: then proceed as in Step 9.
7. If the estimated d/D_O is other than 2.5, follow Step 6, then enter the upper graph on Figure 4.9 at the given D_U/D_O and determine the increment of K_U required to account for the effects of the estimated relative water depth d/D_U.
8. Add K_U from Step 6 and the increment from Step 7 to determine the total value of K_U. Note that negative values of K_U may occur.
9. For a rounded outfall pipe entrance or one consisting of a pipe socket, reduce K_U according to Gen. Instr. 6.
10. Calculate h_U as indicated on the diagram on the Figure and by Gen. Instr. 7.
11. Add h_U to the elevation of the outfall pressure line at the inlet center to obtain a more precise value for the water depth, d.
12. Repeat the above procedure with the improved value of d from Step 11 if necessary. Such repetition may not be necessary if the original estimated d/D_O of Step 5 was reasonably accurate.
13. Check to be sure the water elevation is at least 6 inches below the gutter elevation at the inlet so that inflow may be admitted.

4.2.4.4 Rectangular Manhole—Upstream Main and 90 Degree Lateral Pipe—with or without Inlet Flow: Pressure change coefficients are presented in Figure 4.10 for use in determining the common elevation of the two upstream pipe pressure lines and the water surface in the manhole. Flow into the combination inlet and junction box is supplied by an upstream main, in-line with the outfall and flowing through the short dimension of the manhole, and a 90 degree lateral pipe connected at one end of the box, supplemented by flow through a top inlet. The main graph of Figure 4.10 applies directly for no flow into the manhole through the inlet. Increments of K_U and K_L for inlet flow conditions are shown in the supplementary graphs of the upper portion of the figure.

To use Figure 4.10:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate the velocity head in the outfall (Gen. Instr. 2).
3. Calculate the ratios D_U/D_O, Q_U/Q_O (Gen. Instr. 3).
4. If no inlet flow is involved, enter the lower graph on Figure 4.10 at the pipe size ratio D_U/D_O and read K_U (or K_L) at the curve or interpolated curve for Q_U/Q_O, then proceed as in Step 10.
5. With inlet flow, estimate a value for the water depth, d.
   - » Follow Gen. Instr. 4
   - » Estimate K = 1.5
6. Calculate the corresponding relative water depth d/D_O.
7. Enter the lower graph and obtain K_U (or K_L) as in Step 4, this value applying for $Q_G/Q_O = 0$.

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The line diagrams of Figure 4.5 illustrate the pipe positions and the function of each as supply or outfall for each type of inlet and junction involved. A detail plan is included to show junction and pipe diameter dimensions used. These dimensions may be in inches, feet, or any other unit of linear measurement since they are used in the design figures only as ratios of one to another. The figures included in Section 4.2.4 are based on tests of round pipe, and apply to pipes of circular cross section. However, the figures will apply accurately enough to pipes of any cross section.

One of the diagrams on Figure 4.5 shows a through main at a junction of a 90 degree lateral, with pressure lines and total head lines superimposed. It will be noted that the relative elevations of the various total head lines are not dealt with in this sketch.

The same diagram shows the hydraulic grade lines projected to a point above the "branch point," this being the location in plan of the intersection of the outfall pipe and lateral pipe centerlines. A similar point of reference, the center of the junction box, is used for the upstream in-line pipe and its hydraulic grade line where no lateral is present. The change of pressure at a junction is measured by the difference in elevation between the outfall hydraulic grade line and an upstream hydraulic grade line, along the vertical line through the branch point. The vertical dimensions h_U and h_L indicate the change of pressure for the upstream in-line and lateral pipes, respectively. The adjacent equations on Figure 4.5 state how each is calculated.

There will be situations where flow conditions are not represented in the design figures. In this instance, it is generally acceptable to extrapolate. It is rare to have required extrapolations that are significant.

In those rare instances where more than three pipelines enter the same manhole, it may be necessary to make simplifying assumptions in regard to the flow conditions and utilize the appropriate figure representing these assumptions. This approach is not considered a serious constraint as there are many similarities between the graphics for varying flow conditions. Table 4.2 lists the recommended graphs and assumptions.

At all junctions where a change of pressure occurs, a loss of total head must occur whether the pressure change is positive or negative. This basic fact may be used to check pressure results.

General Instructions for Use of Design Figures: Several operations are common to use of the design figures for various types of junctions. Instructions for performing these recurring procedures are consolidated in the following General Instructions. In the detailed instructions for use of the individual figures, references to these General Instructions are made by number (Gen. Instr. 1, etc.). The general instructions are as follows:

1. Determine and tabulate the elevation of the outfall pipe pressure line at the branch point or catch basin center (refer to Figure 4.5). This elevation is obtained by adding the pipe friction loss to the elevation of the pressure line at the preceding structure downstream. The friction loss, h_f, is given by $S_f \times L$, where S_f is the friction slope in a pipe flowing full.

$$S_f = 2.86n^2V_O^2/D_O^{4/3}$$

Do = Pipe diameter only not Box height

However $D = \frac{R_H}{4}$ ±

2. Calculate the mean velocity head of the flow in the outfall pipe.

$$V_O^2/2g$$

3. Calculate the required flow rate and size ratios. Examples:

Q_U/Q_O, Q_L/Q_U, Q_G/Q_O etc.

D_U/D_O, D_L/D_O, B/D_O etc.

4. Estimate the depth of water, d, in a manhole with flow into the manhole from a top inlet, either along or combining with flow from an upstream pipe.

d = total depth of water, ft

h = (outfall pressure line elevation) – (catch basin bottom elevation) + (K) $V_O^2/2g$

Table 4.2
Summary of Design Figure/Manhole Configuration Application

Case	Design Figure
Catch Basin With Inlet Flow Only	4.7
Flow Straight Through Any Manhole	4.8
Rectangular Manhole, Through Pipe and Inlet Flow	4.9
Rectangular Manhole With In-Line Upstream Main and 90° Lateral Pipe (with or without inlet flow)	4.10
Rectangular Manhole With In-Line Opposed Lateral Pipes Each at 90° to Outfall (with or without inlet flow)	4.11
Rectangular Manhole With Offset Opposed Lateral Pipes each at 90° to Outfall (with or without inlet flow)	4.12
Square Manhole at 90° Deflection	4.13
Round Manhole at 90° Deflection	4.13
Deflectors in Square or Round Manholes at 90° Deflection	4.13
Square Manhole on Through Pipeline at Junction of a 90° Lateral Pipe (large size laterals: $D_L/D_O > 0.6$)	4.13, 4. 14
Round Manhole on Through Pipeline at Junction of a 90° Lateral Pipe (large size lateral: $D_L/D_O > 0.6$)	4.13, 4.14
Deflectors in Square or Round Manholes on Through Pipelines at Junction of a 90° Lateral Pipe (large size laterals: $D_L/D_O > 0.6$)	4.13, 4.14
Square or Round Manhole on Through Pipeline at Junction of a 90° Lateral Pipe (smaller size laterals: $D_L/D_O < 0.6$) or laterals with no manhole.	4.15
Sewer Bends with or without Manhole	4.16
Square or Round Manhole on Through Pipeline with Two Laterals ($D_L/D_O > 0.5$) or ($Q_U/Q_O > 0.3$), Consider Upstream Pipe as Grate Flows	4.10
Square or Round Manhole on Through Pipeline with Two Laterals (($D_L/D_O < 0.5$) or ($Q_U/Q_O < 0.3$), Consider Upstream Pipe as Grate Flows	4.11, 4.12

sheet 8 of 9

K = the pressure change coefficient for the catch basin water depth (This is estimated as detailed for each type of manhole. Such estimates are not necessary for manholes with in-line or offset opposed laterals.)

5. Use the coefficients K from the figures for manholes with square-edged entrance to the outfall pipe (entrance flush with box side, with square edges).

6. Use reduced coefficients K, where applicable, for a rounded entrance to the outfall pipe (rounded on 1/4 circle arc of approximate radius 1/8 D_O) or for an entrance formed by the socket end of a standard tongue-and-groove concrete pipe.

Figure 4.7:	Insignificant effect, make no reduction.
Figure 4.8:	Read directly from the figure.
Figure 4.9:	Reduce K_U by 0.1 for usual proportions of inlet flow; by 0.2 for Q_G about 0.5Q.
Figure 4.10:	Reduce K_U and K_L in same manner as Figure 4.9.
Figure 4.11:	Insignificant effect, make no reduction.
Figure 4.12:	Insignificant effect, make no reduction.
Figure 4.13, 4.14, 4.15:	See specific instructions for each case.
Figure 4.16:	Make no reductions.

7. Calculate pressure change. To calculate the change of pressure at a manhole, working upstream from the outfall pipe to an upstream pipe, the design figure applying to the type of junction involved is selected. The pressure change coefficient for a specific upstream pipe is read from the figure for the particular flow rate and size ratios already calculated. The pressure change is calculated:

$$h = K \times V_O^2 / 2g \qquad (4.1)$$

The coefficient is a dimensionless number, and therefore, the change of pressure will be in feet.

8. Apply the pressure change. The pressure change, in feet, for each upstream pipe is added to the outfall pipe pressure line elevation at the branch point to obtain the elevation of each pressure line for further calculations upstream along the pipe. In some cases, the upstream pressure line at the branch point will be at a lower elevation than the downstream pressure line. Where this less common situation may occur with a particular type of junction, it is mentioned in the instructions for use of the specific figure.

9. Determine the elevation of the water surface. The elevation of the water surface in a manhole (with or without catch basin flow) receiving flow from a pipe or pipes will correspond to that of the upstream in-line pipe pressure line. At a junction with offset opposed laterals, the water surface will correspond to the elevation of the far lateral pipe pressure line. At a junction with in-line opposed laterals, the water surface will correspond to the elevation of the pressure line of the higher-velocity lateral pipe.

   Verify that the water surface is above the crown elevation of all pipe connections to the structures that are being analyzed. Small pipes, such as laterals to catch basins, which carry a small portion of the total flow, may reasonably be constructed to affect a manhole in the same way as catch basin flow from the ground surface.

4.2.3.3 Design Procedure for Open Channel Flow: The hydraulic grade line (HGL) in an upstream pipe that is in line with the outlet will seek normal depth when the slope of the pipe is greater than the slope required for full flow. Should the slope of the pipe be less than that required for full flow, the HGL will be at an elevation greater than the crown of the pipe. Drawdown effects will be observed near the outfall from the pipe. In this case, the depth will pass through critical depth at or near the point of outfall. Backwater or drawdown calculations for large diameter pipes should be made along the length of the pipe to determine whether normal depth or pressure flow is attained before the next manhole.

for open channel flow only

For the size of pipes normally encountered in storm drain design, it is reasonable to assume a straight water surface. It is also assumed that the energy grade line is parallel to the pipe grade, and that any losses other than pipe friction may be accounted for by assuming point losses at each manhole.

The basic approach to design of open channel flow in storm drains should be to calculate the energy grade line along the system. Once the discharge has been determined and a pipe size and slope assumed for a given section, the d/D and v/V_{full} ratios can be determined from a graph of Hydraulic Elements of Circular Conduits (Figure 4.6).

The next step is to calculate the energy grade line:

$$H = Z + d + (v^2/2g) \tag{4.2}$$

At each manhole the energy grade line of all pipes should coincide, allowing for reasonable values of head loss to the junction.

The usual method of stating head losses at manholes is in terms of a constant K times the velocity head of the conduit in question,

$$h_e = (K)\, v^2/2g \tag{4.3}$$

STORM OUTPUT FILE
7SD594.DAT & MAP

For General Storm.

PROJECT NAME: OLD CROSS CUT CANAL

PROJECT NUMBER: FCDMC 94-24; GREINER E002102

RE: Modification To STORM Program Results Due To Inappropriate Internal Loss Calculation for Velocity Changes.

This section contains the output from the STORM Program and a correction table. It was determined that the STORM Program was computing additional losses when the velocity changed without an appropriate reason. The table repeats the information from the STORM Program output along with a energy computation. It also computes the correct energy and hydraulic grade elevations at all locations. The table corrects the problems and the results from the table were used in all calculations.

Refer to the following table and specifically the line for Cambridge which is the first location that this occurs. The STORM Program computes an hydraulic grade elevation (HG2) for the line for Virginia of 1204.74 and velocity head of 2.09'. The energy grade elevation (EG2) for the line for Virginia is computed as 1206.83. The velocity for the line for Cambridge at V1 is 10.9 fps and the STORM Program computes the hydraulic grade elevation (HG1) at Cambridge to be 1205.25. However, this would cause the energy to be 1207.09. This does not equal the energy at EG2 for the line for Virginia which can not be true. The hydraulic grade line elevation at this location should be 1204.99. The losses for the line from Virginia to Cambridge are 0.93' and the energy grade elevation at Cambridge (EG2) should be 1207.76 and the hydraulic grade elevation (HG2) should be 1205.92. The last three columns of the table show the correct computed results. These values should be used for all calculations.

Drop Inlets ~~at~~ are not included because they are not needed for general storm. Only at windsor drop inlet is needed.

See sec. "Catch basin Grate Design"

* Single Inlet Design: General vs. Local C's & Grates

PROJECT NAME: Old Cross Cut Canal
PROJECT NO.: E002102

CORRECTED STORM PROGRAM RESULTS FOR THE GENERAL STORM

DESIGN BY: RHF
DATE: 7-22-94

CHECKED BY: _____
DATE: _____

STREET NAME	BOX STATION	V1	HG1	Hv1	EG1	V2	HG2	Hv2	EG2	CORRECTED EG2	COMPUTED Hv2	COMPUTED HG2
OUTLET	11+00	–	–	–	–	11.6	1196.00	2.09	1198.09	1198.09	2.09	1196.00
GRANADA	19+00	11.6	1196.00	2.09	1198.09	11.6	1197.83	2.09	1199.92	1199.92	2.09	1197.83
HOLLY	27+17	11.6	1197.83	2.09	1199.92	11.6	1199.60	2.09	1201.69	1201.69	2.09	1199.60
S. CEMETARY DR.	33+57	11.6	1199.60	2.09	1201.69	11.6	1200.99	2.09	1203.08	1203.08	2.09	1200.99
OAK	35+81	11.6	1200.99	2.09	1203.08	11.6	1201.62	2.09	1203.71	1203.71	2.09	1201.62
N. CEMETARY DR.	40+57	11.6	1201.62	2.09	1203.71	11.6	1202.65	2.09	1204.74	1204.74	2.09	1202.65
VIRGINIA	50+26	11.6	1202.65	2.09	1204.74	11.6	1204.74	2.09	1206.83	1206.83	2.09	1204.74
CAMBRIDGE	55+17	10.9	1205.25	1.84	1207.09	10.9	1206.18	1.84	1208.02	1207.76	1.84	1205.92
WINDSOR	57+90	10.9	1206.18	1.84	1208.02	10.9	1207.03	1.84	1208.87	1208.61	1.84	1206.77
S. OF THOMAS	62+49	10.8	1207.05	1.81	1208.86	10.8	1207.94	1.81	1209.75	1209.50	1.81	1207.69
N. OF THOMAS	64+34	10.8	1207.94	1.81	1209.75	10.8	1208.31	1.81	1210.12	1209.87	1.81	1208.06
PINCHOT	71+37	10.8	1208.31	1.81	1210.12	10.8	1209.64	1.81	1211.45	1211.20	1.81	1209.39
GRADE BREAK	75+00	10.6	1209.82	1.74	1211.56	10.6	1210.47	1.74	1212.21	1211.85	1.74	1210.11
EARLL	75+85	10.6	1210.47	1.74	1212.21	10.6	1210.73	1.74	1212.47	1212.11	1.74	1210.37
BEGIN TRANSITION	77+00	10.6	1210.73	1.74	1212.47	10.6	1210.94	1.74	1212.68	1212.32	1.74	1210.58
END TRANSISTION	77+25	15.8	1208.86	3.88	1212.74	15.8	1210.82	3.88	1214.70	1214.28	3.88	1210.40
RICHARDSON	83+00	15.8	1210.82	3.88	1214.70	15.8	1213.75	3.88	1217.63	1217.21	3.88	1213.33
S. OF OSBORN	88+61	11.6	1217.37	2.09	1219.46	11.6	1218.99	2.09	1221.08	1218.83	2.09	1216.74
N. OF OSBORN	90+50	11.6	1218.99	2.09	1221.08	11.6	1219.56	2.09	1221.65	1219.38	2.09	1217.29
WHITTON	96+64	11.5	1219.63	2.05	1221.68	11.5	1221.27	2.05	1223.32	1221.02	2.05	1218.97
GRADE BREAK	97+00	10.1	1222.19	1.58	1223.77	10.1	1222.26	1.58	1223.84	1221.09	1.58	1219.51
WELDON	100+22	10.1	1222.26	1.58	1223.84	10.1	1222.96	1.58	1224.54	1221.79	1.58	1220.21
BEGIN TRANSITION	101+00	10	1222.99	1.55	1224.54	10	1223.15	1.55	1224.70	1221.95	1.55	1220.40
END TRANSISTION	101+10	12.1	1222.46	2.27	1224.73	12.1	1222.90	2.27	1225.17	1222.39	2.27	1220.12
S. OF INDIAN SCHOOL	111+25	12.1	1219.05	2.27	1221.32	12.1	1230.84	2.27	1233.11	1230.21	6.56	1223.65
N. OF INDIAN SCHOOL	116+83	20.1	1223.77	6.27	1230.04	15.7	1230.23	3.83	1234.06	1235.00	6.56	1228.44
INLET	117+50	13.3	1231.47	2.75	1234.22	15.6	1230.70	3.78	1234.48	1235.58	6.56	1229.02

LA COUNTY PUBLIC WORKS

STORM DRAIN ANALYSIS
(INPUT)

REPT: PC/RD4412.1
DATE: 07/18/94
PAGE 1

should be 1196.01
see calcs. in the front of report

PROJECT: OCCC EG=1198.02 SUBMIT 7-94 BEGIN STA. 11+00
DESIGNER: RHF 7SD594.DAT

CD	L2	MAX Q	ADJ Q	LENGTH	FL 1	FL 2	CTL/TW	D	W	S	KJ	KE	KM	LC	L1	L3	L4	A1	A3	A4	J	N
8	1						1196.10															
2	2	2085.0	2085.0	800.00	1180.04	1181.96	1197.90	120.	216.	3	0.00	0.00	0.00	1	3	0	0	0.	0.	0.	0.00	0.013
2	3	2085.0	2085.0	817.00	1181.96	1183.92	1199.40	120.	216.	3	0.00	0.00	0.00	0	4	0	0	0.	0.	0.	0.00	0.013
2	4	2085.0	2085.0	640.00	1183.92	1185.46	1201.21	120.	216.	3	0.00	0.00	0.00	0	5	0	0	0.	0.	0.	0.00	0.013
2	5	2085.0	2085.0	224.00	1185.46	1186.00	1202.00	120.	216.	3	0.00	0.00	0.07	0	6	0	0	0.	0.	0.	0.00	0.013
2	6	2085.0	2085.0	476.00	1186.00	1187.14	1202.19	120.	216.	3	0.00	0.00	0.00	0	7	0	0	0.	0.	0.	0.00	0.013
2	7	2085.0	2085.0	969.20	1187.14	1189.46	1205.20	120.	216.	3	0.00	0.00	0.00	0	8	0	0	0.	0.	0.	0.00	0.013
2	8	1955.0	1955.0	490.73	1189.46	1190.64	1206.40	120.	216.	3	0.00	0.00	0.00	0	9	0	0	0.	0.	0.	0.00	0.013
2	9	1955.0	1955.0	273.07	1190.64	1191.30	1206.80	120.	216.	3	0.00	0.00	0.18	0	10	0	0	0.	0.	0.	0.00	0.013
2	10	1950.0	1950.0	459.00	1191.30	1192.40	1208.08	120.	216.	3	0.00	0.00	0.01	0	11	0	0	0.	0.	0.	0.00	0.013
2	11	1950.0	1950.0	185.00	1192.40	1192.84	1208.65	120.	216.	3	0.00	0.00	0.01	0	12	0	0	0.	0.	0.	0.00	0.013
2	12	1950.0	1950.0	703.32	1192.84	1194.53	1210.24	120.	216.	3	0.00	0.00	0.00	0	13	0	0	0.	0.	0.	0.00	0.013
2	13	1900.0	1900.0	362.68	1194.53	1195.40	0.00	120.	216.	3	0.00	0.00	0.00	0	14	0	0	0.	0.	0.	0.00	0.013
2	14	1900.0	1900.0	85.00	1195.40	1195.79	1210.50	120.	216.	3	0.00	0.00	0.06	0	15	0	0	0.	0.	0.	0.00	0.013
2	15	1900.0	1900.0	115.00	1195.79	1196.32	0.00	120.	216.	3	0.00	0.00	0.00	0	16	0	0	0.	0.	0.	0.00	0.013
2	16	1900.0	1900.0	25.00	1196.32	1196.44	0.00	120.	144.	3	0.00	0.00	0.47	0	17	0	0	0.	0.	0.	0.00	0.013
2	17	1900.0	1900.0	575.00	1196.44	1199.08	1218.72	120.	144.	3	0.00	0.00	0.01	0	18	0	0	0.	0.	0.	0.00	0.013
2	18	1390.0	1390.0	561.00	1199.08	1201.66	1222.05	120.	144.	3	0.00	0.00	0.05	0	19	0	0	0.	0.	0.	0.00	0.013
2	19	1390.0	1390.0	189.00	1201.66	1202.53	1222.22	120.	144.	3	0.00	0.00	0.02	0	20	0	0	0.	0.	0.	0.00	0.013
2	20	1375.0	1375.0	614.00	1202.53	1205.35	1227.17	120.	144.	3	0.00	0.00	0.01	0	21	0	0	0.	0.	0.	0.00	0.013
2	21	1210.0	1210.0	36.00	1205.35	1205.52	0.00	120.	144.	3	0.00	0.00	0.00	0	22	0	0	0.	0.	0.	0.00	0.013
2	22	1210.0	1210.0	322.00	1205.52	1208.29	1230.00	120.	144.	3	0.00	0.00	0.03	0	23	0	0	0.	0.	0.	0.00	0.013
2	23	1205.0	1205.0	78.00	1208.29	1208.96	0.00	120.	144.	3	0.00	0.00	0.00	0	24	0	0	0.	0.	0.	0.00	0.013
2	24	1205.0	1205.0	10.00	1208.96	1209.05	0.00	120.	120.	3	0.00	0.00	0.18	0	25	0	0	0.	0.	0.	0.00	0.013
2	25	1205.0	1205.0	1015.00	1209.05	1217.78	1243.31	120.	120.	3	0.00	0.00	0.00	0	26	0	0	0.	0.	0.	0.00	0.013

asked Ron why Q is higher he said they had to put more e in the box at Virginia.

check trans. losses calcs. Q=1390 why?

check trans. losses calcs. Q=1180 cfs why?

