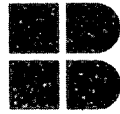


WILLIAM L. KORBER, L.S.

JOHN E. LUNDBLADE, L.S.

N. BRENT WOOTE, P.E.



BAUGHMAN COMPANY, P.A.

SURVEYING & ENGINEERING

316/262-7271 • 315 ELLIS • WICHITA, KANSAS 67211

March 13, 1987

Vicki Huang
City Engineer's Office
455 North Main
Wichita, KS 67202

re: Westwind 2nd Addition
Westwind 3rd Addition

Dear Vicki,

Submitted herewith is a drainage report which includes a comprehensive Drainage Plan for the above plats and adjacent areas. A copy is also being submitted to Sedgwick County.

Please review this report as a Final Drainage Plan for Westwind 2nd and Westwind 3rd Additions. The preliminary plat of Westwind 3rd Addition is being submitted today to MAPD.

If you have any questions, please call.

Sincerely,

Thomas C. Ruggles, P.E.

File
TCR/ksb

cc: Jim Weber

CONTENTS

1. Scope and Location Map
2. General Description

Part I - Westwind C.U.P.

- 3.-4. Description and input data
- 5.-7. Hydrographs
- 8.-9. Output data

Part II - Golf Park West - Westwind 2nd Addition

- 10.-11. Description and input data
- 12.-13. Hydrographs
- 14.-15. Output data

Part III - South part of Westwind 2nd Addition

16. Description and runoff calculations

Storm Water Sewer no. 145 Analysis

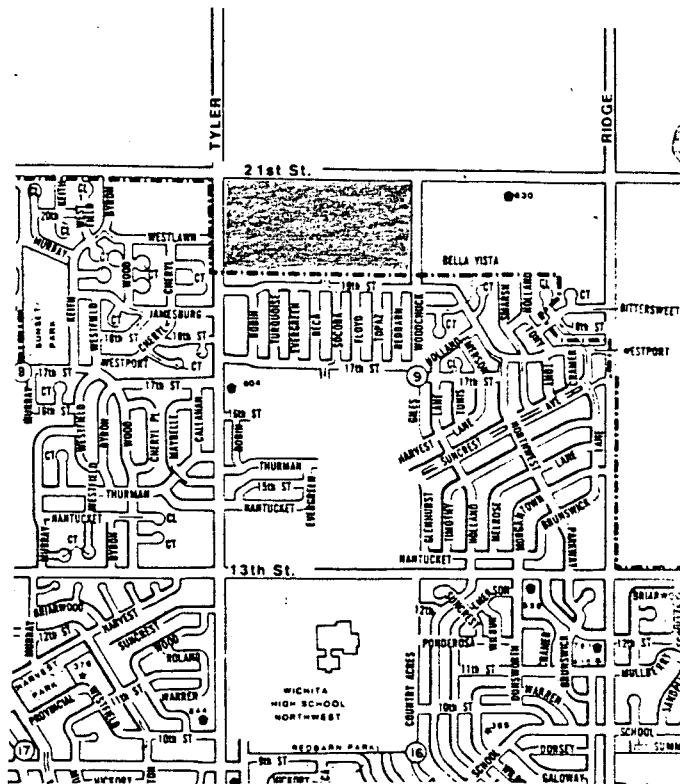
Attachments

- Drainage Plan
- 21st Street ditch plan/profile

DRAINAGE REPORT FOR WESTWIND C.U.P., GOLF PARK WEST,
AND WESTWIND 2ND ADDITION, WICHITA, KANSAS

Scope

This report covers drainage of proposed developments on the south side of 21st Street North, between Tyler and Woodchuck. Areas included in the study are the Westwind Residential C.U.P. and the Westwind Commercial C.U.P., a total of 36 acres; Westwind 2nd Addition, a 51-lot residential subdivision; and Golf Park West, a 20-acre recreational area located between the two above mentioned developments. A Drainage Plan exhibit accompanies this report.



LOCATION MAP

Drainage from the study area flows into an existing storm sewer system, SWS 145, which drains east across Ridge Road to the Big Slough. Capacity of SWS 145 has been analyzed as part of the drainage plan for Westwind 2nd Addition, dated November 1986. A drainage plan for Golf Park West has also been prepared by Professional Engineering Consultants, dated June 1986. Any variations from those plans proposed by this study should be considered as revisions to those previously submitted plans.

Runoff calculations, detention, and routing for this study were performed on the Texas A&M University Watershed Model, run on the IBM AT computer. The A&M model methods are described in the Users Manual provided (See page 1, Chapter 1). Please return the manual to Baughman Co. after review.

The area was studied in three parts:

- I. C.U.P. area only, with on-site detention
- II. Golf Park West and part of Westwind 2nd Addition, with detention on Golf Park site
- III. Part of Westwind 2nd Addition, with no detention

The design storm used for detention basins was the 100-year, 24-hour duration rainfall, per City of Wichita criteria.

A preliminary Drainage Plan sheet included with this report delineates the drainage areas and shows the detention and storm sewer systems.

The entire area included in this study naturally drains to a low area near 19th Street and Woodchuck. An area of approximately 70 acres north of 21st Street also drains to the same point. The drainage from north of 21st Street is to be diverted to the east along 21st Street, per the ditch plan and profile sheets and supporting calculations submitted with the November 1986 drainage plan for Westwind 2nd Addition (see attachments). Other developments north of 21st Street will also divert part of the runoff to the east, north of 21st St.

The low area at 19th and Woodchuck has no natural outlet. However, the design of Storm Water Sewer 145 includes a major sump at that location, with the downstream system sized for 100-year design flow. The previously mentioned analysis of SWS 145 indicates excess capacity of 45 cfs at that point, using current City of Wichita drainage criteria. The Drainage Plan described in this report will provide control of runoff to allow the area to be accommodated by the existing storm sewer system.

I. Westwind C.U.P.

The area referred to as "Westwind C.U.P." includes both the Residential and the Commercial C.U.P. areas as shown on the Drainage Plan, less approximately 5.0 acres along the west side which drain to Tyler Road.

Drainage Area = 31.0 Acres

Soil type Ma, Hydrologic Group B

Existing condition, cultivation: CN = 75, 0 % impervious

Proposed developed condition:

Use	% Imp.	Area	CN	Product
Single-family (1/4 ac.)	38	12.0	75	900.0
Multi-family (attached)	65	8.1	85	688.5
Commercial	70	10.9	87	948.3
				<hr/>
				2536.8

Weighted CN = $2544.3 / 31.0 \text{ Ac.} = 81.8$ (use 82)
(Curve numbers are from SCS tables w/ AMC II).

Input Data for Watershed file (developed condition)

Subbasin file I.D. no. 300
Precipitation file I.D. no. 500
Drainage Area 31.0 Ac. = 0.0484 sq. mi.
Subbasin height = 1358-1350 = 8 feet
Subbasin length = 1550 feet = 0.294 miles
% impervious = 56.3
% watershed slope = 0.7
% channel improved = 100

Input Data for Precipitation

Precipitation file I.D. no. 500
Start time of storm 0 hours
Storm Duration 24 hours
Incremental time 0.25 hours
Total precipitation 7.8 inches (from U.S. Weather Service Hydro-35)
Type storm distribution SCS Type II
Return period 100 years

Inflow Hydrograph

Hydrographs for 100 yr., 24-hour runoff, with Developed and Undeveloped conditions are attached (see pages 5-7).

