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VAN BUREN PHASE VII

DRAINAGE REPORT

UNITED WESTERN ENGINEERS

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Suite 200
4525 Northpark Drive
Colorado Springs, Colo. 80907

(303) 598-3222

September 24, 1973

Mr. DeWitt Miller
City Hall
P.O. Box 1575
Colorado Springs, Colorado

Subject: Van Buren Phase VII
Drainage Report

Dear Deke:

Transmitted herewith is subject drainage report for your review and approval.

This report, based on my discussions with Mr. Bob Martin, shows alternative calculations and facilities for two different methods of analysis.

The first method is that currently used throughout the City and accepted by your department. The second is a considerably more detailed analysis using the most current information available from many different sources. We feel the second method to be the only true representation of storm runoff and hydraulic routing.

We therefore recommend acceptance of the proposed facilities. This report is presented in such a format as to give a direct comparison of the two methods of analysis. We suggest that you examine the City criteria in light of the material presented.

Respectfully submitted,

UNITED WESTERN ENGINEERS

Oliver E. Watts

Oliver E. Watts
Engineering Director

Enclosures

VAN BUREN PHASE VII

DRAINAGE PLAN

Certifications and Approvals

Registered Engineer

I, Oliver E. Watts, a registered engineer in the State of Colorado, hereby certify that the attached drainage plan and report were prepared under my direction and supervision and are correct to the best of my knowledge and belief. I further certify that said drainage report is in accordance with all City of Colorado Springs Ordinances and specifications and criteria.

Colorado PE - LS No. 9853

OLIVER E. WATTS
REGISTERED
9853
P. E. & L. S.
STATE OF COLORADO

Approved:

City of Colorado Springs, Department of Public Works

City Engineer

Date

Comments:

VAN BUREN PHASE VII

DRAINAGE REPORT

INDEX

I. DESCRIPTION

A. Location: The project area is located in primarily in the Northeasterly portion of Section 33, Township 13 South, Range 66 West of the 6th P.M., in Colorado Springs, Colorado. The total drainage area shown on Plate Two was analyzed, totaling 396 acres in Sections 27, 28, 33 and 34, all of which is the Van Buren Area Drainage Basin.

The study area is bounded on the West by the Colorado Springs Country Club, on the North and East by the natural drainage divide within Palmer Park, and on the South by the natural drainage divide located between Grand View Street and Holiday Lane.

Previous subdivision plattings within the project area have occured (mainly in the early 1960's) as follows:

1. Isaac's Addition, North of Paseo Road.
2. Austin Bluffs and Austin Bluffs Filing 39, above Chelton Road.
3. Highland Hills and Highland Hills Filing 2, West of Chelton Road and South of Paseo Road.
4. Country Club Addition Numbers 1, 2 and 3, and Filing 2 of Addition 3, South of Paseo Road and East of Country Club Drive.

B. Existing Runoff: The significant problems with existing drainage facilities may be summarized as follows:

1. Flooding of Paseo Road from Country Club Drive to Leslie Drive, particularly below Lees Lane as shown in the first two photo sheets enclosed.

2. Active erosion in the greenbelt North of Paseo Road from Leslie Drive to the Golf Course, having several exposed utilities.

3. An inadequately designed intersection at Marilyn Road and Leslie Drive, creating unnecessary runoff in the West curb of Leslie Drive.

4. A natural sump along Chelton Road in Basin "D" and inadequate means to route this flow to Austin Drive.

5. Generally inadequate drainage facilities along Chelton Road to positively contain and route the runoff from Palmer Park.

6. Generally inadequate drainage of Isaac's Addition due to the high sediment load in the existing ditch and culvert. Two residents near the top of this development pay several hundred dollars a year in maintenance and repair of storm damage.

The above problems are listed in order of priority for recommending facilities toward the allotted project funds.

II. HYDROLOGIC COMPUTATIONS

A. General: Two methods of analysis are presented, each of which is commonly referred to as the Soil Conservation Service synthetic hydrograph method, as follows:

1. Soil Conservation Service Method: The method used by Soil Conservation Service personnel and enforced in their review of County Subdivision plans under Senate Bill 35 is described in the December, 1972 publication of the Denver office "Procedures for Determining Peak Flows in Colorado". This method is particularly applicable to small basins such as those of the project area. Runoff is determined by the formula:

$$Qp = QAQ$$

Where Qp = storm runoff in CFS
Q = runoff per square mile, in this region, taken from figure 8B. The shorter the time of concentration, the higher the runoff. This method therefore becomes very similar to the various forms of the "rational" method in use throughout the country.
A = Area in square miles
Q = runoff in inches, corresponding to the computed curve number of the basin. The 50 year six hour storm of 3.2 inches was used.

The use of this method, as shown in the computations, gives 1-1/2 to 3 times the runoff in range land and about 170% in developed areas, compared to City criteria.

2. City Method: The method in use by the City is demonstrated in the U.S.B.R. "Small Dams" publication. Runoff is determined by the formula:

$$Qp = \frac{484\ AQ}{Tpo}$$

Where Qp = storm runoff in CFS
A = Area, SM
Q = runoff in inches, corresponding to computed curves. The 50 year one hour storm of 2.0 inches was used. See the comparison in Section II A-1 above.
Tpo = peak time, equal to $Tpo = D/2Tc + D/2$
tc = time of concentration
D = duration (one hour)

This method assumes uniform rainfall over the entire length of the storm, and is recommended for use by the Soil Conservation Service and U.S.B.R. only for determining incremental hydrographs in storms of known, unequal rainfall distribution. A common application is in the computation of hydrographs for the "Maximum Probable Storm".

The runoff curves used in this method come from Figures 3 and 4. Figure 4 gives significantly lower curves than those now in use by the City. For this reason a detailed analysis of composite curves were computed in the Vista Grande Terrace Subdivisions, based on the grading plans and the various house models designed by respective builders. A summary of these calculations follows:

Filing No.	Soil Grouping	% of Impervious Cover	Computed Curve No.	Figure 4 Curve No.	Curve No. City Criteria
9	B	33.0	78.9	79.5	94
10	B	41.2	80.9	80.2	94

The percent of various covers and Curve numbers in the above two subdivisions were as follows:

	Curve #*	Filing 9	Curve #*	Filing 10
Roofs	95	0.0814	95	0.1732
Streets	95	0.1927	95	0.1544
Sidewalks	95	0.0343	95	0.0381
Driveways-Patios	95	0.0181	95	0.0467
House walks	95	0.0035	95	(incl. above)
Total Impervious	95	0.3300	95	0.4124
Lawns-Landscaping	71	0.6700	71	0.5876
TOTAL	78.9	1.0000	80.9	1.0000

* per Soil Conservation Service Engineering Manual, Part IV, Hydrology, January, 1971.

The above comparisons seem to validate Figure 4.

The use of the City method is very dangerous in basins where the time of concentration is less than the storm duration. An example is the computed runoff in El Paso Street at the primary channel of the Van Buren Storm Sewer. This 441 acre basin had a time of concentration of 0.411 hours (25 minutes). In 25 minutes, the 10 year rainfall intensity in this region is 3.5 inches (Drainage Study, Broadmoor Mesa First Filing, August 18, 1969, by Hartzell-Pfeiffenburger). The 50 year computed runoff used a rainfall of 57% of what could be expected for the 10 year storm. As shown below the 50 year runoff by City Criteria should be that of a 17 year storm. A comparison of runoffs by the two methods is as follows:

<u>Soil Conservation Service Method</u>:

50% Imp.,"B" Soil, CN = 81   Tc = 0.411 hr.

10 year storm (6hour) I = 2.5"   Q = 0.94"
Gp = 820 x 0.689 x 0.94 = 531.08 CFS

50 year storm (6hour) I = 3.2"   Q = 1.47"
Gp = 820 x 0.689 x 1.47 = 830.52 CFS

25 year storm (6hour) I = 3.0"   Q = 1.32"
Gp = 820 x 0.689 x 1.32 = 745.77 CFS

<u>City Method</u>:

Area = 0.689 SM   I = 2.0"   CN = 94   Q = 1.41

Area B (1-39)

$$Gp = \frac{484 \times 0.689 \times 1.41}{\underset{(0.747)}{0.5 + 0.6 \times 0.411}} = 675.42 \text{ CFS (as published)}$$

B. <u>Soil Types</u>: Soil Mapping and interpretations were taken from the local Soil Conservation Service for the three soil types within the project area as follows:

<u>Unit R9-C</u>: Bresser Series of deep, dark, moderately coarse textured soils, hydrologic group "B".

<u>Unit RB-1</u>: Stoney steep land of from 6% slope to vertical cliffs, about 20-30% being rock outcrop. Hydrologic group "D".

<u>Unit R7-BD</u>: Blakeland Series of deep, dark, coarse textured soils, hydrologic group "A".

Range covers and vegitation types are shown on Plate One, as taken from figure 3 of the Soil Conservation Service publication referenced above. These covers were taken from field trips and aerial photgraphy of the Soil Conservation Service and this firm.

The following are the descriptions of the various cover types shown on Plate One.

Area No. (See Plate One)	Soil Type & Hydrologic Group	Range Type & Cover Density	Soil Cover Complex No.
1	R9-C "B"	Pine-30%	62
2	R9-C "B"	Herb-40%	74
3	RB-1 "D"	Res.-20%	87
4	RB-1 "D"	Herb-20%	92
5	RB-1 "D"	Pine-10%	84
6	RB-1 "D"	Herb-40%	90
7	RB-1 "D"	Oak -40%	70
8	RB-1 "D"	Pine-60%	75
9	RB-1 "D"	Oak -60%	62
10	RB-1 "D"	Pine-30%	81
11	RB-1 "D"	Pine-40%	79
12	RB-1 "D"	Res.-10%	85
13	RB-1 "D"	Pine-20%	82
14	RB-1 "D"	Oak -80%	54
15	R7-BD "A"	Res.-20%	70
16	R7-BD "A"	Herb-40%	66
17	R7-BD "A"	Res.-40%	72*
18	R7-BD "A"	Herb-80%	54

* Used as Curve Number 92 in the City criteria alternative.

Project Van Buren Phase VII

Calc. by O.E. Watts date 8-31-73

Checked by ______ date ______

HYDROLOGIC COMPUTATIONS

Re: SCS - Procedures for determining Peak Flows in Colorado, Dec. 1972.

Criteria: 50 year Rainfall
Using Most Severe of
6 hr rainfall 3.2" by referenced procedure
1 hr rainfall 2.0" by City Criteria
Below 7000ft - eastern slope - Type II A intensity
Antecedent moisture Condition: AMC II

Part I - Major Inflows

Soil Cover Complexes

See plate 1 & Report for description
Using: SCS photo 1303
Field Trip info 8-28, 8-29, 8-30-73
500 scale photos for 200', 2' Topo

1" = 500'

Basin#	Cover #	Curve#	PR	% Area	% x CN
A1	4	92	1.08	0.3429	31.5
	5	84	1.59	0.5048	42.4
	3	87	0.11	0.0349	3.0
	15	70	0.32	0.1016	7.1
	16	66	0.05	0.0159	1.0
	ΣA1	—	3.15	1.000	85.1 use 85
A2	5	84	0.05	0.0388	3.3
	7	70	0.12	0.0930	6.5
	3	87	0.29	0.2248	19.6
	6	90	0.08	0.0620	5.6
	16	66	0.19	0.1473	9.7
	15	70	0.56	0.4341	30.4
	ΣA2	—	1.29	1.0000	75.0 use 75
A3	5	84	0.02	0.0080	0.7
	7	70	0.49	0.1968	13.8
	6	90	1.19	0.4779	43.0
	16	66	0.79	0.3173	21.0
	ΣA3	—	2.49	1.0000	78.4 use 78
A4	1	62	0.74	0.0516	3.2
	10	81	1.25	0.0871	7.1
	8	75	3.42	0.2383	17.9
	9	62	2.79	0.1944	12.1
	11	79	1.78	0.1240	9.8
	6	90	1.39	0.0969	8.7
	5	84	1.49	0.1038	8.7

Soil Complex - Cont

Basin #	Cover #	Curve #	PR	% A	% x CN
A4-cont	2	74	0.34	0.0237	1.8
	7	70	1.10	0.0766	5.4
	ΣA4	—	14.35	1.00	74.5+ Use 75
B	2	74	1.59	0.0629	4.7
	5	84	4.99	0.1973	16.6
	6	90	1.14	0.0451	4.1
	7	70	0.99	0.0391	2.7
	8	75	3.17	0.1253	9.4
	9	62	2.57	0.1016	6.3
	11	79	2.96	0.1170	9.2
	13/14	82/54	6.06/1.82	0.2396/0.0720	19.6/ 3.9
	ΣB	—	25.29	1.000	76.5+ Use 77
C	5	84	0.70	0.1609	13.5
	6	90	0.17	0.0391	3.5
	7	70	1.61	0.3701	25.9
	11	79	0.14	0.0322	2.5
	12	85	0.59	0.1356	11.5
	13	82	1.14	0.2621	21.5
	ΣC	—	4.35	1.0000	78.5+ Use 78
D	3	87	0.26	0.4127	35.9
	7	70	0.16	0.2540	17.8
	12	85	0.05	0.0794	6.7
	13	82	0.16	0.2540	20.8
	ΣD	—	0.63	1.000	81.3 Use 81
E1	3	87	0.91	0.6107	53.1
	7	70	0.58	0.3893	27.2
	ΣE1	—	1.49	1.0000	80.4 use 80
E2	17		2.97		72
F	Note: For F Sc = 1" = 100' See Plate 3				

F Calculations on p

Time of Concentration

$$T_c = \left(\frac{11.9 L^3}{H}\right)^{0.385}$$

(very close to chart solution) for Overland Flow

For Natural Channels see attached SCS chart (Last Page)

L = miles H = ft T_c = hrs

Channels $T_c = \frac{L_{ft}}{3600 V_{fps}}$

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Checked by date

Time Conc - cont

Basin#	El T ft	El B ft	H ft	L ft	S %	Type of Flow: O'land	Type of Flow: Channel	V-fps	Tc
A_1	6600	6340	260	1300	—	X		—	0.060
	6340	6280	60	875	6.86		II	11.9	0.020
									0.080
A_2	6545	6280	265	1560		X			0.074
A_3	6545	6300	245	1035	—	X		—	0.048
	6300	6285	15	580	2.56		II	4.6	0.035
									0.083
A4	6585	6380	205	1900	—	X		—	0.072
	6380	6320	60	1450	4.14		I	6.2	0.065
									0.137
B	6585	6460	125	900	—	X		—	0.052
	6460	6330	130	2700	4.81		I-III	6.8	0.110
									0.162
C	6600	6460	140	690	—	X		—	0.037
	6460	6365	95	930	10.21		I	11.0	0.023
	6365	6330	35	675	5.19		II	10.2	0.018
									0.079
D	6560	6400	160	520	—	X	—	—	0.025
E1	6590	6390	200	700	—	X	—	—	0.020
	6390	6385	5	650	0.0077		II	4.3	0.042
									0.062
E2	6390	6385	5	650	0.0077		II	4.3	0.042
	6385	6320	65	970	6.70		street	14.7	0.018
									0.060
E1 +	see above								0.062
E2	see above								0.018
									0.080

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Inflow Quanties Comparisons SCS 50yr 6hr
City 50yr 1hr

This Sheet

$$q_p = q A Q$$

A from p 1 & 2 (SM)
q from Fig 8B SCS ref.
Q from 3.2" runoff charts

Page 5 - regular City Method, Curve #'s for Res. used differently

Basin	A - SM	Tc - Hrs	Curve #	q	Q	q_p	q_p p5	q_p/q_p
A1	0.0282	0.080	85	1000	1.76	49.6	19.9	2.49
A2	0.0116	0.079	75	1000	1.10	12.8	5.6	2.29
A3	0.0223	0.083	78	1000	1.27	28.3	9.4	3.01
A4	0.1287	0.137	75	995	1.10	140.9	40.7	3.46
B	0.2268	0.162	77	980	1.21	268.9	183.9	1.46
C	0.0390	0.079	78	1000	1.27	49.5	16.6	2.98
D	0.0056	0.025	81	1000	1.47	8.2	3.2	2.56
E1	0.0134	0.062	80	1000	1.40	18.8	6.8	2.76
E2	0.0266	0.060	72	1000	0.93	24.7	29.8	0.83
ΣE	0.0400	0.080	75	1000	1.10	44.0	34.3	1.28
Total	0.5022	—	—	—	—	602.2	313.6	1.92
C+D	0.0446	0.079	78	1000	1.27	56.6		

Conclusions:

50 yr 6 hr flows higher in range conditions
50 yr 6 hr flows lower in subdivided conditions
See later calc's - is about 70% higher.

Should use 50 yr 6 hour flows throughout

Check: 50yr 6hr on Basin B by std formula

$$q_p = \frac{484 \times 0.2268 \times 1.21}{3 + 0.6 \times 0.162} = 42.9 \text{ CFS}$$

- very much lower due to no consideration of rainfall distribution of intensity in small basins.