PROJECT: OCCC EG=1198.02 SUBMIT 7-94 BEGIN STA. 11+00
DESIGNER: RHF 7SD594.DAT

CD	L2	MAX Q	ADJ Q	LENGTH	FL 1	FL 2	CTL/TW	D	W	S	KJ	KE	KM	LC	L1	L3	L4	A1	A3	A4	J	N
2	26	1205.0	1205.0	558.00	1217.78	1222.57	1242.50	120.	120.	3	0.00	0.00	0.00	0	27	0	0	0.	0.	0.	0.00	0.013
2	27	1180.0	1180.0	67.00	1222.57	1223.15	0.00	120.	120.	1	0.00	0.50	0.00	0	0	0	0	0.	0.	0.	0.00	0.013

PROJECT: OCCC EG=1198.02 SUBMIT 7-94 BEGIN STA. 11+00
DESIGNER: RHF 7SD594.DAT

LINE NO	Q (CFS)	D (IN)	W (IN)	DN (FT)	DC (FT)	FLOW TYPE	SF-FULL (FT/FT)	V 1 (FPS)	V 2 (FPS)	FL 1 (FT)	FL 2 (FT)	HG 1 CALC	HG 2 CALC	D 1 (FT)	D 2 (FT)	TW CALC	TW CK	REMARKS
1	HYDRAULIC GRADE LINE CONTROL = 1196.10																	
2	2085.0	120	216	7.93	7.46	FULL	0.00216	11.6	11.6	1180.04	1181.96	1196.10	1197.83	16.06	15.87	0.00	0.00	
3	2085.0	120	216	7.93	7.46	FULL	0.00216	11.6	11.6	1181.96	1183.92	1197.83	1199.60	15.87	15.68	0.00	0.00	
4	2085.0	120	216	7.92	7.46	FULL	0.00216	11.6	11.6	1183.92	1185.46	1199.60	1200.99	15.68	15.53	0.00	0.00	
5	2085.0	120	216	7.91	7.46	FULL	0.00216	11.6	11.6	1185.46	1186.00	1200.99	1201.62	15.53	15.62	0.00	0.00	
6	2085.0	120	216	7.93	7.46	FULL	0.00216	11.6	11.6	1186.00	1187.14	1201.62	1202.65	15.62	15.51	0.00	0.00	
7	2085.0	120	216	7.93	7.46	FULL	0.00216	11.6	11.6	1187.14	1189.46	1202.65	1204.74	15.51	15.28	0.00	0.00	
8	1955.0	120	216	7.55	7.15	FULL	0.00190	10.9	10.9	1189.46	1190.64	1205.25	1206.18	15.79	15.54	0.00	0.00	
9	1955.0	120	216	7.54	7.15	FULL	0.00190	10.9	10.9	1190.64	1191.30	1206.18	1207.03	15.54	15.73	0.00	0.00	
10	1950.0	120	216	7.55	7.14	FULL	0.00189	10.8	10.8	1191.30	1192.40	1207.05	1207.94	15.75	15.54	0.00	0.00	
11	1950.0	120	216	7.57	7.14	FULL	0.00189	10.8	10.8	1192.40	1192.84	1207.94	1208.31	15.54	15.47	0.00	0.00	
12	1950.0	120	216	7.54	7.14	FULL	0.00189	10.8	10.8	1192.84	1194.53	1208.31	1209.64	15.47	15.11	0.00	0.00	
13	1900.0	120	216	7.40	7.01	FULL	0.00180	10.6	10.6	1194.53	1195.40	1209.82	1210.47	15.29	15.07	0.00	0.00	
14	1900.0	120	216	5.86	7.01	FULL	0.00180	10.6	10.6	1195.40	1195.79	1210.47	1210.73	15.07	14.94	0.00	0.00	
15	1900.0	120	216	5.85	7.01	FULL	0.00180	10.6	10.6	1195.79	1196.32	1210.73	1210.94	14.94	14.62	0.00	0.00	
16	1900.0	120	144	8.62	9.19	FULL	0.00504	15.8	15.8	1196.32	1196.44	1208.86	1210.82	12.54	14.38	0.00	0.00	
17	1900.0	120	144	8.77	9.19	FULL	0.00504	15.8	15.8	1196.44	1199.08	1210.82	1213.75	14.38	14.67	0.00	0.00	
18	1390.0	120	144	6.87	7.46	FULL	0.00269	11.6	11.6	1199.08	1201.66	1217.37	1218.99	18.29	17.33	0.00	0.00	
19	1390.0	120	144	6.87	7.46	FULL	0.00269	11.6	11.6	1201.66	1202.53	1218.99	1219.54	17.33	17.01	0.00	0.00	
20	1375.0	120	144	6.82	7.41	FULL	0.00264	11.5	11.5	1202.53	1205.35	1219.63	1221.27	17.10	15.92	0.00	0.00	
21	1210.0	120	144	6.12	6.80	FULL	0.00204	10.1	10.1	1205.35	1205.52	1222.19	1222.26	16.84	16.74	0.00	0.00	
22	1210.0	120	144	4.91	6.80	FULL	0.00204	10.1	10.1	1205.52	1208.29	1222.26	1222.96	16.74	14.67	0.00	0.00	
23	1205.0	120	144	4.89	6.78	FULL	0.00203	10.0	10.0	1208.29	1208.96	1222.99	1223.15	14.70	14.19	0.00	0.00	
24	1205.0	120	120	5.76	7.66	FULL	0.00328	12.1	12.1	1208.96	1209.05	1222.46	1222.90	13.51	13.85	0.00	0.00	

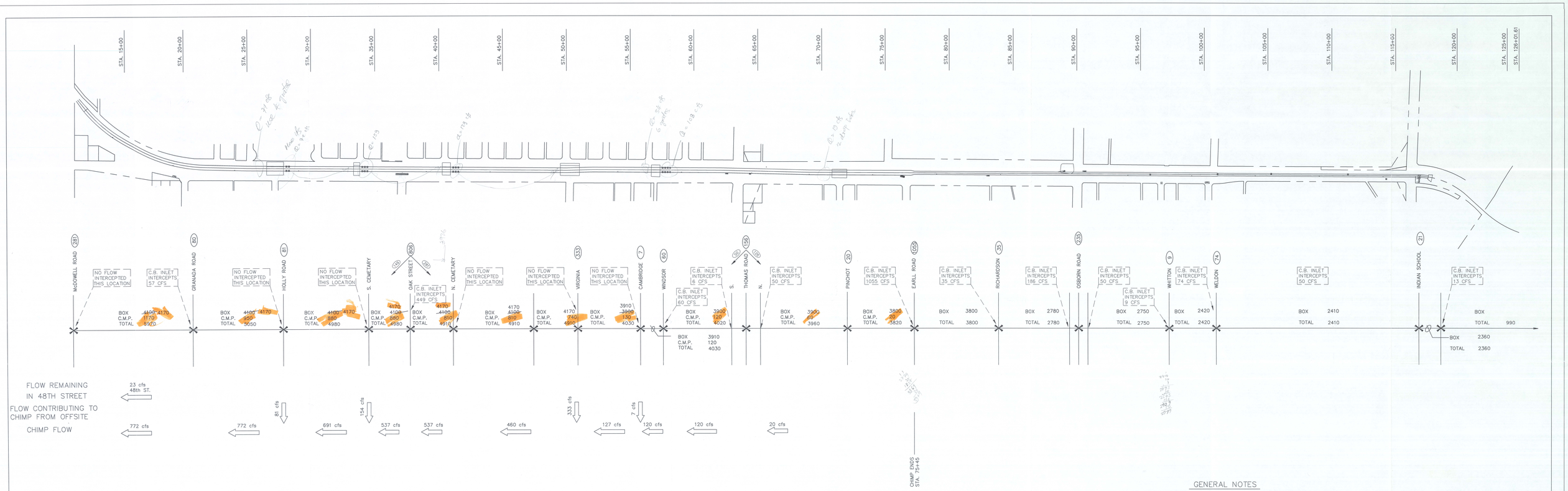
PROJECT: OCCC EG=1198.02 SUBMIT 7-94 BEGIN STA. 11+00
DESIGNER: RHF 7SD594.DAT

LINE NO	Q (CFS)	D (IN)	W (IN)	DN (FT)	DC (FT)	FLOW TYPE	SF-FULL (FT/FT)	V 1 (FPS)	V 2 (FPS)	FL 1 (FT)	FL 2 (FT)	HG 1 CALC	HG 2 CALC	D 1 (FT)	D 2 (FT)	TW CALC	TW CK	REMARKS
25	1205.0	120	120	5.86	7.66	FULL	0.00328	12.1	12.1	1209.05	1217.78	1219.05	1230.84	10.00	13.06	0.00	0.00	HJ @ UJT
26	1205.0	120	120	5.87	7.66	PART	0.00328	20.1	15.7	1217.78	1222.57	1223.77	1230.23	5.99	7.66	0.00	0.00	
27	1180.0	120	120	5.75	7.55	PART	0.00314	13.3	15.6	1222.57	1223.15	1231.47	1230.70	8.90	7.55	1236.39	0.00	HYD JUMP

X = 0.00 X(N) = 0.00 X(J) = 32.61 F(J) = 866.29 D(BJ) = 6.84 D(AJ) = 8.33

V 1, FL 1, D 1 AND HG 1 REFER TO DOWNSTREAM END
V 2, FL 2, D 2 AND HG 2 REFER TO UPSTREAM END
X - DISTANCE IN FEET FROM DOWNSTREAM END TO POINT WHERE HG INTERSECTS SOFFIT IN SEAL CONDITION
X(N) - DISTANCE IN FEET FROM DOWNSTREAM END TO POINT WHERE WATER SURFACE REACHES NORMAL DEPTH BY EITHER DRAWDOWN OR BACKWATER
X(J) - DISTANCE IN FEET FROM DOWNSTREAM END TO POINT WHERE HYDRAULIC JUMP OCCURS IN LINE
F(J) - THE COMPUTED FORCE AT THE HYDRAULIC JUMP
D(BJ) - DEPTH OF WATER BEFORE THE HYDRAULIC JUMP (UPSTREAM SIDE)
D(AJ) - DEPTH OF WATER AFTER THE HYDRAULIC JUMP (DOWNSTREAM SIDE)
SEAL INDICATES FLOW CHANGES FROM PART TO FULL OR FROM FULL TO PART
HYD JUMP INDICATES THAT FLOW CHANGES FROM SUPERCRITICAL TO SUBCRITICAL THROUGH A HYDRAULIC JUMP
HJ @ UJT INDICATES THAT HYDRAULIC JUMP OCCURS AT THE JUNCTION AT THE UPSTREAM END OF THE LINE
HJ @ DJT INDICATES THAT HYDRAULIC JUMP OCCURS AT THE JUNCTION AT THE DOWNSTREAM END OF THE LINE

EOJ 7/21/1994 20: 7



GENERAL NOTES

1. THIS MAP IS A GRAPHICAL REPRESENTATION OF THE FLOW SPLITS, INTERCEPTED FLOWS AND FLOWS CARRIED IN THE CHIMP. PLEASE REFER TO GREINER, MEMORANDUM TO THE FILE DATED MAY 17, 1994 SUBJECT: BOX CULVERT, SURFACE FEATURE AND INLET DESIGN PROCEDURES PRESENTED TO THE FCDMC IN THE WEEKLY MEETING ON MAY 18, 1994 FOR PROCEDURE EXPLANATION.
2. THE FLOW SPLITS GRAPHICALLY SHOWN HERE DEPICT THE WAY THE STORM DRAIN PROGRAM DATA WAS ENTERED INTO FILE 75D594.DAT, AND REPRESENTS THE FLOW INTERCEPTION FOR INLETS ONLY NEEDED FOR THE GENERAL STORM.

GRAPHICAL REPRESENTATION OF THE GENERAL STORM FLOW SPLIT

* C.M.P. = CHIMP MAXIMUM CAPACITY

MAP FOR FILE 75D594.DAT

STORM OUTPUT FILE
8SD594.DAT & MAP

For local Storm

Greiner

Job No. E002102 Project Old Cross Cut Canal Sheet 1 of 1

Subject STORM Model 8SD594.DAT

By RHF Date 7-22-94 Checked By N.W Date 7-26-94

This file uses the general storm discharges in the side streets. The box is full. The inlets in the bottom of the CHIMP have been sized then a clogging factor was applied. The inlets were analyzed using the pressure procedure to determine the discharge intercepted. The FCDMC for drop inlets procedure was used to compute the loss coefficient and this coefficient was input into the storm program. The storm program was run and the output is included with the inlet calculations in this section.

Per 8/15/94 Conversation w/ Ron ✓

* Raised the question about using full flow at the box for local storm which may not happen during the local storm and Ron said the FCD Drop Inlet calcs. requires full flow cond. and he thinks that since the lateral Q's are so small compare to mainline Q's it will not make big difference.

PROJECT NAME:	OLD CROSS CUT CANAL	Grate Net Area:	4.25	No. OF GRATES:	2 (W/O CLOGGING A TOTAL OF 4 ON PLANS)
JOB NUMBER:	E002102	Height of Catch Basin:	4	Ke:	0.50
		Top Slab Thickness:	1.75	Ko:	1.00
GRATE TYPE:	C.O.P. Standard Detail P1565 Type 1 Grate	Length:	5.75	Kf:	0.0611
CATCH BASIN LOCATION:	HOLLY – STA. 28+24 (INLETS IN CHIMP)	Manning's n:	0.014		
CHIMP HG AT INLET:	1203.47	Grate Total Area:	6.67		
BOX CULVERT HG ELEVATION:	1199.83	Total Perimeter:	10.67		
GRATE ELEVATION:	1199.60	Hydraulic Radius:	0.6251		

Q	Hp	Ho	An	Ag	Kl	Kl–Kt	Ktg	Vn	Hvn	Vg	Hvg	Ktn	Ktn	Qo
Assumed Discharge	Pressure Head	Oriface Head	Net Grate Open Area	Total Grate Open Area	Calculated Total Loss Coefficient	Total Loss Coefficient Without Trashrack Loss	Calculated Trashrack Adjusted Coefficient	Net Grate Area Velocity	Net Grate Area Velocity Head	Total Grate Area Velocity	Total Grate Area Velocity Head	Calculated Trashrack Coefficient	"Small Dam's" Trashrack Coefficient	Discharge Oriface Equation
40	3.64	3.87	8.5	13.34	26.0723	1.5611	24.5112	4.7059	0.3439	2.9985	0.1396	9.9516	0.7573	89.91
50	3.64	3.87	8.5	13.34	16.6863	1.5611	15.1252	5.8824	0.5373	3.7481	0.2181	6.1408	0.7573	89.91
60	3.64	3.87	8.5	13.34	11.5877	1.5611	10.0266	7.0588	0.7737	4.4978	0.3141	4.0708	0.7573	89.91
70	3.64	3.87	8.5	13.34	8.5134	1.5611	6.9523	8.2353	1.0531	5.2474	0.4276	2.8227	0.7573	89.91
80	3.64	3.87	8.5	13.34	6.5181	1.5611	4.9570	9.4118	1.3755	5.9970	0.5584	2.0125	0.7573	89.91
90	3.64	3.87	8.5	13.34	5.1501	1.5611	3.5890	10.5882	1.7408	6.7466	0.7068	1.4571	0.7573	89.91
100	3.64	3.87	8.5	13.34	4.1716	1.5611	2.6105	11.7647	2.1492	7.4963	0.8726	1.0599	0.7573	89.91
110	3.64	3.87	8.5	13.34	3.4476	1.5611	1.8865	12.9412	2.6005	8.2459	1.0558	0.7659	0.7573	89.91
110.34	3.64	3.87	8.5	13.34	3.4264	1.5611	1.8653	12.9812	2.6166	8.2714	1.0624	0.7573	0.7573	89.91
	0	0.00	0	0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.00
	0	0.00	0	0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.00
	0	0.00	0	0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.00
	0	0.00	0	0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.00
	0	0.00	0	0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.00
	0	0.00	0	0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.00
	0	0.00	0	0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.00
	0	0.00	0	0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.00
	0	0.00	0	0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.00
	0	0.00	0	0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.00
	0	0.00	0	0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.00
	0	0.00	0	0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.00

NOTE: Qo Should Be > Q (Column 'A' < Column 'O'); Column 'M' Should = Column 'N'

Maximum Flow at Holly is 81 cfs for the general Storm.

PROJECT: Old Cross Cut Canal JOB NUMBER: E002102 DATE: 7-22-94

DISCRIPTION: Inlet in CHIMP Invert Designed for General Storm w/ 2 grates (clogging not considered) (4 provided)

COMPUTED BY: RHF CHECKED BY: NW DATE CHECKED: 7-24-94 SHT ____ OF ____

FCDMC – FIGURE 4–9

INLET STATION: 28+24 DOWNSTREAM HG: 1196.10 Holly

1. DESIGN CONDITION:
   - A. INLET LOCATED OVER THE BOX CULVERT.
   - B. USE FCDMC VOLUME II – HYDRAULICS PP 77 &78; SECTION 4.2.4.3; FIGURE 4.9.
   - C. Qo= 4528 ; Qu= 4447 ; Qg= 81 ; (Qu/Qo)=0.9821 ; (Qg/Qo)=0.0179 ;
   - D. Do= 10 ; Du= 10 ; B= 37 ; (Du/Do)= 1.0 ; (B/Do)= 3.7 ;
   - E. MAINLINE OUTLET STRUCTURE: 2-18 X10
   - F. MAINLINE UPSTREAM STRUCTURE: 2-18 X10

2. GENERAL INSTRUCTION No. 1:
   - A. DOWNSTREAM HG= 1196.10
   - B. BOX REACH LENGTH= 1724
   - C. BOX Sf= 0.002552
   - D. Hf = Sf * L = (0.002552) (1724) = 4.40
   - E. PRESSURE LINE EL. AT D.S. SIDE OF JUNCTION STRUCTURE = 1196.10 + 4.40 = 1200.50

3. GENERAL INSTRUCTION No. 2:
   - A. Vo= 12.58 ; Ho= 2.46 ; [Ho=(Vo^2)/2g]

4. GENERAL INSTRUCTION No. 4:
   - A. STRUCTURE INVERT ELEVATION = 1184.18
   - B. PRESSURE DEPTH = 1200.50 - 1184.18 = 16.32 ; ((2.E) – (4.A))
   - C. K(Vo^2/2g) WHERE K=3(Qg/Qo)= 0.13 ; (PER SECTION 4.2.4.3 No. 4 – pg 4–23)
   - C. d = 16.32 + 0.13 = 16.45 ; ((4.B) + (4.C))
   - D. d/Do = 16.45/10.0 = 1.6 ; USE 1.5

NOTE:

FOR Qg/Qo = 0.1 Du/Do = 1.0

Ku = 0.12

FOR d/Do = 1.5 ; INCREMENT OF Ku = 0.06

THEREFORE Ku = Ku + Ku INCREMENT = 0.18

Hu = Ku(Vo^2/2g) = 0.18 (2.46) = 0.44

TRIAL UPSTREAM OF JUNCTION STRUCTURE PRESSURE LINE EL. = 1200.50 + 0.44 = 1200.94 ((2.E) + (Hu))

CHECK d:

- A. (UPSTREAM PRESSURE LINE EL.) – (STRUCTURE INVERT EL.) = 1200.94 - 1184.18 = 16.76; USE = 16.8
- B. d/Do = 16.8/10 = 1.7 ; USE 1.5

DOES d/Do = THE ASSUMED d/Do Yes

- A. IF YES STOP
- B. IF NO COMPUTE THE CORRECT d/Do AND READ INCREMENT OF Ku AND PROCEED:
  1. d/Do = _____ ; (FROM CHECK d ABOVE)
  2. INCREMENT OF Ku = _____ ; (FROM UPPER GRAPH OF FIGURE 4.9)
  3. CORRECT Ku = Ku + INCREMENT OF Ku = _____

Hu = Ku(Vo^2/2g) = 0.18 (2.46) = 0.44

HGu = (UPSTREAM PRESSURE LINE) + (Hu) = 1200.50 + 0.44 = 1200.94

FILE: DF-INLET.WK3

use K = 0.18

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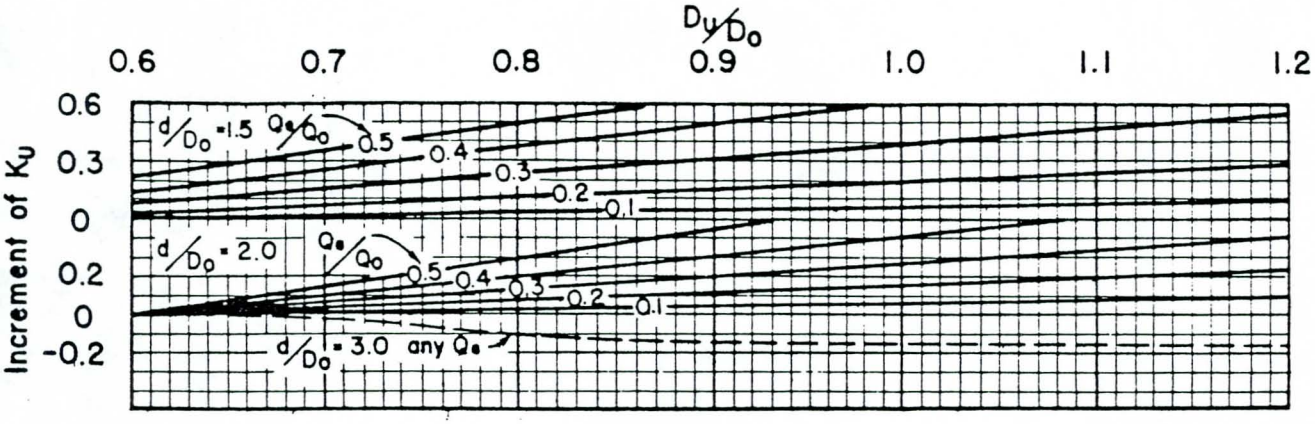

Supplementary Chart for Modification of K_U for Depth in Inlet other than 2.5 D_O