Undeveloped Q peak = 43.3 cfs

Developed Q peak = 61.1 cfs

For comparison of methods, both the Rural and Urban watershed options were run. The Urban model produced a slightly lower Q peak; therefore the higher Q from the Rural Model was used.

Reservoir Routing

The developed Inflow hydrograph was routed through the detention basin shown on the drainage plan. The input stage-storage data for the reservoir is as follows:

Elev.	Area s.f.
1345.2	0
1346.7	35078
1347.7	43286
1348.7	51855
1349.7	60442
1350.7	69390
1351.7	78600
1352.7	100700

Stage-Storage volumes and stage-discharge rates were output, following routing, in the Reservoir file attached (Reservoir I.D. no. 900), page 8.

Routing was performed using the Culvert Design option since the reservoir is to have only one outlet spillway (pipe). Input and output data for the pipe design are listed in the attached Culvert Design printout (see page 8). The pipe selected is a 24" R.C.P., with Q_{peak} outflow = 29.0 cfs at a maximum water surface elevation of 1350.2 (5.0' HW).
(5.0 HW).

Detention Analysis

Inflow Q peak = 61.1 cfs at time 12.5 hours

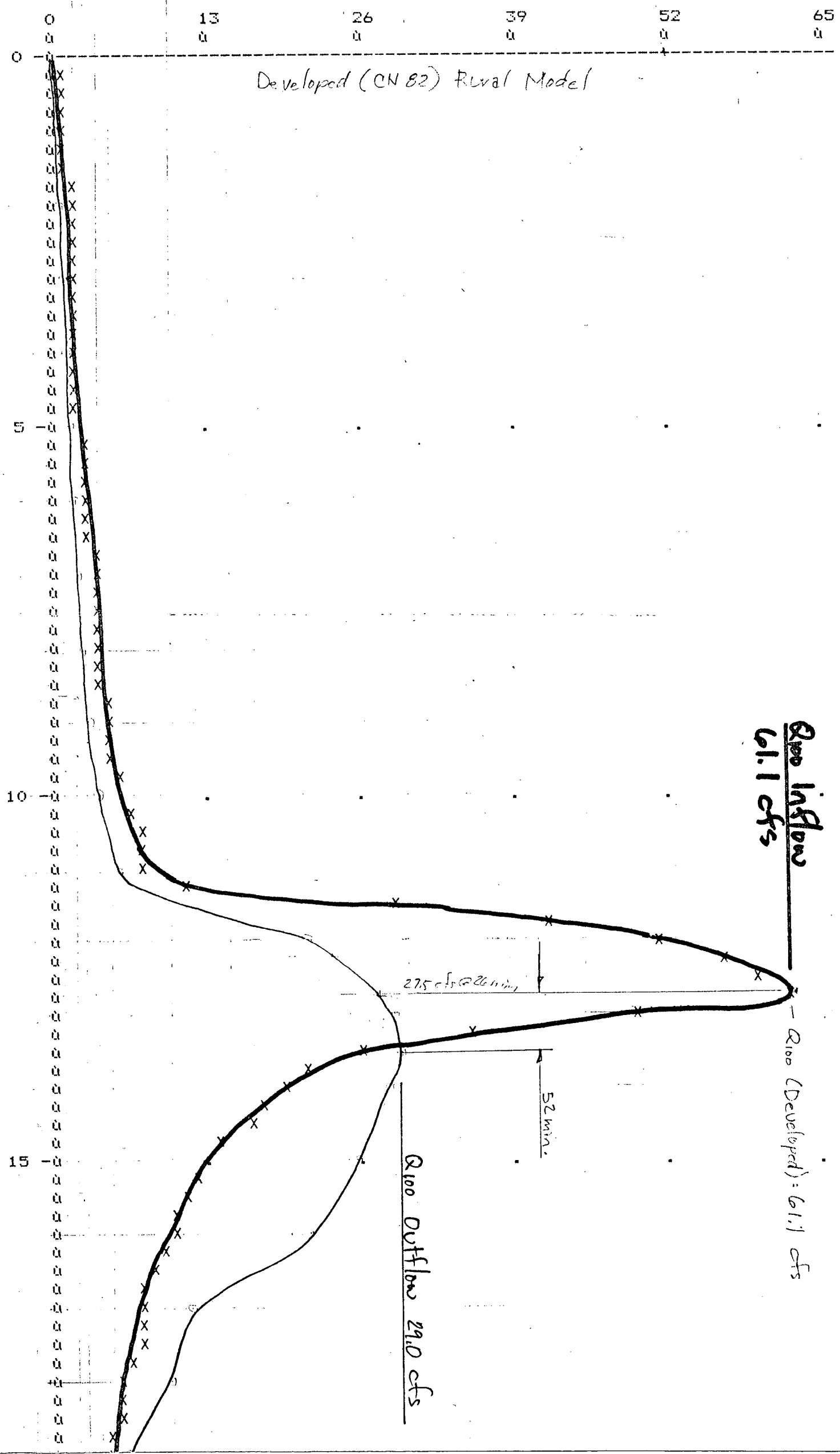
Outflow Q peak = 29.0 cfs at time 13.58 hours

Time delay from detention = 0.88 hrs. = 52 minutes (to be added to the initial tc for the subbasin).

Since the time of concentration (total) in SWS 145 is approximately 30 minutes at the point of inflow, the detention lag produced will allow the storm sewer peak flow to pass before the peak outflow from the detention basin occurs. The outflow hydrograph is shown superimposed on the inflow hydrograph (page 7).

I WESTWIND C.U.P.
A&M WATERSHED MODEL
HYDROGRAPH PLOT

HYDROGRAPH ID NO 0 DRAINAGE AREA IN SQ MI 0.05
INCREMENT TIME STEP HRS 0.25 START TIME HYDRO IN HRS 0.00
BASE FLOW IN CFS 0.00 RIVER STATION IN MILES 0.00
X HYD 0 V IN. AC-FT 17
DISCHARGE IN CFS