$q_p = \frac{484 AQ}{T_{po}}$ $T_{po} = 0.50 + 0.6 T_c$ I = 2"/hr Dur = 1 hr $T_b = 1.67 T_{po}$

MAJOR BASIN	SUB BASIN	AREA Planim. Read	AREA MILE	BASIN LENGTH	BASIN HEIGHT	Tc	DITCH LENGTH	DITCH SLOPE	Curve #	TPO	2" FLOW Q	FLOW qp	Tb
A	1		0.0282	See P3		0.080			85	0.548	0.80	19.9	
	2		0.0116			0.074			75	0.544	0.38	5.6	
	3		0.0223			0.083			78	0.550	0.48	9.4	
	4		0.1287			0.137			75	0.582	0.38	40.7	
B			0.2268			0.162			77	0.597	0.45	183.9	
C			0.0390			0.079			78	0.547	0.48	16.6	
D			0.0056			0.025			81	0.515	0.61	3.2	
E	1		0.0134			0.062			80	0.537	0.56	6.8	
	2		0.0266			0.060			$\boxed{92}$	0.536	1.24	29.8	
	ΣE		0.0400			0.080			$\boxed{88}$	0.548	0.97	34.3	

Note higher Curves Used per City Criteria

2" 1hr rainfall used

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HYDROLOGIC COMPUTATION — BASIC DATA

PROJ: Van Buren VIII

By: GEW
Date: 9-1

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1" = 100' $qp = \frac{484\,AQ}{TPo}$ $TPo = 0.5 + 0.6\,Tc$ Tc from Chart

MAJOR BASIN	SUB BASIN	AREA Planim. Read	AREA MILE	BASIN LENGTH	BASIN HEIGHT	Tc	DITCH LENGTH	DITCH SLOPE	Curve#	TPO	FLOW Q	FLOW qp	Tb
F	1	6.93	0.002306	1040	68	0.075			66	0.545	0.15	0.3	
	2	6.39	0.002921	610	30	0.056			92	0.534	1.24	3.3	
	3	13.11	0.004703	820	33	0.076				0.546		5.2	
	4	13.05	0.004681	640	21	0.070				0.542		5.2	
	5	20.96	0.007518	1140	81	0.078				0.547		8.3	
	6	23.50	0.008429	1175	71	0.085				0.551		9.2	
	7	39.21	0.01406	1510	97	0.100				0.560		15.1	
	8	29.26	0.01050	1640	94	0.110				0.566		11.1	
	9	35.00	0.01255	1470	78	0.106				0.564		13.4	
	10	5.67	0.002034	500	27	0.047				0.528		2.3	
	11	16.58	0.005948	820	47	0.067				0.540		6.6	
	12	10.55	0.003784	670	11	0.092				0.555		4.1	
	13	9.51	0.003411	460 ~~900~~	25	0.044				0.526		3.9	
	14	13.75	0.004932	440	14	0.056				0.534		5.5	
	15	6.26	0.002245	490	29	0.045				0.527		2.6 ~~25.6~~	
	16	19.87	0.007127	1115	65	0.083			92	0.550	1.24	7.8	

PART II - INTERNAL RUNOFF & OUTFALL TOTALS

HYDROLOGIC COMPUTATION — BASIC DATA

PROJ: Van Buren 7

By: BEJ/OEW

Date: 9-6

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MAJOR BASIN	SUB BASIN	AREA		BASIN		Tc	DITCH		~~V~~ Curve #	TPO	FLOW		Tb
		Planim. Read	MILE	LENGTH	HEIGHT		LENGTH	SLOPE			Q	qp	
F	17	19.13	0.006862	1180	49	0.078			92	0.547	1.29	7.5	
	18	13.89	0.004982	1130	45	0.097			92	0.558	1.24	5.4	
	19	11.20	0.004017	870	28	0.086			54	0.552	0.02	0.1	
A	5	4.13	0.001481	440	31	0.038			85	0.523	0.80	1.1	
E	2A	14.86	0.005330	585	18	0.066			92	0.540	1.29	5.9	
F	1+2	–	0.005227			0.095			81	0.557	0.61	2.8	
F	4,7 Thru 10	–	0.04382			0.171			92	0.603	1.29	43.6	
F	12 Thru 19	–	0.03736			0.184			88	0.610	0.99	28.7	
D+F1+F2		–	0.0108			0.120			81	0.572	0.61	5.6	
Hyd. PT	#1		0.2268			0.162			77	0.597	0.45	183.9	
	#2		0.4053			0.185			76	0.611	0.41	131.6	
	#3		0.4325			0.212			77	0.627	0.45	150.2	
	#4		0.4436			0.233			77	0.640	0.45	151.0	
	#5		0.5331			0.238			79	0.643	0.52	208.7	
	#6		0.5720			0.244			79	0.646	0.52	222.7	

HYDROLOGIC COMPUTATION — BASIC DATA

PROJ: Van Buren 7

By: BEJ/DEW
Date: 9-6

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Project VB7
Calc. by OEW date 9-6
Checked by ______ date ______

Internal Hydrology - Cont

SCS method : $I = 3.2"/Hr$ CN developed = 87 (90% imp - 'A' soil)

$$T_c = \left(\frac{11.9 L^3}{H}\right)^{0.385}$$ - neglect & use chart when $T_c < 0.13$

Basins	A - SM	T_c - Hrs	CN	q	Q	q_p	q_p - P697	Δ
F1	0.0023	0.078	66	1000	0.64	1.5	0.3	5.0
F2	0.0029	0.057	87		1.92	5.6	3.3	1.7
F1+2	0.0052	0.095	81		1.47	7.7	2.8	2.7
F3	0.0047	0.076	87		1.92	9.0	5.2	1.7
F4	0.0047	0.070				9.0	5.2	1.7
F3+4	0.0094	0.120				18.0	10.4	1.7
F5	0.0075	0.078				14.9	8.3	1.7
F6	0.0084	0.085				16.1	9.2	1.8
F5+6	0.0160	0.104				30.7	17.5	1.8
F7	0.0141	0.105				27.1	15.1	1.8
F8	0.0105	0.117				20.1	11.1	1.8
F9	0.0126	0.111				24.2	13.4	1.8
F8+9	0.0230	0.131		1000		44.1	24.5	1.8
F7-9	0.0371	0.150		990		70.5	39.6	1.8
F10	0.0020	0.047		1000		3.8	2.3	1.7
F4,7-10	0.0438	0.171		980		82.4	43.6	1.9
F11	0.0059	0.067		1000		11.3	6.6	1.7
F12	0.0038	0.092				7.3	4.1	1.8
F13	0.0034	0.044				6.5	3.9	1.7
F14	0.0049	0.056				9.4	5.5	1.7
F15	0.0022	0.045				4.2	2.6	1.6
F16	0.0071	0.083				13.6	7.8	1.7
F15+16	0.0094	0.083				18.0	10.4	1.7
F17	0.0069	0.078				13.2	7.5	1.8
F15-17	0.0162	0.101				31.1	17.3	1.7
F18	0.0050	0.101				9.6	5.4	1.8
F15-18	0.0212	0.101	87		1.92	40.7	23.3	1.7
F19	0.0040	0.086	54		0.25	1.0	0.1	10.0
F15-19	0.0252	0.101	82		1.54	38.8	23.4	1.7
F13-19	0.0335	0.129	80	1000	1.40	46.[illegible]		
F12-19	0.0374	0.184	80	975	1.40	51.1	28.7	1.8
E2A	0.0053	0.066	87	1000	1.92	10.2	5.9	1.7
E-E2A	0.0347	0.080	79	1000	1.34	46.5	28.4	1.6
A5	0.0015	0.038	85	1000	1.76	2.6	1.1	2.4
D+F1+F2	0.0108	0.120	81	1000	1.47	15.9	5.6	2.8
D+F1	0.0079	0.063	81	1000	1.47	11.6	3.5	3.3

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Project UB7
Calc. by OHW date 9-7
Checked by date

Major Greenbelt Flows - SCS Method see p 2 & p 8

For Curve #'s

Hydrograph Point	Basin Numbers	A -SM-	% Total Area	CN	% x CN
1	B	0.2268	1-	77	76.5
2	B	0.2268	0.5596	76.5	42.8
	A4	0.1287	0.3175	74.5	23.7
	C	0.0390	0.0962	78.5	7.6
	D+F1+F2	0.0108	0.0266	81	2.1
	Σ	0.4053	1.000	—	76.1 use 76
3	pt #2	0.4053	0.9371	76.1	71.3
	½ A3	0.0112	0.0259	78.4	2.0
	F5+F6	0.0160	0.0370	87	3.2
	Σ	0.4325	1.0000	—	76.6 use 77
4	pt 3	0.4325	0.9750	76.6	74.7
	½ A3	0.0111	0.0250	78.4	2.0
	Σ	0.4436	1.0000	—	76.6 use 77
5	pt 4	0.4436	0.8321	76.6	63.7
	A1	0.0282	0.0529	85.1	4.5
	A2	0.0116	0.0218	75.0	1.6
	F4,7-10	0.0438	0.0822	87	7.1
	F11	0.0059	0.0111	87	1.0
	Σ	0.5331	1.000	—	78.0 78
6	pt #5	0.5331	0.9320	78	72.7
	A5	0.0015	0.0026	85	0.2
	F12-19	0.0374	0.0654	80	5.2
	Σ	0.5720	1.0000	—	78.1 use 78

366 Acres

Project VB7

Calc. by OEW date 9-8

Checked by date

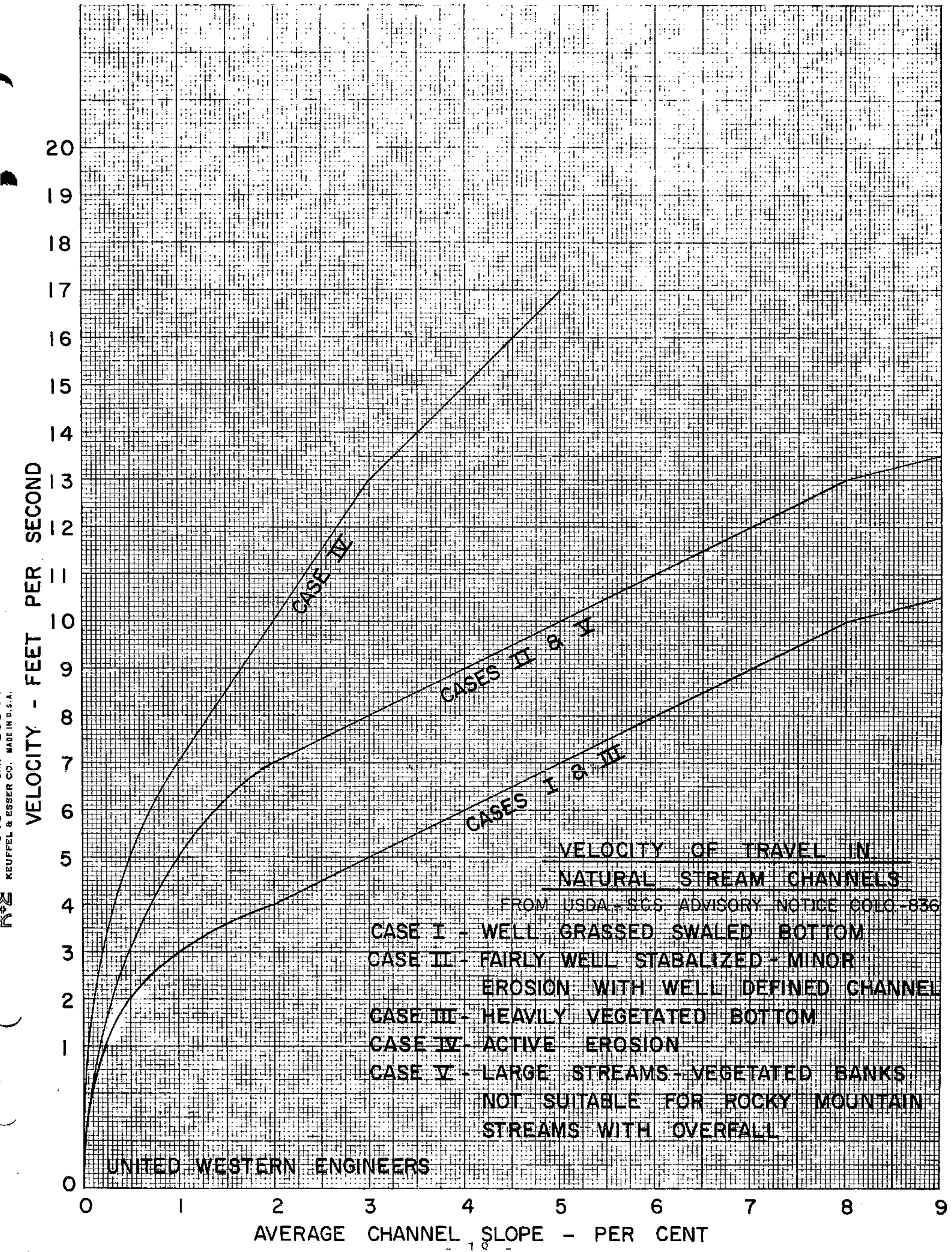
Greenbelt Flows - Cont

Tc from Chart - Lwot Pg

Hyd PT# & channel Case	L ft	H ft	S % & V-fps	Tc Hrs	q	A SM	CN	Q	qp CFS
1				0.162	980	0.2268	77	1.21	268.9
I	380	10	2.63%-4.6	0.023					
2				0.185	970	0.4053	76	1.15	452.1
II	750	20	2.67%-7.7	0.027					
3				0.212	955	0.4325	77	1.21	499.8
II	560	14	2.5%-7.5	0.021					
4				0.233	940	0.4936	77	1.21	504.6
IV	330	18	5.5%-18	0.005					
5				0.238	940	0.5331	78	1.27	636.4
II	180	5	2.78%-7.8	0.006					
6				0.244	935	0.5720	78	1.27	679.2

Comparisons of Greenbelt Flows

Hydrograph Point	Runoff - SCS Method	Runoff - City Method	Factor
1	268.9	183.9	1.46
2	452.1	131.6	3.44
3	499.8	150.2	3.33
4	504.6	151.0	3.34
5	636.4	208.7	3.05
6	679.2	222.7	3.05


VELOCITY - FEET PER SECOND
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
CASE IV
CASES II & V
CASES I & III
VELOCITY OF TRAVEL IN NATURAL STREAM CHANNELS
FROM USDA - SCS ADVISORY NOTICE COLO.-836
CASE I - WELL GRASSED SWALED BOTTOM
CASE II - FAIRLY WELL STABALIZED - MINOR EROSION WITH WELL DEFINED CHANNEL
CASE III - HEAVILY VEGETATED BOTTOM
CASE IV - ACTIVE EROSION
CASE V - LARGE STREAMS - VEGETATED BANKS
NOT SUITABLE FOR ROCKY MOUNTAIN STREAMS WITH OVERFALL
UNITED WESTERN ENGINEERS
0
1
2
3
4
5
6
7
8
9
AVERAGE CHANNEL SLOPE - PER CENT
KEUFFEL & ESSER CO. MADE IN U.S.A.

III. FLOW ROUTING

A. Inflows to Project Area: (Basins A through E) See Plate Two.

1. Basin A1 generates 49.6 cfs, which is concentrated in the shown ditch.

The upper end of this ditch commonly silts full and creates problems to the homeowners. Along the lower end of the ditch is a 36" x 58" x 24' CMP across Leslie Drive. This culvert will handle 54.3 cfs, if clean, however, it is half full of silt. The lower ditch capacity is 43.2 cfs. Below the culvert the sewer dike contains the flow, however severe erosion at the greenbelt is in progress.

A lined ditch is proposed from the culvert to the greenbelt.

2. Basin A2 generates 12.8 cfs into Leslie Drive, which is swaled and has poor hydraulic control. The street will contain 1.8 cfs near the outfall point, where curb control is required to protect the proposed greenbelt.

3. Basin A3 is sheet flow, 28.3 cfs.

4. Basin A4 contributes 140.9 cfs in a natural channel.

5. Basin B generates 268.9 cfs to an existing 6' x 2.5' x 15.5' reinforced concrete culvert, whose capacity is 258 cfs.

6. Basin C contributes 49.5 cfs which should be contained on the East side of Chelton Road. However, through Sections Z-Z and AA-AA it overtops the crown. Low runoff along this edge of pavement will run Westerly along the South side of Paseo Road to an existing 24 inch CMP (See Plate Three). The design runoff will jump Paseo Road and enter the main greenbelt.

7. Basin D flows 8.2 cfs into a natural sump on Chelton Road. This sump overfills in the average worst storm of the year and must flow between the houses @ 3223 and 3227 Austin Drive, which creates problems. Because of severe difficulty in maintaining this routing, we propose that the flow will be transmitted Northerly along Chelton Road.

8. Basin E1 sheet flows 18.8 cfs onto Chelton Road which collects at a low point near Grandview, thence down Grandview. The total basin E will contribute 44.0 cfs through a 14 foot curbed channel to the golf course. The capacity of this channel is 370 cfs.

B. Street Summary: The following is a summary of the ability of existing streets to accomodate the design runoff. Capacities calculated at the cross-section points (Plate Five) were computed to the top of the curb on the low side.

Street	Basin	Type	Slope %	Flow-CFS SCS	Flow-CFS City	Capacity CFS
Country Club	E2A	40' Ramp	0.8	10.2	5.9	14.1
	F18	40' Ramp	3.6	9.6	5.4	29.9
	F15-18	40' Ramp	5.8	40.7	23.3	78.7
Grandview	E1	40' As R	11.0	18.8	6.8	49.8
	E-E2A	40' Dipped	4.2	46.5	28.4	122.8
Marilyn	F16	40' Ramp	2.7	13.6	7.8	62.1
	F16	40' Ramp	2.7	13.6	7.8	62.5
	F16	40' Ramp	5.3	13.6	7.8	36.3
Lees Lane	F15-17	40' Ramp	2.0	31.1	17.9	17.9
	F11	40' Ramp	5.4	11.3	6.6	36.6
Highland	F9	40' Ramp	6.5	24.2	13.4	40.2
Chelton Dr.	F8	40' Ramp	5.6	20.1	11.1	37.3
Austin Dr.	F7	40' VC	1.4	27.1	15.1	23.8
	F5-6	40' R	1.2	30.7	17.5	5.8
	F5-6	40' R	1.2	30.7	17.5	6.2
Leslie Dr.	F9	40' VC	2.6			51.7
South	F9	40' VC	2.2	24.2	13.4	63.2
	F8-9	40' VC	5.7	44.1	24.5	100.7
	F7-9	40' VC	4.6	70.5	39.6	97.4
	F7-10	40' VC	5.8	82.4	41.9	33.3
Chelton	D + F1	Special	7.2	11.6	3.3	94.8
	D + F1	Special	6.4	11.6	3.3	76.2
	C	Special	7.2	49.5	16.6	20.2
	C	Special	6.4	49.5	16.6	153.1
	E	Special	6.0	58.1		38.4
	E	Special	4.0	58.1		3.8
Paseo	F2	Special	2.3	7.7		72.0
	F3+	Special	3.4	9.0	21.7	117.5
	F4+	Special	3.2	18.0	27.9	41.5
	F4+	Special	3.1	18.0	27.9	33.2
Leslie	A2	Special	0.8	12.8	5.6	1.8
North	A2	Special	4.6	12.8	5.6	47.2

Paseo Road from Country Club to Leslie is obviously insufficient as shown on the enclosed photo sheets.

IV. OUTFALL POINTS

Existing structures will accomodate the anticipated flows as follows:

Basin	Structures	Runoff-CFS SCS	Runoff-CFS City	Capacity CFS
HP #1	6'x2.5'x15.5'RCB	268.9	183.9	258
A1	36"x58"x24'CMP	49.6	19.9	54.3
F5+F6	20.5'CO & 24" CMP	30.7	17.5	16.8
F1+F2	24" + 36' CMP		14.1	7.7
HP #4	48" x 20' CMP	504.6	151.0	64.5
F11	3'x5'x30'RCB	11.3	6.6	169.1
F11	23' curb outlet	11.3	6.6	50.9
F12	Unlined ditch	51.1	28.7	27.2
HP #6	Unlined ditch	679.2	222.7	OK

V. INTERNAL DESIGN

The following is a summary of internal design computations for the two alternatives shown. Computations are enclosed.

A. By City Criteria: See Plate No. Three.

1. Catch Basin Sizing: All catch basins are standard D-10R, used with CMP outlet pipes, min. S=1%.

Street	Slope %	Throat Width Feet	Design Capacity CFS	CMP Size In.
Chelton	Sub'd	4	3.2	18
Chelton	6.4	12	9.3	21
Austin Dr.	1.4	10	15.1	21
Leslie	4.6	8(2 ea)	6.0(ea)	24
Leslie	5.8	16	16.5	30
Lees Lane	2.0	10	16.0	21
Country Club	5.8	10	7.3	21
Paseo Rd. at Country Cl.	Sub'd	4	4.0	18
Paseo Rd. at F2	2.0	10	14.1	24

Existing outlets on Austin Drive (F5 +F6) and Paseo Road (F11) will be used without revision.

2. Storm Sewers are to be CMP, std corrugations, n = 0.024 with a one foot minimum cover. Gage will be determined by resistivity testing. The following is a summary of the calculations.

Location	Size In	Design Flow CFS	Minimum Slope -%-
Chelton	18	3.2	0.32
Chelton	21	9.3	1.3
Austin Dr.	21	15.1	3.1
Leslie Dr.	24	23.1	4.9
Leslie Dr.	42	43.6	0.64
Country Club	21	16.0	3.48
Country Club	30	23.3	1.10
Country Club	36	24.6	0.46
F12	30	28.7	1.67
Paseo	24	14.1	1.3

3. RCB: The 6' x 3' x 20' reinforced concrete box culvert on Leslie Drive will accomodate the design flow of 151.0 cfs under inlet control with

a minimum headwater depth of 1.05 feet. The water surface is well below the crown at the outlet, so that the Leslie Drive storm sewer may stub into the box for energy dissipation.