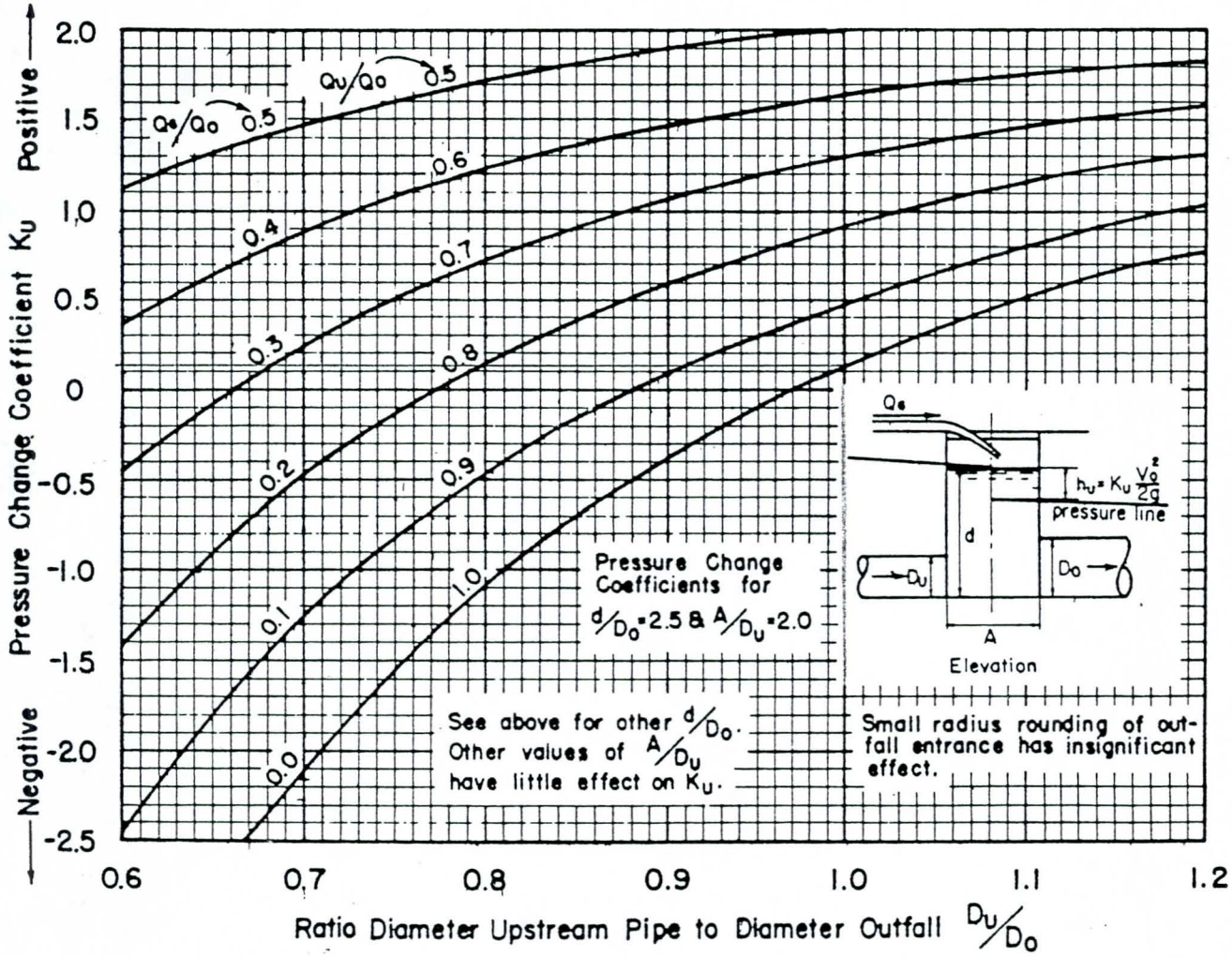

Figure 4.9
Rectangular Manhole with Through Pipeline and Inlet Flow

To use Figure 4.8:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate the velocity head in the outfall (Gen. Instr. 2).
3. Calculate the size ratios D_U/D_O and A/D_U (Gen. Instr. 3).
4. Note whether the outfall pipe entrance is to be square-edged or rounded smooth (note Gen. Instr. 6).
5. Enter Figure 4.8 at the pipe size ratio D_U/D_O and read K_U at the curve for the proper value of A/D_U for a square-edged entrance condition, or at the dashed curve for a rounded entrance.
6. Calculate h_U (positive or negative) as indicated on the diagrams on Figure 4.8 and by Gen. Instr. 7.
7. Add a positive h_U to (or subtract the absolute value of a negative h_U from) the elevation of the outfall pressure at the junction center to obtain the elevation of the upstream pipe pressure line at the same location.
8. The water surface elevation in the junction corresponds to that of the upstream pipe, whether above or below the outfall pressure line.
9. Check to be sure the water surface elevation in the junction is at least 6 inches below the top of the junction box so that overflow may not occur.

Comments: For a square-edged entrance to the outfall pipe, values of A/D_U less than 1 do not appreciably reduce the values of K_U shown for $A/D_U = 1$. K_U increases for distances A/D_U greater than 3, but such values are not usual in storm drain construction. For rounded entrances, the curve shown will apply with sufficient accuracy for all values of A/D_U up to 3.

4.2.4.3 Rectangular Manhole—Through Pipeline—with Inlet Flow: Coefficients for pressure changes are presented in Figure 4.9 for use in determining the common elevation of the upstream in-line pipe pressure line and the water surface in the manhole. The in-line pipes connect at the manhole sides (long dimension) and must meet the alignment requirement stated for Figure 4.9. As much as half the total flow may enter through a top inlet. The main graph of Figure 4.9 includes effects of various portions of grate flow for a relative water depth d/D_O of 2.5. Increments of K_U for other relative water depths are shown in the supplemental graphs; positive increments for d/D_O less than 2.5 and negative for greater depths.

To use Figure 4.9:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate velocity head in the outfall (Gen. Instr. 2).
3. Calculate the ratios of D_U/D_O and Q_U/Q_O (Gen. Instr. 3). (The inlet flow ratio $Q_G/Q_O = 1 - Q_U/Q_O$).
4. Estimate a value for the water depth, d.
   - » Follow Gen. Instr. 4.
   - » Estimate $K = 3\ Q_G/Q_O$.
5. Calculate the corresponding relative water depth d/D_O.

6. If the estimated d/D_O is approximately 2.5, enter the lower graph on Figure 4.9 at the pipe size ratio D_U/D_O and read K_U at the curve of interpolated curve for Q_U/Q_O: then proceed as in Step 9.
7. If the estimated d/D_O is other than 2.5, follow Step 6, then enter the upper graph on Figure 4.9 at the given D_U/D_O and determine the increment of K_U required to account for the effects of the estimated relative water depth d/D_U.
8. Add K_U from Step 6 and the increment from Step 7 to determine the total value of K_U. Note that negative values of K_U may occur.
9. For a rounded outfall pipe entrance or one consisting of a pipe socket, reduce K_U according to Gen. Instr. 6.
10. Calculate h_U as indicated on the diagram on the Figure and by Gen. Instr. 7.
11. Add h_U to the elevation of the outfall pressure line at the inlet center to obtain a more precise value for the water depth, d.
12. Repeat the above procedure with the improved value of d from Step 11 if necessary. Such repetition may not be necessary if the original estimated d/D_O of Step 5 was reasonably accurate.
13. Check to be sure the water elevation is at least 6 inches below the gutter elevation at the inlet so that inflow may be admitted.

4.2.4.4 Rectangular Manhole—Upstream Main and 90 Degree Lateral Pipe—with or without Inlet Flow: Pressure change coefficients are presented in Figure 4.10 for use in determining the common elevation of the two upstream pipe pressure lines and the water surface in the manhole. Flow into the combination inlet and junction box is supplied by an upstream main, in-line with the outfall and flowing through the short dimension of the manhole, and a 90 degree lateral pipe connected at one end of the box, supplemented by flow through a top inlet. The main graph of Figure 4.10 applies directly for no flow into the manhole through the inlet. Increments of K_U and K_L for inlet flow conditions are shown in the supplementary graphs of the upper portion of the figure.

To use Figure 4.10:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate the velocity head in the outfall (Gen. Instr. 2).
3. Calculate the ratios D_U/D_O, Q_U/Q_O (Gen. Instr. 3).
4. If no inlet flow is involved, enter the lower graph on Figure 4.10 at the pipe size ratio D_U/D_O and read K_U (or K_L) at the curve or interpolated curve for Q_U/Q_O, then proceed as in Step 10.
5. With inlet flow, estimate a value for the water depth, d.
   - » Follow Gen. Instr. 4
   - » Estimate K = 1.5
6. Calculate the corresponding relative water depth d/D_O.
7. Enter the lower graph and obtain K_U (or K_L) as in Step 4, this value applying for $Q_G/Q_O = 0$.

PROJECT NAME:	OLD CROSS CUT CANAL	Grate Net Area:	4.25	No. OF GRATES:	4 (W/O CLOGGING A TOTAL OF 6 ON PLANS)
JOB NUMBER:	E002102	Height of Catch Basin:	4	Ke:	0.50
		Top Slab Thickness:	1.75	Ko:	1.00
GRATE TYPE:	C.O.P. Standard Detail P1565 Type 1 Grate	Length:	5.75	Kf:	0.0611
CATCH BASIN LOCATION:	S. CEMETARY – STA. 34+39 (INLETS IN CHIMP)	Manning's n:	0.014		
CHIMP HG AT INLET:	1204.80	Grate Total Area:	6.67		
BOX CULVERT HG ELEVATION:	1201.22	Total Perimeter:	10.67		
GRATE ELEVATION:	1201.08	Hydraulic Radius:	0.6251		

Q Assumed Discharge	Hp Pressure Head	Ho Orifice Head	An Net Grate Open Area	Ag Total Grate Open Area	Kl Calculated Total Loss Coefficient	Kl–Kt Total Loss Coefficient Without Trashrack Loss	Ktg Calculated Trashrack Adjusted Coefficient	Vn Net Grate Area Velocity	Hvn Net Grate Area Velocity Head	Vg Total Grate Area Velocity	Hvg Total Grate Area Velocity Head	Ktn Calculated Trashrack Coefficient	Ktn "Small Dam's" Trashrack Coefficient	Qo Discharge Orifice Equation
40	3.58	3.72	17	26.68	102.5700	1.5611	101.0090	2.3529	0.0860	1.4993	0.0349	41.0097	0.7573	176.29
50	3.58	3.72	17	26.68	65.6448	1.5611	64.0838	2.9412	0.1343	1.8741	0.0545	26.0180	0.7573	176.29
60	3.58	3.72	17	26.68	45.5867	1.5611	44.0256	3.5294	0.1934	2.2489	0.0785	17.8744	0.7573	176.29
70	3.58	3.72	17	26.68	33.4923	1.5611	31.9312	4.1176	0.2633	2.6237	0.1069	12.9641	0.7573	176.29
80	3.58	3.72	17	26.68	25.6425	1.5611	24.0815	4.7059	0.3439	2.9985	0.1396	9.7771	0.7573	176.29
90	3.58	3.72	17	26.68	20.2608	1.5611	18.6997	5.2941	0.4352	3.3733	0.1767	7.5921	0.7573	176.29
100	3.58	3.72	17	26.68	16.4112	1.5611	14.8502	5.8824	0.5373	3.7481	0.2181	6.0292	0.7573	176.29
110	3.58	3.72	17	26.68	13.5630	1.5611	12.0019	6.4706	0.6501	4.1229	0.2640	4.8728	0.7573	176.29
120	3.58	3.72	17	26.68	11.3967	1.5611	9.8356	7.0588	0.7737	4.4978	0.3141	3.9933	0.7573	176.29
130	3.58	3.72	17	26.68	9.7108	1.5611	8.1497	7.6471	0.9080	4.8726	0.3687	3.3088	0.7573	176.29
140	3.58	3.72	17	26.68	8.3731	1.5611	6.8120	8.2353	1.0531	5.2474	0.4276	2.7657	0.7573	176.29
150	3.58	3.72	17	26.68	7.2939	1.5611	5.7328	8.8235	1.2089	5.6222	0.4908	2.3275	0.7573	176.29
160	3.58	3.72	17	26.68	6.4106	1.5611	4.8496	9.4118	1.3755	5.9970	0.5584	1.9689	0.7573	176.29
170	3.58	3.72	17	26.68	5.6786	1.5611	4.1176	10.0000	1.5528	6.3718	0.6304	1.6717	0.7573	176.29
180	3.58	3.72	17	26.68	5.0652	1.5611	3.5041	10.5882	1.7408	6.7466	0.7068	1.4227	0.7573	176.29
190	3.58	3.72	17	26.68	4.5460	1.5611	2.9850	11.1765	1.9397	7.1214	0.7875	1.2119	0.7573	176.29
200	3.58	3.72	17	26.68	4.1028	1.5611	2.5418	11.7647	2.1492	7.4963	0.8726	1.0320	0.7573	176.29
210	3.58	3.72	17	26.68	3.7214	1.5611	2.1603	12.3529	2.3695	7.8711	0.9620	0.8771	0.7573	176.29
218.858	3.58	3.72	17	26.68	3.4262	1.5611	1.8652	12.8740	2.5736	8.2031	1.0449	0.7573	0.7573	176.29

NOTE: Qo Should Be > Q (Column 'A' < Column 'O'); Column 'M' Should = Column 'N'

The Maximum Flow at this location is 149 cfs for the general storm.

PROJECT: Old Cross Cut Canal JOB NUMBER: E002102 DATE: 7-22-94

DISCRIPTION: Inlet in CHIMP Invert Designed for General Storm w/ 4-grates (clogging not considered) (6 provided)

COMPUTED BY: RHF CHECKED BY: NW DATE CHECKED: 7-24-94 SHT____ OF____

FCDMC – FIGURE 4–9

INLET STATION: 34+39 DOWNSTREAM HG: 1200.94 S. Cemetery

1. DESIGN CONDITION:
   - A. INLET LOCATED OVER THE BOX CULVERT.
   - B. USE FCDMC VOLUME II – HYDRAULICS PP 77 &78; SECTION 4.2.4.3; FIGURE 4.9.
   - C. Qo= 4528 ; Qu= 149 ; Qg= 4379 ; (Qu/Qo)= 0.9671 ; (Qg/Qo)= 0.0329 ;
   - D. Do= 10 ; Du= 10 ; B= 37 ; (Du/Do)= 1.0 ; (B/Do)= 3.7 ;
   - E. MAINLINE OUTLET STRUCTURE: 2-10'X10'
   - F. MAINLINE UPSTREAM STRUCTURE: 2-10'X10'

2. GENERAL INSTRUCTION No. 1:
   - A. DOWNSTREAM HG= 1200.94
   - B. BOX REACH LENGTH= 615 ft.
   - C. BOX Sf= 0.002552
   - D. Hf = Sf * L = (0.002552)(615) = 1.57 ft.
   - E. PRESSURE LINE EL. AT D.S. SIDE OF JUNCTION STRUCTURE = 1.57 + 1200.94 = 1202.51

3. GENERAL INSTRUCTION No. 2:
   - A. Vo= 12.58 ; Ho= 2.46 ; [Ho=(Vo^2)/2g]

4. GENERAL INSTRUCTION No. 4:
   - A. STRUCTURE INVERT ELEVATION = 1185.65
   - B. PRESSURE DEPTH = 1202.51 - 1185.65 = 16.86 ; ((2.E) – (4.A))
   - C. K(Vo^2/2g) WHERE K=3(Qg/Qo)= 0.10 ; (PER SECTION 4.2.4.3 No. 4 – pg 4–23)
   - C. d = 16.86 + 0.10 = 16.96 ; ((4.B) + (4.C))
   - D. d/Do = 16.96/10 = 1.70 ; USE 1.5

NOTE:

FOR Qg/Qo = 0.0 Du/Do = 1.0
Ku = 0.12
FOR d/Do = 1.5 ; INCREMENT OF Ku = 0.06
THEREFORE Ku = Ku + Ku INCREMENT = 0.18
Hu = Ku(Vo^2/2g) = 0.18 (2.46) = 0.44
TRIAL UPSTREAM OF JUNCTION STRUCTURE PRESSURE LINE EL. = 1202.51 + 0.44 = 1202.95 ((2.E) + (Hu))

CHECK d:

- A. (UPSTREAM PRESSURE LINE EL.) – (STRUCTURE INVERT EL.) = 1202.95 - 1185.65 = 17.30 ; USE = 17.30
- B. d/Do = 17.30 / 10.0 = 1.73 ; USE 1.7

DOES d/Do = THE ASSUMED d/Do Yes

- A. IF YES STOP
- B. IF NO COMPUTE THE CORRECT d/Do AND READ INCREMENT OF Ku AND PROCEED:
  1. d/Do = _____ ; (FROM CHECK d ABOVE)
  2. INCREMENT OF Ku = _____ ; (FROM UPPER GRAPH OF FIGURE 4.9)
  3. CORRECT Ku = Ku + INCREMENT OF Ku = _____

Hu = Ku(Vo^2/2g) = 0.18 (2.46) = 0.44

HGu = (UPSTREAM PRESSURE LINE) + (Hu) = 1202.51 + 0.44 = 1202.95

FILE: DF-INLET.WK3

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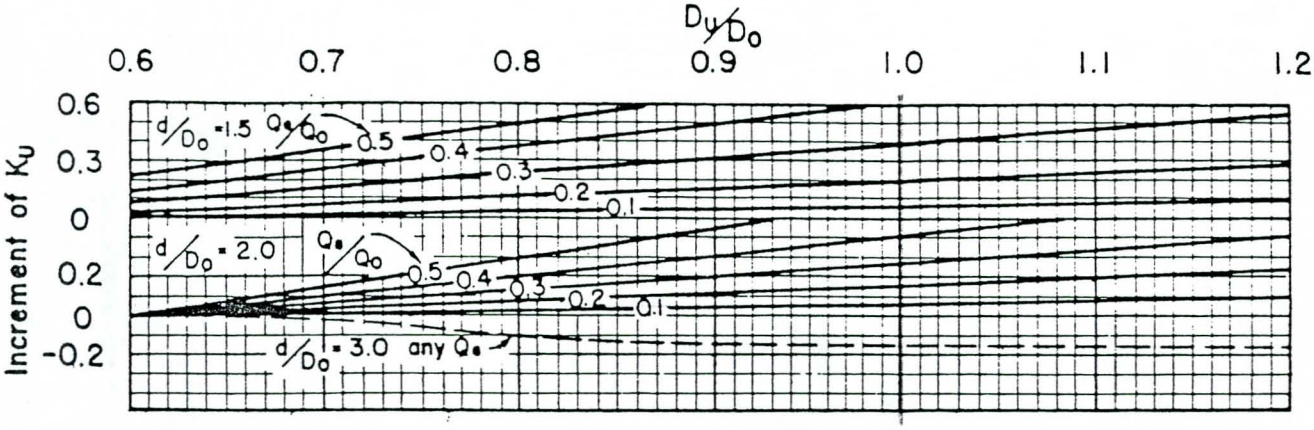


Supplementary Chart for Modification of K_u for Depth in Inlet other than $2.5\,D_o$

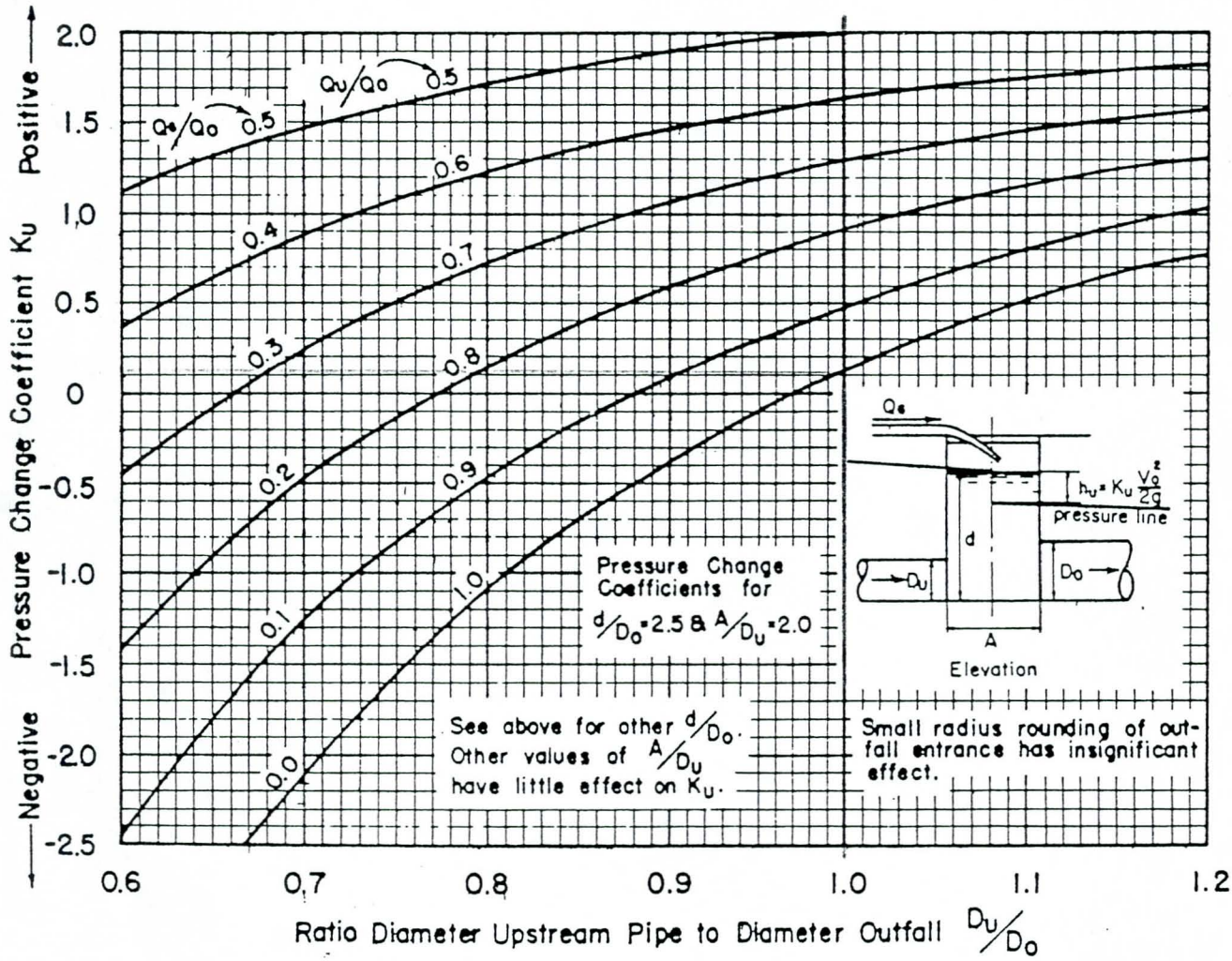


Figure 4.9
Rectangular Manhole with Through Pipeline and Inlet Flow

To use Figure 4.8:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate the velocity head in the outfall (Gen. Instr. 2).
3. Calculate the size ratios D_U/D_O and A/D_U (Gen. Instr. 3).
4. Note whether the outfall pipe entrance is to be square-edged or rounded smooth (note Gen. Instr. 6).
5. Enter Figure 4.8 at the pipe size ratio D_U/D_O and read K_U at the curve for the proper value of A/D_U for a square-edged entrance condition, or at the dashed curve for a rounded entrance.
6. Calculate h_U (positive or negative) as indicated on the diagrams on Figure 4.8 and by Gen. Instr. 7.
7. Add a positive h_U to (or subtract the absolute value of a negative h_U from) the elevation of the outfall pressure at the junction center to obtain the elevation of the upstream pipe pressure line at the same location.
8. The water surface elevation in the junction corresponds to that of the upstream pipe, whether above or below the outfall pressure line.
9. Check to be sure the water surface elevation in the junction is at least 6 inches below the top of the junction box so that overflow may not occur.

Comments: For a square-edged entrance to the outfall pipe, values of A/D_U less than 1 do not appreciably reduce the values of K_U shown for $A/D_U = 1$. K_U increases for distances A/D_U greater than 3, but such values are not usual in storm drain construction. For rounded entrances, the curve shown will apply with sufficient accuracy for all values of A/D_U up to 3.