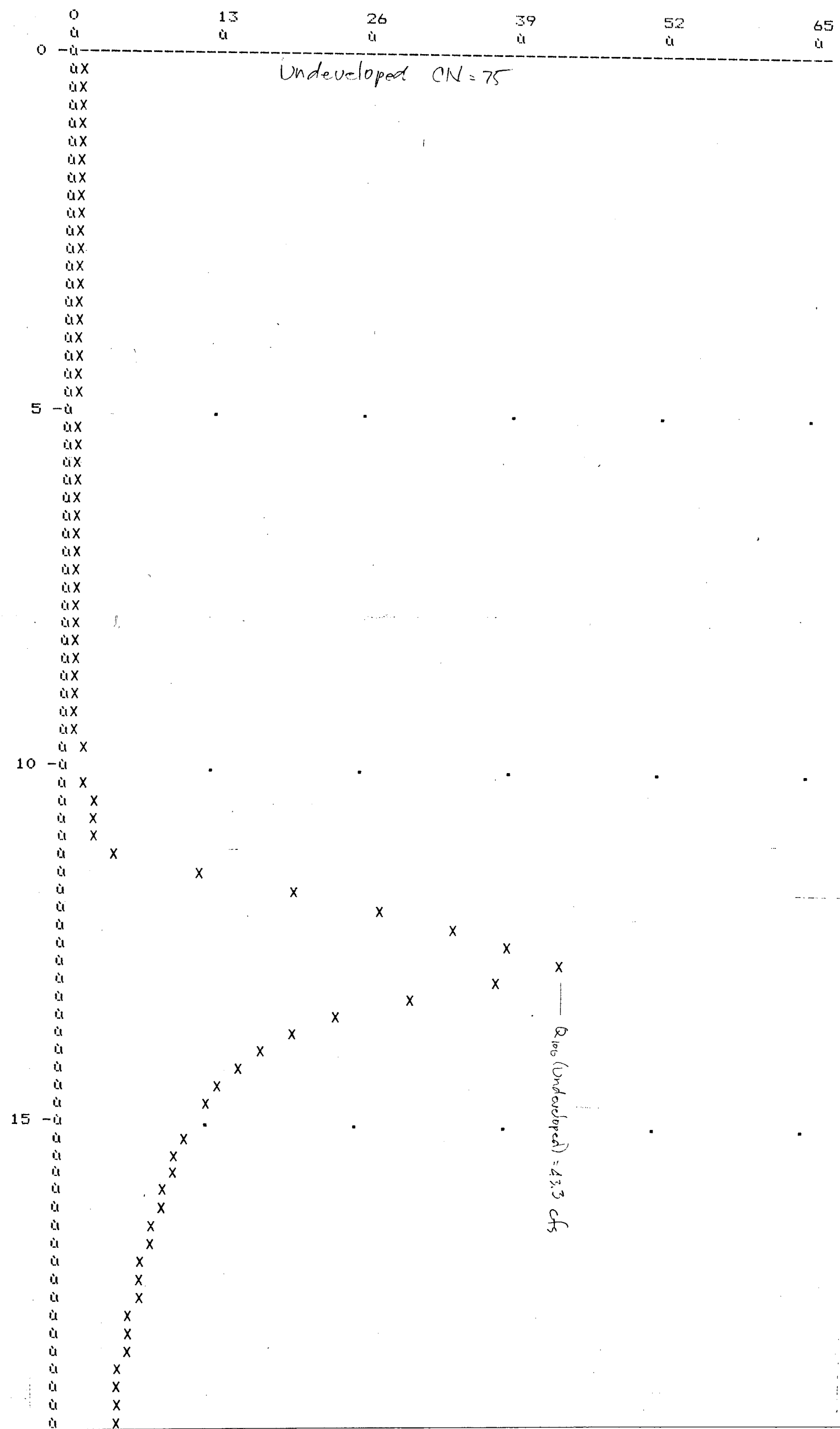


1 WESTWIND C.U.P.

A&M WATERSHED MODEL

HYDROGRAPH PLOT

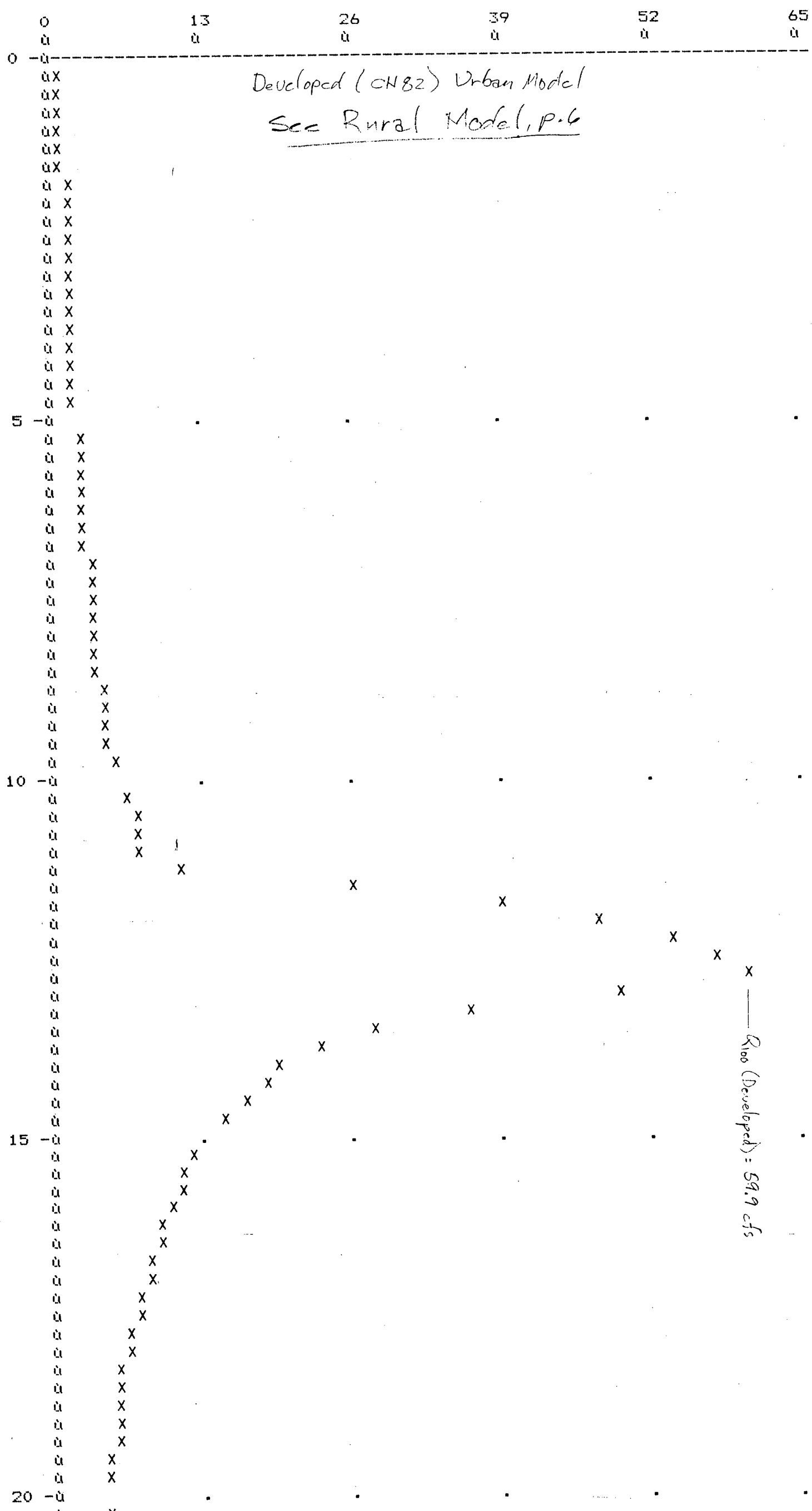
HYDROGRAPH ID NO 3 DRAINAGE AREA IN SQ MI 0.05
INCREMENT TIME STEP HRS 0.25 START TIME HYDRO IN HRS 0.00
BASE FLOW IN CFS 0.00 RIVER STATION IN MILES 0.00
X HYD 3 V IN AC-FT 12
DISCHARGE IN CFS



A&M WATERSHED MODEL

HYDROGRAPH PLOT

HYDROGRAPH ID NO 1 DRAINAGE AREA IN SQ MI 0.05
 INCREMENT TIME STEP HRS 0.25 START TIME HYDRO IN HRS 0.00
 BASE FLOW IN CFS 0.00 RIVER STATION IN MILES 0.70
 X HYD 1 V IN AC-FT 17
 DISCHARGE IN CFS