4. Concrete Lined Channel: n = 0.015. The following is the design summary of the various concrete channels. Curvature and transition details are in the calculations.

Location	Size(bxdxz) Ft	Design Flow CFS	Velocity fps	Freeboard Ft
Main Greenbelt	4 x 2.5 x 1.5	208.7	22.8	1.03
Main Greenbelt	4 x 3 x 1.5	222.7	19.5	1.27
F12	2 x 2 x 1.5	28.7	6.9	0.87
Paseo Inlet	2 x 2 x 1.5	say 30	14.3	1.31
A1 inlet	2 x 2 x 1	19.9	13.0	1.45
Lees Ln. inlet	5'x8"x Vert	6.6	10.0 ±	0.54

5. Riprap Channel: n=0.035, will be grouted, six feet wide, four feet deep and 40 feet long in order to dissipate the greenbelt flow to 6.6 feet per second. Details of a down stream drawdown are not known at this time.

B. By Soil Conservation Service Criteria: The Soil Conservation Service hydrology previously discussed was used for design runoff. Other departures from current criteria are discussed below. See Plate No. Four.

1. Catch Basin Sizing: All catch basins are standard D-10R, sized in accordance with the Denver Urban Storm Drainage Criteria Manual. Since the deflector slots and throat depth is constant in all D-10R basins, the capacity on sloping streets is a simple function of gutter flow depth (in a normal gutter above the opening) and the slope of the street. On submerged catch basins, the LA Country Flood Control District Criteria was used. Outlet pipes are an entirely separate problem. The sizing of catch basins is summarized as follows. Design sheets are included.

Normal Gutter Flow				Basin Width	Catch Basin Flow	
Street	CFS	%	Depth	Ft.	Actual	Capacity
Chelton	8.2	Sub'd	0.67	4	8.2	8.9
Chelton*	56.6			5' special*	56.6	
Paseo	7.7	2.5	0.59	4	7.7	8.5
Austin	30.7	Sub'd	1.07	6	17.6	17.6
Austin	27.1	1.4	0.6	16	10.2	10.2
Austin	16.9	1.4	0.52	16	8.6	8.6

Leslie Dr.(RT)	24.7	4.6	0.36	16'	7.9	7.9
Leslie Dr.(LT)	11.1	4.6	0.16	16'	2.3	2.3**
Leslie Dr.(RT)	28.6	5.8	0.36	16'	9.9	9.9
Leslie Dr. (LT)	12.9	5.8	0.16	16'	2.5	2.5**
Leslie Dr.(RT)	7.1	5.8	0.11	16'	1.4	1.4**
Leslie Dr.(LT)	33.9	5.8	0.61	16'	19.1	19.1
Lees Ln.	31.1	2.0	0.68	16'	12.0	12.0
Lees Ln.	19.1	2.0	0.49	16'	8.2	8.2
Country Club	20.5	5.8	0.21	16'	4.2	4.2**
Paseo	22.5	Sub'd	1.00	8'	22.5	22.6

* Special design to accomodate ditch flow.
**These may as well be eliminated due to their low capacity, if the resulting street flows do not appear excessive.

In order to double the capacity of the above inlets, the opening would have to be quadrupled. The use of the cities new "high velocity" catch basin may not be feasible for this reason.

2. Storm Sewers are to be standard CMP as in the City criteria, designed to flow as full as possible under no head. The summary of the storm sewer trunk lines is as follows:

Street	Size of Pipe In.	Flow CFS	Slope %	Normal Depth of Flow Ft.
Austin outlet	24	13.1	2.5	1.20
Austin outlet	24	30.7	6.0	1.68
Austin outlet	30	30.7	2.2	1.90
Austin	18	10.2	3.9	1.12
Austin	24	18.8	3.9	1.32
Leslie	24	29.0	7.3	1.46
Leslie	30	41.4	5.9	1.65
Leslie	48	61.9	0.8	2.92
Lees Lane	24	12.0	0.9	1.70
Lees Lane	24	20.2	5.3	1.26
Country Club	30	24.4	2.0	1.65
Paseo	48	46.9	0.6	2.72

3. Connector Pipes are to be CMP as before. They are designed to accomodate the required flow operating under the maximum head (H) that will permit the catch basin to act as if it were empty. The depth of the catch basin alows six inches of freeboard above the head (hi) required to cause the outlet pipe to flow full. (See Page 14 of the enclosed calculations.) The sizes may be summarized as follows. Note that the required depths under this criteria would make 6 out of the 13 catch basins on Dwg.D-10R of insufficient depth.

Street	Design Flow CFS	Pipe Length Ft	Head H Ft.	Head hi Ft.	Required Size in	Resulting Depth Ft.
Austin outlet	17.6	63	1.04	0.28	30	4.00
Austin #1	10.2			0.73	18	4.30
Austin #2	8.6	25	4.40	0.52	18	3.52
Leslie #3	2.3	28	4.74	0.04	18	3.04
Leslie #4	7.9	28	3.94	0.44	18	4.94
Leslie #5	2.5	28	0.85	0.04	18	3.04
Leslie #6	9.9	28	0.85	0.37	21	3.62
Leslie #7	19.1	28	0.40	0.16	36	4.66*
Leslie #8	1.4	28	0.90	0.01	18	3.01*
Lees #1	12.0	20	1.30	0.55	21	3.80
Lees #2	8.2	20	1.74	0.47	18	3.47
Country Cl.#3	4.2	28	1.25	0.12	18	3.12
Paseo #4	22.5	30	0.78	0.46	30	4.46
Chelton	8.2	360	3.00	0.47	18	3.47
Paseo	7.7	45	0.50	0.13	24	2.13*

* Outlet is submerged by design water levels in the open channel or RCB designs.

4. Culvert Design: The two cell 8' x 4' reinforced concrete box culvert on Leslie Lane is controlled by the required inlet head of 1.00 ft. - although a double 7-1/2' x 4' would work at the inlet. However, the 8' x 4' would allow the Leslie Drive culvert to stub in midway down without affecting the headwater.

5. Open Channel Design may be summarized as follows, where channel size is b x d x z. Curvature and transition design are in the calculations.

Location	Size Ft	Design Flow CFS	Maximum Velocity fps	Minimum Freeboard Ft
Chelton	2x1.5x2	56.6	15.6	0.48
Greenbelt	5x3.5x1.5	636.4	32.1	1.06
Greenbelt	5x3.5x1.5	679.2	30.6	0.97
F12	4x2x1.5	51.1	8.1	0.49
A1	2x2x1.5	49.6	27.9	1.10
Lees Lane	5'x8"xVert	11.3	15.5	0.52
Paseo Inlet	2x2x2	38.5	9.9	1.02

The grouted riprap section at the end will be 9 x 6 x 1.5 and must be 85 feet long to provide a velocity of 6.3 fps on a level slope. Details on downstream drawdown will be taken into account on the final design.

UNITED WESTERN ENGINEERS

Page Index of

Project Van Buren 7

Calc. by OEW date

Checked by date

HYDRAULIC COMPUTATIONS

UNITED WESTERN ENGINEERS

Project Van Buren VII
Calc. by OEWatts date 9-4-73/9-8
Checked by ______ date ______

Hydraulic Computations

Part I - Capacities of Existing Structures

Basin A1 $Q = 49.6$ CFS

Culvert: 36" × 58" × 24' CMP equiv size = 48"ϕ

$h_i = 0.5'$

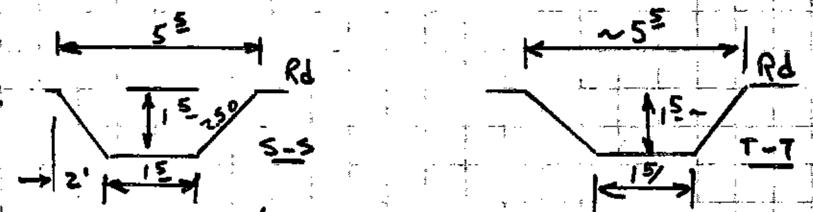
$H' = 0.75'$ half full of silt

$H'\ cap = 66/2 = 33$ CFS

$h_i\ cap = 0.022 V^2 \quad 0.022 \frac{Q^2}{11.4^2} = 0.5' \quad Q = 54.3/2 = 27.2$

h_i limits - would handle 54.3 CFS if clean

Ditch



Grouted Stone
$n = 0.035$
$S = 5\%$
$S^{1/2} = 0.2236$

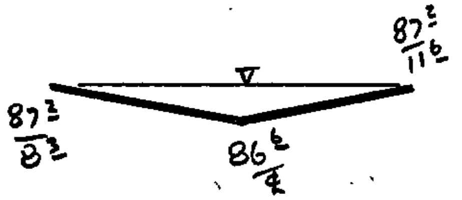
$A = 5.25$ □'
$WP = 6.50$
$R = 0.8077$
$R^{2/3} = 0.8673$

$$Q = \frac{1.486}{0.035} \times 5.25 \times 0.8673 \times 0.2236 = \underline{43.2} \text{ CFS}$$

6.4 CFS under capacity - need ditch control upstream of culvert

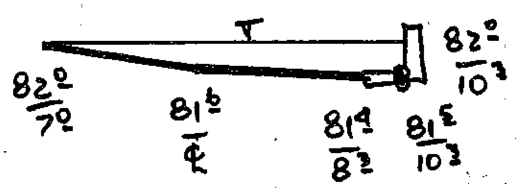
Basin A2 Runoff is in street $Q = 12.8$ CFS

asphalt $n = 0.018$



Sec R-R
$S = 4.6\%$

$Q = 47.2$ CFS



Sec Q-Q
needs Curb
$S = 0.8\%$

w/o curb $Q = 1.8$ CFS
$Q = 53.2$ CFS w/curb

Existing - cont

Basin A3 - sheet Flow

Basin A4 - sheet Flow - wide gulley no problem

Basin B $Q = 268.9$ CFS $6' \times 2.5' \times 15.5'$ RCB

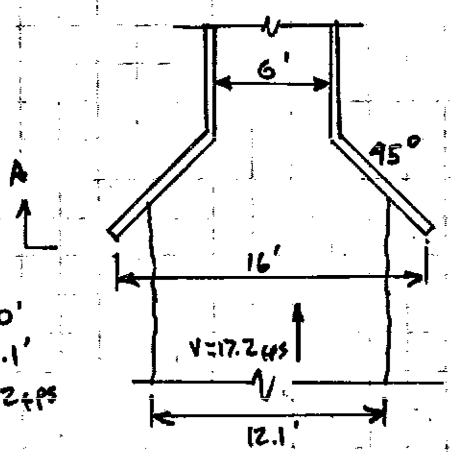
Under head $h_i = 0.1'$ as it $h_i = 2.2'$ if road To Top of parapet
$H' = 2.2'$ max

w/road To Top of parapet Q by $H' = 166$ CFS Q by $h_i = 170.6$ CFS

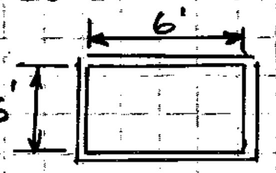
as is: h_i cap. $= 0.017\, V^2 = 0.017 \dfrac{Q^2}{(6 \times 2.5)^2} = 0.1$ $Q = 36.4$ cfs if clear

Max. Capacity To be determined by velocity flow. Check for 268.9 CFS

Re: Sec 4 Denver Manual



Sec AA: d=2.0'
T=12.1'
V=17.2fps



$V_n = \dfrac{Q}{A}$ $d_1 = \dfrac{268.9}{17.2 \times 6} = 2.61'$

$> 2.50'$

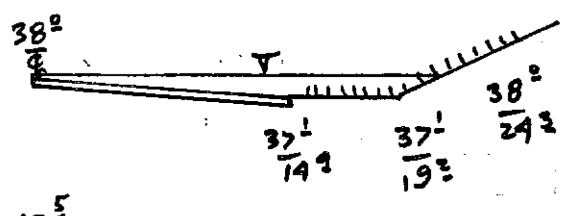
For Max Capacity: $Q \approx 17.2 \times 6 \times 2.5 = 258$ cfs with no backwater

need 5.4' Road fill above culvert - no way

Basin C 49.5 cfs - RT side of Chelton Rd

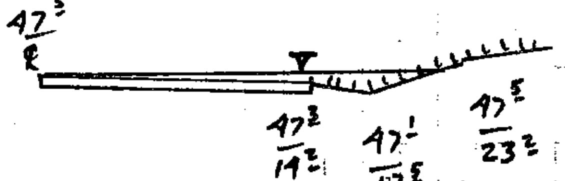
Note: on section AA-AA 46.5% of Total flow is on RT side

sec C-C:



n=0.020
S = 6.35%
Q = 153.1 CFS OK
V = 11.7 fps

sec D-D:



n=0.020
S = 7.2%
Q = 20.2 cfs NG
water over crown
V = 5.7 fps

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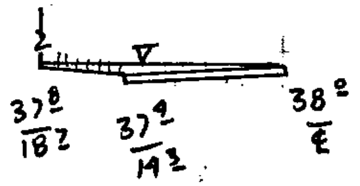
existing — cont

Basin D + F1 + F2 w side Chelton Rd & ~~So.~~ North Side Paseo Rd

Note: on Section AA-AA - 53.5% of Total flow goes To LT side of Road

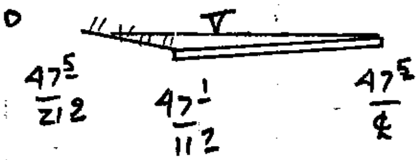
D+F1 — Q = 11.6 CFS Lt side Shelton

Sec C-C



S = 6.35%
n = 0.020
Q = 76.2 ok
V = 10.2 fps

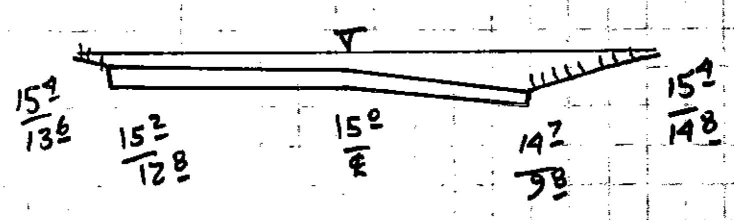
Sec D-D



S = 7.2%
n = 0.020
Q = 94.8 cfs OK
V = 10.8 fps

D+F1+F2 — Paseo Rd Q = 15.9 cfs

Sec E-E

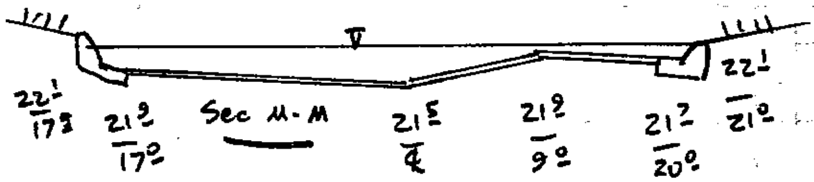


S = 3.2%
n = 0.018
Q = 72.0 ok
water on North Side
V = 6.5 fps

Basin E

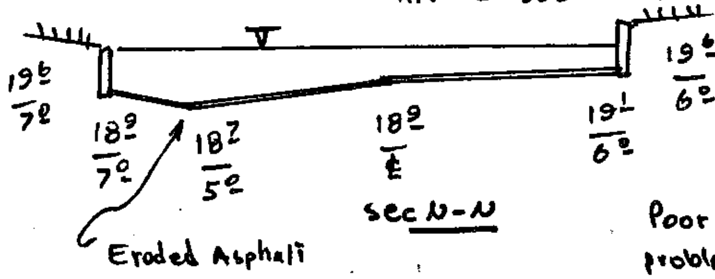
Very Poor Control on E1 (18.8cfs) onto Grandview

E(-) E2(A) 46.5 cfs on Lower Grandview



n = 0.018
S = 4.4%
Cap = 122.8 cfs
V = 8.8 fps

EE 44.0 cfs Thru Curbed outlet To Golf Course



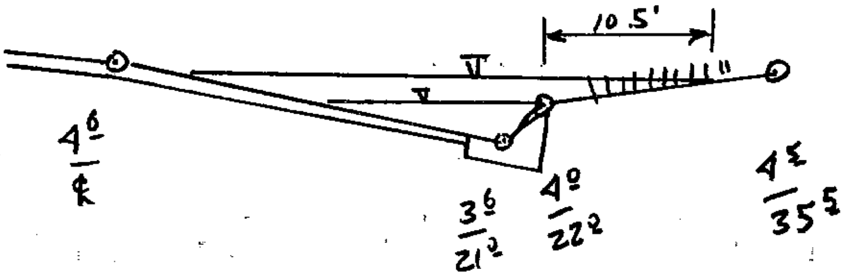
n = 0.018
S = 4.2%
Cap = 370.0 cfs
V = 20.1

Backwater will prevent This Poor Inlet Control — no problems in 5-6yrs To Mrs Smith, 2906 Country Club Ln

Existing - Cont

Basins F5 + F6 - 20.5' Curb outlet on Austin Drive

$Q = 30.7$ CFS



Cap To T/C $n = 0.018$
$Q = 5.78$ CFS
$V = 3.07$ fps

Cap of 30.7 CFS $n = 0.023$
WS elev = 4.34
$V = 3.41$ fps
Water on lawn as shown

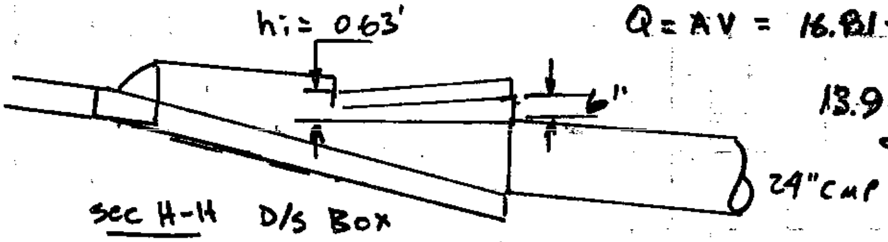
Sec II - 50' above D/s end of Outlet $S = 1.2\%$

Capacity of curb outlet
$d = 0.79'$
$w = 20.5'$
$S = 1.2\%$

$Q = 3.25\,W^{0.83}\,D^{2.0}$
DC - 5.12 LA Manual
$D = 0.79$
$Q_{cap} = 21.83$ CFS ← N6

8.9 CFS continues downstream ← N6
Mrs Harrison (3262) had basement water 2 yrs ago!

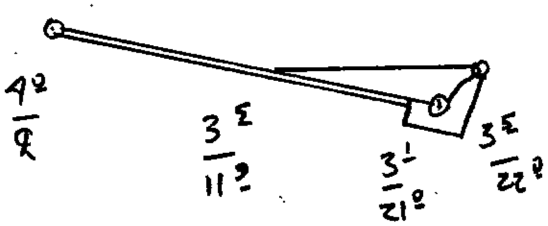
Inlet Capacity of 24" CMP outlet
$h_i = 0.022V^2 = 0.63'$
inlet $V = 5.35$ fps
$Q = AV = 16.81$ - say 16.8 CFS ←



13.9 CFS goes downstream ←

Sec H-H D/S Box

For Downstream Street Capacity:



$n = 0.018$ $S = 1.2\%$
$Q = 6.2$ CFS $V = 3.09'$
Curb R = 140'

No wonder Mrs Harrisson had a flooded basement.