4.2.4.3 Rectangular Manhole—Through Pipeline—with Inlet Flow: Coefficients for pressure changes are presented in Figure 4.9 for use in determining the common elevation of the upstream in-line pipe pressure line and the water surface in the manhole. The in-line pipes connect at the manhole sides (long dimension) and must meet the alignment requirement stated for Figure 4.9. As much as half the total flow may enter through a top inlet. The main graph of Figure 4.9 includes effects of various portions of grate flow for a relative water depth d/D_O of 2.5. Increments of K_U for other relative water depths are shown in the supplemental graphs; positive increments for d/D_O less than 2.5 and negative for greater depths.

To use Figure 4.9:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate velocity head in the outfall (Gen. Instr. 2).
3. Calculate the ratios of D_U/D_O and Q_U/Q_O (Gen. Instr. 3). (The inlet flow ratio $Q_G/Q_O = 1 - Q_U/Q_O$).
4. Estimate a value for the water depth, d.
   - » Follow Gen. Instr. 4.
   - » Estimate $K = 3\ Q_G/Q_O$.
5. Calculate the corresponding relative water depth d/D_O.

6. If the estimated d/D_O is approximately 2.5, enter the lower graph on Figure 4.9 at the pipe size ratio D_U/D_O and read K_U at the curve of interpolated curve for Q_U/Q_O: then proceed as in Step 9.
7. If the estimated d/D_O is other than 2.5, follow Step 6, then enter the upper graph on Figure 4.9 at the given D_U/D_O and determine the increment of K_U required to account for the effects of the estimated relative water depth d/D_U.
8. Add K_U from Step 6 and the increment from Step 7 to determine the total value of K_U. Note that negative values of K_U may occur.
9. For a rounded outfall pipe entrance or one consisting of a pipe socket, reduce K_U according to Gen. Instr. 6.
10. Calculate h_U as indicated on the diagram on the Figure and by Gen. Instr. 7.
11. Add h_U to the elevation of the outfall pressure line at the inlet center to obtain a more precise value for the water depth, d.
12. Repeat the above procedure with the improved value of d from Step 11 if necessary. Such repetition may not be necessary if the original estimated d/D_O of Step 5 was reasonably accurate.
13. Check to be sure the water elevation is at least 6 inches below the gutter elevation at the inlet so that inflow may be admitted.

4.2.4.4 Rectangular Manhole—Upstream Main and 90 Degree Lateral Pipe—with or without Inlet Flow: Pressure change coefficients are presented in Figure 4.10 for use in determining the common elevation of the two upstream pipe pressure lines and the water surface in the manhole. Flow into the combination inlet and junction box is supplied by an upstream main, in-line with the outfall and flowing through the short dimension of the manhole, and a 90 degree lateral pipe connected at one end of the box, supplemented by flow through a top inlet. The main graph of Figure 4.10 applies directly for no flow into the manhole through the inlet. Increments of K_U and K_L for inlet flow conditions are shown in the supplementary graphs of the upper portion of the figure.

To use Figure 4.10:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate the velocity head in the outfall (Gen. Instr. 2).
3. Calculate the ratios D_U/D_O, Q_U/Q_O (Gen. Instr. 3).
4. If no inlet flow is involved, enter the lower graph on Figure 4.10 at the pipe size ratio D_U/D_O and read K_U (or K_L) at the curve or interpolated curve for Q_U/Q_O, then proceed as in Step 10.
5. With inlet flow, estimate a value for the water depth, d.
   - » Follow Gen. Instr. 4
   - » Estimate K = 1.5
6. Calculate the corresponding relative water depth d/D_O.
7. Enter the lower graph and obtain K_U (or K_L) as in Step 4, this value applying for $Q_G/Q_O = 0$.

PROJECT NAME:	OLD CROSS CUT CANAL	Grate Net Area:	4.25	No. OF GRATES:	3 (W/O CLOGGING A TOTAL OF 6 ON PLANS)
JOB NUMBER:	E002102	Height of Catch Basin:	4	Ke:	0.50
		Top Slab Thickness:	1.75	Ko:	1.00
GRATE TYPE:	C.O.P. Standard Detail P1565 Type 1 Grate	Length:	5.75	Kf:	0.0611
CATCH BASIN LOCATION:	NORTH CEMETARY – STA. 41+49 (INLETS IN CHIMP)	Manning's n:	0.014		
CHIMP HG ELEVATION:	1206.24	Grate Total Area:	6.67		
BOX CULVERT HG ELEVATION:	1202.85	Total Perimeter:	10.67		
GRATE ELEVATION:	1202.78	Hydraulic Radius:	0.6251		

Q Assumed Discharge	Hp Pressure Head	Ho Oriface Head	An Net Grate Open Area	Ag Total Grate Open Area	Kl Calculated Total Loss Coefficient	Kl–Kt Total Loss Coefficient Without Trashrack Loss	Ktg Calculated Trashrack Adjusted Coefficient	Vn Net Grate Area Velocity	Hvn Net Grate Area Velocity Head	Vg Total Grate Area Velocity	Hvg Total Grate Area Velocity Head	Ktn Calculated Trashrack Coefficient	Ktn "Small Dam's" Trashrack Coefficient	Qo Discharge Oriface Equation
50	3.39	3.46	12.75	20.01	34.9655	1.5611	33.4044	3.9216	0.2388	2.4988	0.0970	13.5622	0.7573	127.52
75	3.39	3.46	12.75	20.01	15.5402	1.5611	13.9792	5.8824	0.5373	3.7481	0.2181	5.6755	0.7573	127.52
100	3.39	3.46	12.75	20.01	8.7414	1.5611	7.1803	7.8431	0.9552	4.9975	0.3878	2.9152	0.7573	127.52
125	3.39	3.46	12.75	20.01	5.5945	1.5611	4.0334	9.8039	1.4925	6.2469	0.6060	1.6376	0.7573	127.52
150	3.39	3.46	12.75	20.01	3.8851	1.5611	2.3240	11.7647	2.1492	7.4963	0.8726	0.9435	0.7573	127.52
155	3.39	3.46	12.75	20.01	3.6384	1.5611	2.0774	12.1569	2.2949	7.7461	0.9317	0.8434	0.7573	127.52
156	3.39	3.46	12.75	20.01	3.5920	1.5611	2.0309	12.2353	2.3246	7.7961	0.9438	0.8245	0.7573	127.52
157	3.39	3.46	12.75	20.01	3.5463	1.5611	1.9853	12.3137	2.3545	7.8461	0.9559	0.8060	0.7573	127.52
158	3.39	3.46	12.75	20.01	3.5016	1.5611	1.9405	12.3922	2.3846	7.8961	0.9681	0.7879	0.7573	127.52
159	3.39	3.46	12.75	20.01	3.4577	1.5611	1.8966	12.4706	2.4148	7.9460	0.9804	0.7700	0.7573	127.52
159.724	3.39	3.46	12.75	20.01	3.4264	1.5611	1.8654	12.5274	2.4369	7.9822	0.9894	0.7573	0.7573	127.52

NOTE: Qo Should Be > Q (Column 'A' < Column 'O'); Column 'M' Should = Column 'N'

128 cfs is intercepted at this location.

PROJECT: Old Cross Cut Canal JOB NUMBER: E002102 DATE: 7-22-94

DISCRIPTION: Inlet in CHIMP Invert Designed for General Storm w/3 grates (clogging not considered) (6 on plans)

COMPUTED BY: RHF CHECKED BY: NW DATE CHECKED: 7-26-94 SHT____ OF____

FCDMC – FIGURE 4–9

INLET STATION: 41+49 DOWNSTREAM HG: 1204.20 N. Cemetary

1. DESIGN CONDITION:
   - A. INLET LOCATED OVER THE BOX CULVERT.
   - B. USE FCDMC VOLUME II – HYDRAULICS PP 77 &78; SECTION 4.2.4.3; FIGURE 4.9.
   - C. Qo= 4298; Qu= 4170; Qg= 128; (Qu/Qo)= 0.9702; (Qg/Qo)= 0.0298;
   - D. Do= 10; Du= 10; B= 37; (Du/Do)= 1.0; (B/Do)= 3.7;
   - E. MAINLINE OUTLET STRUCTURE: 2-18X10
   - F. MAINLINE UPSTREAM STRUCTURE: 2-18X10

2. GENERAL INSTRUCTION No. 1:
   - A. DOWNSTREAM HG= 1204.20
   - B. BOX REACH LENGTH= 345
   - C. BOX Sf= 0.002300
   - D. Hf = Sf * L = (0.002300)(345) = 0.79
   - E. PRESSURE LINE EL. AT D.S. SIDE OF JUNCTION STRUCTURE = 1204.20 + 0.79 = 1204.99

3. GENERAL INSTRUCTION No. 2:
   - A. Vo= 11.9; Ho= 2.20; [Ho=(Vo^2)/2g]

4. GENERAL INSTRUCTION No. 4:
   - A. STRUCTURE INVERT ELEVATION = 1187.36
   - B. PRESSURE DEPTH = 1204.99 - 1187.36 = 17.63; ((2.E) – (4.A))
   - C. K(Vo^2/2g) WHERE K=3(Qg/Qo)= 0.07; (PER SECTION 4.2.4.3 No. 4 – pg 4–23)
   - C. d = 17.63 + 0.07 = 17.70; ((4.B) + (4.C))
   - D. d/Do = 17.7/10 = 1.77; USE 2.0

NOTE:

FOR Qg/Qo = 0.0 Du/Do = 1.0
Ku = 0.12
FOR d/Do = 2.0; INCREMENT OF Ku = 0.05
THEREFORE Ku = Ku + Ku INCREMENT = 0.12 + 0.05
Hu = Ku(Vo^2/2g) = 0.17 (2.20) = 0.37
TRIAL UPSTREAM OF JUNCTION STRUCTURE PRESSURE LINE EL. = 1204.99 + 0.37 = 1205.36 ((2.E) + (Hu))

CHECK d:

- A. (UPSTREAM PRESSURE LINE EL.) – (STRUCTURE INVERT EL.) = 1205.36 - 1187.36 = 18; USE = 18.0
- B. d/Do = 18.0/10.0 = 1.80; USE 2.0

DOES d/Do = THE ASSUMED d/Do Yes

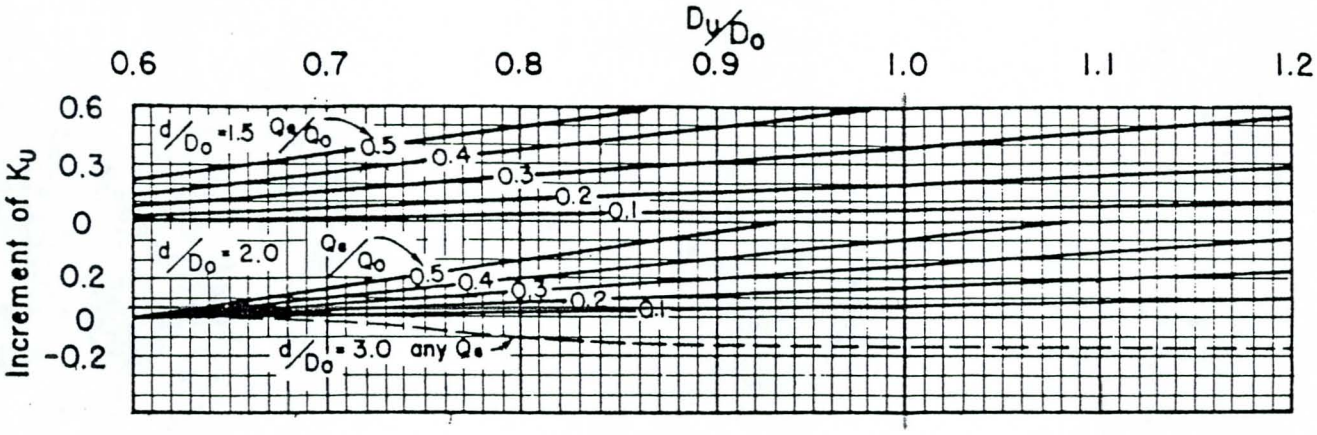
- A. IF YES STOP
- B. IF NO COMPUTE THE CORRECT d/Do AND READ INCREMENT OF Ku AND PROCEED:
  1. d/Do =_____; (FROM CHECK d ABOVE)
  2. INCREMENT OF Ku =_____; (FROM UPPER GRAPH OF FIGURE 4.9)
  3. CORRECT Ku = Ku + INCREMENT OF Ku =_____

Hu = Ku(Vo^2/2g) = 0.17 (2.20) = 0.37

HGu = (UPSTREAM PRESSURE LINE) + (Hu) = 1204.99 + 0.37 = 1205.36

FILE: DF-INLET.WK3

GREINER, INC.



Supplementary Chart for Modification of K_u for Depth in Inlet other than $2.5\,D_o$

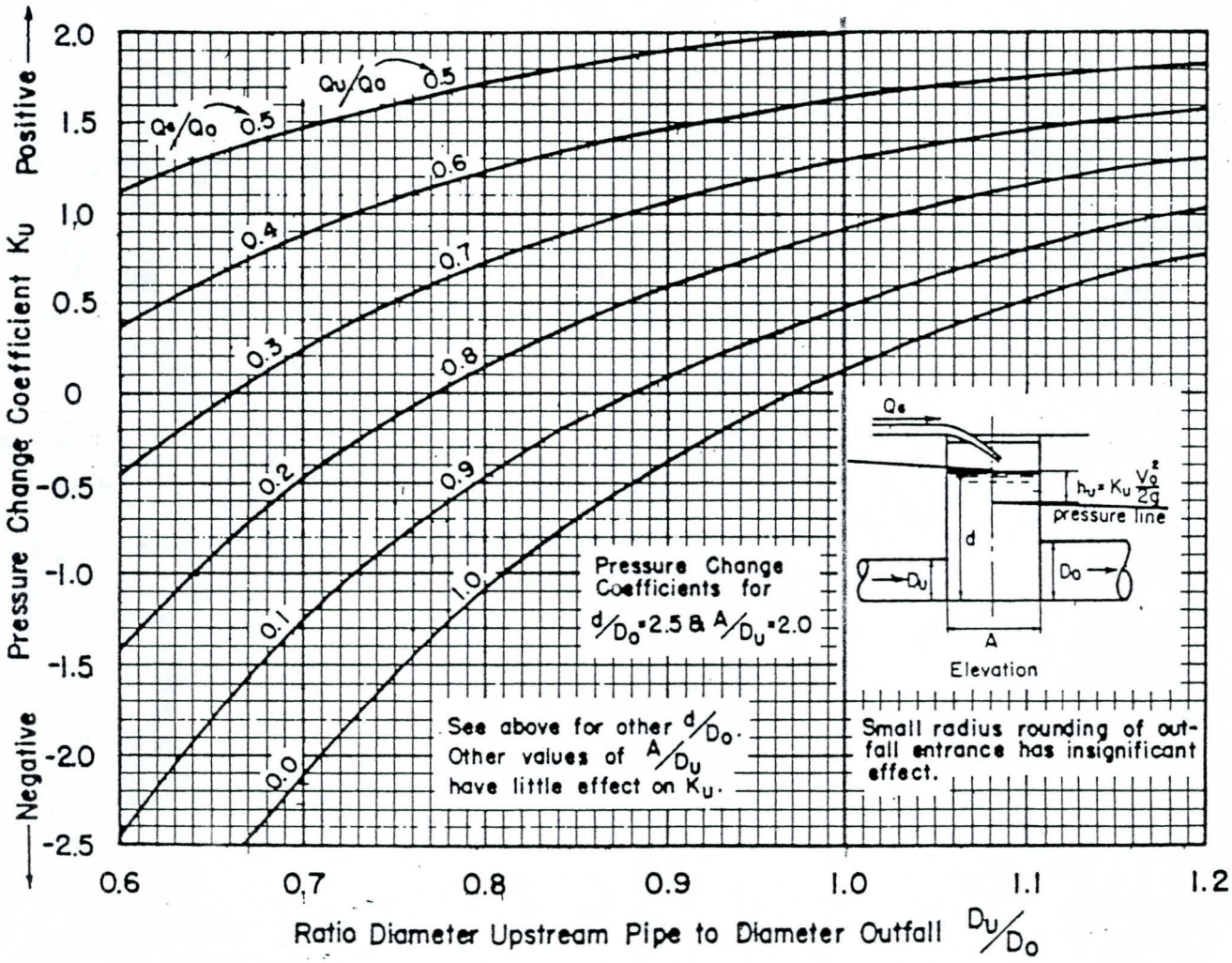


Figure 4.9
Rectangular Manhole with Through Pipeline and Inlet Flow

To use Figure 4.8:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate the velocity head in the outfall (Gen. Instr. 2).
3. Calculate the size ratios D_U/D_O and A/D_U (Gen. Instr. 3).
4. Note whether the outfall pipe entrance is to be square-edged or rounded smooth (note Gen. Instr. 6).
5. Enter Figure 4.8 at the pipe size ratio D_U/D_O and read K_U at the curve for the proper value of A/D_U for a square-edged entrance condition, or at the dashed curve for a rounded entrance.
6. Calculate h_U (positive or negative) as indicated on the diagrams on Figure 4.8 and by Gen. Instr. 7.
7. Add a positive h_U to (or subtract the absolute value of a negative h_U from) the elevation of the outfall pressure at the junction center to obtain the elevation of the upstream pipe pressure line at the same location.
8. The water surface elevation in the junction corresponds to that of the upstream pipe, whether above or below the outfall pressure line.
9. Check to be sure the water surface elevation in the junction is at least 6 inches below the top of the junction box so that overflow may not occur.

Comments: For a square-edged entrance to the outfall pipe, values of A/D_U less than 1 do not appreciably reduce the values of K_U shown for $A/D_U = 1$. K_U increases for distances A/D_U greater than 3, but such values are not usual in storm drain construction. For rounded entrances, the curve shown will apply with sufficient accuracy for all values of A/D_U up to 3.

4.2.4.3 Rectangular Manhole—Through Pipeline—with Inlet Flow: Coefficients for pressure changes are presented in Figure 4.9 for use in determining the common elevation of the upstream in-line pipe pressure line and the water surface in the manhole. The in-line pipes connect at the manhole sides (long dimension) and must meet the alignment requirement stated for Figure 4.9. As much as half the total flow may enter through a top inlet. The main graph of Figure 4.9 includes effects of various portions of grate flow for a relative water depth d/D_O of 2.5. Increments of K_U for other relative water depths are shown in the supplemental graphs; positive increments for d/D_O less than 2.5 and negative for greater depths.

To use Figure 4.9:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate velocity head in the outfall (Gen. Instr. 2).
3. Calculate the ratios of D_U/D_O and Q_U/Q_O (Gen. Instr. 3). (The inlet flow ratio $Q_G/Q_O = 1 - Q_U/Q_O$).
4. Estimate a value for the water depth, d.
   - » Follow Gen. Instr. 4.
   - » Estimate $K = 3\ Q_G/Q_O$.
5. Calculate the corresponding relative water depth d/D_O.

6. If the estimated d/D_O is approximately 2.5, enter the lower graph on Figure 4.9 at the pipe size ratio D_U/D_O and read K_U at the curve of interpolated curve for Q_U/Q_O: then proceed as in Step 9.
7. If the estimated d/D_O is other than 2.5, follow Step 6, then enter the upper graph on Figure 4.9 at the given D_U/D_O and determine the increment of K_U required to account for the effects of the estimated relative water depth d/D_U.
8. Add K_U from Step 6 and the increment from Step 7 to determine the total value of K_U. Note that negative values of K_U may occur.
9. For a rounded outfall pipe entrance or one consisting of a pipe socket, reduce K_U according to Gen. Instr. 6.
10. Calculate h_U as indicated on the diagram on the Figure and by Gen. Instr. 7.
11. Add h_U to the elevation of the outfall pressure line at the inlet center to obtain a more precise value for the water depth, d.
12. Repeat the above procedure with the improved value of d from Step 11 if necessary. Such repetition may not be necessary if the original estimated d/D_O of Step 5 was reasonably accurate.
13. Check to be sure the water elevation is at least 6 inches below the gutter elevation at the inlet so that inflow may be admitted.

4.2.4.4 Rectangular Manhole—Upstream Main and 90 Degree Lateral Pipe—with or without Inlet Flow: Pressure change coefficients are presented in Figure 4.10 for use in determining the common elevation of the two upstream pipe pressure lines and the water surface in the manhole. Flow into the combination inlet and junction box is supplied by an upstream main, in-line with the outfall and flowing through the short dimension of the manhole, and a 90 degree lateral pipe connected at one end of the box, supplemented by flow through a top inlet. The main graph of Figure 4.10 applies directly for no flow into the manhole through the inlet. Increments of K_U and K_L for inlet flow conditions are shown in the supplementary graphs of the upper portion of the figure.

To use Figure 4.10:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate the velocity head in the outfall (Gen. Instr. 2).
3. Calculate the ratios D_U/D_O, Q_U/Q_O (Gen. Instr. 3).
4. If no inlet flow is involved, enter the lower graph on Figure 4.10 at the pipe size ratio D_U/D_O and read K_U (or K_L) at the curve or interpolated curve for Q_U/Q_O, then proceed as in Step 10.
5. With inlet flow, estimate a value for the water depth, d.
   - » Follow Gen. Instr. 4
   - » Estimate K = 1.5
6. Calculate the corresponding relative water depth d/D_O.
7. Enter the lower graph and obtain K_U (or K_L) as in Step 4, this value applying for $Q_G/Q_O = 0$.