RESERVOIR FILE

RESERVOIR ID NO. 900
 RESERVOIR W.S. EL. 1345.2 FEET
 DOWNSTREAM CH EL 1344.2 FEET

LINE NO.	ELEVATION FEET	STORAGE AC-FT	DISCHARGE CFS
1	1345.2	0.0	0.0
2	1345.6	0.0	1.3
3	1346.0	0.2	2.9
4	1346.4	0.4	6.0
5	1346.8	0.7	9.7
6	1347.2	1.0	11.2
7	1347.6	1.4	12.6
8	1348.0	1.8	21.7
9	1348.4	2.2	23.2
10	1348.8	2.7	24.6
11	1349.1	3.1	26.0
12	1349.5	3.7	27.2
13	1349.9	4.2	28.4
14	1350.3	4.8	29.6
15	1350.7	5.4	30.7
16	1351.1	6.1	31.8
17	1351.5	6.7	32.8
18	1351.9	7.5	33.9
19	1352.3	8.3	34.8
20	1352.7	9.1	35.8

CULVERT DESIGN

OUTLET CONDUIT 900
 HEADLOSS COEFFICIENTS

- 1. MANNING N .013
- 2. K EXIT 1
- 3. K ENTRANCE .5
- 4. K OTHER 0

CULVERT

- 5. LENGTH 180
- 6. NO. OF PIPES 1
- 7. DIAMETER 2
- 9. DESIGN DISCHARGE (Q) 30
- 10. RESERVOIR W.S. EL AT DESIGN Q 1351.7
- 11. MAX TAILWATER ELEVATION 1345.1
- 12. CENTERLINE ELEV OF INLET 1346.2
- 13. CENTERLINE ELEV OF OUTLET 1345.2

PEAK INFLOW DISCHARGE 61.10025

RESULTS OF RESERVOIR ROUTING

- PEAK OUTFLOW DISCHARGE 29.02421
- MAX W.S. ELEV IN RESERVOIR 1350.135
- STARTING W.S. ELEV IN RES. 1345.2

DESIGN STORMS

- A. RAINFALL IN. 7.80 DURATION 24.00

I WESTWIND C.U.P.

OUTFLOW HYDROGRAPH LISTING

HYDROGRAPH ID NO 900 DRAINAGE AREA IN SQ MI .0484
 INCREMENT TIME STEP HRS .25 MAX. WATER SURFACE EL FEET 1350.1
 RUNOFF IN. 6.197136 PEAK DISCHARGE CFS 29.02421
 TIME TO CENTROID HRS 14.59

ITEM	FLOW	ITEM	FLOW	ITEM	FLOW	ITEM	FLOW	ITEM	FLOW	ITEM	FLOW
1	0.0	51	27.5	101	1.4	151	0.0	201	0.0	251	0.0
2	0.1	52	28.6	102	1.0	152	0.0	202	0.0	252	0.0
3	0.3	53	29.0	103	0.6	153	0.0	203	0.0	253	0.0
4	0.4	54	29.0	104	0.4	154	0.0	204	0.0	254	0.0
5	0.6	55	28.6	105	0.3	155	0.0	205	0.0	255	0.0
6	0.7	56	28.2	106	0.2	156	0.0	206	0.0	256	0.0
7	0.8	57	27.7	107	0.2	157	0.0	207	0.0	257	0.0
8	0.9	58	27.1	108	0.1	158	0.0	208	0.0	258	0.0
9	1.0	59	26.5	109	0.1	159	0.0	209	0.0	259	0.0
10	1.1	60	25.7	110	0.1	160	0.0	210	0.0	260	0.0
11	1.2	61	24.9	111	0.0	161	0.0	211	0.0	261	0.0
12	1.2	62	23.9	112	0.0	162	0.0	212	0.0	262	0.0
13	1.3	63	23.0	113	0.0	163	0.0	213	0.0	263	0.0
14	1.3	64	21.9	114	0.0	164	0.0	214	0.0	264	0.0
15	1.3	65	17.4	115	0.0	165	0.0	215	0.0	265	0.0
16	1.4	66	13.2	116	0.0	166	0.0	216	0.0	266	0.0
17	1.4	67	12.2	117	0.0	167	0.0	217	0.0	267	0.0
18	1.4	68	11.7	118	0.0	168	0.0	218	0.0	268	0.0
19	1.4	69	11.2	119	0.0	169	0.0	219	0.0	269	0.0
20	1.5	70	10.8	120	0.0	170	0.0	220	0.0	270	0.0
21	1.6	71	10.3	121	0.0	171	0.0	221	0.0	271	0.0
22	1.7	72	9.8	122	0.0	172	0.0	222	0.0	272	0.0
23	1.8	73	8.7	123	0.0	173	0.0	223	0.0	273	0.0
24	1.9	74	7.7	124	0.0	174	0.0	224	0.0	274	0.0
25	2.0	75	6.8	125	0.0	175	0.0	225	0.0	275	0.0
26	2.1	76	6.2	126	0.0	176	0.0	226	0.0	276	0.0
27	2.1	77	5.7	127	0.0	177	0.0	227	0.0	277	0.0
28	2.2	78	5.2	128	0.0	178	0.0	228	0.0	278	0.0
29	2.3	79	4.8	129	0.0	179	0.0	229	0.0	279	0.0
30	2.4	80	4.4	130	0.0	180	0.0	230	0.0	280	0.0
31	2.4	81	4.2	131	0.0	181	0.0	231	0.0	281	0.0
32	2.5	82	4.0	132	0.0	182	0.0	232	0.0	282	0.0
33	2.6	83	3.8	133	0.0	183	0.0	233	0.0	283	0.0
34	2.7	84	3.7	134	0.0	184	0.0	234	0.0	284	0.0
35	2.9	85	3.5	135	0.0	185	0.0	235	0.0	285	0.0
36	3.1	86	3.4	136	0.0	186	0.0	236	0.0	286	0.0
37	3.3	87	3.4	137	0.0	187	0.0	237	0.0	287	0.0
38	3.4	88	3.3	138	0.0	188	0.0	238	0.0	288	0.0
39	3.6	89	3.3	139	0.0	189	0.0	239	0.0	289	0.0
40	4.0	90	3.2	140	0.0	190	0.0	240	0.0	290	0.0
41	4.4	91	3.2	141	0.0	191	0.0	241	0.0	291	0.0
42	4.9	92	3.1	142	0.0	192	0.0	242	0.0	292	0.0
43	5.3	93	3.0	143	0.0	193	0.0	243	0.0	293	0.0
44	5.6	94	2.9	144	0.0	194	0.0	244	0.0	294	0.0
45	6.4	95	2.9	145	0.0	195	0.0	245	0.0	295	0.0
46	9.6	96	2.9	146	0.0	196	0.0	246	0.0	296	0.0
47	11.9	97	2.7	147	0.0	197	0.0	247	0.0	297	0.0
48	21.8	98	2.4	148	0.0	198	0.0	248	0.0	298	0.0
49	23.9	99	2.1	149	0.0	199	0.0	249	0.0	299	0.0
50	25.8	100	1.7	150	0.0	200	0.0	250	0.0	300	0.0

II. GOLF PARK WEST / PART OF WESTWIND 2ND ADDITION

This drainage area includes Golf Park West, except the north 150'; and Lot 1, except the North 150', Lots 5-15 and 24-30 in Westwind 2nd Addition. The north 150' will drain to the 21st Street ditch previously described. Adjacent Street R/W areas are also included in the drainage area.