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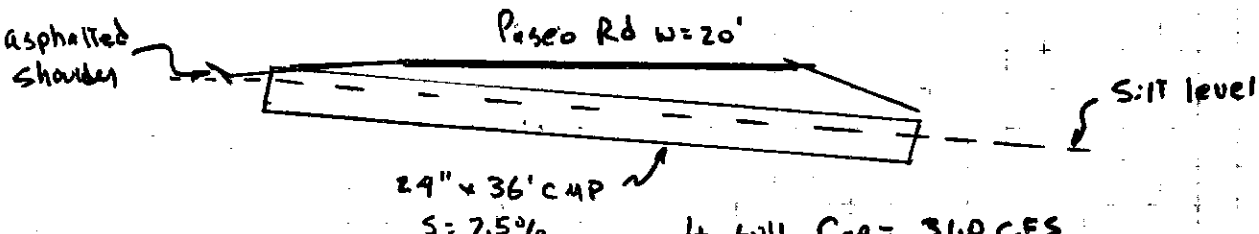
Existing - Cont

Basins F1 & F2 Q = 7.7 CFS

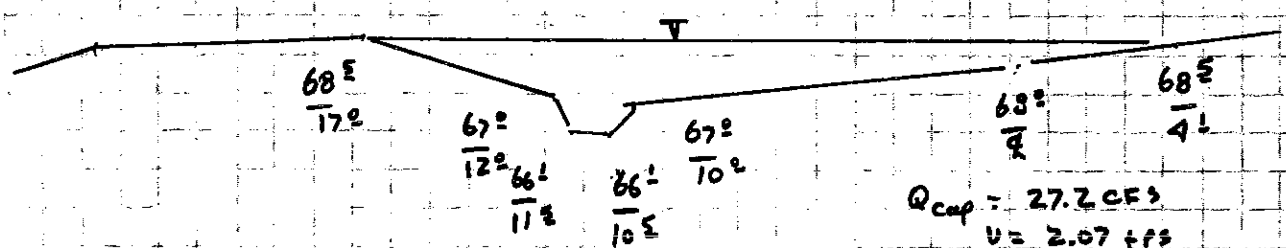


$\frac{1}{2}$ full Cap = 31.0 CFS
req'd $h_i = 0.022 V^2 = 2.14'$ – none available

for Q = 7.7 CFS we need $h_i = 0.13'$
(if its clean)

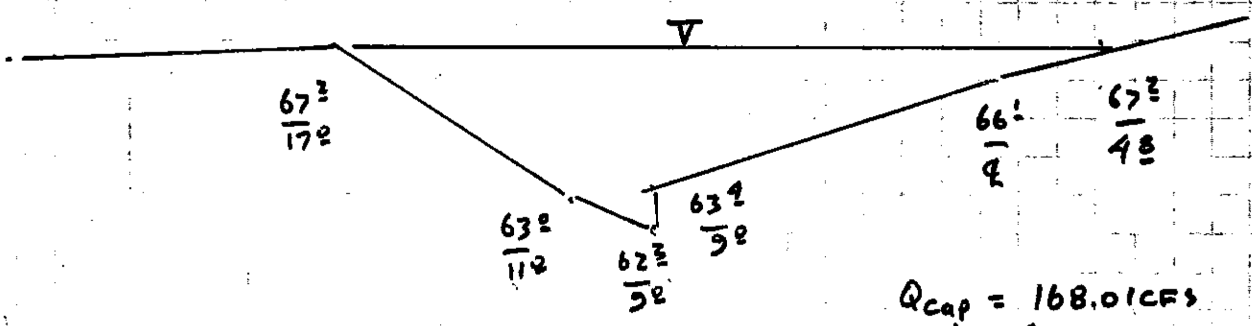
Basins F4, 7 to 10 Q = 82.4 CFS inlet works but is badly flooded

Basins F12-19 Q = 51.1 CFS Unlined outfall from Paseo Rd



Q_{cap} = 27.2 CFS
V = 2.07 fps

Sec O-O S = 0.633%
n = 0.040



Q_{cap} = 168.01 CFS
V = 4.11 fps

Sec P-P S = 0.633%
n = 0.040

36" x 58" CMP:

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Existing - Cont

Lees Lane & Paseo Road Outlet

23-foot Curb opening, Submerged Condition

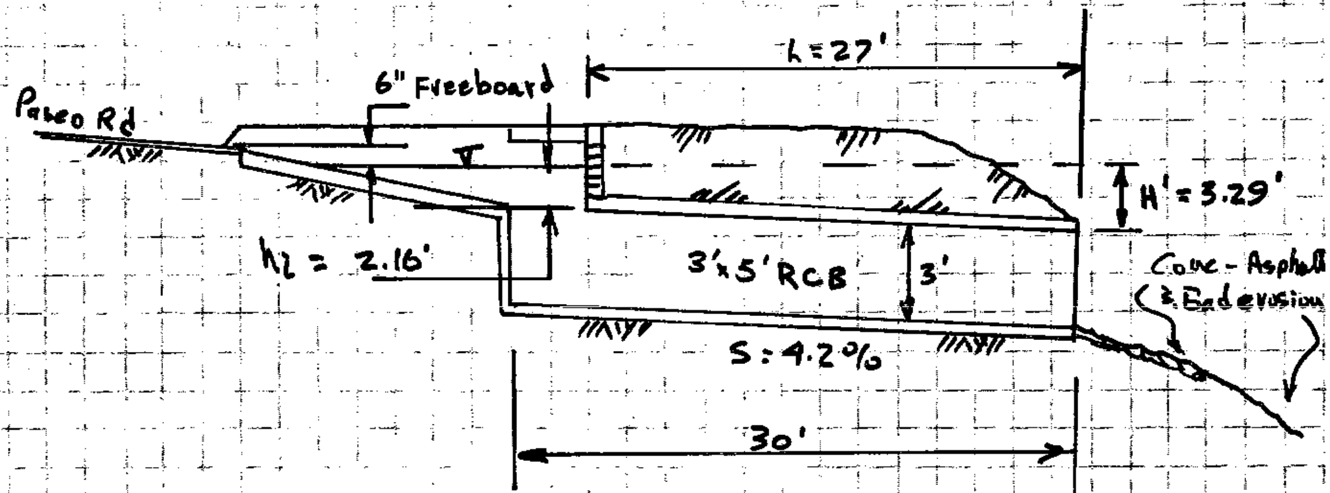
$Q = 4.3 A D^{0.6}$ DC-5.13 LA Manual

where $A = w \times 0.656$

$w = 23'$

for $D = 8''$ $Q = 50.9$ CFS ← Surface flow

for Maximum Capacity of Outlet



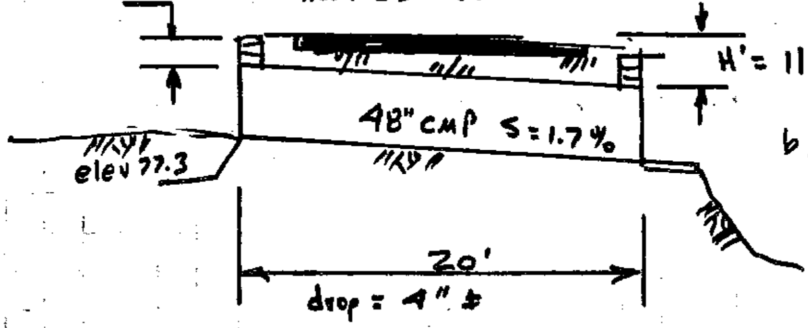
H' $Q = 205$ CFS $h_i = 0.017 V^2 = 3.17'$ NG

flow limited by h_i $Q^2 = \frac{2.16}{0.017} \times 15^2 = 28{,}588$

$Q = 169.1$ CFS ← Max. Total flow

Leslie Drive Culvert

Hydrograph PT #4 $Q = 504.6$ CFS



by H' $Q = 72$ CFS

$h_i = 0.022 V^2 = 0.72'$ NG - could be filled to get this

by h_i: $Q^2 = \frac{\cancel{0.42}\ 0.58 \times 12.57^2}{0.022} = \cancel{3016.46}\ 4165.58$

$Q = \cancel{54.9}$ CFS ←
64.5

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Project Van Buren 7
Calc. by OEW date 9-13-73
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Existing - Cont

Summary of Street Capacities

Full Capacities by Cross-Section Calculated To Top of Curb on Low side

Street	Basin	Type	Slope %	Capacity By Chart	Capacity By X-Sec	Flow - CFS SCS	Flow - CFS City	Capacity - CFS
Country Cl.	E2A	40' Ramp	0.8	X		10.2	5.9	14.1
	F18	✓	3.6	X		9.6	5.4	29.9
	F15-18	✓	5.8		W-W	40.7	23.3	78.7
Grandview	E1	40' As R	11.0	X		18.8	6.8	49.8
	E-E2A	40' Dipped	4.2		M-M	46.5	28.4	122.8
Marilyn	F16	40' Ramp	2.7		K-K	13.6	7.8	62.1
	✓	✓	2.7		L-L	✓	✓	62.5
	✓	✓	5.3	X		✓	✓	36.3
Lees Ln	F15-17	✓	2.0		X-X	31.1	17.9	17.9
	F11	✓	5.4	X		11.3	6.6	36.6
Highland	F9	✓	6.5	X		24.2	13.4	40.2
Chelton Dr	F8	✓	5.6	X		20.1	11.1	37.3
Austin Dr	F7	✓ VC	1.4		V-V	27.1	15.1	23.8
	F 5-6	40' R	1.2		I-I	30.7	17.5	5.8
	✓	✓	1.2		H-II	30.7	17.5	6.2
Leslie Dr South	F9	40' VC	2.6		J-J	—	—	51.7
	F9	✓	2.2	X		24.2	13.4	63.2
	F8-9	✓	5.7	X		44.1	24.5	100.7
	F7-9	✓	4.6		U-U	70.5	39.6	97.4
	F7-10	✓	5.8		T-T	82.4	41.9	33.3
Chelton	D+F1	Special	7.2		D-D	11.6	3.3	94.8
	✓	✓	6.4		C-C	11.6	3.3	76.2
	C	✓	7.2		D-D	49.5	16.6	20.2
	C	✓	6.4		C-C	49.5	16.6	153.1
	Σ	✓	6.0		Z-Z	58.1	—	38.9
	Σ	✓	4.0		AA-AA	58.1	—	3.8
Paseo	F2	✓	2.3		E-E	7.7	—	72.0
	F3+	✓	3.4		H-H	9.0	21.7	117.5
	F4+	✓	3.2		Y-Y	18.0	27.9	41.5
	F4+	✓	3.1		S-S	18.0	27.9	33.2
Leslie North	A2	✓	0.8		Q-Q	12.8	5.6	1.8
	A2	✓	4.6		R-R	12.8	5.6	47.2

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Project Van Buren 7
Calc. by OEW date 9-13-73
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BY CITY CRITERIA

Street and Storm Sewer Calculations

$Q = \frac{0.463}{n} D^{8/3} S^{1/2}$ n=0.024 cmp

Elev's from Topo

STREET	LOCATION	DIST	ELEVATION & SLOPE	TOTAL RUNOFF	STREET FLOW CAPACITY	PIPE FLOW	TYPE PIPE, CATCH BASIN & SLOPE %
Chelton Rd	Basin D		88.0/In 84.0				4' CB d=4'
		360'	0.56%	3.2		3.2	18" CMP min S = 0.0032
	End Pipe		82.0				
	Paseo Rd Right		6334/inv 30	9.3	9.3/153.1		12' CB (50% full) d=4'
		150'	1.33%			9.3	21" CMP (18" 14 min S = 1.17%)
	Greenbelt		6328				
Paseo Rd	Basin F2		19.0/Inv 15.0	14.1	14.1		10' CB d=4'
		60'	2.0%			14.1	24" CMP min S = 1.32%
	Hyd Pt #2		14.0				
				27.9	27.9/33.2		
	Leslie Dr						
Austin Dr	Sec I-I						
			1.2%	17.5	17.5/5.8		
	Sec H-H					17.5	exist 20.5' Outlet
		116	2.5%				24" CMP S=2.5% (exist.)
	Outlet						
	Leslie Dr - Sec VV		1.9%	15.1	15.1/23.8	15.1	10' Catch Basin d=4'
Leslie Dr	From Leslie CB		98.6/Inv 94.3				10' CB min hi=0.86 min d = 3.61'
		75'	4.13%	39.6	24.5/97.3	15.1	24" CMP min S = 3.10%
	Section U-U		Inv 91.2				2-8' CB's with 24" CMP's
		175'	7.54%	43.6	16.5/33.3	27.1	24" CMP min S = 4.9%
	Section T-T		Inv 78.0		Dried up		16' CB with 30" CMP @ 0.6% min
		120'	0.75%	43.6		43.6	42" CMP min S = 0.69%
	Greenbelt		Inv 77.1				RCB - see greenbelt design
Country Club Ln	Lees Ln Sec X-X		84.5/Inv 80.5		17.9/17.9 !!	16 to CB	10' CB
		255'	5.69%	23.3	7.3/78.7	16.0	21" CMP min S = 3.48%
TC 70.9	Section W-W		Inv 66.0				10' CB (50%) 21" CMP min S = 3%
		50'	2%	23.3	0	23.3	30" CMP min S = 1.10%
	Paseo Rd		Inv 65.0		4.0		4' CB w 18" CMP 1% L=12'
		210'	0.98%	24.6		24.6	36" CMP min S = 0.96%
	End Culvert		Inv 64.0				See open channel design

Culvert & Channel Calculations

$K = \frac{Qn}{b^{8/3} S^{1/2}}$ n=0.015 conc, n=0.035 grr $b^{8/3} = \frac{Qn}{K S^{1/2}}$

Elev's from Topo for D/b=1, z=1, K=1.93, A=zb² ① for D/b=0.5, z=1.5, K=0.599, A=0.875 b² ②

Freeboard

AREA	LOCATION & DISTANCE	ELEV & S%	S 1/2	Q50	b 8/3	b	S F AREA	USE DITCH	CULVERT ETC.	~~TIME HRS~~
Main Greenbelt	Hyd PT #4	77.3								
	20'	1.50%	0.1225	151.0	Special	Culvert	Design	Con	6x3 RCB	h_i=2'
	End Culvert	77.0								
	270'	5.19%	0.2278	208.7	② 22.94	3.29 V=22.26	9.17	4'x2.5' Conc	z=1.5	1.03'
	Hyd PT #5	63.0								
	205'	3.27%	0.1808	222.7	② 30.85	3.62 V=19.53	11.45	4'x3' Conc	z=1.5	1.27'
	Grade Break	56.3								
Use	40' min → 30'	1.00%	0.1000	222.7	② 130.13	6.21 V=6.61	33.71	6'x4' grot RR	z=1.5	0.86'
	Hyd PT #6	56.0								
F12 Ditch	End Pipe	62.64			② 8.012	2.18 V=6.88	4.17	2'x2' conc	z=1.5	0.87'
	460'	0.80%	0.0897	28.7	~~① 2.487~~	~~1.41~~	~~3.96~~	~~1.5'x2' Conc~~	~~z=1~~	~~0.62'~~
	Inlet	60.3								
	63'	3.65%	S=1.67 % min	28.7					30" CMP	h_i=0.75'
	Hyd PT #6	58.0								
Paseo Road Inlet	End Asph Swale	70.0								
	62'	5.48%	0.2342	Say 30.0	② 3.208	1.548 V=14.29	2.10	2'x2' Conc	z=1.5	1.31'
	Greenbelt	66.6								
A1 Ditch	End Culvert	78.0								
	142'	5.63%	0.2374	19.9	① 2.099	1.321 V=13.01	1.53	2'x2' Conc	z=1	1.45'
	Grade Break	70.0								
	30'	0.2337	0.4830	19.9	① —	— V=22.20	0.896	2'x2' Conc	z=1	1.66'
	Greenbelt	63.0								
Lees Lane Inlet	End 5x3 RCB	70.0								
	43'	0.1628	0.4035	6.6	~~② 0.412~~	~~0.716~~	~~0.448~~	5'x8" Conc	~~z=1.5~~ Vert	0.66'
	Greenbelt	63.0			③ 25.908	3.389	0.057	Corbel Slab		

③ D/b=0.05, K=0.00947, A=db, z=0, n=0.015

City Criteria - Cont

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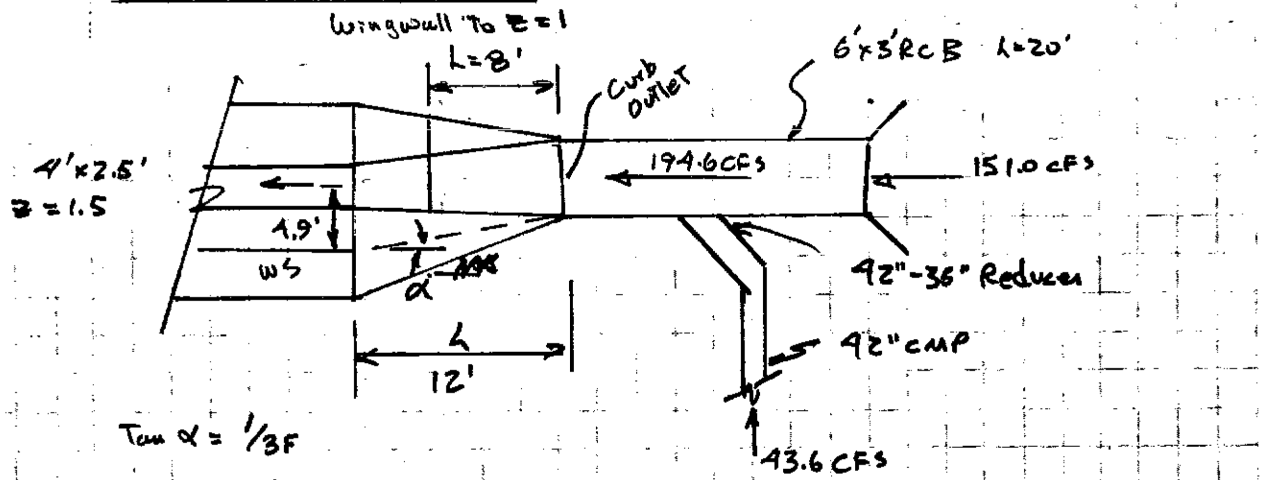
City Criteria Hydraulics - Cont

Hydraulic Details of Greenbelts

Outlet Leslie Drive Culvert



Tan α = 1/3F

$F = \frac{V}{\sqrt{gd}}$

$V_1 = 194.6/18 = 10.81$ fps

$V_2 \cong 22.\cancel{39}$ fps Ave = 16.8 fps $d_{ave} = 2.2'$

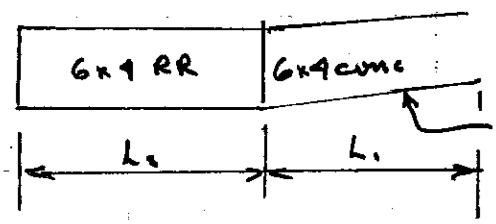
d = 1.4'