PROJECT NAME:	OLD CROSS CUT CANAL	Grate Net Area:	4.25	No. OF GRATES:	5 (W/O CLOGGING A TOTAL OF 8 ON PLANS)
JOB NUMBER:	E002102	Height of Catch Basin:	4	Ke:	0.50
		Top Slab Thickness:	1.75	Ko:	1.00
GRATE TYPE:	C.O.P. Standard Detail P1565 Type 1 Grate	Length:	5.75	Kf:	0.0611
CATCH BASIN LOCATION:	WINDSOR STA. 57+90 (INLETS IN CHIMP)	Manning's n:	0.014		
CHIMP HG AT INLET:	1208.31	Grate Total Area:	6.67		
MAINLINE HG AT DROP INLET:	1206.77	Total Perimeter:	10.67		
GRATE ELEVATION:	1206.76	Hydraulic Radius:	0.6251		

Q	Hp	Ho	An	Ag	Kl	Kl−Kt	Ktg	Vn	Hvn	Vg	Hvg	Ktn	Ktn	Qo
Assumed Discharge	Pressure Head	Oriface Head	Net Grate Open Area	Total Grate Open Area	Calculated Total Loss Coefficient	Total Loss Coefficient Without Trashrack Loss	Calculated Trashrack Adjusted Coefficient	Net Grate Area Velocity	Net Grate Area Velocity Head	Total Grate Area Velocity	Total Grate Area Velocity Head	Calculated Trashrack Coefficient	"Small Dam's" Trashrack Coefficient	Discharge Oriface Equation
100	1.54	1.55	21.25	33.35	11.0306	1.5611	9.4695	4.7059	0.3439	2.9985	0.1396	3.8446	0.7573	142.25
120	1.54	1.55	21.25	33.35	7.6601	1.5611	6.0991	5.6471	0.4952	3.5982	0.2010	2.4762	0.7573	142.25
140	1.54	1.55	21.25	33.35	5.6278	1.5611	4.0668	6.5882	0.6740	4.1979	0.2736	1.6511	0.7573	142.25
160	1.54	1.55	21.25	33.35	4.3088	1.5611	2.7478	7.5294	0.8803	4.7976	0.3574	1.1156	0.7573	142.25
170	1.54	1.55	21.25	33.35	3.8168	1.5611	2.2558	8.0000	0.9938	5.0975	0.4035	0.9158	0.7573	142.25
175	1.54	1.55	21.25	33.35	3.6018	1.5611	2.0408	8.2353	1.0531	5.2474	0.4276	0.8286	0.7573	142.25
176	1.54	1.55	21.25	33.35	3.5610	1.5611	2.0000	8.2824	1.0652	5.2774	0.4325	0.8120	0.7573	142.25
177	1.54	1.55	21.25	33.35	3.5209	1.5611	1.9598	8.3294	1.0773	5.3073	0.4374	0.7957	0.7573	142.25
178	1.54	1.55	21.25	33.35	3.4814	1.5611	1.9204	8.3765	1.0895	5.3373	0.4423	0.7797	0.7573	142.25
179	1.54	1.55	21.25	33.35	3.4426	1.5611	1.8816	8.4235	1.1018	5.3673	0.4473	0.7639	0.7573	142.25
179.423	1.54	1.55	21.25	33.35	3.4264	1.5611	1.8654	8.4434	1.1070	5.3800	0.4494	0.7573	0.7573	142.25

NOTE: Qo Should Be > Q (Column 'A' < Column 'O'); Column 'M' Should = Column 'N'

Max Flow at windsor is 60 cfs for the general storm

PROJECT: Old Cross Cut Canal JOB NUMBER: E002102 DATE: 7-22-94

DISCRIPTION: Inlet in CHIMP Invert Designed for General Storm w/ 5 grates (clogging non considered) (8 on Plans)

COMPUTED BY: RHF CHECKED BY: NW DATE CHECKED: 7-26-94 SHT____ OF ____

FCDMC – FIGURE 4–9

INLET STATION: 57+90 DOWNSTREAM HG: 1205.36

1. DESIGN CONDITION:
   - A. INLET LOCATED OVER THE BOX CULVERT.
   - B. USE FCDMC VOLUME II – HYDRAULICS PP 77 &78; SECTION 4.2.4.3; FIGURE 4.9.
   - C. Qo= 4298 ; Qu= 4238 ; Qg= 60 ; (Qu/Qo)= 0.9860; (Qg/Qo)= 0.0140;
   - D. Do= 10.0 ; Du= 10.0 ; B= 37 ; (Du/Do)= 1.0 ; (B/Do)= 3.7 ;
   - E. MAINLINE OUTLET STRUCTURE: 2-18x10
   - F. MAINLINE UPSTREAM STRUCTURE: 2-18x10

2. GENERAL INSTRUCTION No. 1:
   - A. DOWNSTREAM HG= 1205.36
   - B. BOX REACH LENGTH= 1641
   - C. BOX Sf= 0.002300
   - D. Hf = Sf * L =(0.002300)(1641)=3.77
   - E. PRESSURE LINE EL. AT D.S. SIDE OF JUNCTION STRUCTURE = 1205.36+3.77 =1209.13

3. GENERAL INSTRUCTION No. 2:
   - A. Vo= 11.94 ; Ho= 2.21 ; [Ho=(Vo^2)/2g]

4. GENERAL INSTRUCTION No. 4:
   - A. STRUCTURE INVERT ELEVATION = 1191.30
   - B. PRESSURE DEPTH =1209.13 - 1191.30 = 17.83 ; ((2.E) – (4.A))
   - C. K(Vo^2/2g) WHERE K=3(Qg/Qo)= 0.04 ; (PER SECTION 4.2.4.3 No. 4 – pg 4–23)
   - C. d = 17.83+0.04 = 17.87 ; ((4.B) + (4.C))
   - D. d/Do = 17.87/10.0 = 1.8 ; USE 2.0

NOTE:

FOR Qg/Qo = 0.0140 Du/Do = 1.0
Ku = 0.12
FOR d/Do = 2.0 ; INCREMENT OF Ku = 0.05
THEREFORE Ku = Ku + Ku INCREMENT = 0.17
Hu = Ku(Vo^2/2g) = 0.17 (2.21) = 0.38
TRIAL UPSTREAM OF JUNCTION STRUCTURE PRESSURE LINE EL. = 1209.13 + 0.38 = 1209.51 ((2.E) + (Hu))

CHECK d:

- A. (UPSTREAM PRESSURE LINE EL) – (STRUCTURE INVERT EL) = 1209.51 - 1191.30=18.2 ; USE = 18.0
- B. d/Do = 18.0 /10.0 = 1.80 ; USE 2.0

DOES d/Do = THE ASSUMED d/Do Yes

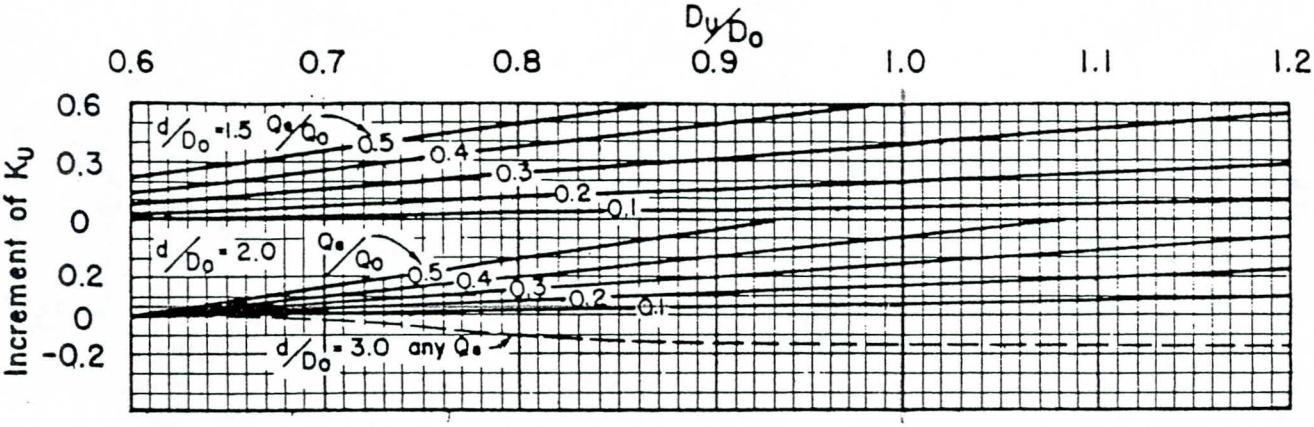
- A. IF YES STOP
- B. IF NO COMPUTE THE CORRECT d/Do AND READ INCREMENT OF Ku AND PROCEED:
  1. d/Do =_____; (FROM CHECK d ABOVE)
  2. INCREMENT OF Ku =_____; (FROM UPPER GRAPH OF FIGURE 4.9)
  3. CORRECT Ku = Ku + INCREMENT OF Ku =_____

Hu = Ku(Vo^2/2g) = 0.17 (2.21) = 0.38

HGu = (UPSTREAM PRESSURE LINE) + (Hu) = 1209.13 + 0.38 = 1209.51

FILE: DF-INLET.WK3

GREINER, INC.



Supplementary Chart for Modification of K_u for Depth in Inlet other than 2.5 D_o

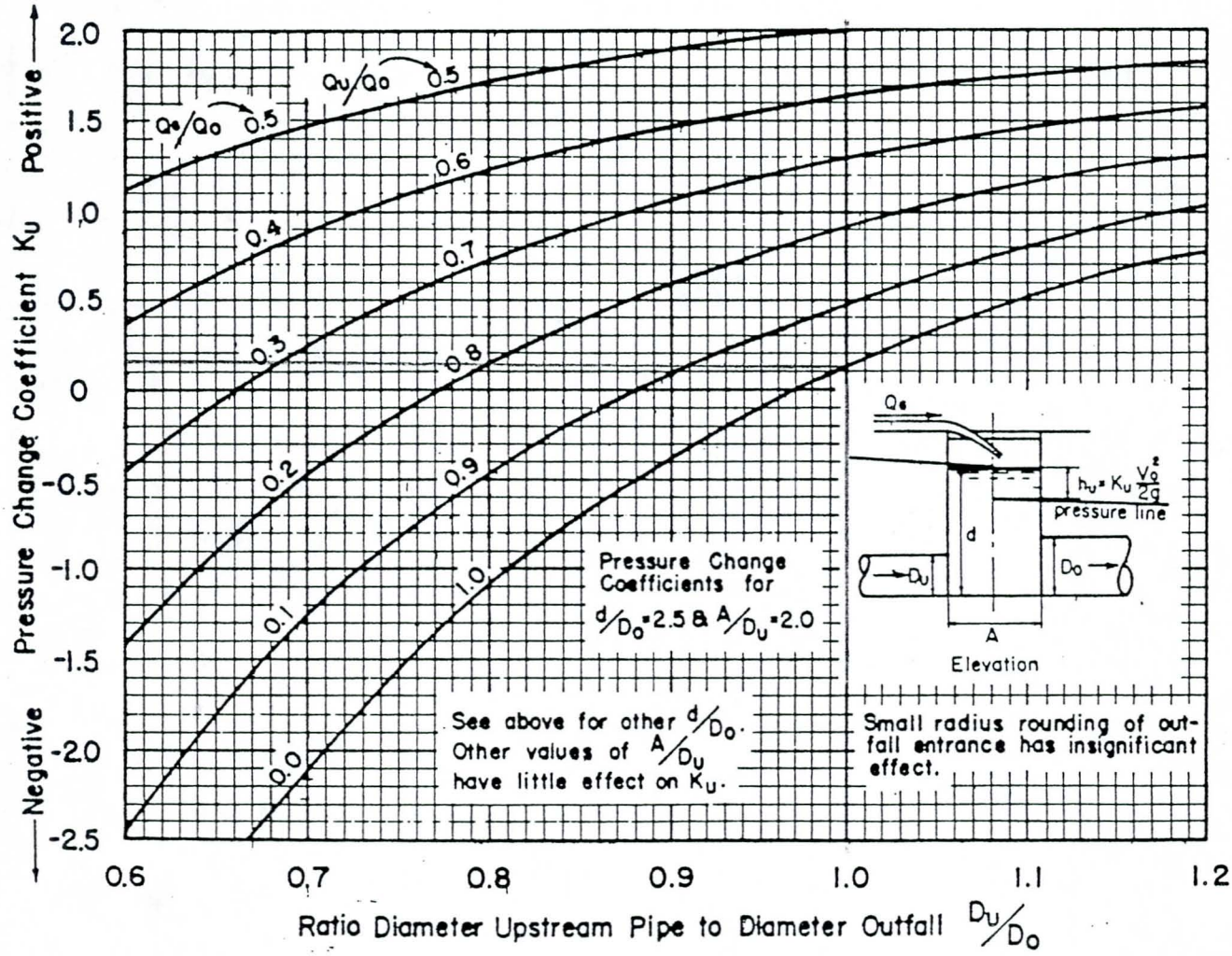


Figure 4.9
Rectangular Manhole with Through Pipeline and Inlet Flow

To use Figure 4.8:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate the velocity head in the outfall (Gen. Instr. 2).
3. Calculate the size ratios D_U/D_O and A/D_U (Gen. Instr. 3).
4. Note whether the outfall pipe entrance is to be square-edged or rounded smooth (note Gen. Instr. 6).
5. Enter Figure 4.8 at the pipe size ratio D_U/D_O and read K_U at the curve for the proper value of A/D_U for a square-edged entrance condition, or at the dashed curve for a rounded entrance.
6. Calculate h_U (positive or negative) as indicated on the diagrams on Figure 4.8 and by Gen. Instr. 7.
7. Add a positive h_U to (or subtract the absolute value of a negative h_U from) the elevation of the outfall pressure at the junction center to obtain the elevation of the upstream pipe pressure line at the same location.
8. The water surface elevation in the junction corresponds to that of the upstream pipe, whether above or below the outfall pressure line.
9. Check to be sure the water surface elevation in the junction is at least 6 inches below the top of the junction box so that overflow may not occur.

Comments: For a square-edged entrance to the outfall pipe, values of A/D_U less than 1 do not appreciably reduce the values of K_U shown for $A/D_U = 1$. K_U increases for distances A/D_U greater than 3, but such values are not usual in storm drain construction. For rounded entrances, the curve shown will apply with sufficient accuracy for all values of A/D_U up to 3.

4.2.4.3 Rectangular Manhole—Through Pipeline—with Inlet Flow: Coefficients for pressure changes are presented in Figure 4.9 for use in determining the common elevation of the upstream in-line pipe pressure line and the water surface in the manhole. The in-line pipes connect at the manhole sides (long dimension) and must meet the alignment requirement stated for Figure 4.9. As much as half the total flow may enter through a top inlet. The main graph of Figure 4.9 includes effects of various portions of grate flow for a relative water depth d/D_O of 2.5. Increments of K_U for other relative water depths are shown in the supplemental graphs; positive increments for d/D_O less than 2.5 and negative for greater depths.

To use Figure 4.9:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate velocity head in the outfall (Gen. Instr. 2).
3. Calculate the ratios of D_U/D_O and Q_U/Q_O (Gen. Instr. 3). (The inlet flow ratio $Q_G/Q_O = 1 - Q_U/Q_O$).
4. Estimate a value for the water depth, d.
   - » Follow Gen. Instr. 4.
   - » Estimate $K = 3\ Q_G/Q_O$.
5. Calculate the corresponding relative water depth d/D_O.

6. If the estimated d/D_O is approximately 2.5, enter the lower graph on Figure 4.9 at the pipe size ratio D_U/D_O and read K_U at the curve of interpolated curve for Q_U/Q_O: then proceed as in Step 9.
7. If the estimated d/D_O is other than 2.5, follow Step 6, then enter the upper graph on Figure 4.9 at the given D_U/D_O and determine the increment of K_U required to account for the effects of the estimated relative water depth d/D_U.
8. Add K_U from Step 6 and the increment from Step 7 to determine the total value of K_U. Note that negative values of K_U may occur.
9. For a rounded outfall pipe entrance or one consisting of a pipe socket, reduce K_U according to Gen. Instr. 6.
10. Calculate h_U as indicated on the diagram on the Figure and by Gen. Instr. 7.
11. Add h_U to the elevation of the outfall pressure line at the inlet center to obtain a more precise value for the water depth, d.
12. Repeat the above procedure with the improved value of d from Step 11 if necessary. Such repetition may not be necessary if the original estimated d/D_O of Step 5 was reasonably accurate.
13. Check to be sure the water elevation is at least 6 inches below the gutter elevation at the inlet so that inflow may be admitted.

4.2.4.4 Rectangular Manhole—Upstream Main and 90 Degree Lateral Pipe—with or without Inlet Flow: Pressure change coefficients are presented in Figure 4.10 for use in determining the common elevation of the two upstream pipe pressure lines and the water surface in the manhole. Flow into the combination inlet and junction box is supplied by an upstream main, in-line with the outfall and flowing through the short dimension of the manhole, and a 90 degree lateral pipe connected at one end of the box, supplemented by flow through a top inlet. The main graph of Figure 4.10 applies directly for no flow into the manhole through the inlet. Increments of K_U and K_L for inlet flow conditions are shown in the supplementary graphs of the upper portion of the figure.

To use Figure 4.10:

1. Determine the outfall pipe pressure line elevation (Gen. Instr. 1).
2. Calculate the velocity head in the outfall (Gen. Instr. 2).
3. Calculate the ratios D_U/D_O, Q_U/Q_O (Gen. Instr. 3).
4. If no inlet flow is involved, enter the lower graph on Figure 4.10 at the pipe size ratio D_U/D_O and read K_U (or K_L) at the curve or interpolated curve for Q_U/Q_O, then proceed as in Step 10.
5. With inlet flow, estimate a value for the water depth, d.
   - » Follow Gen. Instr. 4
   - » Estimate K = 1.5
6. Calculate the corresponding relative water depth d/D_O.
7. Enter the lower graph and obtain K_U (or K_L) as in Step 4, this value applying for $Q_G/Q_O = 0$.

PROJECT: OCCC EG=1198.02 SUBMIT 7-94 BEGIN STA. 11+00
DESIGNER: RHF 8SD594.DAT

CD	L2	MAX Q	ADJ Q	LENGTH	FL 1	FL 2	CTL/TW	D	W	S	KJ	KE	KM	LC	L1	L3	L4	A1	A3	A4	J	N
8	1						1196.10															
2	2	2264.0	2264.0	810.00	1180.04	1181.98	1197.90	120.	216.	3	0.00	0.00	0.03	1	3	0	0	0.	0.	0.	0.00	0.013
2	3	2264.0	2264.0	807.00	1181.98	1183.92	1199.40	120.	216.	3	0.00	0.00	0.18	0	4	0	0	0.	0.	0.	0.00	0.013
2	4	2224.0	2224.0	640.23	1183.92	1185.46	1201.21	120.	216.	3	0.00	0.00	0.18	0	5	0	0	0.	0.	0.	0.00	0.013
2	5	2149.0	2149.0	354.94	1185.46	1186.31	1202.00	120.	216.	3	0.00	0.00	0.07	0	6	0	0	0.	0.	0.	0.00	0.013
2	6	2149.0	2149.0	344.83	1186.31	1187.14	1202.19	120.	216.	3	0.00	0.00	0.17	0	7	0	0	0.	0.	0.	0.00	0.013
2	7	2149.0	2149.0	969.20	1187.14	1189.46	1205.20	120.	216.	3	0.00	0.00	0.17	0	8	0	0	0.	0.	0.	0.00	0.013
2	8	2007.0	2007.0	490.73	1189.46	1190.64	1206.40	120.	216.	3	0.00	0.00	0.00	0	9	0	0	0.	0.	0.	0.00	0.013
2	9	2007.0	2007.0	158.39	1190.64	1191.02	1206.80	120.	216.	3	0.00	0.00	0.00	0	10	0	0	0.	0.	0.	0.00	0.013
2	10	2007.0	2007.0	579.68	1191.02	1192.41	1208.08	120.	216.	3	0.00	0.00	0.02	0	11	0	0	0.	0.	0.	0.00	0.013
2	11	2007.0	2007.0	182.50	1192.41	1192.85	1208.65	120.	216.	3	0.00	0.00	0.01	0	12	0	0	0.	0.	0.	0.00	0.013
2	12	1900.0	1900.0	699.82	1192.85	1194.53	1210.24	120.	216.	3	0.00	0.00	0.00	0	13	0	0	0.	0.	0.	0.00	0.013
2	13	1900.0	1900.0	362.68	1194.53	1195.40	0.00	120.	216.	3	0.00	0.00	0.00	0	14	0	0	0.	0.	0.	0.00	0.013
2	14	1900.0	1900.0	163.93	1195.40	1196.15	1210.50	120.	216.	3	0.00	0.00	0.06	0	15	0	0	0.	0.	0.	0.00	0.013
2	15	1900.0	1900.0	36.07	1196.15	1196.32	0.00	120.	216.	3	0.00	0.00	0.00	0	16	0	0	0.	0.	0.	0.00	0.013
2	16	1900.0	1900.0	25.00	1196.32	1196.44	0.00	120.	144.	3	0.00	0.00	0.47	0	17	0	0	0.	0.	0.	0.00	0.013
2	17	1900.0	1900.0	575.77	1196.44	1199.08	1218.72	120.	144.	3	0.00	0.00	0.02	0	18	0	0	0.	0.	0.	0.00	0.013
2	18	1390.0	1390.0	599.73	1199.08	1201.84	1222.05	120.	144.	3	0.00	0.00	0.05	0	19	0	0	0.	0.	0.	0.00	0.013
2	19	1390.0	1390.0	159.50	1201.84	1202.58	1222.22	120.	144.	3	0.00	0.00	0.02	0	20	0	0	0.	0.	0.	0.00	0.013
2	20	1375.0	1375.0	603.06	1202.58	1205.35	1227.17	120.	144.	3	0.00	0.00	0.01	0	21	0	0	0.	0.	0.	0.00	0.013
2	21	1210.0	1210.0	36.94	1205.35	1205.52	0.00	120.	144.	3	0.00	0.00	0.00	0	22	0	0	0.	0.	0.	0.00	0.013
2	22	1210.0	1210.0	332.72	1205.52	1208.38	1230.00	120.	144.	3	0.00	0.00	0.03	0	23	0	0	0.	0.	0.	0.00	0.013
2	23	1205.0	1205.0	67.28	1208.38	1208.96	0.00	120.	144.	3	0.00	0.00	0.00	0	24	0	0	0.	0.	0.	0.00	0.013
2	24	1205.0	1205.0	10.00	1208.96	1209.05	0.00	120.	120.	3	0.00	0.00	0.18	0	25	0	0	0.	0.	0.	0.00	0.013
2	25	1205.0	1205.0	1015.00	1209.05	1217.78	1243.31	120.	120.	3	0.00	0.00	0.00	0	26	0	0	0.	0.	0.	0.00	0.013

why trans. losses has calculated for 1390 cfs?

why trans. losses has cal for 1180 cfs

PROJECT: OCCC EG=1198.02 SUBMIT 7-94 BEGIN STA. 11+00
DESIGNER: RHF 8SD594.DAT

CD	L2	MAX Q	ADJ Q	LENGTH	FL 1	FL 2	CTL/TW	D	W	S	KJ	KE	KM	LC	L1	L3	L4	A1	A3	A4	J	N
2	26	1205.0	1205.0	558.00	1217.78	1222.57	1242.50	120.	120.	3	0.00	0.00	0.00	0	27	0	0	0.	0.	0.	0.00	0.013
2	27	1180.0	1180.0	67.00	1222.57	1223.15	0.00	120.	120.	1	0.00	0.50	0.00	0	0	0	0	0.	0.	0.	0.00	0.013