Drainage Area - 26.4 Acres

Soil type Ma, Hydrologic Group B

Existing condition, cultivation: CN = 75, 0% Impervious

Proposed developed condition

Use	% imp.	Area ac.	CN	Product
Single-family (1/4 Ac.)	38	3.7	75	277.5
Commercial (Church)	70	3.0	87	295.8
Commercial (Golf)	70	7.5	87	652.5
Open space (Golf)	0	10.0	61	610.0

Weighted CN = 75 (Note: In this case the developed and undeveloped Curve Numbers are equal).

Input Data for Watershed file (developed condition)

Subbasin file I.D. no. 305
Precipitation file I.D. no. 500
Drainage Area 24.6 Ac. = 0.0384 sq. mi.
Subbasin height 6.6 feet
Subbasin length 1450 feet = 0.275 miles
% Impervious 36.7
% Watershed slope 0.5
% Channel improved 100

The Input Data for Precipitation are identical to those for the previous basin (file no. 500), page 3.

Inflow Hydrograph

Hydrographs for 100-yr., 24-hour runoff, with Developed vs. Undeveloped conditions, are attached (see pages 12-13).

Undeveloped Q peak = 34.5 cfs

Developed Q peak = 45.5 cfs

Both hydrographs were generated on the Rural Model.

Reservoir Routing

The developed Inflow hydrograph was routed through the detention basin in the south part of Golf Park West. The input stage-storage data for the reservoir are as follows:

Elev.	Area s.f.
1341.4	0
1342.0	1533
1343.0	17733
1344.0	41026
1345.0	68643
1346.0	98523

Stage-storage volumes and stage-discharge rates were output following routing, in the Reservoir file attached (Reservoir I.D. no. 902).

Routing was performed using the Culvert Design option. Input and output data for the pipe design are listed in the Culvert Design printout. The pipe selected is a 24" R.C.P., with Q peak outflow = 22.6 cfs at a maximum water surface elevation of 1345.2 (3.8' HW).

Detention Analysis

Inflow Q peak = 45.5 cfs at time 12.75 hours

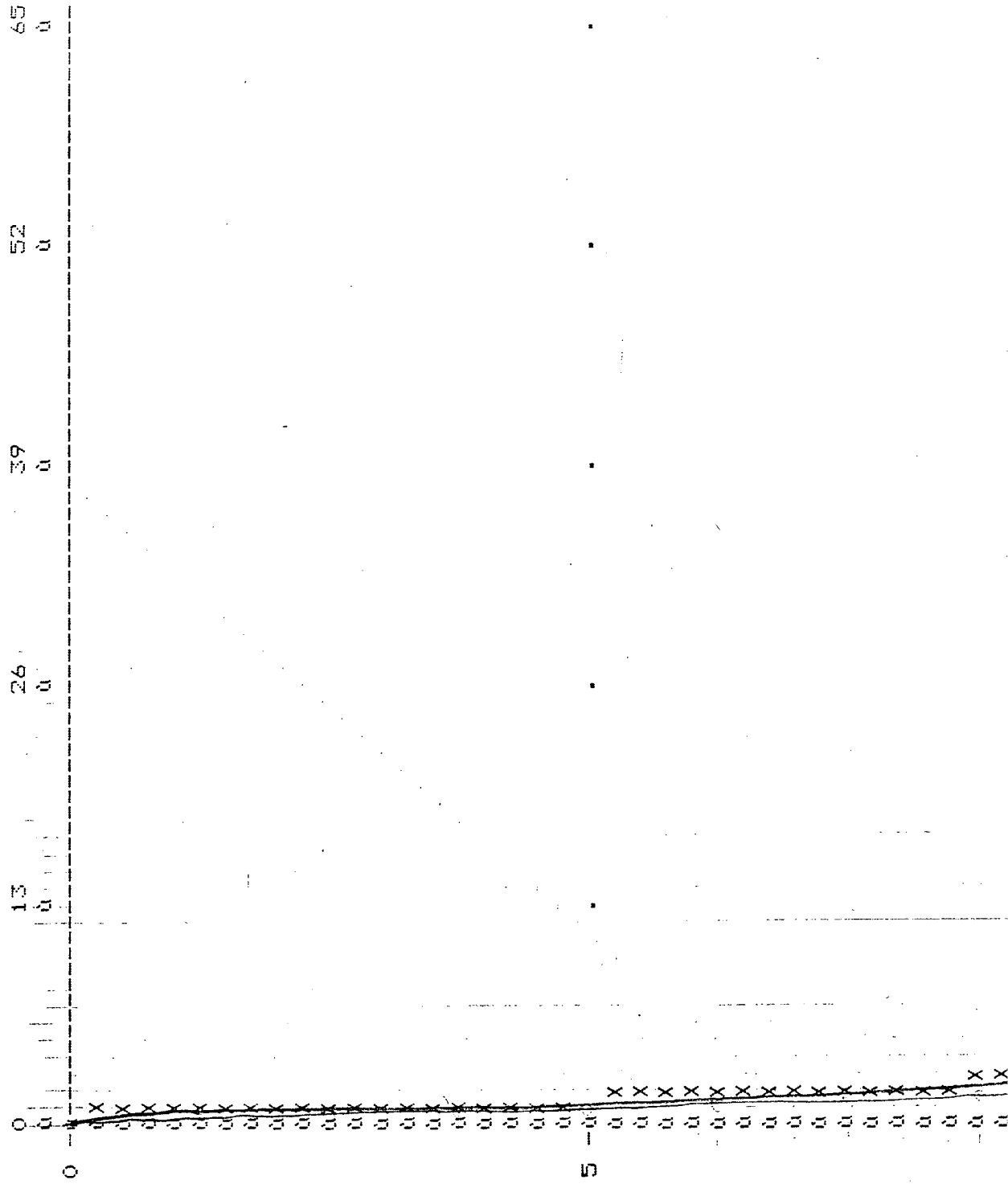
Outflow Q peak = 22.6 cfs at time 13.25 hours

Time delay from detention = 0.5 hrs. = 30 minutes (to be added to the initial tc for the subbasin).

The detention lag produced will allow the storm sewer peak flow (at approximate time 30 minutes) to pass before the peak outflow from the detention basin occurs. The outflow hydrograph is shown superimposed on the inflow hydrograph (page 13).

HYDROGRAPH PLOT

HYDROGRAPH ID NO 1 DRAINAGE AREA IN SQ MI 0.04
 INCREMENT TIME STEP HRS 0.25 START TIME HYDRO IN HRS 0.00
 BASE FLOW IN CFS 0.00 RIVER STATION IN MILES 0.00
 X HYD 1 V IN AC-FT 11 DISCHARGE IN CFS



RESERVOIR FILE

RESERVOIR ID NO. 902
 RESERVOIR W.S. EL. 1341.4 FEET
 DOWNSTREAM CH EL 1341.2 FEET

LINE NO.	ELEVATION FEET	STORAGE AC-FT	DISCHARGE CFS
1	1341.4	0.0	0.0
2	1341.6	0.0	0.6
3	1341.9	0.0	1.1
4	1342.1	0.0	1.7
5	1342.4	0.1	2.9
6	1342.6	0.1	4.4
7	1342.9	0.2	6.2
8	1343.1	0.3	8.1
9	1343.3	0.4	10.1
10	1343.6	0.6	12.0
11	1343.8	0.7	16.6
12	1344.1	1.0	17.8
13	1344.3	1.2	19.0
14	1344.5	1.5	20.0
15	1344.8	1.9	21.0
16	1345.0	2.2	22.0
17	1345.3	2.6	22.9
18	1345.5	3.1	23.8
19	1345.8	3.6	24.6
20	1346.0	4.1	25.5