A = 8.54 ft² Fr = 1.996

S = 5.19%

L = (4.9 - 3.0) 3F = 11.37' - use 12'

Length of riprap section by backwater curve



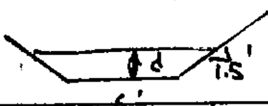
4x3 conc Q = 222.7

$Fr = \frac{19.53}{(32.2 \times 1.73)^{1/2}} = 2.62$

$L_1 = 2 \times 3F = 15.7'$ use 16'

normal depth in 6x4 conc

$AR^{2/3} = \frac{222.7 \times 0.015}{1.486 \times 0.1808} = 12.43$



$A = 6d + 1.5d^2$ $wP = 2d\sqrt{3.25} + 6$

d	A	wP	R	$R^{2/3}$	$AR^{2/3}$
1.5	12.375	11.41	1.085	1.056	13.06
1.4	11.34	11.03	1.026	1.018	11.54
1.45	11.854	11.23	1.056	1.037	12.29
1.46	11.957	11.26	1.062	1.041	12.44

V = 18.63 fps

City Criteria - Cont

Hyd Details - cont

$Q = 222.7$

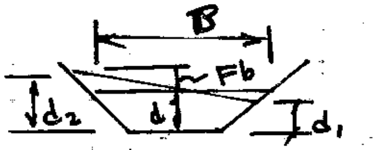
For h_2 $\quad \Delta h = \dfrac{\Delta H}{S_o - S_{ave}}$ where $S = \dfrac{n^2 Q^2}{2.25 A^2 R^{4/3}}$

$S_o = 1\%$ $\quad H = V^2/2g + d$ $\quad n = 0.035$

Loc	d	A	V	$V^2/2g$	H	wP	$R^{4/3}$	S	S_{ave}	h
Top	1.96	11.96	18.63	5.39	6.85	11.26	1.084 ~~1.267~~	0.17419		
					3.03				0.09195	36.97
Bottom	3.14	33.71	6.61	0.68	3.82	17.32	2.430	0.00978		

Note - use h = 40' min
check on final design

For radius of Curvatures:



$d_2 - d_1 = \dfrac{V^2 B}{gR}$ or $R = \dfrac{V^2 B}{g(d_2 - d_1)}$

where $d = \dfrac{d_2 + d_1}{2}$ & $\dfrac{d_2 - d_1}{2} + d \not> d + Fb$

For min R $\quad \dfrac{(d + Fb) + d_1}{2} = d$

Loc	V	d	Fb	max $d_2 - d_1$	B	Min R	Use
Hyd PT 5	22.76	1.47	1.53	3.00	8.41	45.10'	100'
dit A1	22.20	0.34	1.66	2.00	2.68	20.50	wait

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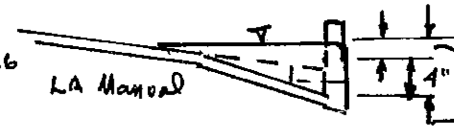
Part III - Detailed Hydraulic Design

Using SCS hydrology,
Catch Basin Charts,
Maximum pipe head - 6" min FB
on all catch basins

Catch Basin Sizing

Sub'd $w = Q/2.8208D^{0.6}$ LA Manual



(for design ≯ 0.6' on slopes

See D-10R charts per Denver Manual for D < 0.6'
~~For D > 0.6' + Basin acts submerged~~ D's from X-Sec's or Calc'd from Topo

Street	Basin	Total Q (Surface)	Slope %	D -FT-	Use Catch Basin	Catch Basin Q-CFS	Catch Basin Cap-CFS
Chelton	D	8.2	sub'd	0.67	4' D-10R	8.2	8.9
	C	56.6	—	—	5' Special	56.6	56.6
Paseo	F2+	7.7	2.5	0.59	4' D-10R	7.7	8.5
Austin Dr	F5+F6	30.7	sub'd	1.07	6' D-10R	17.6	17.6
	✓	13.1	see	p 4	Exist 20.5' outlet	13.1	16.8
Austin Dr	F7	27.1	1.9	0.60	16' D-10R	10.2	10.2
	F7	16.9	1.9	0.52	16' D-10R	8.6	8.6
Leslie Dr	F7-9 RT	24.7	4.6	0.36	16' D-10R	7.9	7.9
	F7-9 LT	11.1	4.6	0.16	16' D-10R	2.3 *	2.3
	F7-9 RT	28.6	5.8	0.36	16' D-10R	9.9	9.9
	F7-9 LT	12.9	5.8	0.16	16' D-10R	2.5 *	2.5
	F7-10 LT	33.9	5.8	0.61	16' D-10R	19.1	19.1
	F7-10 RT	7.1	5.8	0.11	16' D-10R	1.4 *	1.4
Lees Ln	F15-17	31.1	2.0	0.68	16' D-10R	12.0	12.0
	F15-17	19.1	2.0	0.49	16' D-10R	8.2	8.2
Country Cl.	F15-19	20.5	5.8	0.21	16' D-10R	4.2 *	4.2
Paseo	F14-19	22.5	Sub'd	1.00	8' D-10R	22.5	22.6

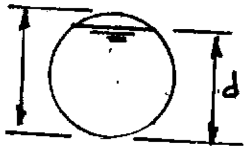
* Normal used

Part III - SCS hydraulics - Cont

For Size & hydraulic Gradient of Trunk Lines

Pipe Sized To flow under no head $Q = \frac{0.463}{0.024} D^{8/3} S^{1/2}$

min 18" D
CMP
n=0.024



$\frac{Q \times 0.024}{D^{8/3} S^{1/2}} = K$

See Tbl 21 USBR for d/D

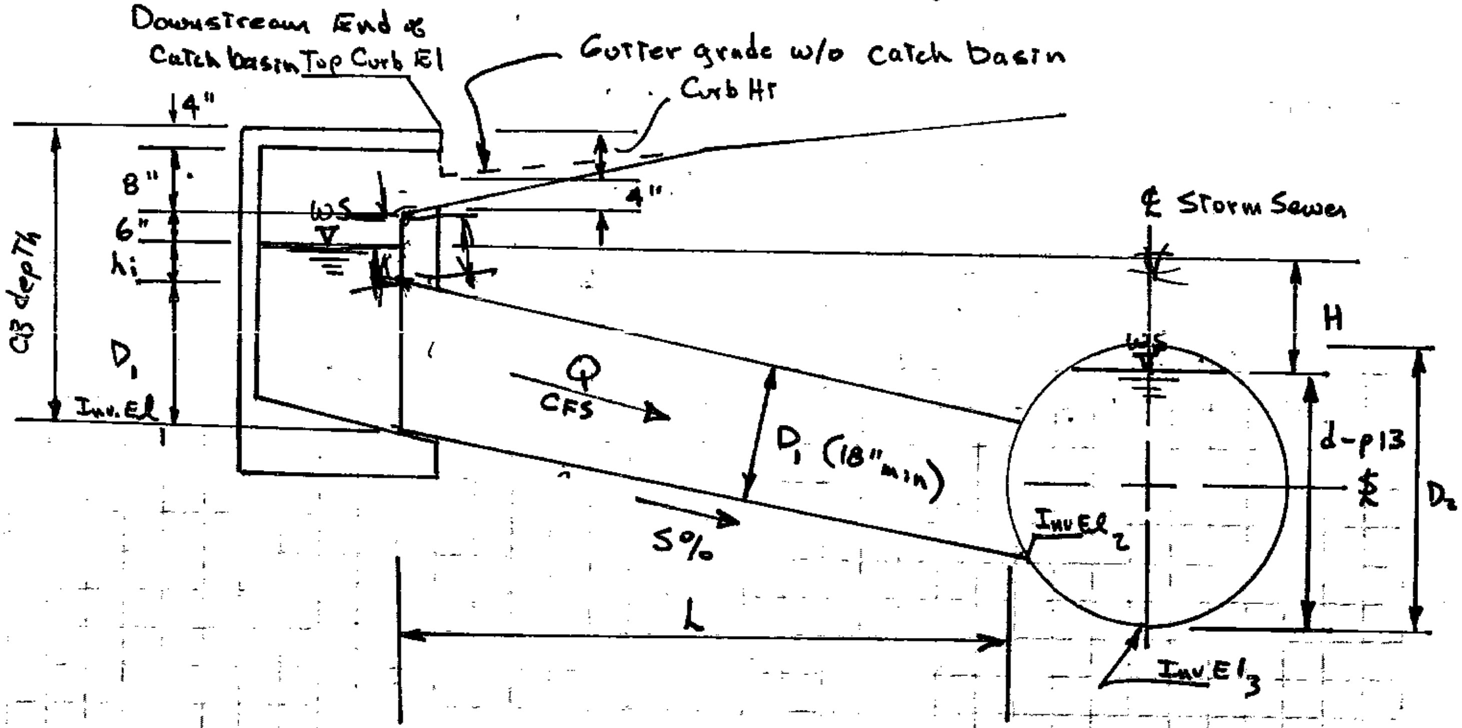
Using Ground Surface Elev's / Inv Elev

Street	Loc & Dist	Elev & S%	$S^{1/2}$	Q50	Pipe Size	$D^{8/3}$	K	d/D	d GT
Chelton	4' CB ~~360'~~ Outlet	88/84 ~~0.56%~~ /6382	~~0.07453~~ See p 15	~~8.2~~	~~24"~~	~~6.350~~	~~0.416~~	~~0.74~~	~~1.48~~
Austin	20.5 CO exist 32' Tie In	6300.57 2.5% /99.78	0.1581	13.1	24"	6.350	0.313	0.60	1.20
	Tie In	~~2.5%~~	~~0.1581~~	~~30.7~~ Won't work	~~24"~~	~~6.350~~	~~0.733~~	Under Pressure	
	0 99.5' C.B Ex.	6.01% /93.8	0.2451	30.7	24"	6.350	0.473	0.84	1.68
	172.5 Greenbelt	2.20% /90.0	0.1483	30.7	30"	11.51	0.431	0.76	1.90
Austin	CB #1	99.3/95.0							
	92' CB #2	3.94% /91.38	0.1984	10.2	18"	2.948	0.419	0.795	1.12
Leslie	35' CB #3-4	3.94% /90.0	0.1984	18.8	24"	6.350	0.358	0.66	1.32
	96' CB #5-6	7.29% /83.0	0.2700	29.0	24"	6.350	0.406	0.73	1.46
	85' CB #7-8	5.88% /78.0	0.2425	41.4	30"	11.51	0.356	0.66	1.65
	110' Greenbelt	0.818% /77.1	0.0905	61.9	48"	40.32	0.407	0.73	2.92*
Lees Ln	CB #1	/81.0							
	110' CB #2	0.91% /80.0	0.0953	12.0	24"	6.350	0.476	0.85	1.70
Country Club	255' CB #3	5.29% /66.5	0.2301	20.2	24"	6.350	0.332	0.63	1.26
Paseo	75' CB #4	2.00% /65.0	0.1414	24.4	30"	11.51	0.360	0.66	1.65
Ditch	180' End Pipe	0.556% /64.0	0.07454	46.9	48"	40.32	0.375	0.68	2.72

* depth of outlet to be elev 81.1 - min depth of flow @ CB 7-8 must be elev 81.1, not 80.92.

Part III - SCS Hydraulics - Cont

For Catch Basin Depths & Connector Pipe Sizes



For D_1: (1) Max WS in CB = TC - 1.50'
(2) Max WS in Storm Sewer (from p 13) = Inv El + d For Downstream Flow!
(3) Using h & H & Q, D Taken from California Chart

For CB Depth:
(1) $h_i = 0.022V^2$ for CMP
(2) CB Depth = $D + h_i + 1.50$ (minimum) May use 4'-0"

For Connector Pipe Slope:

(1) $Inv\ El_1 = TC - CB\ Depth$
(2) $Inv\ El_2 = Inv\ El_3 + \frac{1}{2} D_2 - \frac{1}{2} D_1$
(3) $S = (El_1 - El_2)/L$

Note: If Catch Basin is Submerged The above does not apply.

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Project Van Buren VII
Calc. by DEW date 9-15-73
Checked by ______ date ______

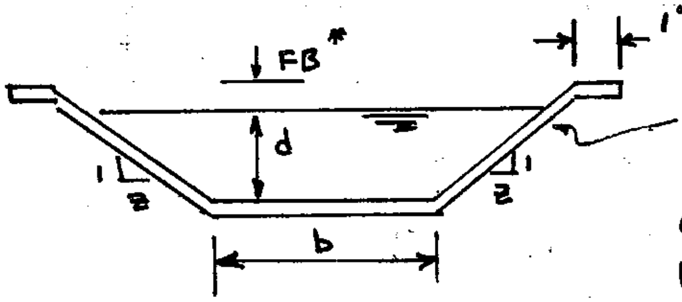
Part III - Cont

Catch basins - Cont * Catch basin Submerged

Street	Loc	T/c El	Water Surface CB	Water Surface SS	Q_{50} cfs	L	H	D,	A_i	CB Inv.	S %
Austin	I-I *	309.0	302.5	301.46	17.6	63	1.09	30"	0.28	300.0 *	0.35
Austin	CB#1	99.3	97.8	96.12	10.2	—	~~1.68~~	18"	0.73	95.0	3.99
	CB#2	98.6	97.1	92.70	8.6	25	4.40	18"	0.52	95.08	13.2
Leslie	CB#3 LT	97.7	96.2	91.46	2.3	28	4.74	18"	0.04	94.66	16.6
	CB#4 RT	96.9	95.4	91.46	7.9	28	3.94	18"	0.49	91.96	7.0
	#5 LT	87.0	85.5	84.65	2.5	28	0.85	18"	0.04	83.96	3.9
	#6 RT	87.0	85.5	84.65	9.9	28	0.85	21"	0.37	83.38	1.4
	#7 LT	83.0	81.5	81.1 (1)	19.1	28	0.40	36"	0.16	78.34	1.2
	#8 RT	83.5	82.0	81.1 (1)	1.4	28	0.90	18"	0.01	80.49	8.9
Lees Ln	#1	85.5	84.0	82.70	12.0	20	1.30	21"	0.55	81.30	3.5
	#2	84.5	83.0	81.26	8.2	20	1.74	18"	0.47	81.03	5.2
Country Club	#3	70.9	69.4	68.15	4.2	28	1.25	18"	0.12	67.78	4.6
Paseo	#4 *	70.0	68.5	67.72	22.5	30	0.78	30"	0.46	65.59	1.8
Chelton	D	88.0	86.5	83.5 Inv = 82.0	8.2	360	3.0	18"	0.47	84.53	0.70
Paseo	F2	18.5	18.0 (1)	17.5 (1)	7.7	95	0.5	24"	0.13	16.37	0.82

(1) Special Case - Outlet of Pipe is submerged by flood plain as deeper depth of flow in greenbelt channel

Greenbelt Designs see following Page



4" conc lining w/ 6x6-6/6 wwf
$n = 0.015$

$$\frac{Q n}{b^{8/3} S^{1/2}} = K$$

Labled as b x d , z = ?

See Table 18 USBR for K

* FB = 0.5' where against a street which helps contain the flow
FB = 1.0' elsewhere

Culvert & Channel Calculations

$K = \frac{Qn}{b^{8/3} S^{1/2}}$ or $b^{8/3} = \frac{Qn}{K S^{1/2}}$

n = 0.015 Conc 0.035 grout riprap

Elev's from Topo

AREA	LOCATION & DISTANCE	ELEV & S%	S 1/2	Q50	b 8/3	b	S F AREA	USE DITCH	CULVERT ETC.	Freeboard ~~TIME~~ ~~HRS~~
C+D	Z-Z	6351.9								
	50'	3.60%	0.1897	56.6	6.59	2.03	4.11	2'x1.5' conc	V= 13.77	0.98'
d/B = 0.5	A-A	49.6								
z = 2	250'	5.00%	0.2236	56.6	5.59	1.91	3.69	2'x1.5' conc	V= 15.55	0.56'
K = 0.679	C-C	37.1								
A = 6z²	85'	4.82%	0.2196	56.6	5.69	1.92	3.69	2'x1.5' conc	V= 15.34	0.56'
n=0.015	Special Inlet	33.0/Inv 29.9				0.76'	5' box	d = 3.34' min		
	120'	1.38%	min S =	56.6					58"x31" CMP	h_i = 0.76'
	Greenbelt	6328								
Main Greenbelt	Hyd pT #4	72.3								
	20'	1.50%	0.1225	504.6	Special Culvert Design				Dbl 8'x9' RCB	h_i = 1.0'
z = 1.5	End Culvert	72.0								
A = 0.875b²	270'	5.19%	0.2277	636.4	69.99	4.919	21.17	5'x3.5' conc	V=32.0 fps	1.06'
n=0.015	Hyd pT #5	63.0								
d/b = 0.5	135'	5.19%	0.2277	679.2	74.70	5.091	22.23	5'x3.5' conc	V= 30.[illegible] fps	0.97'
K=0.599	Grade Break	56.0								
n=0.035	100'	0.00%	0.000	679.2	See Page #18		108.00	9'x7' grout riprap	V= 6.3 fps	1.80'
	Hyd pT #6	56.0								
F12 Ditch	End 48" CMP	64.0								
	530'	0.283%	0.0532	51.1	29.05	3.30	9.50	4'x2' Conc	V=5.4 fps	0.49'
d/b = 0.5, z = 1.5	Inlet C.B.	62.50								
K=0.599, n=0.035	60'	0.833%	0.0913	51.1	14.02	2.69	6.39	4'x2' conc	V=8.1 fps	0.88'
	Hyd PT #6	56.0/ws 62.00								
A1 DiT	End Culvert	78.0								
d/b = 0.5, z = 1.5	142'	5.63%	0.2374	49.6	5.232	1.86	3.027	2'x2' conc	V=16.4 fps	1.10'
K = 0.599	Grade Break	70.0								
A = 0.875b²	30'	0.2337	0.4830	49.6	2.572	1.43	1.78	2'x2' conc	V=27.9 fps	1.39'
	Hyd pT #5	63.0								

III SCS Method - Cont

UNITED WESTERN ENGINEERS

Project Van Buren 7 Page 16 of 20

Calc. by JEW date 9-16-73

Checked by ______ date ______

Culvert & Channel Calculations

$b^{8/3} = \frac{Qn}{KS^{1/2}}$

Part III - Cont

(1) $n = 0.018$ $d/b = 0.2$ $Z = 0$ $K = 0.0813$ $A = 0.2b^2$

(2) $n = 0.015$ $d/b = 0.5$ $Z = 2$ $K = 0.679$ $A = b^2$

AREA	LOCATION & DISTANCE	ELEV & S %	S 1/2	Q50	b 8/3	b	S F AREA	USE DITCH	CULVERT ETC.	~~TIME HRS~~ Freeboard
Lees Ln Ditch (1)	End 5x3 RCB 43' Hyd PT #5	70.0 0.1040 ~~0.1628~~ 63.0/ws 66.53	0.3224	[illegible]	5.604	1.908	0.728	5'x8" Curbed Swale	V= 15.5 fps	0.52'
Paseo Inlet (2)	End asphalt 62' Greenbelt	72.0 1.94% 68.9/ws 70.8	0.1391	38.5	6.113	1.972	3.89	2'x 2' Conc	V= 9.9 fps	1.02'