PROJECT: OCCC EG=1198.02 SUBMIT 7-94 BEGIN STA. 11+00
DESIGNER: RHF 8SD594.DAT

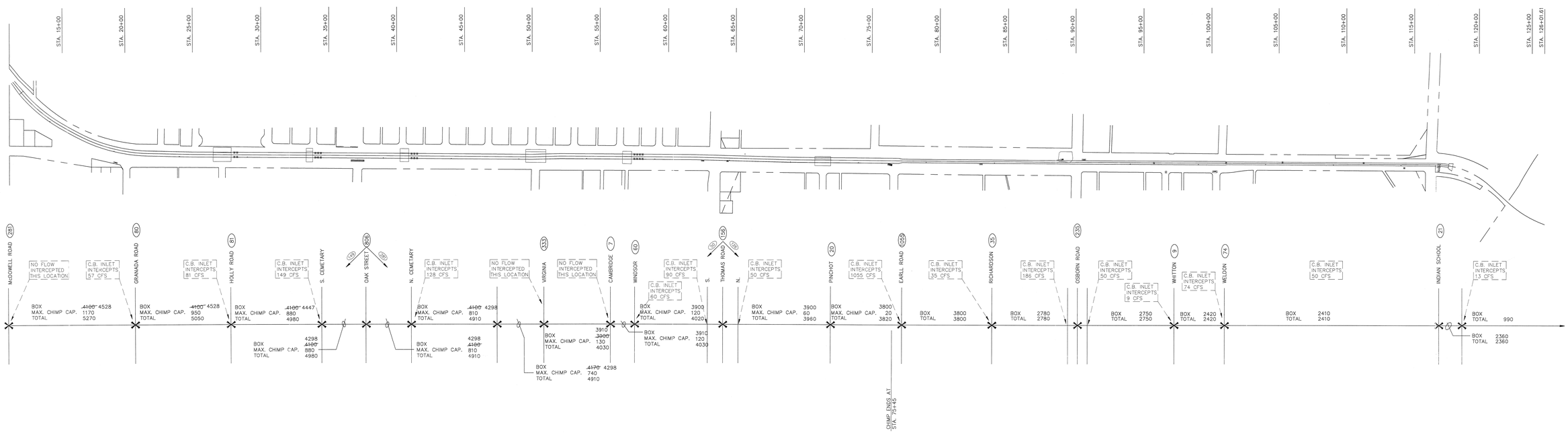
LINE NO	Q (CFS)	D (IN)	W (IN)	DN (FT)	DC (FT)	FLOW TYPE	SF-FULL (FT/FT)	V 1 (FPS)	V 2 (FPS)	FL 1 (FT)	FL 2 (FT)	HG 1 CALC	HG 2 CALC	D 1 (FT)	D 2 (FT)	TW CALC	TW CK	REMARKS
1	HYDRAULIC GRADE LINE CONTROL = 1196.10																	
2	2264.0	120	216	8.43	7.88	FULL	0.00255	12.6	12.6	1180.04	1181.98	1196.10	1198.24	16.06	16.26	0.00	0.00	
3	2264.0	120	216	8.42	7.88	FULL	0.00255	12.6	12.6	1181.98	1183.92	1198.24	1200.74	16.26	16.82	0.00	0.00	
4	2224.0	120	216	8.31	7.79	FULL	0.00246	12.4	12.4	1183.92	1185.46	1200.92	1202.92	17.00	17.46	0.00	0.00	
5	2149.0	120	216	8.11	7.61	FULL	0.00230	11.9	11.9	1185.46	1186.31	1203.23	1204.20	17.77	17.89	0.00	0.00	
6	2149.0	120	216	8.10	7.61	FULL	0.00230	11.9	11.9	1186.31	1187.14	1204.20	1205.37	17.89	18.23	0.00	0.00	
7	2149.0	120	216	8.11	7.61	FULL	0.00230	11.9	11.9	1187.14	1189.46	1205.37	1207.98	18.23	18.52	0.00	0.00	
8	2007.0	120	216	7.70	7.27	FULL	0.00201	11.1	11.1	1189.46	1190.64	1208.54	1209.53	19.08	18.89	0.00	0.00	
9	2007.0	120	216	7.71	7.27	FULL	0.00201	11.1	11.1	1190.64	1191.02	1209.53	1209.85	18.89	18.83	0.00	0.00	
10	2007.0	120	216	7.71	7.27	FULL	0.00201	11.1	11.1	1191.02	1192.41	1209.85	1211.05	18.83	18.64	0.00	0.00	
11	2007.0	120	216	7.69	7.27	FULL	0.00201	11.1	11.1	1192.41	1192.85	1211.05	1211.43	18.64	18.58	0.00	0.00	
12	1900.0	120	216	7.40	7.01	FULL	0.00180	10.6	10.6	1192.85	1194.53	1211.83	1213.09	18.98	18.56	0.00	0.00	
13	1900.0	120	216	7.40	7.01	FULL	0.00180	10.6	10.6	1194.53	1195.40	1213.09	1213.74	18.56	18.34	0.00	0.00	
14	1900.0	120	216	5.87	7.01	FULL	0.00180	10.6	10.6	1195.40	1196.15	1213.74	1214.14	18.34	17.99	0.00	0.00	
15	1900.0	120	216	5.80	7.01	FULL	0.00180	10.6	10.6	1196.15	1196.32	1214.14	1214.21	17.99	17.89	0.00	0.00	
16	1900.0	120	144	8.62	9.19	FULL	0.00504	15.8	15.8	1196.32	1196.44	1212.13	1214.09	15.81	17.65	0.00	0.00	
17	1900.0	120	144	8.78	9.19	FULL	0.00504	15.8	15.8	1196.44	1199.08	1214.09	1217.06	17.65	17.98	0.00	0.00	
18	1390.0	120	144	6.87	7.46	FULL	0.00269	11.6	11.6	1199.08	1201.84	1220.68	1222.40	21.60	20.56	0.00	0.00	
19	1390.0	120	144	6.85	7.46	FULL	0.00269	11.6	11.6	1201.84	1202.58	1222.40	1222.87	20.56	20.29	0.00	0.00	
20	1375.0	120	144	6.82	7.41	FULL	0.00264	11.5	11.5	1202.58	1205.35	1222.96	1224.57	20.38	19.22	0.00	0.00	
21	1210.0	120	144	6.18	6.80	FULL	0.00204	10.1	10.1	1205.35	1205.52	1225.49	1225.57	20.14	20.05	0.00	0.00	
22	1210.0	120	144	4.91	6.80	FULL	0.00204	10.1	10.1	1205.52	1208.38	1225.57	1226.30	20.05	17.92	0.00	0.00	
23	1205.0	120	144	4.89	6.78	FULL	0.00203	10.0	10.0	1208.38	1208.96	1226.32	1226.46	17.94	17.50	0.00	0.00	
24	1205.0	120	120	5.76	7.66	FULL	0.00328	12.1	12.1	1208.96	1209.05	1225.78	1226.21	16.82	17.16	0.00	0.00	

PROJECT: OCCC EG=1198.02 SUBMIT 7-94 BEGIN STA. 11+00
DESIGNER: RHF 8SD594.DAT

LINE NO	Q (CFS)	D (IN)	W (IN)	DN (FT)	DC (FT)	FLOW TYPE	SF-FULL (FT/FT)	V 1 (FPS)	V 2 (FPS)	FL 1 (FT)	FL 2 (FT)	HG 1 CALC	HG 2 CALC	D 1 (FT)	D 2 (FT)	TW CALC	TW CK	REMARKS
25	1205.0	120	120	5.86	7.66	FULL	0.00328	12.1	12.1	1209.05	1217.78	1226.21	1229.54	17.16	11.76	0.00	0.00	
26	1205.0	120	120	5.87	7.66	FULL	0.00328	12.1	12.1	1217.78	1222.57	1227.78	1233.43	10.00	10.86	0.00	0.00	HJ @ UJT
27	1180.0	120	120	5.75	7.55	FULL	0.00314	11.8	11.8	1222.57	1223.15	1233.62	1234.10	11.05	10.95	1236.80	0.00	

V 1, FL 1, D 1 AND HG 1 REFER TO DOWNSTREAM END
V 2, FL 2, D 2 AND HG 2 REFER TO UPSTREAM END
X - DISTANCE IN FEET FROM DOWNSTREAM END TO POINT WHERE HG INTERSECTS SOFFIT IN SEAL CONDITION
X(N) - DISTANCE IN FEET FROM DOWNSTREAM END TO POINT WHERE WATER SURFACE REACHES NORMAL DEPTH BY EITHER DRAWDOWN OR BACKWATER
X(J) - DISTANCE IN FEET FROM DOWNSTREAM END TO POINT WHERE HYDRAULIC JUMP OCCURS IN LINE
F(J) - THE COMPUTED FORCE AT THE HYDRAULIC JUMP
D(BJ) - DEPTH OF WATER BEFORE THE HYDRAULIC JUMP (UPSTREAM SIDE)
D(AJ) - DEPTH OF WATER AFTER THE HYDRAULIC JUMP (DOWNSTREAM SIDE)
SEAL INDICATES FLOW CHANGES FROM PART TO FULL OR FROM FULL TO PART
HYD JUMP INDICATES THAT FLOW CHANGES FROM SUPERCRITICAL TO SUBCRITICAL THROUGH A HYDRAULIC JUMP
HJ @ UJT INDICATES THAT HYDRAULIC JUMP OCCURS AT THE JUNCTION AT THE UPSTREAM END OF THE LINE
HJ @ DJT INDICATES THAT HYDRAULIC JUMP OCCURS AT THE JUNCTION AT THE DOWNSTREAM END OF THE LINE

EOJ 7/22/1994 16:17



GENERAL NOTES:

1.) THIS MAP IS A GRAPHICAL REPRESENTATION OF THE FLOW SPLITS AND INTERCEPTED FLOWS. THE LATERAL SURFACE FLOWS ARE FOR THE GENERAL STORM. THE INTERCEPTED FLOWS ARE BASED ON THE TOTAL NUMBER OF GRATES PROVIDED FOR THE LOCAL STORM WITHOUT CLOGGING AND THE HYDRAULIC GRADE LINE IN CHIMP FOR THE GENERAL STORM. THIS MAP GRAPHICALLY REPRESENTS THE WAY THE STORM PROGRAM DATA WAS ENTERED INTO FILE 8SD594.DAT.

GRAPHICAL REPRESENTATION OF GENERAL STORM w/ADDITIONAL INLETS

MAP FOR FILE 8SD594.DAT

CALCULATIONS FOR MAXIMUM SPACING OF INLETS FOR THE TWO-YEAR STORM

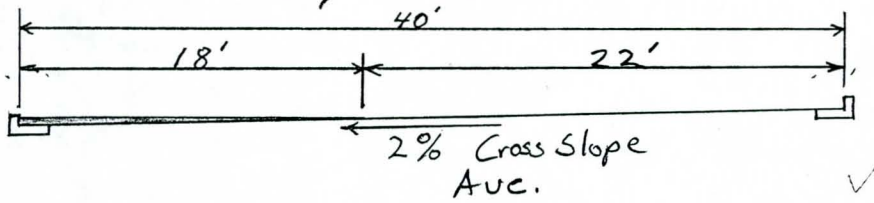
Greiner

Job No. F002102 Project OCCC Sheet 1 of

Subject Curb openings along 48TH ST - 2YR STorm

By JVG Date 6/21/94 Checked By N. W. Date 7-22-94

Determine Spacing & opening length of Curb Cuts. to Maintain 2 - 11' Dry lanes.



Location	Ave. Longitudinal Slope
Earll to Osborn	0.7 %
Osborn to I.S.R.	0.9 %

This may change

Determine Q & depth of flow in 48TH ST. For Max Spread of 18' - Compound MAG Curb & Gutter

Location	Q	Depth
Earll to Osborn	10.5 CFS	0.42'
Osborn to I.S.R.	11.9 CFS	0.42'

Determine length of Curb Opening Req. for 100% Flow.

Weir Eq. without depression. $Q_i = C_w L d^{1.5}$ $C_w = 3.0$ (Pg 3-18 Drainage Design Manual - M.C. Hydraulics VOL II)

$$L = \frac{Q_i}{C_w d^{1.5}}$$

$$L = \frac{10.5}{(3.0)(0.42)^{1.5}} = 12.85' \quad \text{use } \underline{13'}$$

$$L = \frac{11.9}{(3.0)(0.42)^{1.5}} = 14.57' \quad \text{use } \underline{15'}$$

length of opening needed

Greiner

Job OCCC | Project No. E002102 | Sheet 2 of
Description Curb Openings | Computed By JUG | Date 6/21/94
Checked By N.W | Date 7-22-94

C U R B   A N D   G U T T E R   F L O W   C A L C U L A T I O N S
DDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDD
Location: 48TH STREET - EARLL DRIVE TO OSBORN ROAD
Compound section geometry data:
  Manning's n = .015 ✓
  Gutter width = 1.416667 feet ✓
  Cross slope = .02 ft/ft ✓
  Slope of Curb face = 0 Horiz : 1 Vert
  Gutter drop = 1 inch absolute ✓
  Roadway Slope = .007 ft/ft ✓
Calculate flow capacity of section:
  Flow = 10.46933 cfs ✓
  Pvmt Spread = 18 feet ✓
  depth of flow = .415 feet ✓
  Average Velocity = 3.192884 fps ✓
DDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDD

C U R B   A N D   G U T T E R   F L O W   C A L C U L A T I O N S
DDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDD
Location: 48TH STREET - OSBORN ROAD TO INDIAN SCHOOL ROAD
Compound section geometry data:
  Manning's n = .015
  Gutter width = 1.416667 feet
  Cross slope = .02 ft/ft
  Slope of Curb face = 0 Horiz : 1 Vert
  Gutter drop = 1 inch absolute
  Roadway Slope = .009 ft/ft
Calculate flow capacity of section:
  Flow = 11.87111 cfs ✓
  Pvmt Spread = 18 feet ✓
  depth of flow = .415 feet ✓
  Average Velocity = 3.62039 fps ✓
DDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDDD

W/ Pvmt Spread of 18'

Greiner

Job No. E002102 Project OCCC Sheet 3 of

Subject Curb Openings

By JUG Date 6/21/94 Checked By N.W. Date 7-22-94

Determine length of Roadway to Produce a Q w/18' spread.

Rational Method $Q = CiA$

REF: Drainage Design Manual Maricopa Cnty. - Volume I Hydrology - Section 3

$C = 0.85$ $i = 3.8$ in/Hr. ($T_c = 5$ min)

$Q = CiA = Ci\left(\frac{L \times W}{43{,}560'}\right)$ ($W = 41' + 12' = 53'$ (Road) (East R/W) use $55'$ Ave)

$L = \frac{Q\ 43560}{C\ i\ W}$

$L = \frac{(0.5)(43560)}{(0.85)(3.8)(55')} = 2{,}574.61'$ ✓

$L = \frac{(11.9)(43{,}560)}{(0.85)(3.8)(55')} = 2{,}917.89'$ ✓

Hydraulics Manual (Vol. II) pg. 3-15

For continuous Grade Conditions (3.3.2.2) use 660' Max spacing for collection of nuisance flows.

$Q = (0.85)(3.8)\frac{(660)(55')}{43{,}560} = 2.70$ CFS ✓

Determine Length of Curb opening for 2.70 CFS (Complete interception)

$L = \frac{Q_i}{C_w\ d^{1.5}}$

$L = \frac{2.70}{(3.0)(0.28)^{1.5}} = 6.07'$ ✓

$L = \frac{2.70}{(3.0)(0.26)^{1.5}} = 6.79'$ ✓

Use a 3' Curb Opening To eliminate the need for Bollards.

Inlet spacing along the park swale invert shall Corespond with the 660' Opening Spacing.

Ron will chk

Greiner

Job No. E002102 Project OCCC Sheet 4 of

Subject Curb Openings

By JVG Date 6/21/94 Checked By N.W. Date 7-22-94

Check 2-year Q for R/W corridor for the 660' spacing using $i = 3.8$ in/hr ($T_c = 5$ min)

$$Q = CiA \qquad C = \underset{\text{Pvmt}}{(0.85}\times\underset{28\%}{.28)} + \underset{\text{Parks}}{(0.25}\times\underset{72\%}{.72)} = 0.42$$

Should total 100 CHK. 0.35 OK!

Earll To Osborn

Ave R/W Width $= \frac{210 + 205}{2} = 207.8$ use 208

$$Q = (0.42)(3.8)\left(\frac{208 \times 660}{43{,}560}\right) = 5.03 \text{ CFS}$$

0.35 4.14 CFS OK

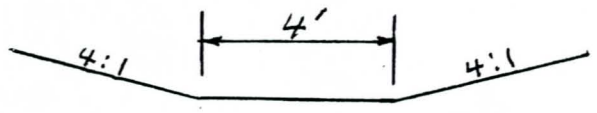
Osborn To ISR

Ave R/W Width $= \frac{93 + 118 + 143 + 182}{4} = 134$ use 135

$$Q = (0.42)(3.8)\left(\frac{135 \times 660}{43{,}560}\right) = 3.26 \text{ CFS}$$

2.72 cfs OK

Check depth of flow for both reaches



18 22 40 $\frac{18}{40}$, $\frac{22}{40}$ 0.45, 0.55

Earll To Osborn → $d_n = 0.40'$ $U = 2.3$ FPS

Orsborn to ISR → $d_n = 0.29$ $U = 2.2$ FPS.

Design swale min. 0.5' deep

DRAINAGE SWALE - EARLL TO OSBORN
CHANNEL HYDRAULICS CHARACTERISTICS

```
**                          **
 **-------------------------**
  **          0.40          **
   **       FT DEEP        **
    **                    **
     ******************
```

Q=	5.03	cfs	
S=	0.007	ft/ft	
n=	0.025		
b=	4	ft	
z=	4	:1	
TRIAL DEPTH=	0.40	ft	

CALCULATED

FLOW=	5.03	CFS	CRITICAL DEPTH=	0.33	FT
NORMAL DEPTH=	0.40	FT	VELOCITY=	2.9	FPS
VELOCITY=	2.3	FPS	SLOPE=	0.0145	FT/FT
FROUDE #=	0.6				
TOP WIDTH WS=	7.2	FT			

VALUES = FOR CRITICAL DEPTH

5.20	5.20

-Jun-94 11:10 AM NUM CAPS

DRAINAGE SWALE - OSBORN TO INDIAN SCHOOL
CHANNEL HYDRAULICS CHARACTERISTICS

```
**                          **                Q=        3.26 cfs
 **----------------------**                   S=       0.009 ft/ft
  **          0.29       **                   n=       0.025
   **      FT DEEP     **                     b=           4 ft
    **                **                      z=           4 :1
     ****************                TRIAL DEPTH=       0.29 ft

      CALCULATED
     FLOW=     3.26 CFS          CRITICAL  DEPTH=       0.25 FT
NORMAL DEPTH=  0.29 FT              VELOCITY=           2.6 FPS
   VELOCITY=    2.2 FPS               SLOPE=        0.0156 FT/FT
   FROUDE #=    0.7
TOP WIDTH WS=   6.3 FT
                                  VALUES = FOR CRITICAL DEPTH
                                          1.98        1.98
Jun-94  11:09 AM                                      NUM CAPS
```

✓

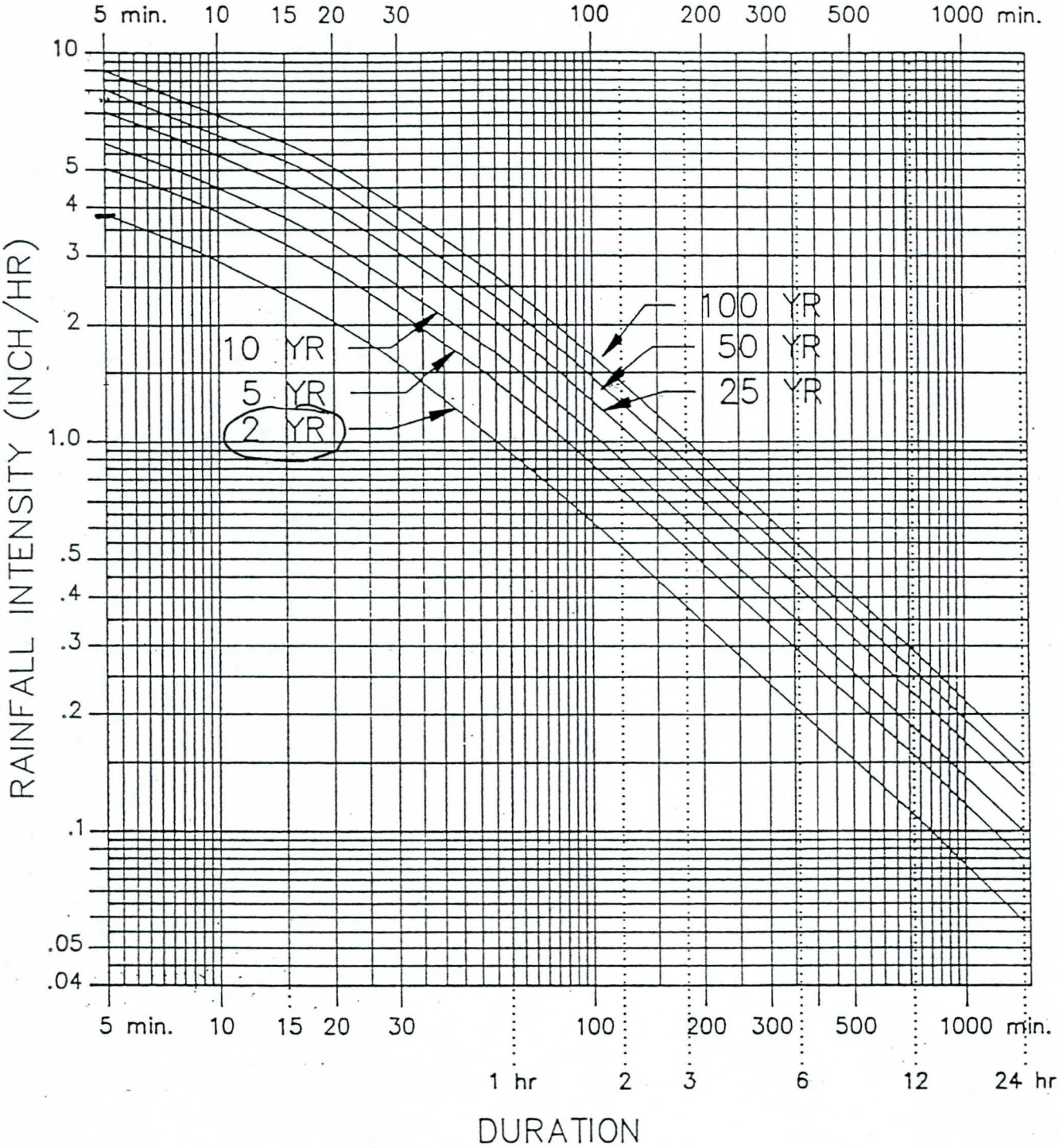


Figure 3.2
Rainfall Intensity-Duration-Frequency Relation
(Phoenix Metro Area)

Table 3.2
C Coefficients for Use with the Rational Method

Land Use	Return Period			
	2-10 Year	25 Year	50 Year	100 Year
Streets and Roads				
Paved Roads	0.75 – 0.85	0.83 – 0.94	0.90 – 0.95	0.94 – 0.95
Gravel Roadways & Shoulders	0.60 – 0.70	0.66 – 0.77	0.72 – 0.84	0.75 – 0.88
Industrial Areas				
Heavy	0.70 – 0.80	0.77 – 0.88	0.84 – 0.95	0.88 – 0.95
Light	0.60 – 0.70	0.66 – 0.77	0.72 – 0.84	0.75 – 0.88
Business Areas				
Downtown	0.75 – 0.85	0.83 – 0.94	0.90 – 0.95	0.94 – 0.95
Neighborhood	0.55 – 0.65	0.61 – 0.72	0.66 – 0.78	0.69 – 0.81
Residential Areas				
Lawns – Flat	0.10 – 0.25	0.11 – 0.28	0.12 – 0.30	0.13 – 0.31
– Steep	0.25 – 0.40	0.28 – 0.44	0.30 – 0.48	0.31 – 0.50
Suburban	0.30 – 0.40	0.33 – 0.44	0.36 – 0.48	0.38 – 0.50
Single Family	0.45 – 0.55	0.50 – 0.61	0.54 – 0.66	0.56 – 0.69
Multi-Unit	0.50 – 0.60	0.55 – 0.66	0.60 – 0.72	0.63 – 0.75
Apartments	0.60 – 0.70	0.66 – 0.77	0.72 – 0.84	0.75 – 0.88
Parks/Cemetaries	0.10 – 0.25	0.11 – 0.28	0.12 – 0.30	0.13 – 0.31
Playgrounds	0.40 – 0.50	0.44 – 0.55	0.48 – 0.60	0.50 – 0.63
Agricultural Areas	0.10 – 0.20	0.11 – 0.22	0.12 – 0.24	0.13 – 0.25
Bare Ground	0.20 – 0.30	0.22 – 0.33	0.24 – 0.36	0.25 – 0.38
Undeveloped Desert	0.30 – 0.40	0.33 – 0.44	0.36 – 0.48	0.38 – 0.50
Mountain Terrain (Slopes > 10%)	0.60 – 0.80	0.66 – 0.88	0.72 – 0.95	0.75 – 0.95

Note: Values of C for 25, 50 and 100 Year were derived using frequency adjustment factors of 1.10, 1.20, and 1.25, respectively, with an upper limit of 0.95 for C for the 2-10 Year values.