CULVERT DESIGN

OUTLET CONDUIT 902
 HEADLOSS COEFFICIENTS

1. MANNING N .013
2. K EXIT 1
3. K ENTRANCE .5
4. K OTHER 0

CULVERT

5. LENGTH 180
6. NO. OF PIPES 1
7. DIAMETER 2
9. DESIGN DISCHARGE (Q) 21
10. RESERVOIR W.S. EL AT DESIGN Q 1344.8
11. MAX TAILWATER ELEVATION 1342.02
12. CENTERLINE ELEV OF INLET 1342.4
13. CENTERLINE ELEV OF OUTLET 1342.2

PEAK INFLOW DISCHARGE 43.41179

RESULTS OF RESERVOIR ROUTING

PEAK OUTFLOW DISCHARGE 22.63171

MAX W.S. ELEV IN RESERVOIR 1345.201

STARTING W.S. ELEV IN RES. 1341.4

DESIGN STORMS

- A. RAINFALL IN. 7.80 DURATION 24.00

OUTFLOW HYDROGRAPH LISTING

HYDROGRAPH ID NO. 902	DRAINAGE AREA IN SQ MI .0384
INCREMENT TIME STEP HRS .25	MAX. WATER SURFACE EL FEET 1345.2
RUNOFF IN. 5.340816	PEAK DISCHARGE CFS 22.63171
TIME TO CENTROID HRS 14.45	

ITEM	FLOW	ITEM	FLOW	ITEM	FLOW	ITEM	FLOW	ITEM	FLOW	ITEM	FLOW
1	0.0	51	21.6	101	0.3	151	0.0	201	0.0	251	0.0
2	0.1	52	22.4	102	0.3	152	0.0	202	0.0	252	0.0
3	0.3	53	22.6	103	0.2	153	0.0	203	0.0	253	0.0
4	0.3	54	22.5	104	0.2	154	0.0	204	0.0	254	0.0
5	0.4	55	22.2	105	0.1	155	0.0	205	0.0	255	0.0
6	0.4	56	21.7	106	0.1	156	0.0	206	0.0	256	0.0
7	0.4	57	21.2	107	0.1	157	0.0	207	0.0	257	0.0
8	0.5	58	20.7	108	0.1	158	0.0	208	0.0	258	0.0
9	0.6	59	20.0	109	0.0	159	0.0	209	0.0	259	0.0
10	0.6	60	19.2	110	0.0	160	0.0	210	0.0	260	0.0
11	0.6	61	18.3	111	0.0	161	0.0	211	0.0	261	0.0
12	0.6	62	17.2	112	0.0	162	0.0	212	0.0	262	0.0
13	0.7	63	14.4	113	0.0	163	0.0	213	0.0	263	0.0
14	0.7	64	11.5	114	0.0	164	0.0	214	0.0	264	0.0
15	0.7	65	10.2	115	0.0	165	0.0	215	0.0	265	0.0
16	0.7	66	8.8	116	0.0	166	0.0	216	0.0	266	0.0
17	0.7	67	7.6	117	0.0	167	0.0	217	0.0	267	0.0
18	0.8	68	6.5	118	0.0	168	0.0	218	0.0	268	0.0
19	0.8	69	5.7	119	0.0	169	0.0	219	0.0	269	0.0
20	0.8	70	5.1	120	0.0	170	0.0	220	0.0	270	0.0
21	0.9	71	4.7	121	0.0	171	0.0	221	0.0	271	0.0
22	1.0	72	4.3	122	0.0	172	0.0	222	0.0	272	0.0
23	1.1	73	3.9	123	0.0	173	0.0	223	0.0	273	0.0
24	1.1	74	3.6	124	0.0	174	0.0	224	0.0	274	0.0
25	1.1	75	3.4	125	0.0	175	0.0	225	0.0	275	0.0
26	1.1	76	3.3	126	0.0	176	0.0	226	0.0	276	0.0
27	1.1	77	3.2	127	0.0	177	0.0	227	0.0	277	0.0
28	1.1	78	3.1	128	0.0	178	0.0	228	0.0	278	0.0
29	1.2	79	2.9	129	0.0	179	0.0	229	0.0	279	0.0
30	1.2	80	2.8	130	0.0	180	0.0	230	0.0	280	0.0
31	1.2	81	2.7	131	0.0	181	0.0	231	0.0	281	0.0
32	1.3	82	2.6	132	0.0	182	0.0	232	0.0	282	0.0
33	1.3	83	2.6	133	0.0	183	0.0	233	0.0	283	0.0
34	1.5	84	2.5	134	0.0	184	0.0	234	0.0	284	0.0
35	1.6	85	2.5	135	0.0	185	0.0	235	0.0	285	0.0
36	1.8	86	2.4	136	0.0	186	0.0	236	0.0	286	0.0
37	1.8	87	2.4	137	0.0	187	0.0	237	0.0	287	0.0
38	1.9	88	2.4	138	0.0	188	0.0	238	0.0	288	0.0
39	2.1	89	2.4	139	0.0	189	0.0	239	0.0	289	0.0
40	2.4	90	2.4	140	0.0	190	0.0	240	0.0	290	0.0
41	2.8	91	2.3	141	0.0	191	0.0	241	0.0	291	0.0
42	3.1	92	2.2	142	0.0	192	0.0	242	0.0	292	0.0
43	3.3	93	2.2	143	0.0	193	0.0	243	0.0	293	0.0
44	3.5	94	2.1	144	0.0	194	0.0	244	0.0	294	0.0
45	4.3	95	2.1	145	0.0	195	0.0	245	0.0	295	0.0
46	7.2	96	2.1	146	0.0	196	0.0	246	0.0	296	0.0
47	11.4	97	1.9	147	0.0	197	0.0	247	0.0	297	0.0
48	17.2	98	1.4	148	0.0	198	0.0	248	0.0	298	0.0
49	18.9	99	0.8	149	0.0	199	0.0	249	0.0	299	0.0
50	20.3	100	0.4	150	0.0	200	0.0	250	0.0	300	0.0

III. SOUTH PART OF Westwind 2ND ADDITION

This drainage area includes Lots 2-4, 16-23, 31-52, and adjacent streets, in Westwind 2nd Addition. This area will drain directly to the storm sewer system with no detention required. Therefore the peak discharge was calculated by the Rational Method, using the 100-yr frequency rainfall, per City of Wichita drainage criteria.

Drainage area	8.5 Acres
C100	0.67
Time of concentration	15 minutes
I100	7.37 in./hr.
Q100	42.0 cfs

A major sump will be located at Woodchuck and Westlawn Circle, with inlet and outflow pipe capacity for the Q100. The outflow system will discharge directly into the 72" SWS 145 at Woodchuck and 19th Street. Extension of the storm sewer system north of Westlawn Circle will be designed for the initial storm (Q2).

The time of concentration for this area is 15 minutes, compared to 35 minutes in the SWS 145 system. By reducing the Q100 by the ratio of the times of concentration, the adjusted Q100 into the storm sewer is $42 \text{ cfs} \times \frac{15}{35} = 18 \text{ cfs}$.

Rainfall Intensity

$$5.0 / 7.37 = 28.5 \text{ cfs}$$

ANALYSIS OF SWS 145 USING CURRENT CRITERIA Revised 2-16-87

Note: Original City of Wichita design was based on old rainfall frequency-intensity-duration curves.