UNITED WESTERN ENGINEERS

Project Van Buren 7

Calc. by JBW date 9-17-73

Checked by ____ date ____

Page 17 of 20

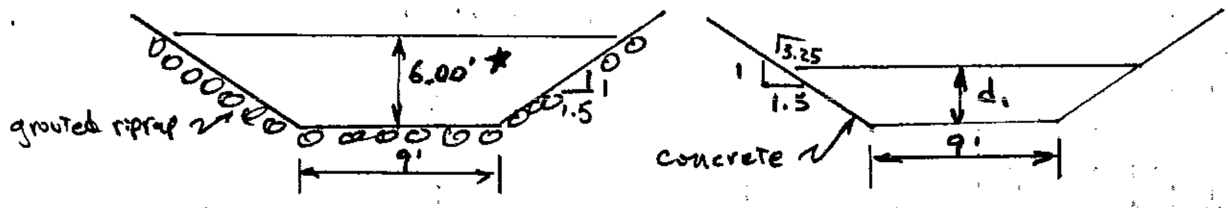
UNITED WESTERN ENGINEERS

Project: Van Buren VII
Calc. by: OE&W date: 9-17-77
Checked by: ______ date: ______

Part III - cont

Open Channel Details

For Length of Riprap Section $Q = \frac{1.486}{n} A R^{2/3} S^{1/2} = 679.2$ cfs



Sec 2-2 n = 0.035
Hyd pt # 6

Sec 1-1 n = 0.015/0.035
@ Top of riprap section

Length sufficient to obtain normal depth of flow in riprap section

$\Delta L = \frac{\Delta H}{S_0 - S_{ave}}$ $H = \frac{V^2}{2g} + d$ $S = \frac{n^2 Q^2}{2.25 A^2 R^{4/3}}$ $S_0 = 0\%$ - flat

For d_1: $S = \frac{77.0 - 56.0}{405'} = 5.19\%$ $S^{1/2} = 0.2277$ $AR^{2/3} = 30.11$

d_1	$A = 9d + 1.5d^2$	$WP = 2\sqrt{3.25}\,d + 9$	R	$R^{2/3}$	$AR^{2/3}$
2.00	24.000	16.211	1.480	1.299	31.17
1.90	22.515	15.861	1.420	1.264	28.45
1.96	23.402	16.067	1.457	1.285	30.07
1.97	23.551	16.103	1.463	1.288	30.39

← V = 29.02 fps

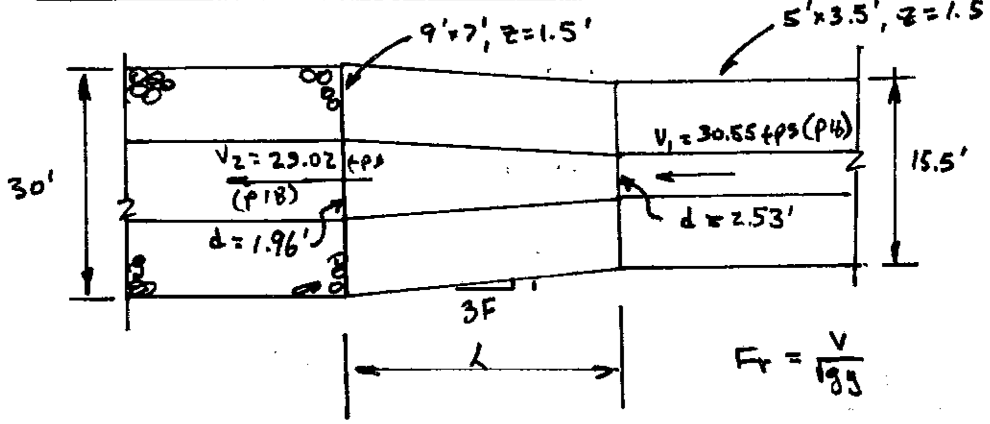
For d_2 @ L = 100' by backwater curve

* Velocity of 6.3 fps is sufficiently low - water surface will draw down to meet velocity downstream.

y	A	V	$V^2/2g$	H	WP	$R^{4/3}$	n	S	S_{ave}	ΔL	L
1.96	23.40	29.03	13.08	15.04	16.067	1.652	0.035	0.27765			0
				7.67					0.16834	45.56	
3.00	40.50	16.77	4.37	7.37	—	2.594	0.035	0.05904			45.56
				1.38					0.03948	34.95	
4.00	60.00	11.32	1.99	5.99	—	3.502	0.035	0.01992			80.51
				0.06					0.01413	4.25	
5.00	82.50	8.23	1.05	6.05	—	4.423	0.035	0.00834			84.76
									0.00617	0.01	
6.00 *	108.00	6.29	0.61	6.61	—	5.375	0.035	0.00901			84.77

Part III - cont

For Transition Length - Channel to Riprap

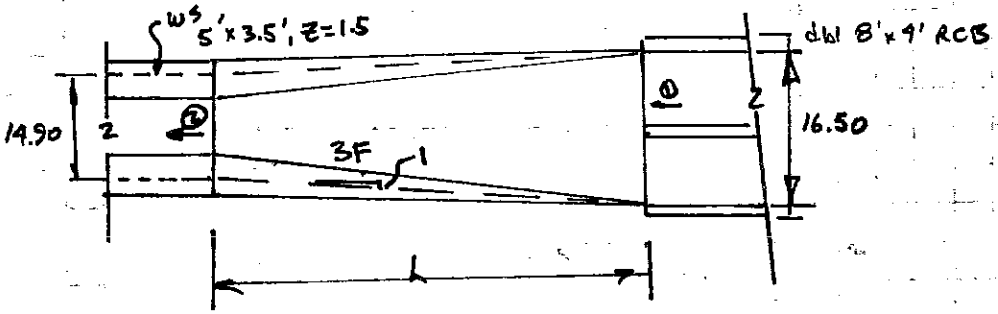


$Fr = \frac{V}{\sqrt{gy}}$

$Fr_1 = 3.38$ $Fr_2 = 3.65$ $Fr_{ave} = 3.52$

$L = \frac{(30' - 15.5')}{2 \times 3 F_{ave}} = 0.69'$ Use 10' min

For Transition Length - RCB to channel



$d_2 = 3.30'$
$V_2 = 32.08$ fps
$Fr = 3.11$

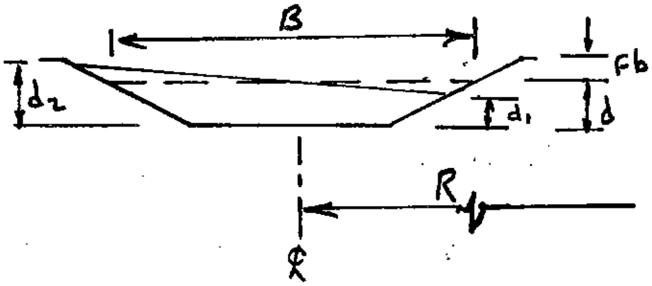
$d_1 = 4'$
$V_1 = 9.99$ fps
$Fr = 0.876$

$Fr_{ave} = 1.993$

$L = \frac{16.5 - 14.9}{6 F_{ave}} = 0.13'$ Use 10' min

III SCS Method - cont

Minimum Radius of Curves



$$d_2 - d_1 = \frac{V^2 B}{gR}$$

or

$$R_{min} = \frac{V^2 B}{g(d_2 - d_1)}$$

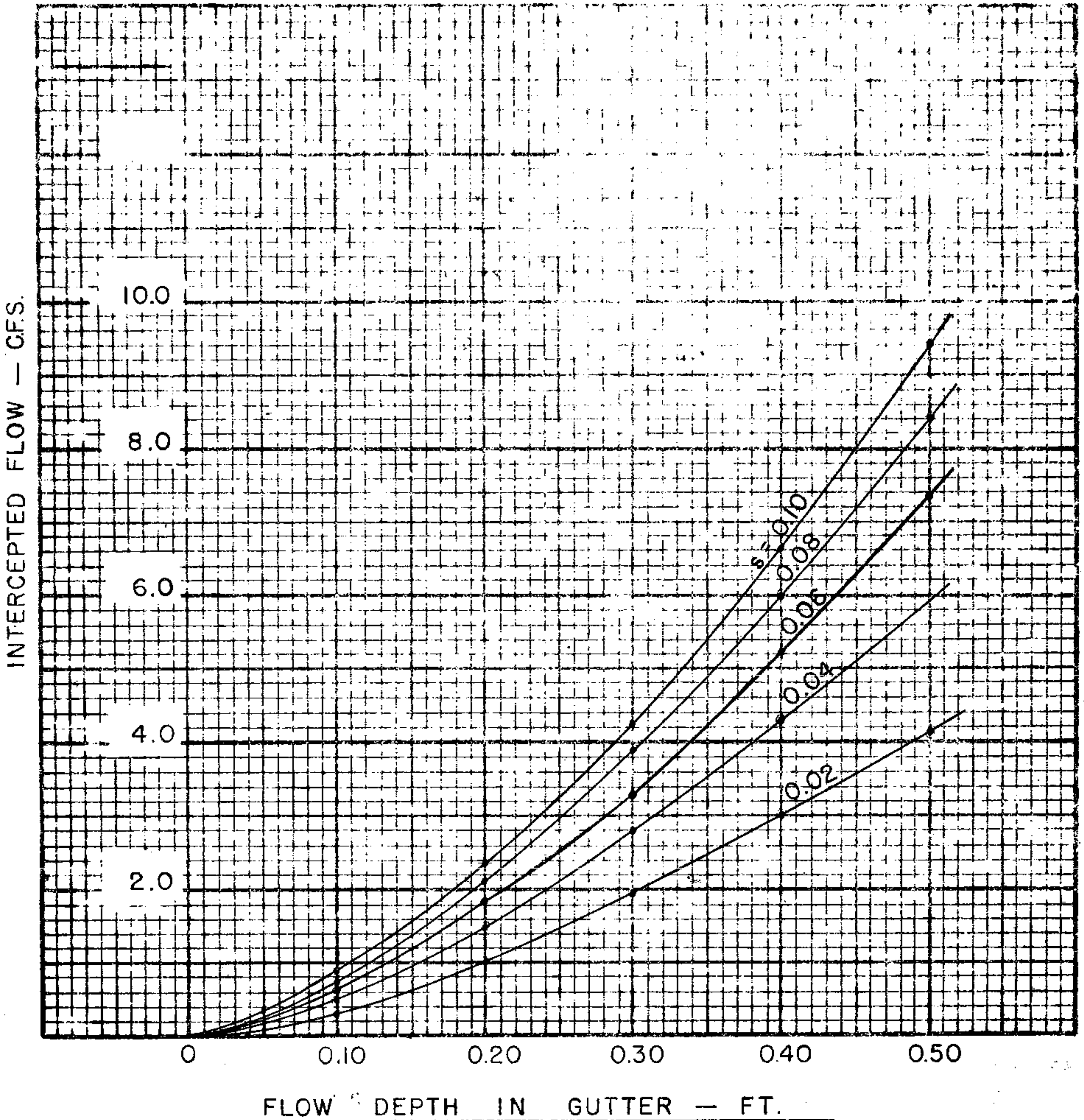
$B = b + 2zd$

where: $\frac{(d_2 + d_1)}{2} = d$

$d_2 - d = Fb$

$\therefore \frac{d_2 - d_1}{2} = Fb$

Ditch	Fb	d_2-d_1	V	B	min R
A1	1.39	1.22	27.9	3.83	75.89' Use 100'
	1.10	1.80	16.4	4.70	35.69' Use 50'
GB e HP #5	0.97	1.94	30.55	12.59	188.10 Use 200'



h = 4'
w = 4'-0"
c/b = 2
n = 0.016

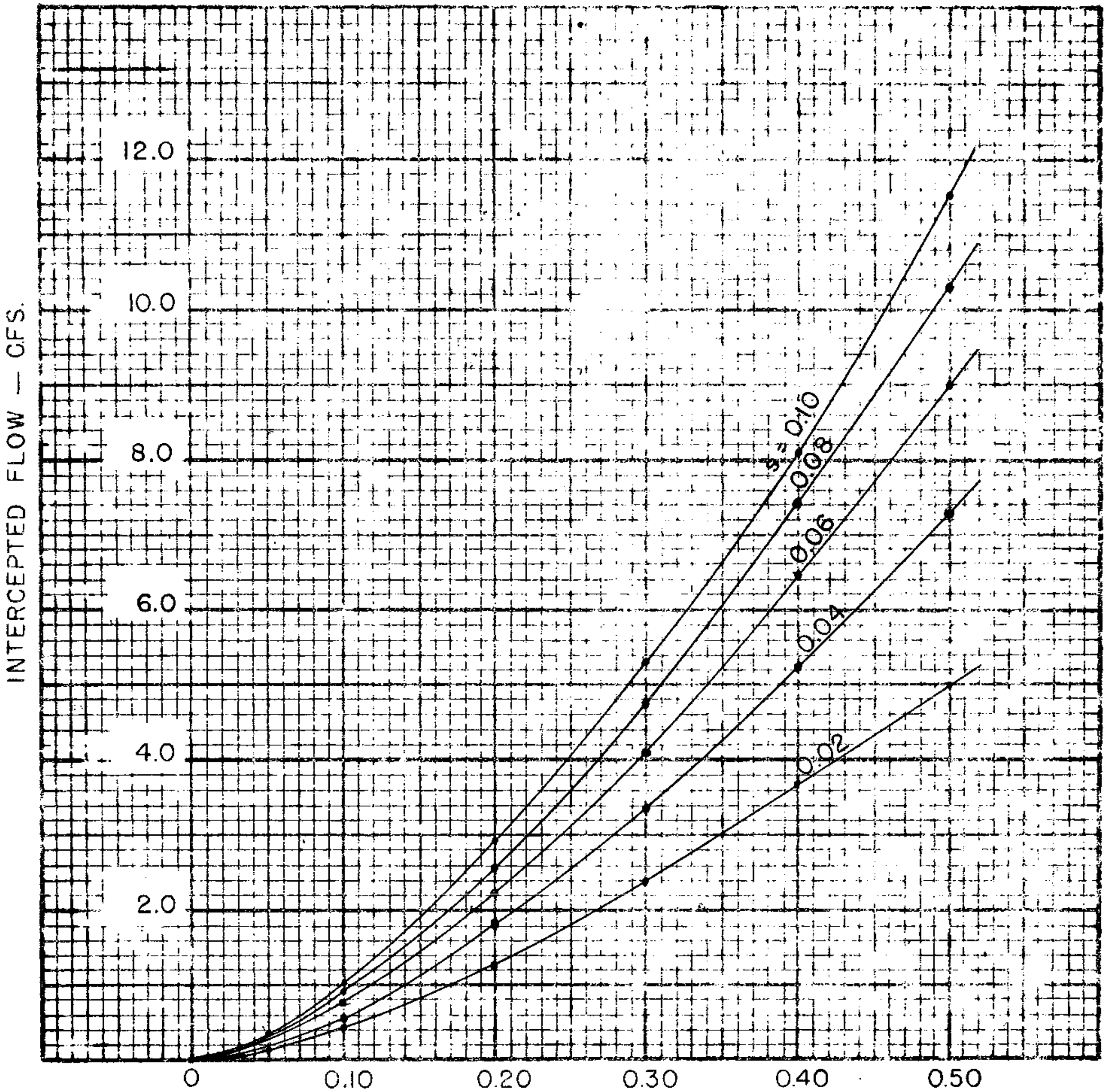
CITY OF COLORADO SPRINGS
STD. DRAWING NO. D-10 R

Compiled from Urban Storm Drainage Criteria Manual, City of Denver

Subdivision ______________ Street ______________ Station ______

Calc'd. by ______________ Date ______

4 FOOT CATCH BASIN CAPACITY	UNITED WESTERN ENGINEERS COLORADO SPRINGS, COLORADO	Fig. 2:1



FLOW DEPTH IN GUTTER — FT.

h = 6'
w = 4' - 0"
c/b = 2
n = 0.016

CITY OF COLORADO SPRINGS
STD. DRAWING NO. D-10 R

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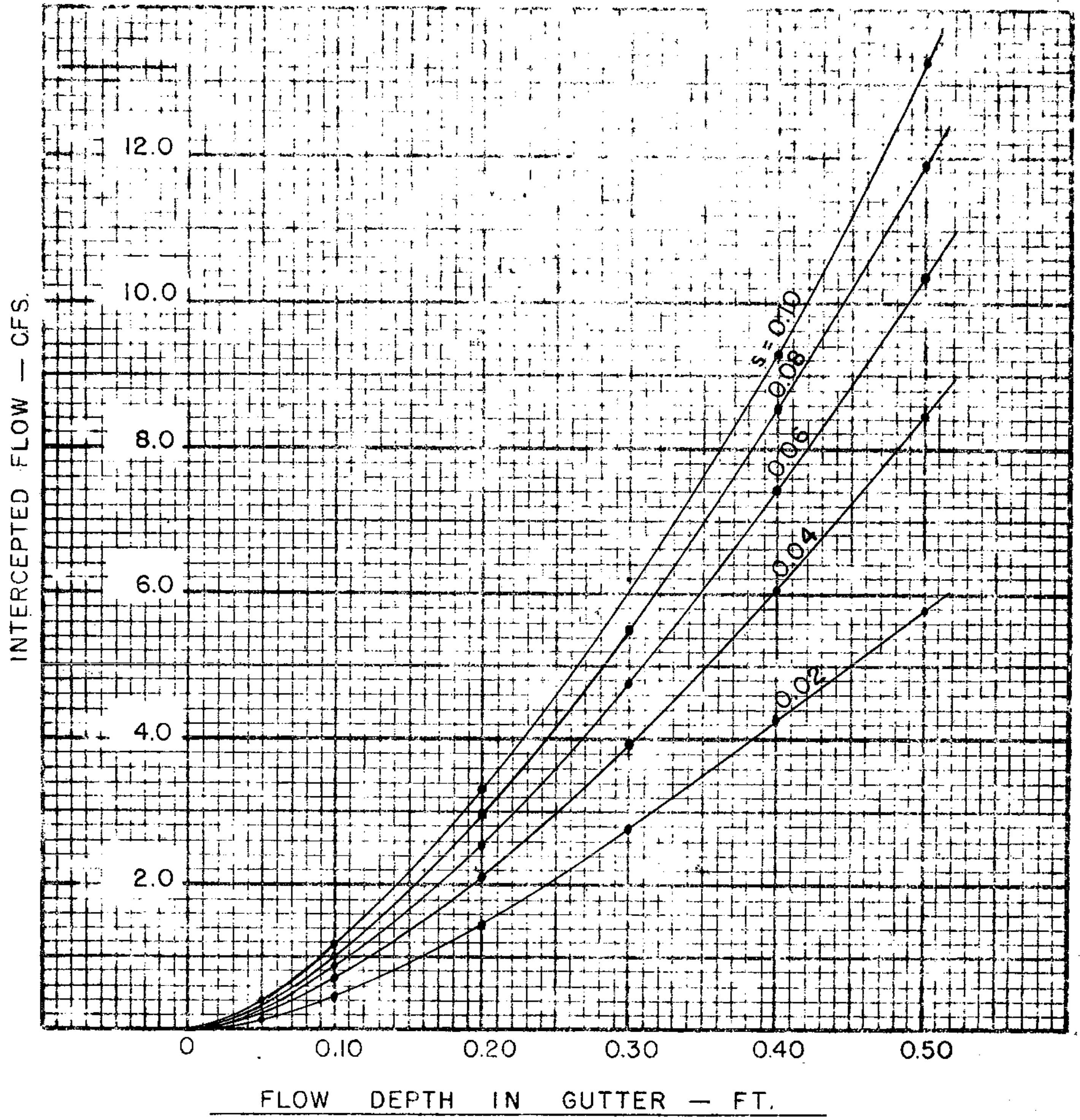
Subdivision ______ Street ______ Station ______
Calc'd. by ______ Date ______