LATEST GREINER HEC-2

```
T1       Old Cross Cut Canal - CHIMP FILE: CMPMDL22.DAT: n=0.045
T2       Preliminary Model with 4:1 Side Slopes, 20' Notch 20+00-11+00
T3       (6:1 Side Slope-East,4:1 Side Slope-West)
T4       Analysis for Stations 75+00 to 0+00 with weir flow @ Thomas Rd.
T5       CHIMP Analysis for Design Discharges (Draft Letter Q to FDCMC 10-7-93)

T6       Some of the cross sections are revised. McDowell Rd. weir = 1194.0
T7       Q @ 5270
J1     0        2        0        0  .00532        0        0        0      180        0
J2     1        0       -1        0        0        0       -1        0        0        0
J3    38       43       39        4       25       26       23       24        1       42
J3     3
J5   -10      -10
NC  .035     .035     .015       .1       .3
QT     1     5270
X1     0        7   999.99  1016.01        0        0        0
X3    10                     1000                1016
GR184.67   950.00   184.67   999.99   170.00  1000.00   169.67  1008.00   170.00  1016.00
GR184.67  1016.01   184.67  1066.00
X1   384        7   999.99  1016.01      374      394      384
X3    10
GR186.71   950.00   186.71   999.99   172.04  1000.00   171.71  1008.00   172.04  1016.00
GR186.71  1016.01   186.71  1066.00
X1   434        7   999.99  1016.01       49       51       50
X3    10
GR188.36   950.00   188.36   999.99   172.31  1000.00   171.98  1008.00   172.31  1016.00
GR188.36  1016.01   188.36  1066.00
X1   484        7   999.99  1016.01       49       51       50
X3    10
GR190.00   950.00   190.00   999.99   173.13  1000.00   172.80  1008.00   173.13  1016.00
GR190.00  1016.01   190.00  1066.00
NC                       3       .5
X1   684        7   999.99  1026.01      200      200      200
GR190.00   950.00   190.00   999.99   176.50  1000.00   176.07  1013.00   176.50  1026.00
GR190.00  1026.01   190.00  1076.00
X1   726        7     1000     1026       42       42       42
X3    10                                                    189      189
GR192.00   950.00   189.00   999.99   177.11  1000.00   177.11  1013.00   177.11  1026.00
GR189.00  1026.01   192.00  1076.00
* The discharges d/s of McDowell Road are 500, 750, 1000 cfs less than the u/s
*    discharges for 25, 50, 100-yr event, respectively.  This is due to the
*    assumption that those amount of discharges do not join the d/s channel
*    flowing west along the McDowell Road.
SC 2.014        2      2.6                10       12      274      9.2   179.85   177.11
X1  1000       10      987     1013      274      274      274
X2                       2   189.85
X3    10                                                    194      194
BT    -8      966   194.00              972   194.00              975   194.00
BT            985   194.00             1027   194.00             1041   194.00
BT           1070   194.00             1116   194.00
GR   193      960      193      966      194   984.99   193.87      985   179.85      987
GR179.85     1013   193.87     1027      194  1027.01    193.9     1041    193.9     1070
NC  .035     .035     .045       .1       .3
QT     1     1170
X1  1099        9   928.0  1028.00       99       99       99
GR195.20   900.00   195.20   928.00   189.79      960   189.79   982.00   180.00    982.0
GR180.00     1000   180.00     1018   189.80   1018.1   195.20     1028
X1  1100        8    911.0   1036.5        1        1        1
GR 195.3      905    195.3    911.0    189.9    947.0    189.9      980    192.2    980.1
GR 192.2     1000    192.2   1018.0    194.8   1036.5
X1  1200        8   905.00   1042.1       92       85       90
GR 195.8      905   190.00   940.00   190.00   960.00   194.03    975.1   194.03     1000

GR194.03     1042   196.00  1042.10   195.00  1059.00
X1  1300        9      894  1042.10      104       96      100
GR 196.1      894   190.27   939.00   190.27   959.00   194.27    975.1   194.27     1000
GR194.27     1042    197.3   1042.1    195.8     1050    196.1     1058
X1  1400        8      899   1039.6      107       96      100
GR 196.3      899    190.5      940    190.5      960    194.5    975.1    194.5     1000
GR 194.5   1039.5    197.9   1039.6    196.6   1055.6
X1  1500        9      895   1039.6      107       96      100
GR 198.2      895    190.7      940    190.7      960    194.7    975.1    194.7     1000

GR 194.7   1039.5    197.2   1039.6    197.2   1044.0    197.2   1055.6
X1  1600        8   906.00  1037.10      104       98      100
GR199.40   906.00   192.20   945.00   192.20   965.00   194.80   975.10   194.80     1000
GR194.80   1037.0   198.60  1037.10   198.60  1053.00
X1  1700        8    940.0   1041.1      105       96      100
GR 201.5      940    193.6      959    193.6      975   195.93    975.1   195.93     1000
GR195.93     1041    200.5   1041.1    199.2     1057
X1  1800        8      942   1044.1      105       96       99
GR 202.3      942    195.1      964    195.1    970.3    196.7    976.1    196.7     1000
GR 196.7     1044    200.7   1044.1    200.7     1060
QT     1      950
X1  1900        9    949.0   1050.1      104       96      100
GR 203.0      925    203.5    949.0    196.5    969.5    196.5    973.0    197.3    977.0
GR 197.3     1000    197.3     1050    201.5   1050.1    199.7     1063
X1  2000        8      957   1050.1      104       96      100
GR 204.0      940    204.0      949    203.3      957    197.9      978    197.9     1000
GR 197.9     1050    201.7   1050.1    200.9     1066
X1  2100        8      959     1051      104       95      100
GR 203.4      955    203.4      959   198.19      981   198.19     1000   198.19     1035

GR 202.0     1051    201.4     1053    200.8     1070
X1  2200        7      951     1054      102       97      100
GR 204.3      944    204.0      951   198.43      973   198.43     1000   198.43     1037
GR 202.4     1054    200.9     1070
X1  2300        9      951     1056      100      100      100
GR 204.0      922    204.3      945    204.0      951    198.6      973    198.6     1000
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GR 198.4	1041	202.0	1056	201.0	1065	200.7	1072		
X1 2400	6	947	1045	100	100	100			
GR 204.0	947	198.9	970	198.9	1000	198.9	1026.5	202.9	1045
GR 202.8	1050								
X1 2500	7	946	1054	100	100	100			
GR 204.1	939.99	203.8	946	199.2	967	199.2	1000	199.2	1014
GR 203.8	1028	203.5	1054						
X1 2600	7	950	1055	100	100	100			
GR 203.1	940	203.0	950	199.2	966	199.2	1000	199.2	1040
GR 202.4	1055	202.2	1058						
QT 1	880								
X1 2700	7	947	1054	100	100	100			
GR 204.0	940	203.8	947	199.4	966	199.4	1000	199.4	1041
GR 202.4	1054	201.8	1061						
X1 2800	7	948	1051	100	100	100			
GR203.98	940	204.0	948	199.6	968	199.6	1000	199.6	1034
GR 203.4	1051	202.0	1060						
X1 2900	7	950	1052	100	100	100			
GR 204.0	940	204.0	950	200.1	969	200.1	1000	200.1	1034
GR 203.8	1052	201.6	1060						
X1 3000	7	952	1049	100	100	100			
GR 204.0	940	204.0	952	200.3	970	200.3	1000	200.3	1031
GR 204.1	1049	203.0	1054						
X1 3100	7	951	1051	100	100	100			
GR 203.5	940	203.5	951	200.5	967	200.5	1000	200.5	1035
GR 204.0	1051	202.4	1063						
X1 3200	6	950	1049	100	100	100			
GR 204.0	950	200.8	966	200.8	1000	200.8	1033	204.0	1049
GR 203.4	1055								
X1 3300	7	952	1051	100	100	100			
GR 205.0	950	205.0	952	201.1	970	201.1	1000	201.1	1036
GR 204.2	1051	203.4	1055						
X1 3400	7	953.5	1051	100	100	100			
GR 204.5	950	204.4	953.5	201.3	968	201.3	1000	201.3	1032
GR 204.6	1051	203.5	1054						
QT 1	810								
X1 3500	6	953	1052	100	100	100			
GR 205.0	953	201.5	969	201.5	1000	201.5	1035	205.0	1052
GR 204.0	1055								
X1 3600	7	954	1042.5	100	100	100			
GR 205.0	950	205.0	954	201.8	969	201.8	1000	201.8	1028
GR 204.7	1042.5	203.5	1055						
X1 3700	7	957	1043	100	100	100			
GR 205.0	950	205.0	957	202.0	971	202.0	1000	202.0	1032
GR 205.7	1043	204.0	1055						
X1 3800	7	957	1047	100	100	100			
GR 205.0	950	205.0	957	202.3	971	202.3	1000	202.3	1036
GR 204.4	1047	203.0	1056						
X1 3900	7	958	1045	100	100	100			
GR 205.0	950	205.0	958	202.5	970	202.5	1000	202.5	1036
GR 204.2	1045	203.9	1051						
X1 4000	7	954	1046	100	100	100			
GR 204.6	950	204.1	954	202.8	967	202.8	1000	202.8	1037
GR 205.5	1046	203.0	1055						
X1 4100	7	957	1046	100	100	100			
GR 205.1	950	205.1	957	203.0	968	203.0	1000	203.0	1036
GR 204.8	1046	206.1	1056						
X1 4200	7	958	1047	100	100	100			
GR 203.7	950	204.8	958	203.2	969	203.2	1000	203.2	1038
GR 205.0	1047	205.0	1056						
X1 4300	7	959	1047	100	100	100			
GR 205.0	950	205.0	959	203.5	968	203.5	1000	203.5	1038
GR 204.8	1047	203.8	1056						
X1 4400	7	958	1044	100	100	100			
GR 205.0	950	205.0	958	203.7	967	203.7	1000	203.7	1037
GR 204.7	1044	203.8	1056						
X1 4500	7	960	1042	100	100	100			
GR 205.3	950	205.1	960	204.0	967	204.0	1000	204.0	1035
GR 205.1	1042	206.4	1051						
X1 4600	7	958	1041	100	100	100			
GR 206.0	950	206.0	958	204.0	968	204.0	1000	204.2	1032
GR 205.3	1041	203.9	1054						
QT 1	740								
X1 4700	7	961	1041	100	100	100			
GR 204.0	950	205.4	961	204.4	971	204.4	1000	204.4	1034
GR 205.1	1041	205.1	1057						
X1 4800	7	966	1044	100	100	100			
GR 206.1	950	205.4	966	204.7	970	204.7	1000	204.7	1039
GR 206.0	1044	207.8	1053						
X1 4900	7	962	1045	100	100	100			
GR 206.2	950	206.2	962	204.9	970	204.9	1000	204.9	1039
GR 205.4	1045	204.8	1057						
X1 5000	6	964	1051	100	100	100			
GR 206.3	950	206.3	964	205.2	968	205.2	1000	205.2	1042
GR 205.8	1051								
QT 1	130								
X1 5100	6	961	1047.7	100	100	100			
GR 206.2	955	206.2	961	205.4	965	205.4	1000	205.4	1036
GR 206.7	1047.7								
X1 5200	6	950	1043	100	100	100			
GR 207.1	950	205.6	957	205.6	1000	205.6	1038	206.4	1043
GR 204.8	1051								
X1 5300	6	950	1039	100	100	100			
GR 207.2	950	205.9	958	205.9	1000	205.9	1032	206.3	1039
GR 206.6	1050								
X1 5400	7	954	1038	100	100	100			
GR 207.2	950	207.0	954	206.1	961	206.1	1000	206.1	1032
GR 207.0	1038	207.0	1046						
QT 1	120								

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X1   5500       [illegible]      954     1039      100      100      100
GR 207.3      950    207.0      954    206.4      960    206.4     1000    206.4     1033
GR 207.0     1039    207.0     1048
X1   5600        7    952.5     1037      100      100      100
GR 207.9      949    207.9    952.5    206.6      960    206.6     1000    206.6     1033

GR 207.1     1037    207.3     1050
X1   5700        6    949.0     1038      100      100      100
GR 208.8      949    206.8      959    206.8     1000    206.8     1030    207.6     1038
GR 207.5     1050
X1   5800        7      947     1038      100      100      100
GR 209.2      943    208.9      947    207.1      958    207.1     1000    207.1     1032
GR 208.0     1038    208.0     1050
X1   5900        7    954.2     1038      100      100      100
GR 209.1      951    209.1    954.2    207.4      960    207.4     1000    207.4     1032

GR 208.0     1038    208.0     1050
X1   6000        7    960.5     1036      100      100      100
GR 209.2      958    209.2    960.5    207.6      968    207.6     1000    207.6     1028

GR 208.6     1036    208.5     1050
X1   6100        5      972   1024.5      100      100      100
GR 210.8      972   207.84      982   207.84     1000   207.84     1018     210.2    1024.5
X1   6200        5      971   1026.5      100      100      100
GR 210.7      971   208.08      982   208.08     1000   208.08     1018     210.5    1026.5
X1   6300        5      973   1027.5      100      100      100
GR 210.5      973   208.32      982   208.32     1000   208.08     1018     210.7    1027.5
* The following 4 stations (6305,6307,6386 & 6387) are for the weir flow
*    condition at Thomas Road.
X1   6305        5      973   1027.5        5        5        5
GR 210.5      973   208.32      982   208.32     1000   208.08     1018     210.7    1027.5
X1   6307        5      968     1025        2        2        2
GR210.61      968   210.61   989.25   211.22   989.26   211.22   1010.8    210.61     1025
X1   6386        5      968     1025       79       79       79
GR210.61      968   210.61   989.25   211.22   989.26   211.22   1010.8    210.61     1025
X1   6387        5      973     1025        1        1        1
GR 209.4      973   208.50      982   208.50     1000   208.50     1018     209.9     1025
OT      1       60
X1   6400        5      973   1027.3       13       13       13
GR 209.5      973   208.56      982   208.56     1000   208.56     1018     210.0    1027.3
X1   6500        5      972     1025      100      100      100
GR 211.1      972    208.8      982    208.8     1000    208.8     1018     210.6     1025
X1   6600        5      972   1027.5      100      100      100
GR 211.5      972   209.04      982   209.04     1000   209.04     1018     211.5    1027.5
X1   6700        5      974   1027.4      100      100      100
GR 211.3      974   209.28      982   209.28     1000   209.28     1018     211.4    1027.4
X1   6800        5      972   1026.3      100      100      100
GR 211.8      972   209.52      982   209.52     1000   209.52     1018     211.5    1026.3
X1   6900        5    972.5     1028      100      100      100
GR 212.0    972.5   209.76      982   209.76     1000   209.76     1018     212.1     1028
X1   7000        5      971     1028      100      100      100
GR 212.7      971    210.0      982    210.0     1000    210.0     1018     212.5     1028
OT      1       20
X1   7100        5      971     1026      100      100      100
GR 212.8      971   210.24      982   210.24     1000   210.24     1018     212.4     1026
X1   7200        5      971     1026      100      100      100
GR 213.0      971   210.48      982   210.48     1000   210.48     1018     212.6     1026
X1   7300        5      971     1024      100      100      100
GR 213.5      971   210.72      982   210.72     1000   210.72     1018     212.5     1024
X1   7400        5    971.4     1025      100      100      100
GR 213.5    971.4   210.96      982   210.96     1000   210.96     1018     213.0     1025
X1   7500        5      961   1026.5      100      100      100
GR 213.6      961    211.2      970    211.2     1000    211.2     1018     213.2    1026.5
EJ


ER
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1*****************************************
 * HEC-2 WATER SURFACE PROFILES           *
 *                                        *
 * Version   4.6.2;  May 1991             *
 *                                        *
 * RUN DATE    07JUL94    TIME   11:38:03 *
 *****************************************
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*************************************
* U.S. ARMY CORPS OF ENGINEERS      *
* HYDROLOGIC ENGINEERING CENTER     *
* 609 SECOND STREET, SUITE D        *
* DAVIS, CALIFORNIA 95616-4687      *
*          (916) 756-1104           *
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X     X  XXXXXXX   XXXXX              XXXXX
X     X  X       X     X            X     X
X     X  X       X                        X
XXXXXXX  XXXX     X          XXXXX   XXXXX
X     X  X       X                  X
X     X  X       X     X            X
X     X  XXXXXXX   XXXXX            XXXXXXX
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1
   07JUL94      11:38:03                                                                                    PAGE    1


                                                                                THIS RUN EXECUTED 07JUL94    11:38:03
 ************************************
  HEC-2 WATER SURFACE PROFILES

  Version   4.6.2;  May 1991
 ************************************


T1       Old Cross Cut Canal - CHIMP FILE: CMPMDL22.DAT; n=0.045
T2       Preliminary Model with 4:1 Side Slopes, 20' Notch 20+00-11+00
T3       (6:1 Side Slope-East,4:1 Side Slope-West)
T4       Analysis for Stations 75+00 to 0+00 with weir flow @ Thomas Rd.
T5       CHIMP Analysis for Design Discharges (Draft Letter Q to FDCMC 10-7-93)
T6       Some of the cross sections are revised. McDowell Rd. weir = 1194.0
T7       Q @ 5270

J1  ICHECK    INQ       NINV      IDIR      STRT      METRIC    HVINS     Q         WSEL      FQ

       0        2         0         0    .00532         0         0         0       180         0

J2  NPROF     IPLOT     PRFVS     XSECV     XSECH     FN        ALLDC     IBW       CHNIM     ITRACE

       1        0        -1         0         0         0        -1         0         0         0

J3  VARIABLE CODES FOR SUMMARY PRINTOUT

      38        43        39         4        25        26        23        24         1        42
       3

J5  LPRNT     NUMSEC                    ********REQUESTED SECTION NUMBERS********

     -10       -10

NC     .035      .035      .015        .1        .3
OT        1      5270
X1        0         7    999.99   1016.01         0         0         0
X3       10                           1000                  1016
GR   184.67    950.00    184.67    999.99    170.00   1000.00    169.67   1008.00    170.00   1016.00
GR   184.67   1016.01    184.67   1066.00

X1      384         7    999.99   1016.01       374       394       384
X3       10
GR   186.71    950.00    186.71    999.99    172.04   1000.00    171.71   1008.00    172.04   1016.00
GR   186.71   1016.01    186.71   1066.00

X1      434         7    999.99   1016.01        49        51        50
X3       10
GR   188.36    950.00    188.36    999.99    172.31   1000.00    171.98   1008.00    172.31   1016.00
GR   188.36   1016.01    188.36   1066.00
1
   07JUL94      11:38:03                                                                                    PAGE    2


X1      484         7    999.99   1016.01        49        51        50
X3       10
GR   190.00    950.00    190.00    999.99    173.13   1000.00    172.80   1008.00    173.13   1016.00
GR   190.00   1016.01    190.00   1066.00

NC                                     .3        .5
X1      684         7    999.99   1026.01       200       200       200
GR   190.00    950.00    190.00    999.99    176.50   1000.00    176.07   1013.00    176.50   1026.00
GR   190.00   1026.01    190.00   1076.00

X1      726         7      1000      1026        42        42        42
X3       10                                                          189       189
GR   192.00    950.00    189.00    999.99    177.11   1000.00    177.11   1013.00    177.11   1026.00
GR   189.00   1026.01    192.00   1076.00
  The discharges d/s of McDowell Road are 500, 750, 1000 cfs less than the u/s
     discharges for 25, 50, 100-yr event, respectively.  This is due to the
     assumption that these amount of discharges do not join the d/s channel
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assumption that those amount of discharges do not join the d/s channel flowing west along the McDowell Road.

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SC    2.014        .2       2.6                  10        12       274       9.2    179.85    177.11
X1     1000        10       987      1013       274       274       274
X2                              2    189.85
X3       10                                                          194       194
BT       -8       966    194.00                972    194.00                 975    194.00
BT                985    194.00               1027    194.00                1041    194.00
BT               1070    194.00               1116    194.00
GR      193       960       193       966       194    984.99    193.87       985    179.85       987
GR   179.85      1013    193.87      1027       194   1027.01     193.9      1041     193.9      1070

NC     .035      .035      .045        .1        .3
OT        1      1170
X1     1099         9     928.0   1028.00        99        99        99
GR   195.20    900.00    195.20    928.00    189.79       960    189.79    982.00    180.00     982.0
GR   180.00      1000    180.00      1018    189.80    1018.1    195.20      1028

X1     1100         8     911.0    1036.5         1         1         1
GR    195.3       905     195.3     911.0     189.9     947.0     189.9       980     192.2     980.1
GR    192.2      1000     192.2    1018.0     194.8    1036.5

X1     1200         8    905.00    1042.1        92        85        90