Per current design standards, initial design storm frequency is 2-yr. (use Q2). Runoff coefficients (for Rational Method) are 0.41 for 2-yr and 0.61 for 100-yr, based on 1/3 acre residential lot size, 50% Soil Group B and 50% Soil Group C.

New I and Q values are based on Rainfall Intensity Table for Sedgwick County (April 15, 1986). C of W Drainage Criteria Manual, Attachment A.

Times of concentration and drainage areas are per original design by C of W (Kerby).

Analysis I used Civilsoft "Storm" program on IEM-AT Computer. Q values used are for 2-yr frequency (Q2) under existing conditions (no development north of 19th Street).

Analysis II was for existing system with effect of Q100. Excess flow was routed through streets.

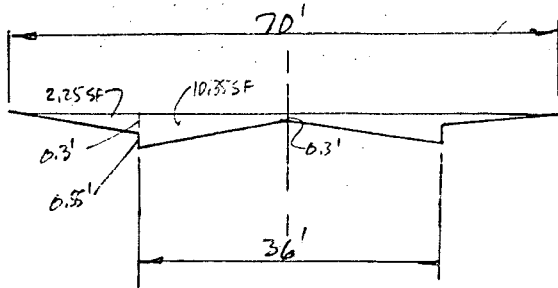
These analyses considered the flow in laterals for the first section only above their junctions with the main line (points 7, 3, and 2 in the original design), since Q (Max Q) at those points was added to mainline flow as adjusted Q (ADJ Q). Laterals were not analyzed in detail upstream from these points.

(See attached printout for Q2 analysis and worksheet for Q100 analysis.)

STREET FLOW CAPACITIES FOR ANALYSIS II

Ignoring storage in sumps,
Pipe Q = Q100 - overflow in street

Street section (70' R/W), typical for 19th, Westport, and Northwest Parkway:



$$A=25.2 \text{ sf} \quad wp=67.4' \quad r=0.295 \quad r^{2/3}=0.44$$

$$n = 0.015$$

$$V=1.486 r^{2/3} s^{1/2}$$

$$V = \frac{1.486 (.44) s^{1/2}}{0.015}$$

$$V = \frac{Q}{A} = \frac{Q}{25.2}$$

$$Q = 43.7 \sqrt{s}$$

$$Q = 1101.2 \sqrt{s}$$

Capacity various slopes:

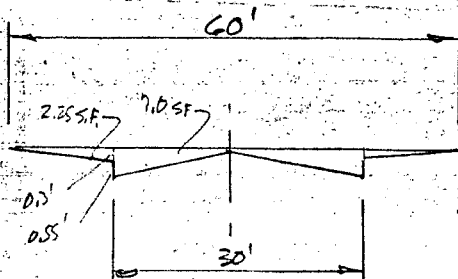
(Sump at Reca)

- for S = 0.004, Q = 22.0
- for S = 0.0107, Q = 114.1
- for S = 0.0070, Q = 92.4
- for S = 0.0120, Q = 120.8

(Sump at Woodchuck)

- for S = 0.0077, Q = 96.6
- for S = 0.0035, Q = 65.2
- for S = 0.0018, Q = 46.3
- for S = 0.0011, Q = 36.4
- for S = 0.0115, Q = 118.2
- for S = 0.0020, Q = 49.4
- for S = 0.0033, Q = 63.3

Street Section (60' R/W), for Cramer Street:



$$A=18.5 \quad wp=61.2 \quad r=0.30 \quad r^{2/3}=0.45$$

$$n=0.015$$

$$Q = \frac{1.486 (0.45) \sqrt{s} (18.5)}{0.015}$$

$$Q = 827.0 \sqrt{s}$$

ANALYSIS II SUMMARY

Line identification same as 2-yr analysis (see printout attached)

Values for Q100 based on current City of Wichita drainage design criteria. See plan worksheet for Q calculations. Storage in sumps ignored.

Note: Original design of system provided for Q100 of 154.21 cfs in pipe from sump at Reca & 19th. Current value calculated for Q100 at same point = 136 cfs.

Original design of system provided for Q100 of 263.19 cfs in pipe from sump at Woodchuck & 19th. Current value calculated for Q100 at same point = 218 cfs.

Based on this analysis, the available capacity for additional area to drain into the system is 45 cfs at the 19th and Woodchuck sump location.

Line	Q cfs	SIZE	Slope in/ft	LENGTH	V ₁	V ₂	FL ₁	FL ₂	HG ₁	h _f	HG ₂	Top of Curb
1	—	8x45	—	—	—	—	—	—	—	—	136.57	
2	494		.00886	608.61	13.7	13.7	128.64	130.95	136.57	5.36	141.93	
3	494		.00886	213.90	13.7	13.7	130.95	131.80	141.93	1.90	143.83	
4	494		.00886	437.19	13.7	13.7	131.80	133.80	143.83	3.37	147.70	148.70
5	494		.00886	272.50	13.7	13.7	133.80	134.86	147.70	2.41	150.11	
6	410		.00611	100.53	11.4	11.4	134.86	135.25	150.11	0.41	150.72	
7	404		.00607	262.20	11.4	11.4	135.25	136.45	150.72	1.59	152.31	
8	339	✓	.00417	234.31	9.4	9.4	136.45	138.50	152.31	0.98	153.29	
9	234	72°	.00305	549.94	8.3	8.3	138.52	140.96	153.29	1.60	154.97	
10	234		.00305	283.02	8.3	8.3	140.96	142.21	154.97	0.46	155.88	
11	222		.00275	276.44	7.9	7.9	142.23	143.45	155.88	0.76	156.59	
12	218		.00265	137.19	7.7	7.7	143.47	144.07	156.59	0.36	156.95	155.20
13	218	✓	.00265	397.67	7.7	7.7	144.12	144.96	156.95	1.05	158.00	
14	137	60°	.00277	320.98	7.0	7.0	145.09	145.73	158.00	2.09	158.29	
15	137		.00277	269.50	7.0	7.0	148.02	149.84	158.89	0.75	159.64	
16	137		.00277	270.00	7.0	7.0	149.87	151.22	159.64	0.75	160.39	
17	137	✓	.00277	508.63	7.0	7.0	151.24	153.78	160.39	1.41	161.80	
18	420	42°	.00179	36.84	4.4	4.4	155.23	156.49	161.80	0.96	161.86	161.45
19	263	36°	.00112	167.00	3.2	3.2	156.00	157.14	161.86	0.15	162.06	
20	223	30°	.00296	274.00	4.5	4.5	157.71	158.75	162.04	0.81	162.85	
21	13.1	20°	.00535	234.40	4.2	4.2	159.25	161.23	162.85	0.99	163.64	
22	13.1	21	.00684	69.50	5.4	5.4	161.48	162.67	163.64	0.47	164.11	164.0

Check Culvert Capacity Sta. 9+74

(outlet control)

1. try 2 - 30"x 19" pipe dc = 1.57' dsfl = 152.59 TW = 154.16
H = 157.1 - 154.16 = 2.9', Q = 32 cfs x 21 = 64 cfs > 52 cfs (Q10)

2. try 3 - 36"x 22" pipe dc = 1.33' DSFL = 152.59 TW = 153.92
H = 157.1 - 153.9 = 3.2', Q = 34 cfs x 3 = 102 cfs > Q50

3. try 2 - 36"x 22" pipe dc = 1.43' dsfl = 152.59 TW = 154.22
H = 157.1 - 154.2 = 2.9', Q = 33 cfs x 2 = 66 cfs