6 FOOT CATCH BASIN CAPACITY	UNITED WESTERN ENGINEERS COLORADO SPRINGS, COLORADO	Fig. 2.2



h = 8'
w = 4' - 0"
c/b = 2
n = 0.016

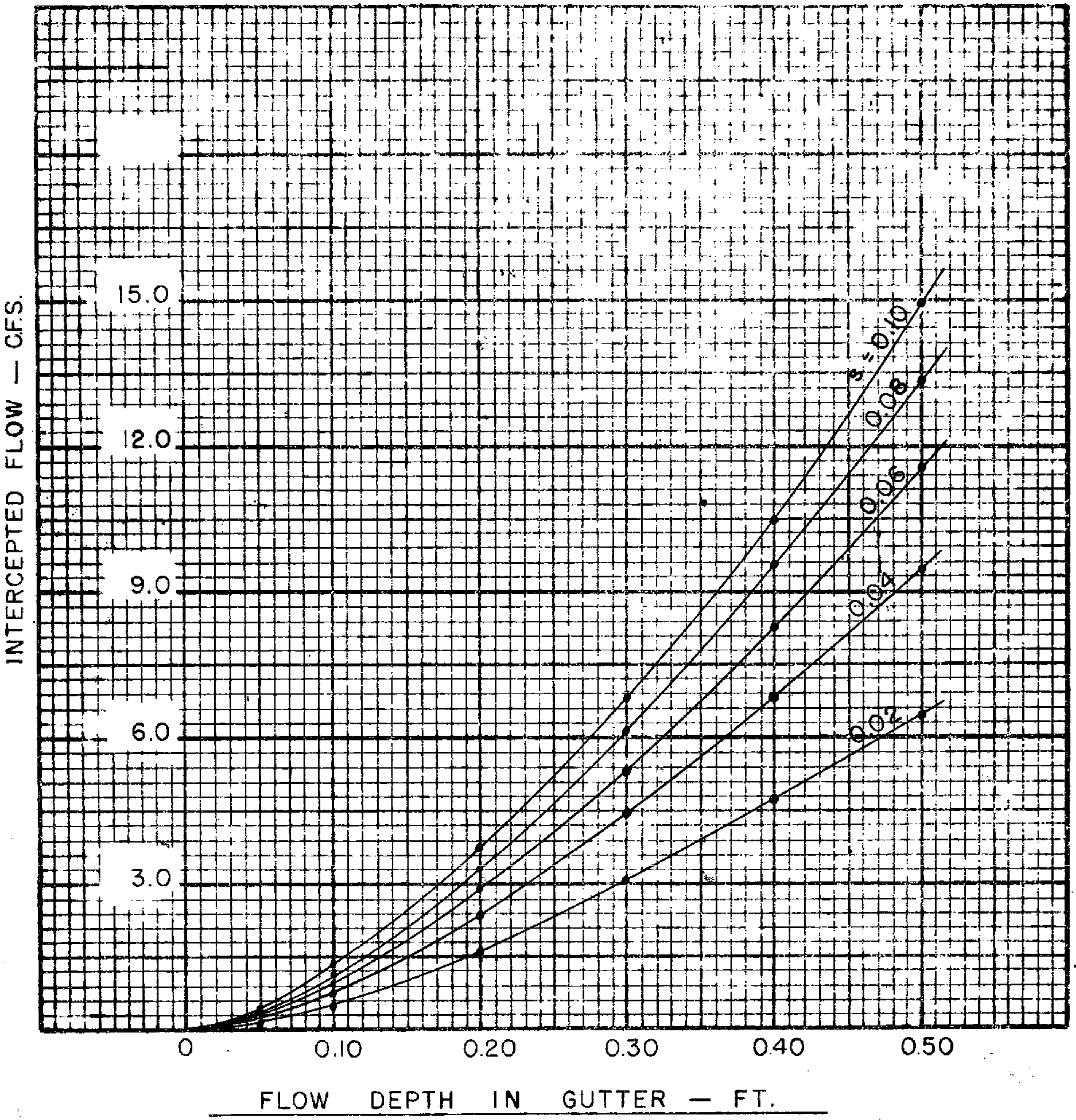
CITY OF COLORADO SPRINGS
STD. DRAWING NO. D-10 R

Compiled from Urban Storm Drainage Criteria Manual, City of Denver

Subdivision ________ Street ________ Station ________

Calc'd. by ________ Date ________

8 FOOT CATCH BASIN CAPACITY	UNITED WESTERN ENGINEERS COLORADO SPRINGS, COLORADO	Fig. 2-3



h = 10'
w = 4' - 0"
$c/b = 2$
n = 0.016

CITY OF COLORADO SPRINGS
STD. DRAWING NO. D-10 R

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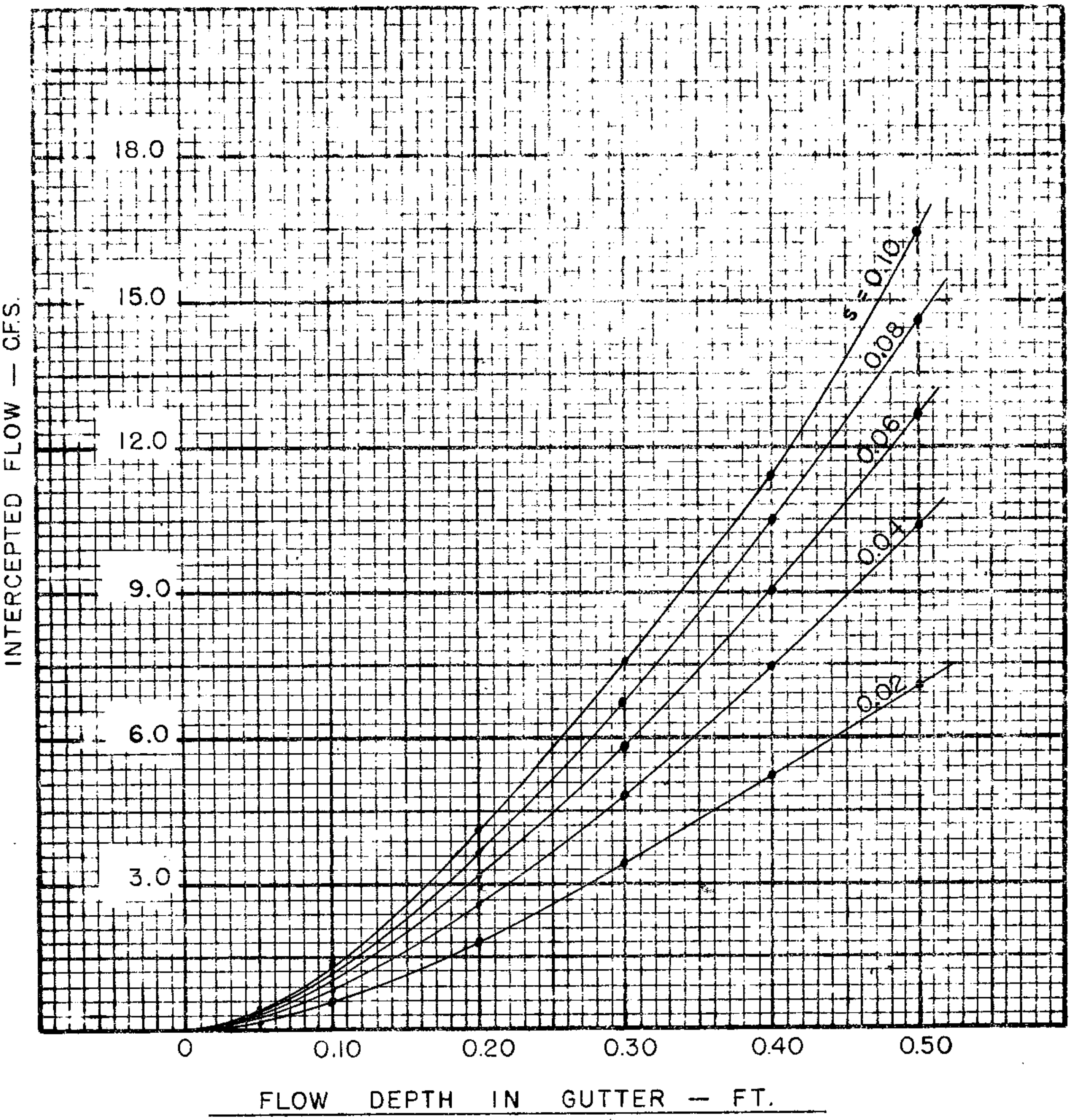
Subdivision ______ Street ______ Station ______

Calc'd. by ______ Date ______

10 FOOT CATCH BASIN CAPACITY	UNITED WESTERN ENGINEERS COLORADO SPRINGS, COLORADO	Fig. 2.4



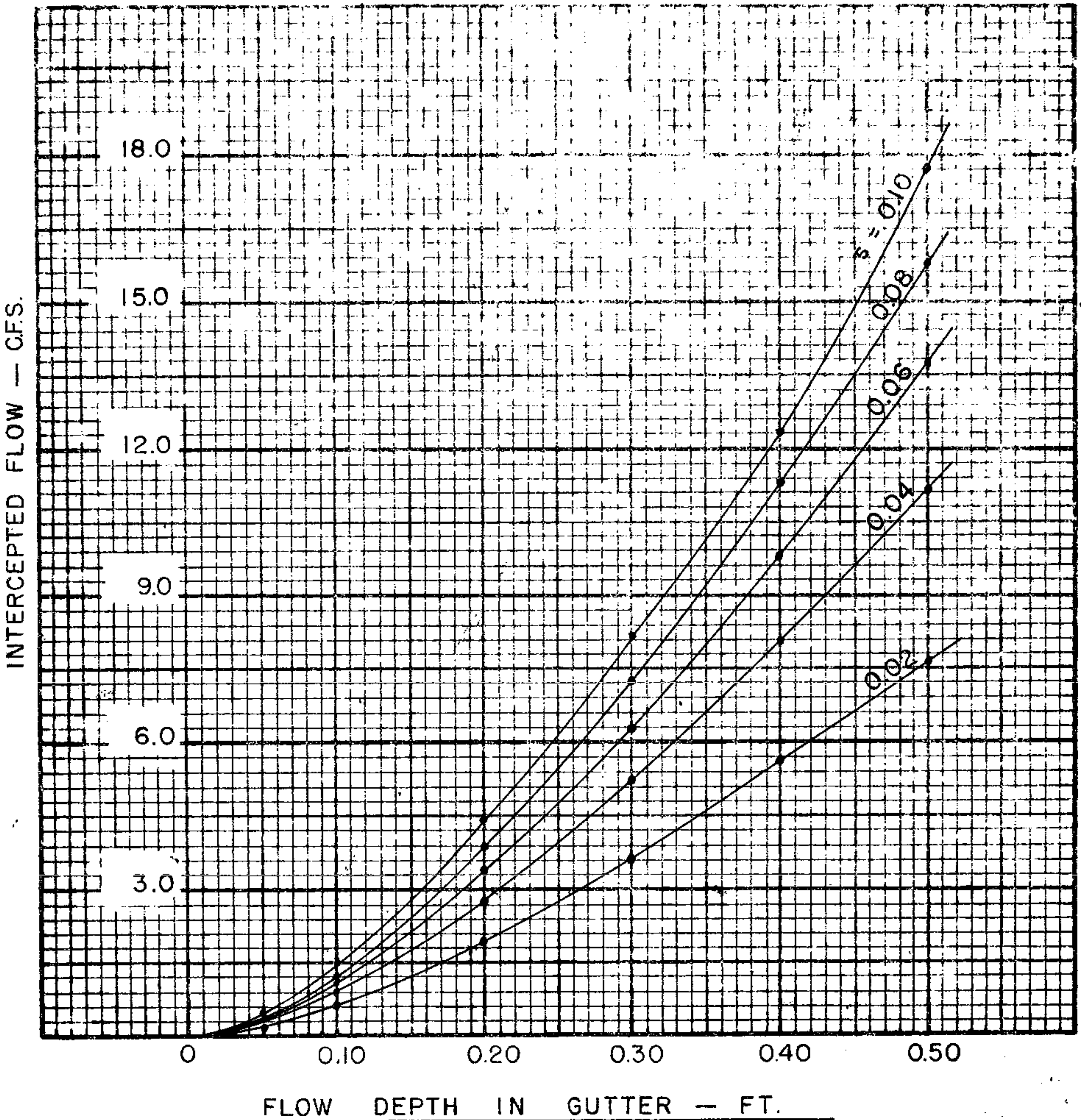
h = 12'
w = 4' - 0"
$c/b = 2$
n = 0.016

CITY OF COLORADO SPRINGS
STD. DRAWING NO. D-10 R

Compiled from Urban Storm Drainage Criteria Manual, City of Denver

Subdivision ________ Street ________ Station ________
Calc'd. by ________ Date ________

12 FOOT CATCH BASIN CAPACITY	UNITED WESTERN ENGINEERS COLORADO SPRINGS, COLORADO	Fig. 2.5



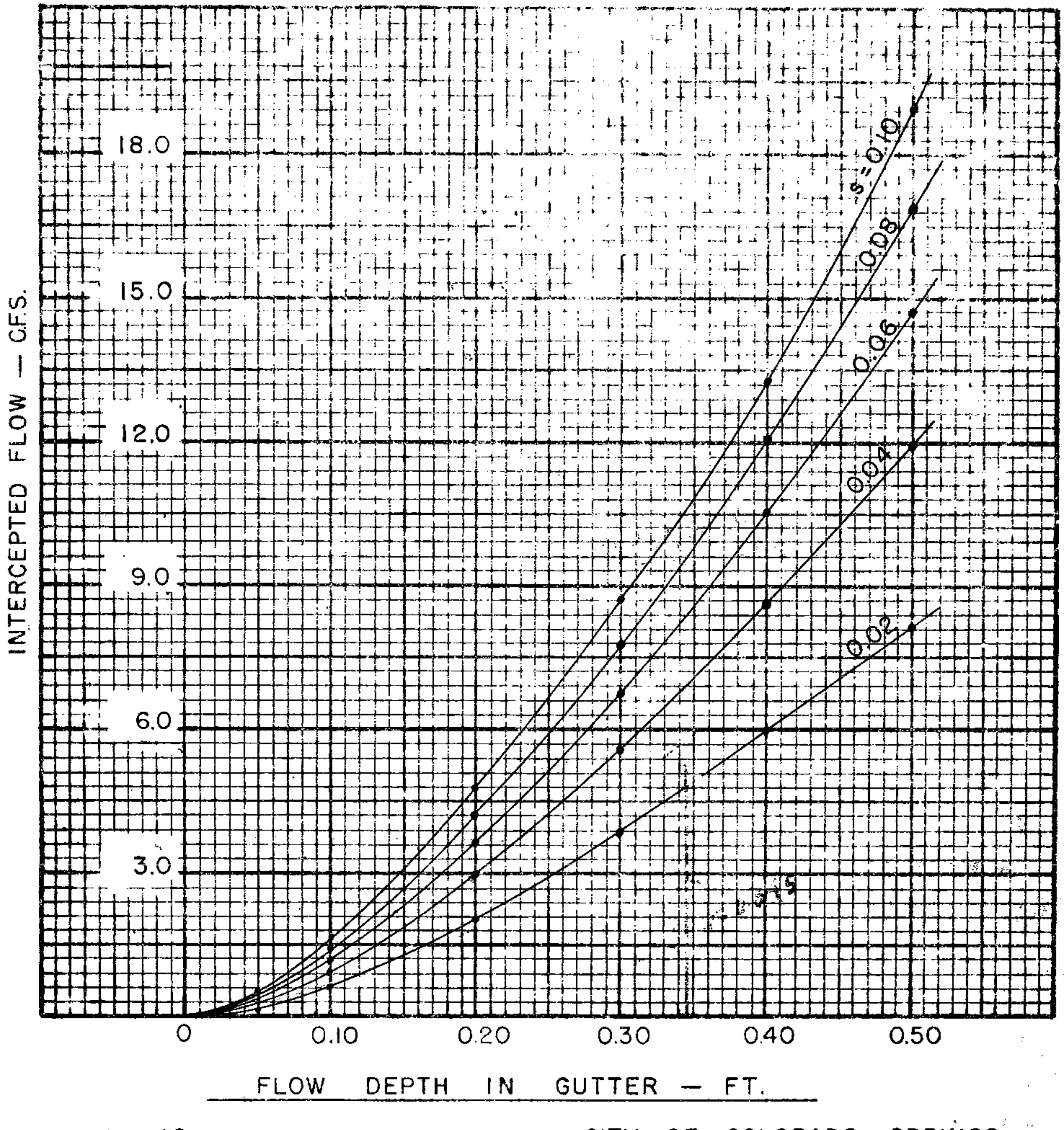
h = 14'
w = 4' - 0"
c/b = 2
n = 0.016

CITY OF COLORADO SPRINGS
STD. DRAWING NO. D-10R

Compiled from Urban Storm Drainage Criteria Manual, City of Denver

Subdivision ______________ Street ______________ Station ______________
Calc'd. by ______________ Date ______________

14 FOOT CATCH BASIN CAPACITY	UNITED WESTERN ENGINEERS COLORADO SPRINGS, COLORADO	Fig. 2.6



h = 16
w = 4' - 0"
c/b = 2
n = 0.016

CITY OF COLORADO SPRINGS
STD. DRAWING NO. D-10 R

Compiled from Urban Storm Drainage Criteria Manual, City of Denver

Subdivision ______ Street ______ Station ______
Calc'd. by ______ Date ______

16 FOOT CATCH BASIN CAPACITY	UNITED WESTERN ENGINEERS COLORADO SPRINGS, COLORADO	Fig. 2.7

VI. COST ESTIMATE

A. City Criteria: The following is the estimated cost of the facilities shown on Plate Number Three, as computed by existing City criteria. This is given for comparitive purposes only and is not our recommended design.

Item	Quantity	Unit Cost	Total
18" CMP	375 LF	$ 25.00	$ 9,375.00
21" CMP	500 LF	27.50	13,750.00
24" CMP	255 LF	30.00	7,650.00
30" CMP	140 LF	30.00	4,200.00
36" CMP	210 LF	25.00	5,250.00
42" CMP	120 LF	25.00	3,000.00
6'x3'x20' RCB	Lump Sum	5000.00	5,000.00
4' catch basin	2 ea	800.00	1,600.00
8' catch basin	3 ea	1300.00	3,900.00
10' catch basin	3 ea	1500.00	4,500.00
12' catch basin	1 ea	1600.00	1,600.00
16' catch basin	1 ea	2000.00	2,000.00
Ditch Paving	1670 SY	9.00	15,030.00
Ditch excavation & embankment	3000 CY	2.00	6,000.00
Grouted Riprap	50 CY	35.00	1,750.00
Utility relocations	Lump Sum	10000.00	10,000.00
8" vertical curb	500 LF	6.00	3,000.00
2" paving & base	250 SY	6.00	1,500.00
Sub total			$ 99,105.00
10% Contingency			9,910.50
TOTAL			$109,015.50

B. Detailed Analysis: The following is the estimated cost of facilities shown on Plate Number Four, which is our recommended preliminary design. Substantial revision may be required due to the extensive utilities now in place, however, the cost should be realistic.