GR    195.8       905    190.00    940.00    190.00    960.00    194.03     975.1    194.03      1000
GR   194.03      1042    196.00   1042.10    195.00   1059.00

X1     1300         9       894   1042.10       104        96       100
GR    196.1       894    190.27    939.00    190.27    959.00    194.27     975.1    194.27      1000
GR   194.27      1042     197.3    1042.1     195.8      1050     196.1      1058

X1     1400         8       899    1039.6       107        96       100
GR    196.3       899     190.5       940     190.5       960     194.5     975.1     194.5      1000
GR    194.5    1039.5     197.9    1039.6     196.6    1055.6
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X1     1500         9       895    1039.6       107        96       100
GR    198.2       895     190.7       940     190.7       960     194.7     975.1     194.7      1000
GR    194.7    1039.5     197.2    1039.6     197.2    1044.0     197.2    1055.6

X1     1600         8    906.00   1037.10       104        98       100
GR   199.40    906.00    192.20    945.00    192.20    965.00    194.80    975.10    194.80      1000
GR   194.80    1037.0    198.60   1037.10    198.60   1053.00

X1     1700         8     940.0    1041.1       105        96       100
GR   201.5       940     193.6       959     193.6       975    195.93     975.1    195.93      1000
GR   195.93      1041     200.5    1041.1     199.2      1057

X1     1800         8       942    1044.1       105        96        99
GR   202.3       942     195.1       964     195.1     970.3     196.7     976.1     196.7      1000
GR   196.7      1044     200.7    1044.1     200.7      1060

OT        1       950
X1     1900         9     949.0    1050.1       104        96       100
GR   203.0       925     203.5     949.0     196.5     969.5     196.5     973.0     197.3     977.0
GR   197.3      1000     197.3      1050     201.5    1050.1     199.7      1063

X1     2000         8       957    1050.1       104        96       100
GR   204.0       940     204.0       949     203.3       957     197.9       978     197.9      1000
GR   197.9      1050     201.7    1050.1     200.9      1066

X1     2100         8       959      1051       104        95       100
GR   203.4       955     203.4       959    198.19       981    198.19      1000    198.19      1035
GR   202.0      1051     201.4      1053     200.8      1070

X1     2200         7       951      1054       102        97       100
GR   204.3       944     204.0       951    198.43       973    198.43      1000    198.43      1037
GR   202.4      1054     200.9      1070

X1     2300         9       951      1056       100       100       100
GR   204.0       922     204.3       945     204.0       951     198.6       973     198.6      1000
GR   198.4      1041     202.0      1056     201.0      1065     200.7      1072

X1     2400         6       947      1045       100       100       100
GR   204.0       947     198.9       970     198.9      1000     198.9    1026.5     202.9      1045
GR   202.8      1050

X1     2500         7       946      1054       100       100       100
GR   204.1    939.99     203.8       946     199.2       967     199.2      1000     199.2      1014
GR   203.8      1028     203.5      1054

X1     2600         7       950      1055       100       100       100
GR   203.1       940     203.0       950     199.2       966     199.2      1000     199.2      1040
GR   202.4      1055     202.2      1058
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OT        1       880
X1     2700         7       947      1054       100       100       100
GR   204.0       940     203.8       947     199.4       966     199.4      1000     199.4      1041
GR   202.4      1054     201.8      1061

X1     2800         7       948      1051       100       100       100
GR   203.98       940     204.0       948     199.6       968     199.6      1000     199.6      1034
GR   203.4      1051     202.0      1060
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GR	203.4	1051	202.0	1060						
X1	2900	7	950	1052	100	100	100			
GR	204.0	940	204.0	950	200.1	969	200.1	1000	200.1	1034
GR	203.8	1052	201.6	1060						
X1	3000	7	952	1049	100	100	100			
GR	204.0	940	204.0	952	200.3	970	200.3	1000	200.3	1031
GR	204.1	1049	203.0	1054						
X1	3100	7	951	1051	100	100	100			
GR	203.5	940	203.5	951	200.5	967	200.5	1000	200.5	1035
GR	204.0	1051	202.4	1063						
X1	3200	6	950	1049	100	100	100			
GR	204.0	950	200.8	966	200.8	1000	200.8	1033	204.0	1049
GR	203.4	1055								
X1	3300	7	952	1051	100	100	100			
GR	205.0	950	205.0	952	201.1	970	201.1	1000	201.1	1036
GR	204.2	1051	203.4	1055						
X1	3400	7	953.5	1051	100	100	100			
GR	204.5	950	204.4	953.5	201.3	968	201.3	1000	201.3	1032
GR	204.6	1051	203.5	1054						
OT	1	810								
X1	3500	6	953	1052	100	100	100			
GR	205.0	953	201.5	969	201.5	1000	201.5	1035	205.0	1052
GR	204.0	1055								
X1	3600	7	954	1042.5	100	100	100			
GR	205.0	950	205.0	954	201.8	969	201.8	1000	201.8	1028
GR	204.7	1042.5	203.5	1055						
X1	3700	7	957	1043	100	100	100			
GR	205.0	950	205.0	957	202.0	971	202.0	1000	202.0	1032
GR	205.7	1043	204.0	1055						
X1	3800	7	957	1047	100	100	100			
GR	205.0	950	205.0	957	202.3	971	202.3	1000	202.3	1036
GR	204.4	1047	203.0	1056						

X1	3900	7	958	1045	100	100	100			
GR	205.0	950	205.0	958	202.5	970	202.5	1000	202.5	1036
GR	204.2	1045	203.9	1051						
X1	4000	7	954	1046	100	100	100			
GR	204.6	950	204.1	954	202.8	967	202.8	1000	202.8	1037
GR	205.5	1046	203.0	1055						
X1	4100	7	957	1046	100	100	100			
GR	205.1	950	205.1	957	203.0	968	203.0	1000	203.0	1036
GR	204.8	1046	206.1	1056						
X1	4200	7	958	1047	100	100	100			
GR	203.7	950	204.8	958	203.2	969	203.2	1000	203.2	1038
GR	205.0	1047	205.0	1056						
X1	4300	7	959	1047	100	100	100			
GR	205.0	950	205.0	959	203.5	968	203.5	1000	203.5	1038
GR	204.8	1047	203.8	1056						
X1	4400	7	958	1044	100	100	100			
GR	205.0	950	205.0	958	203.7	967	203.7	1000	203.7	1037
GR	204.7	1044	203.8	1056						
X1	4500	7	960	1042	100	100	100			
GR	205.3	950	205.1	960	204.0	967	204.0	1000	204.0	1035
GR	205.1	1042	206.4	1051						
X1	4600	7	958	1041	100	100	100			
GR	206.0	950	206.0	958	204.0	968	204.0	1000	204.2	1032
GR	205.3	1041	203.9	1054						
OT	1	740								
X1	4700	7	961	1041	100	100	100			
GR	204.0	950	205.4	961	204.4	971	204.4	1000	204.4	1034
GR	205.1	1041	205.1	1057						
X1	4800	7	966	1044	100	100	100			
GR	206.1	950	205.4	966	204.7	970	204.7	1000	204.7	1039
GR	206.0	1044	207.8	1053						
X1	4900	7	962	1045	100	100	100			
GR	206.2	950	206.2	962	204.9	970	204.9	1000	204.9	1039
GR	205.4	1045	204.8	1057						
X1	5000	6	964	1051	100	100	100			
GR	206.3	950	206.3	964	205.2	968	205.2	1000	205.2	1042
GR	205.8	1051								

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QT        1       130
X1     5100         6       961    1047.7       100       100       100
GR    206.2       955     206.2       961     205.4       965     205.4      1000     205.4      1036
GR    206.7    1047.7

X1     5200         6       950      1043       100       100       100
GR    207.1       950     205.6       957     205.6      1000     205.6      1038     206.4      1043
GR    204.8      1051

X1     5300         6       950      1039       100       100       100
GR    207.2       950     205.9       958     205.9      1000     205.9      1032     206.3      1039
GR    206.6      1050

X1     5400         7       954      1038       100       100       100
GR    207.2       950     207.0       954     206.1       961     206.1      1000     206.1      1032
GR    207.0      1038     207.0      1046

QT        1       120
X1     5500         7       954      1039       100       100       100
GR    207.3       950     207.0       954     206.4       960     206.4      1000     206.4      1033
GR    207.0      1039     207.0      1048

X1     5600         7     952.5      1037       100       100       100
GR    207.9       949     207.9     952.5     206.6       960     206.6      1000     206.6      1033
GR    207.1      1037     207.3      1050

X1     5700         6     949.0      1038       100       100       100
GR    208.8       949     206.8       959     206.8      1000     206.8      1030     207.6      1038
GR    207.5      1050

X1     5800         7       947      1038       100       100       100
GR    209.2       943     208.9       947     207.1       958     207.1      1000     207.1      1032
GR    208.0      1038     208.0      1050

X1     5900         7     954.2      1038       100       100       100
GR    209.1       951     209.1     954.2     207.4       960     207.4      1000     207.4      1032
GR    208.0      1038     208.0      1050

X1     6000         7     960.5      1036       100       100       100
GR    209.2       958     209.2     960.5     207.6       968     207.6      1000     207.6      1028
GR    208.6      1036     208.5      1050

X1     6100         5       972    1024.5       100       100       100
GR    210.8       972    207.84       982    207.84      1000    207.84      1018     210.2    1024.5

X1     6200         5       971    1026.5       100       100       100
GR    210.7       971    208.08       982    208.08      1000    208.08      1018     210.5    1026.5
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X1     6300         5       973    1027.5       100       100       100
GR    210.5       973    208.32       982    208.32      1000    208.08      1018     210.7    1027.5
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The following 4 stations (6305,6307,6386 & 6387) are for the weir flow condition at Thomas Road.

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X1     6305         5       973    1027.5         5         5         5
GR    210.5       973    208.32       982    208.32      1000    208.08      1018     210.7    1027.5

X1     6307         5       968      1025         2         2         2
GR   210.61       968    210.61    989.25    211.22    989.26    211.22    1010.8    210.61      1025

X1     6386         5       968      1025        79        79        79
GR   210.61       968    210.61    989.25    211.22    989.26    211.22    1010.8    210.61      1025

X1     6387         5       973      1025         1         1         1
GR    209.4       973    208.50       982    208.50      1000    208.50      1018     209.9      1025

QT        1        60
X1     6400         5       973    1027.3        13        13        13
GR    209.5       973    208.56       982    208.56      1000    208.56      1018     210.0    1027.3

X1     6500         5       972      1025       100       100       100
GR    211.1       972     208.8       982     208.8      1000     208.8      1018     210.6      1025

X1     6600         5       972    1027.5       100       100       100
GR    211.5       972    209.04       982    209.04      1000    209.04      1018     211.5    1027.5

X1     6700         5       974    1027.4       100       100       100
GR    211.3       974    209.28       982    209.28      1000    209.28      1018     211.4    1027.4

X1     6800         5       972    1026.3       100       100       100
GR    211.8       972    209.52       982    209.52      1000    209.52      1018     211.5    1026.3

X1     6900         5     972.5      1028       100       100       100
GR    212.0     972.5    209.76       982    209.76      1000    209.76      1018     212.1      1028

X1     7000         5       971      1028       100       100       100
GR    212.7       971     210.0       982     210.0      1000     210.0      1018     212.5      1028

QT        1        20
X1     7100         5       971      1026       100       100       100
GR    212.8       971    210.24       982    210.24      1000    210.24      1018     212.4      1026

X1     7200         5       971      1026       100       100       100
GR    213.0       971    210.48       982    210.48      1000    210.48      1018     212.6      1026

X1     7300         5       971      1024       100       100       100
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X1	7500	5	971	1024	100	100	100			
GR	213.5	971	210.72	982	210.72	1000	210.72	1018	212.5	1024

X1	7400	5	971.4	1025	100	100	100			
GR	213.5	971.4	210.96	982	210.96	1000	210.96	1018	213.0	1025
X1	7500	5	961	1026.5	100	100	100			
GR	213.6	961	211.2	970	211.2	1000	211.2	1018	213.2	1026.5

THIS RUN EXECUTED 07JUL94 11:38:07

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 HEC-2 WATER SURFACE PROFILES

 Version   4.6.2;  May 1991
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NOTE- ASTERISK (*) AT LEFT OF CROSS-SECTION NUMBER INDICATES MESSAGE IN SUMMARY OF ERRORS LIST

(6:1 Side Slope-East,4:1

SUMMARY PRINTOUT

	SECNO	Q	XLCH	TOPWID	AREA	VCH	XLBEL	RBEL	CWSEL	ELMIN	EG
	.000	5270.00	.00	16.00	242.32	21.75	100000.00	100000.00	184.98	169.67	192.33
*	384.000	5270.00	384.00	116.00	909.70	9.97	186.71	186.71	192.51	171.71	193.54
	434.000	5270.00	50.00	116.00	668.32	12.81	188.36	188.36	191.88	171.98	193.88
*	484.000	5270.00	50.00	116.00	545.31	14.59	190.00	190.00	192.35	172.80	195.21
*	684.000	5270.00	200.00	126.00	994.99	8.43	190.00	190.00	195.07	176.07	195.95
	726.000	5270.00	42.00	126.00	897.07	9.91	177.11	177.11	194.85	177.11	196.18
	1000.000	5270.00	274.00	110.00	706.28	11.39	179.85	179.85	195.86	179.85	197.68
*	1099.000	1170.00	99.00	128.00	1121.68	1.07	195.20	195.20	197.86	180.00	197.88
*	1100.000	1170.00	1.00	131.50	762.81	1.55	195.30	194.80	197.85	189.90	197.89
	1200.000	1170.00	90.00	154.00	715.77	1.66	195.80	196.00	197.87	190.00	197.91
	1300.000	1170.00	100.00	164.00	724.08	1.64	196.10	197.30	197.90	190.27	197.94
	1400.000	1170.00	100.00	156.60	649.04	1.82	196.30	197.90	197.93	190.50	197.98
	1500.000	1170.00	100.00	159.22	606.76	1.95	198.20	197.20	197.98	190.70	198.03
*	1600.000	1170.00	100.00	123.63	453.75	2.58	199.40	198.60	198.02	192.20	198.12
*	1700.000	1170.00	100.00	92.76	235.37	4.97	201.50	200.50	198.05	193.60	198.44
	1800.000	1170.00	99.00	91.41	205.26	5.70	202.30	200.70	198.81	195.10	199.32
	1900.000	950.00	100.00	91.00	222.26	4.28	203.50	201.50	199.81	196.50	200.09

	SECNO	Q	XLCH	TOPWID	AREA	VCH	XLBEL	RBEL	CWSEL	ELMIN	EG
	2000.000	950.00	100.00	81.66	189.62	5.01	203.30	201.70	200.37	197.90	200.76
	2100.000	950.00	100.00	87.26	194.66	4.91	203.40	202.00	201.11	198.19	201.48
*	2200.000	950.00	100.00	100.12	260.26	3.69	204.00	202.40	201.74	198.43	201.95
	2300.000	950.00	100.00	113.08	304.82	3.18	204.00	202.00	202.06	198.40	202.21
	2400.000	950.00	100.00	87.32	242.65	3.92	204.00	202.90	202.27	198.90	202.51
	2500.000	950.00	100.00	73.12	206.17	4.61	203.80	203.50	202.63	199.20	202.96
*	2600.000	950.00	100.00	118.00	358.16	2.67	203.00	202.40	203.10	199.20	203.21
	2700.000	880.00	100.00	111.58	358.25	2.48	203.80	202.40	203.24	199.40	203.33
	2800.000	880.00	100.00	108.68	318.04	2.80	204.00	203.40	203.36	199.60	203.48
	2900.000	880.00	100.00	105.31	286.18	3.11	204.00	203.80	203.52	200.10	203.67
	3000.000	880.00	100.00	97.45	268.30	3.29	204.00	204.10	203.74	200.30	203.91

3000.000	880.00	100.00	97.45	280.30	3.29	204.00	204.10	203.74	200.30	203.91
3100.000	880.00	100.00	122.88	311.87	2.90	203.50	204.00	204.00	200.50	204.13
3200.000	880.00	100.00	105.00	286.76	3.09	204.00	204.00	204.18	200.80	204.33
3300.000	880.00	100.00	100.25	272.07	3.25	205.00	204.20	204.40	201.10	204.57
3400.000	880.00	100.00	104.00	274.85	3.22	204.40	204.60	204.65	201.30	204.81
3500.000	810.00	100.00	100.75	280.09	2.90	205.00	205.00	204.90	201.50	205.04
3600.000	810.00	100.00	105.00	260.14	3.18	205.00	204.70	205.10	201.80	205.26
3700.000	810.00	100.00	101.25	253.43	3.26	205.00	205.70	205.33	202.00	205.50
3800.000	810.00	100.00	106.00	285.10	2.90	205.00	204.40	205.58	202.30	205.71
3900.000	810.00	100.00	101.00	277.86	2.98	205.00	204.20	205.77	202.50	205.90
4000.000	810.00	100.00	105.00	291.67	2.84	204.10	205.50	205.96	202.80	206.08
4100.000	810.00	100.00	106.00	273.64	3.03	205.10	204.80	206.14	203.00	206.28
4200.000	810.00	100.00	106.00	291.22	2.84	204.80	205.00	206.35	203.20	206.47
4300.000	810.00	100.00	106.00	288.09	2.86	205.00	204.80	206.53	203.50	206.65
4400.000	810.00	100.00	106.00	292.29	2.80	205.00	204.70	206.71	203.70	206.83
4500.000	810.00	100.00	101.00	256.27	3.24	205.10	205.10	206.89	204.00	207.05

	SECNO	Q	XLCH	TOPWID	AREA	VCH	XLBEL	RBEL	CWSEL	ELMIN	EG
	4600.000	810.00	100.00	104.00	282.06	2.90	206.00	205.30	207.13	203.90	207.26
	4700.000	740.00	100.00	107.00	292.65	2.52	205.40	205.10	207.33	204.00	207.43
	4800.000	740.00	100.00	101.44	246.26	3.07	205.40	206.00	207.48	204.70	207.62
	4900.000	740.00	100.00	107.00	276.12	2.69	206.20	205.40	207.71	204.80	207.82
	5000.000	740.00	100.00	101.00	252.74	2.96	206.30	205.80	207.89	205.20	208.03
	5100.000	130.00	100.00	92.70	238.13	.55	206.20	206.70	208.12	205.40	208.12
	5200.000	130.00	100.00	101.00	248.09	.52	207.10	206.40	208.13	204.80	208.13
	5300.000	130.00	100.00	100.00	210.94	.62	207.20	206.30	208.14	205.90	208.14
	5400.000	130.00	100.00	96.00	179.44	.74	207.00	207.00	208.15	206.10	208.16
	5500.000	120.00	100.00	98.00	161.17	.75	207.00	207.00	208.17	206.40	208.18
	5600.000	120.00	100.00	101.00	142.90	.85	207.90	207.10	208.20	206.60	208.21
	5700.000	120.00	100.00	98.20	123.98	.98	208.80	207.60	208.25	206.80	208.26
	5800.000	120.00	100.00	99.45	103.27	1.18	208.90	208.00	208.32	207.10	208.34
	5900.000	120.00	100.00	93.58	87.44	1.40	209.10	208.00	208.45	207.40	208.48
	6000.000	120.00	100.00	86.96	72.13	1.69	209.20	208.60	208.66	207.60	208.70
*	6100.000	120.00	100.00	43.12	45.92	2.61	210.80	210.20	209.00	207.84	209.11
	6200.000	120.00	100.00	46.64	57.00	2.11	210.70	210.50	209.46	208.08	209.53
	6300.000	120.00	100.00	47.91	62.61	1.92	210.50	210.70	209.74	208.08	209.80
	6305.000	120.00	5.00	48.03	63.34	1.89	210.50	210.70	209.76	208.08	209.81
*	6307.000	120.00	2.00	57.00	29.37	4.09	210.61	210.61	211.43	210.61	211.69
*	6386.000	120.00	79.00	57.00	69.29	1.73	210.61	210.61	212.14	210.61	212.18
*	6387.000	120.00	1.00	52.00	182.46	.66	209.40	209.90	212.18	208.50	212.19
	6400.000	60.00	13.00	54.30	186.53	.32	209.50	210.00	212.19	208.56	212.19
	6500.000	60.00	100.00	53.00	162.00	.37	211.10	210.60	212.19	208.80	212.19
	6600.000	60.00	100.00	55.50	150.94	.40	211.50	211.50	212.19	209.04	212.19
	6700.000	60.00	100.00	53.40	137.67	.44	211.30	211.40	212.20	209.28	212.20

SECNO	Q	XLCH	TOPWID	AREA	VCH	XLBEL	RBEL	CWSEL	ELMIN	EG
6800.000	60.00	100.00	54.30	125.98	.48	211.80	211.50	212.20	209.52	212.21
6900.000	60.00	100.00	55.50	113.55	.53	212.00	212.10	212.21	209.76	212.21

7000.000	60.00	100.00	53.91	99.74	.60	212.70	212.50	212.22	210.00	212.23
7100.000	20.00	100.00	51.95	87.67	.23	212.80	212.40	212.23	210.24	212.24
7200.000	20.00	100.00	50.35	76.12	.26	213.00	212.60	212.24	210.48	212.24
7300.000	20.00	100.00	47.14	63.20	.32	213.50	212.50	212.24	210.72	212.24
7400.000	20.00	100.00	45.78	52.61	.38	213.50	213.00	212.25	210.96	212.25
7500.000	20.00	100.00	56.46	55.26	.36	213.60	213.20	212.26	211.20	212.26

SUMMARY OF ERRORS AND SPECIAL NOTES

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WARNING SECNO=    384.000  PROFILE=  1  CONVEYANCE CHANGE OUTSIDE ACCEPTABLE RANGE

CAUTION SECNO=    484.000  PROFILE=  1  CRITICAL DEPTH ASSUMED
CAUTION SECNO=    484.000  PROFILE=  1  MINIMUM SPECIFIC ENERGY

WARNING SECNO=    684.000  PROFILE=  1  CONVEYANCE CHANGE OUTSIDE ACCEPTABLE RANGE

WARNING SECNO=   1099.000  PROFILE=  1  CONVEYANCE CHANGE OUTSIDE ACCEPTABLE RANGE

WARNING SECNO=   1100.000  PROFILE=  1  CONVEYANCE CHANGE OUTSIDE ACCEPTABLE RANGE

WARNING SECNO=   1600.000  PROFILE=  1  CONVEYANCE CHANGE OUTSIDE ACCEPTABLE RANGE

WARNING SECNO=   1700.000  PROFILE=  1  CONVEYANCE CHANGE OUTSIDE ACCEPTABLE RANGE

WARNING SECNO=   2200.000  PROFILE=  1  CONVEYANCE CHANGE OUTSIDE ACCEPTABLE RANGE

WARNING SECNO=   2600.000  PROFILE=  1  CONVEYANCE CHANGE OUTSIDE ACCEPTABLE RANGE

WARNING SECNO=   6100.000  PROFILE=  1  CONVEYANCE CHANGE OUTSIDE ACCEPTABLE RANGE

CAUTION SECNO=   6307.000  PROFILE=  1  CRITICAL DEPTH ASSUMED
CAUTION SECNO=   6307.000  PROFILE=  1  MINIMUM SPECIFIC ENERGY

WARNING SECNO=   6386.000  PROFILE=  1  CONVEYANCE CHANGE OUTSIDE ACCEPTABLE RANGE

WARNING SECNO=   6387.000  PROFILE=  1  CONVEYANCE CHANGE OUTSIDE ACCEPTABLE RANGE
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