Item	Quantity	Unit Cost	Total
18" CMP	700 LF	$ 25.00	$ 17,500.00
21" CMP	20 LF	27.50	550.00
24" CMP	650 LF	30.00	19,500.00
30" CMP	500 LF	30.00	15,000.00
36" CMP	65 LF	30.00	1,950.00
48" CMP	280 LF	30.00	8,400.00
50"x31" CMP	120 LF	40.00	4,800.00
Dbl 8x4x20 RCB	1 ea	Lump Sum	7,500.00
4' catch basin	2 ea	800.00	1,600.00
6' catch basin	1 ea	1000.00	1,000.00
8' catch basin	1 ea	1300.00	1,300.00
16' catch basin	11 ea	2000.00	22,000.00
5' special catch basin	1 ea	1000.00	1,000.00

Ditch Paving	2300 SY	9.00	20,700.00
Ditch excavation & embankment	3000 CY	2.00	6,000.00
Grouted Riprap	190 CY	35.00	6,650.00
Fence relocation	385 LF	0.50	192.50
Utility relocations	Lump Sum	Lump Sum	10,000.00
8" vertical curb	500 LF	6.00	3,000.00
2" paving & base	250 SY	6.00	1,500.00
Sub total			$150,142.50
10% Contingency			15,014.25
TOTAL			$165,156.75

As the above estimate exceeds the contract amount of $120,000.00, the following items are specified for consideration of elimination from the project, listed in order of least priority.

Deleted Items	Total Cost	Revised Project Cost
1. 4-16' catch basins	$ 8,800.00	$156,356.75
2. Lower Chelton Road	11,387.86	144,968.89
3. Austin Drive outlet	11,489.50	133,479.39
4. F12 Ditch	9,247.37	124,232.02
5. Upper Chelton Road	10,780.00	113,452.02
6. A2 curbing & outlet	1,408.00	112,044.02
7. Marilyn-Leslie Intersection Revision		

The merits of these deletions were previously discussed, but will be summarized here:

1. These basins do not catch enough water to be feasible, however, the 38.5 cfs in Paseo Road at Leslie would become 44.7 cfs and the 22.5 cfs at Country Club would become 26.7 cfs.

2. These facilities prevent 56.6 cfs from crossing Paseo Road, however, this road serves only the Park and the caretaker's dwelling.

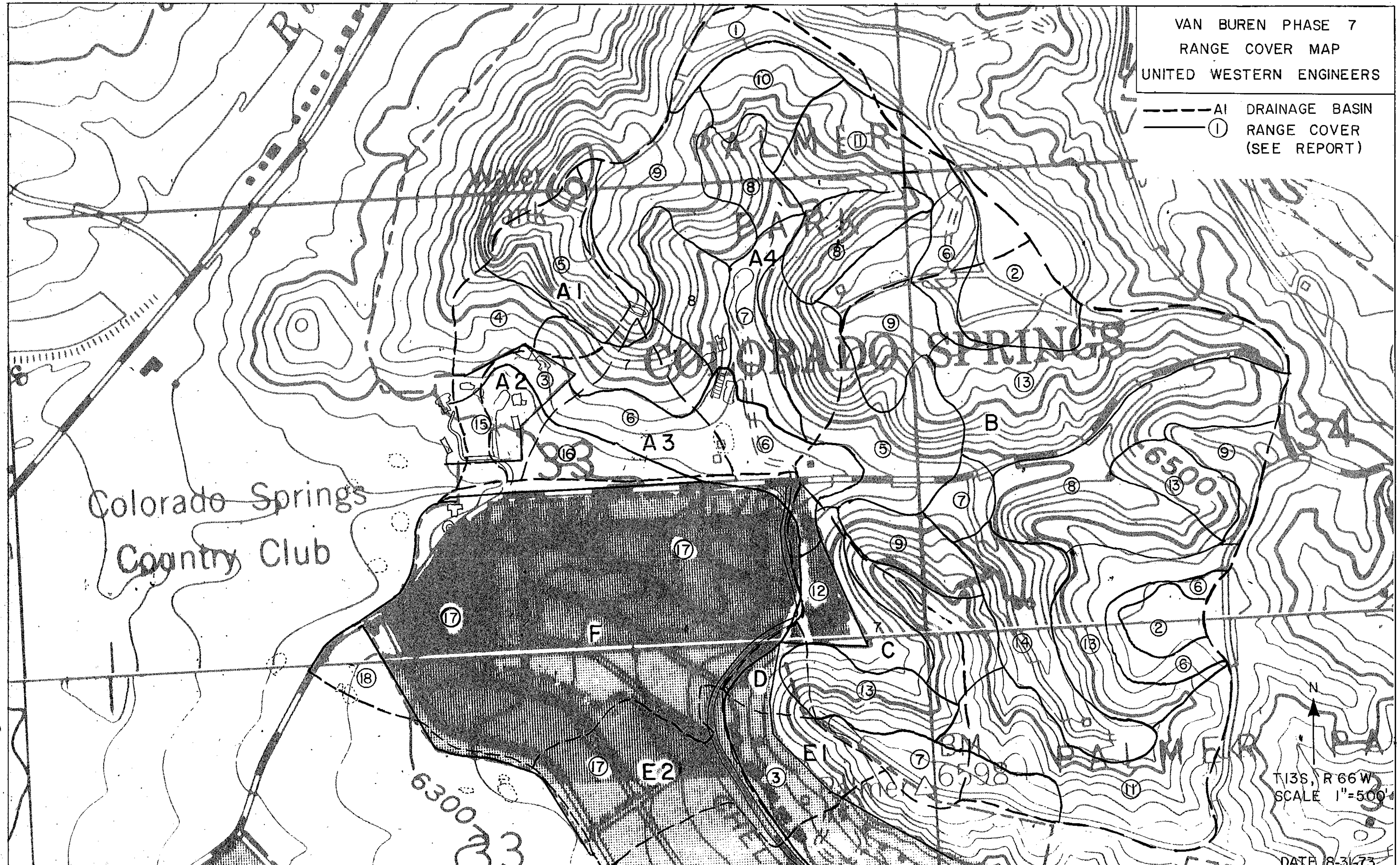
3. The existing outlet works for the 10 year storm, but two years ago the house downstream got water in the basement.

4. Deletion of this item would create a flow of 23.9 cfs over the top of the existing ditch onto the golf course.

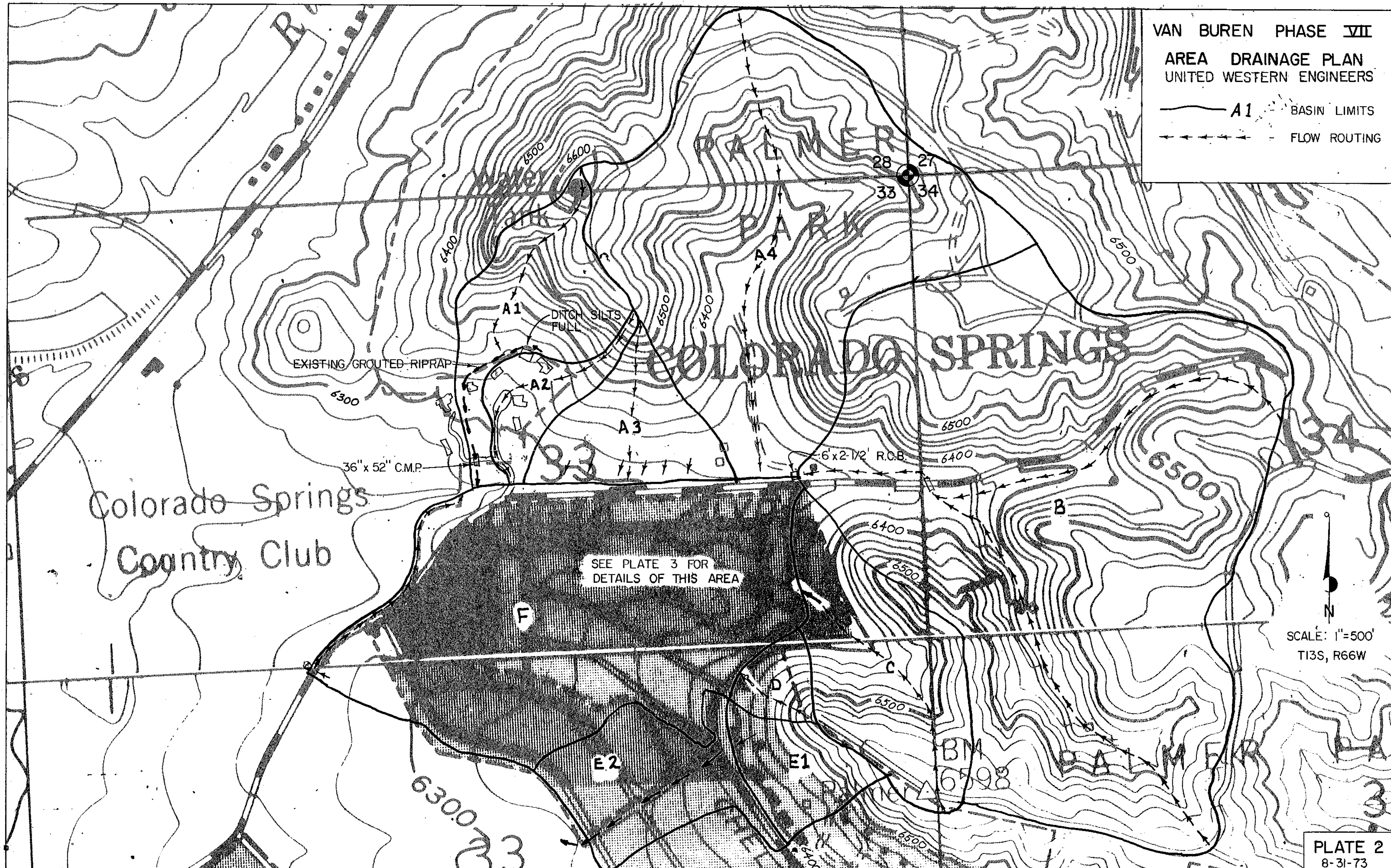
5. Deletion of this item would create an 8.2 cfs flow across two very beautifully landscaped lots.

6. Deletion of this item would very possibly result in a washout of the proposed concrete ditch.

7. Deletion of this would create a severe problem in the flow routing - the runoff splits here and goes down both streets.



VAN BUREN PHASE 7
RANGE COVER MAP
UNITED WESTERN ENGINEERS
A1 DRAINAGE BASIN
1 RANGE COVER (SEE REPORT)
PALMER PARK
COLORADO SPRINGS
Colorado Springs Country Club
A1
A2
A3
A4
B
C
D
E1
E2
F
6500
6300
6598
34
33
N
T13S, R 66 W
SCALE 1"=500'
DATE 8-31-73
PLATE 1


VAN BUREN PHASE VII
AREA DRAINAGE PLAN
UNITED WESTERN ENGINEERS
A1 BASIN LIMITS
FLOW ROUTING
PALMER PARK
COLORADO SPRINGS
Colorado Springs Country Club
A1
A2
A3
A4
B
C
D
E1
E2
F
DITCH SILTS FULL
EXISTING GROUTED RIPRAP
36" x 52" C.M.P.
6' x 2-1/2' R.C.B.
SEE PLATE 3 FOR DETAILS OF THIS AREA
28
27
33
34
6300
6400
6500
6600
BM 6598
N
SCALE: 1"=500'
T13S, R66W
PLATE 2
8-31-73


VAN BUREN VII
DRAINAGE PLAN (CITY CRITERIA)
A —— MAJOR BASIN LIMITS
F6 – – – MINOR BASIN LIMITS
⊗ 4 HYDROGRAPH POINT
—— —— 50 YR. 6 HOUR FLOOD PLAIN LIMITS
EXISTING STRUCTURES
EXISTING DIP SLABS
70/40 R.O.W./STREET WIDTH
16.6 RUNOFF IN C.F.S. – 50 YR. STORM, I = 2"/ 1 HR.
CONC. DITCH
STORM SEWER
CATCH BASIN
UNITED WESTERN ENGINEERS
PLATE NO. 3
73-1067
7-21-73
SCALE 1" = 100'
CONTOUR INTERVAL 2 FEET
TYPICAL DITCH SECTION
Labeled as b x d, Z = as shown
SEE PLATE NO. 5 FOR CROSS-SECTIONS
EXIST. 6' x 2.5' x 15.5' RCB
CAP. = 258 CFS
21" CMP S=1.3%
EXIST. ASPHALT SWALE
PLUG EXIST. 14" STEEL PIPE INLET
EXIST. 36"X58"X24" CMP, S=1%
2'X2' CONC. DITCH, Z=1
4'X2.5' CONC. DITCH, Z=1.5
ADD 8" CURB
4' CURB OUTLET
PROVIDE TEMPORARY CROSSING
6'X3' RCB MIN. hi=1.05'
(EXIST. 48" CMP TO BE REMOVED)
42" CMP Y S=0.64% MIN.
24" CMP, S=1.3%
EXIST. PIPE TO BE REMOVED
EXIST. 24" CMP S=2.5% TO REMAIN
TO REMAIN EXIST. 20.5' OUTLET
40 LF 6' X 4' GROUTED RIPRAP Z=1.5
4'X3' CONC. DITCH Z=1.5
5'X8' CURBED SLAB
EXIST. 3'X5' RCB
EXIST. SWALE TO REMAIN
16' C.B. W/30" CMP, S=0.6% MIN.
2'X2' CONC. DITCH, Z=1.5
30" CMP hi=0.75' S=1.67% MIN.
EXIST. 23' CURB OPENING
24" CMP S=4.9% MIN.
2-8' C.B. W/24" CMP S=1% MIN.
21" CMP MIN. S=3.1%
2'X2' CONC. DITCH Z=1.5
4' C.B. W/18" CMP S=1%
38" CMP MIN. S=0.46%
30" CMP MIN. S=1.10%
10' C.B. W/21" CMP MIN. S=0.7%
21" CMP MIN. S=3.48%
18" CMP MIN. S=0.32%
REMOVE & REPLACE CURB
INSTALL 4' DIPSLAB
NO CURBS
NO CURB
ASPH. RAMP
A1 A2 A3 A4 A5 B C D E1 E2 E2(A)
F1 F2 F3 F4 F5 F6 F7 F8 F9 F10 F11 F12 F13 F14 F15 F16 F17 F18 F19
PASEO ROAD
AUSTIN DR. 60/40
AUSTIN PL. 60/40
CHELTON DR. 60/40
CHELTON ROAD
HIGHLAND DR.
LESLIE DR. 60/40
LEES LN. 60/40
MARILYN RD. 60/40
COUNTRY CLUB LN. 70/40
GRANDVIEW 60/40
6" ASPH.


VAN BUREN VII
DRAINAGE PLAN (SCS HYDROGRAPH METHOD)
A — MAJOR BASIN LIMITS
F6 — MINOR BASIN LIMITS
⊗ 4 HYDROGRAPH POINT
50 YR. 6 HOUR FLOOD PLAIN LIMITS
EXISTING STRUCTURES
EXISTING DIP SLABS
70/40 R.O.W./STREET WIDTH
16.2 RUNOFF IN C.F.S. – 50 YR. STORM, I = 3.2"/6 HRS.
CONCRETE DITCH
STORM SEWER
CATCH BASIN
UNITED WESTERN ENGINEERS
PLATE NO. 4
73-1067
7-21-73
SCALE 1" = 100'
CONTOUR INTERVAL 2 FEET
TYPICAL DITCH SECTION
Labeled as b x d, Z = as shown
SEE PLATE NO. 5 FOR CROSS-SECTIONS
EXIST. 6' x 2.5' RCB, CAP = 258 CFS
50" x 31" CMP
SPECIAL 5' BOX INLET
EXISTING 24" x 36" CMP TO BE REMOVED
2' x 1' CONCRETE CHANNEL Z = 2
PLUG EXIST. 14" STEEL PIPE INLET
DOUBLE 8' x 4' x 20' RCB, hi = 1.0'
EXIST. 48" CMP TO BE REMOVED, PROVIDE TEMP. ACCESS
9' x 7' GROUTED PIPRAP, Z = 1.5 L = 100', LEVEL
5' x 6" CURBED SWALE
EXIST. 3' x 5' RCB
4' x 2' CONC. DITCH Z = 1.5, S = 0.83 %
4' x 2' CONC. DITCH Z = 1.5, S = 0.28 %
13.9 CFS WILL BYPASS EXISTING 20.5' INLET BY ITSELF
REBUILD CURB LINES TO CONTAIN FLOWS IN MARILYN
PASEO ROAD
AUSTIN DR.
AUSTIN PL.
CHELTON DR.
CHELTON ROAD
HIGHLAND DR.
LESLIE DR.
LEE LN.
MARILYN RD.
COUNTRY CLUB LN.
GRANDVIEW
A1
A2
A3
A4
A5
B
C
D
E1
E2
E2(A)
F1
F2
F3
F4
F5
F6
F7
F8
F9
F10
F11
F12
F13
F14
F15
F16
F17
F18
F19


Section S-S Note: 13' U/S begin 48" cmp.
Green Belt (See plan)
Section 2-2
Green Belt (See plan) from Topo
Section 1-1
Green Belt (See plan) from Topo
Section G-G
Green Belt (See plan)
Section F-F
Green Belt (See plan)
Section B-B
Green Belt (See plan)
Section A-A
Green Belt (See plan)
Scale: 1" = 10'
Section Z-Z
Chelton Road
Section AA-AA
Chelton Road
Section M-M S: 4.4%
Grandview 10' below Country Club Ln. C.R.
Section L-L S: 2.72%
Marilyn Road 10' below Leslie Dr. C.R.
Section K-K S: 2.72%
Marilyn Road 10' above Leslie Drive C.R.
Section J-J
Leslie Drive 10' below Marilyn Road C.R.
Section I-I S: 1.2%
Austin Drive 50' East of Section H-H
Section H-H
Austin Drive
Section E-E S: 2.3%
Paseo Road
Section D-D S: 7.2%
Chelton Road
Proposed ditch
Section C-C
Chelton Road
Section Y-Y S: 3.2%
Paseo Road 33' above East Curb on Leslie
Section X-X S: 2.0%
Lees Lane 10' above Country Club Lane C.R.
Section W-W S: 5.8%
Country Club Lane 10' above Paseo Road C.R.
Section V-V S: 1.4%
Austin Drive 10' above Leslie Drive C.R.
Section U-U S: 4.6%
Leslie Drive 10' below Austin Drive C.R.
Section T-T
Leslie Drive 10' above Paseo Road C.R.
Section R-R
Section Q-Q Leslie Dr. S: 0.8%
Section P-P
Section O-O
Section N-N S: 4.2%
JOB NO. 73-1067
VAN BUREN PHASE 7
CROSS SECTIONS
SCALE: Vert. Horiz. AS SHOWN
APPROVED BY: Allan D. Miller P.E. 5076
PLAN BY O.E.W. N.H.P.
DRAWN BY N.H.P.
SURVEYED BY:
CHECKED BY:
UNITED WESTERN ENGINEERS
4525 Northpark Drive
Colorado Springs, Colorado
PLATE 5