4. try 3 - 30"x 19" pipe dc = 1.50' dsfl = 152.59 TW = 154.1
H = 157.1 - 154.1 = 3.0', Q = 32 cfs x 3 = 96 cfs > Q50

Use option 2 or 4. for Q50

Use 1 or 3 for Q10 only

Check Culvert Capacity Sta. 14+57

(outlet control)

1. try 2 - 30"x 19" pipe dc = 1.42' dsfl = 146.2 TW = 147.8
H = 149.6 - 147.8 = 1.8', Q = 24 cfs x 2 = 48 cfs < Q10

2. try 3 - 29"x 18" pipe dc = 1.38' dsfl = 146.2 TW = 147.6
H = 149.6 - 147.6 = 2.0', Q = 16.5 cfs x 3 = 49.5 cfs < Q10

3. try 2 - 38"x 24" pipe dc = 1.75' dsfl = 146.2 TW = 148.0
H = 149.6 - 148.0 = 1.6', Q = 37 cfs x 2 = 74 cfs Q50

4. try 2 - 43"x 27" pipe dc = 1.56' dsfl = 146.2 TW = 147.8
H = 149.6 - 147.8 = 1.8', Q = 40 cfs x 2 = 80 cfs > Q50

Use option 3 or 4.

CHECK DITCH CAPACITY - S. side 21st St. (Westwind 2nd Addition)
 TCR Revised 2-20-87 *Note: elev. are city datum*

Drainage Area north of 21st Street flowing to culverts = 68 acres
 (from Nov 86 calculations) $Q_{10} = 52$ cfs
 $Q_{50} = 81$ cfs
 $Q_{100} = 103$ cfs

Existing culverts are 2 - 49" x 32" rcphe, length = 102'
 $usfl = 159.40$ $dsfl = 159.26$ $s = 0.0014$
 Max. HW = $162.71 - 159.40 = 3.31'$

~~w/ inlet control: HW/D = $3.31/2.67 = 1.24$
 from FHWA chart 3, $Q = 60$ cfs per pipe~~

w/ outlet control: (chart 10) $K_e = 0.5$ $H = 3.31 - 2.67 = 0.64'$
 $Q = 38$ cfs per pipe

Outlet control governs. $Q = 76$ cfs $\approx Q_{50}$. Note: When HW
 elevation exceeds 162.0, water will flow east along n. side of 21st.

DITCH CAPACITY @ MINIMUM SECTION

Minimum ditch section is at Sta. 8+00

Slope = 0.00125 $A = 18.1$ sq $w_p = 18.2'$

$r = 0.995$ $r^{2/3} = 0.997$ $n = 0.030$

$$V = \frac{1.486 (r^{2/3}) (s^{1/2})}{n}$$

$$V = \frac{1.486 (0.977) (0.035)}{0.030} = 1.75 \text{ fps}$$

$$Q = AV = 18.1 (1.75) = 31.6 \text{ cfs}$$

Existing ditch section is insufficient for Q_{50} .

Lower ditch Sta. 8+00 to elev. 154.75

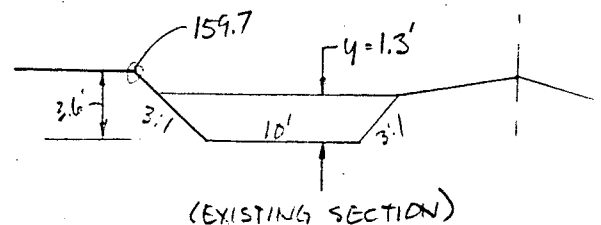
$y = 2.5'$ $s = 0.00532$ $s^{1/2} = 0.0730$

$A = 43.75$ $w_p = 25.6$ $r = 1.695$ $r^{2/3} = 1.422$

$$V = \frac{1.486 (1.422) (0.073)}{0.030} = 5.14 \text{ fps}$$

$$Q = 5.14 (43.8) = 225 \text{ cfs} \gg Q_{50}$$

Actual depth $y @ 52$ cfs $\approx 1.5'$



Line No	Q (cfs)	D (in)	W (in)	Dn (ft)	Dc (ft)	Flow Type	St-full (ft/ft)	V 1 (fps)	V 2 (fps)	FL 1 (ft)	FL 2 (ft)	HG 1 Calc	HG 2 Calc	
1	Hydraulic grade line control = 136.57													
2	173.0	54	96	2.35	2.44	Full	.00109	4.8	4.8	128.64	130.95	136.57	137.23	
3	171.0	54	96	2.30	2.42	Full	.00106	4.8	4.8	130.95	131.80	137.28	137.50	
4	163.0	54	96	2.12	2.34	Seal	.00097	4.5	5.0	131.80	133.80	137.58	137.91	
	X =	353.57		X(N) =	.00									
5	157.0	54	96	2.18	2.28	Seal	.00090	4.4	5.0	133.80	134.86	138.64	138.75	
	X =	112.85		X(N) =	.00									
6	151.0	54	96	2.13	2.23	Part	.00083	4.7	5.2	134.86	135.25	138.88	138.88	
7	151.0	54	96	2.01	2.23	Part	.00083	5.1	10.3	135.25	136.45	138.96	138.28	
	X =	.00		X(N) =	.00	X(J) =	214.91	F(J) =		61.02		1.90	D(AJ)	
8	147.0	54	96	1.58	2.19	Part	.00078	11.5	8.4	136.45	138.50	138.05	140.69	
9	81.3	72	0	2.21	2.41	Part	.00037	4.3	7.6	138.52	140.94	142.35	143.35	
	X =	.00		X(N) =	409.98	X(J) =	239.52	F(J) =		30.43		2.21	D(AJ)	
10	76.1	72	0	2.13	2.33	Part	.00032	5.6	7.5	140.96	142.21	143.85	144.54	
	X =	.00		X(N) =	190.01	X(J) =	56.85	F(J) =		27.97		2.13	D(AJ)	
11	72.2	72	0	2.07	2.27	Part	.00029	5.8	7.4	142.23	143.45	144.96	145.72	
	X =	.00		X(N) =	171.16	X(J) =	40.70	F(J) =		26.15		2.07	D(AJ)	
12	72.2	72	0	2.08	2.27	Part	.00029	5.8	7.4	143.47	144.07	146.19	146.34	
	X =	.00		X(N) =	.00	X(J) =	40.09	F(J) =		26.14		2.08	D(AJ)	
13	71.5	72	0	2.48	2.26	Part	.00028	6.1	6.5	144.10	144.98	146.70	147.46	
	X =	.00		X(N) =	368.53									
14	58.1	60	0	2.49	2.13	Part	.00050	5.0	15.9	145.09	145.73	147.93	146.93	
	X =	.00		X(N) =	.00	X(J) =	187.59	F(J) =		22.01		1.67	D(AJ)	
15	54.4	60	0	1.71	2.07	Part	.00044	9.1	7.1	148.00	149.84	149.71	151.90	
	X =	.00		X(N) =	66.34									
16	51.2	60	0	1.80	2.01	Part	.00039	5.4	7.0	149.87	151.22	152.30	153.23	
	X =	.00		X(N) =	174.24	X(J) =	29.48	F(J) =		17.56		1.80	D(AJ)	
17	48.9	60	0	1.76	1.96	Part	.00035	5.5	13.5	151.24	153.78	153.56	154.98	
	X =	.00		X(N) =	134.99	X(J) =	20.82	F(J) =		16.55		1.76	D(AJ)	

3 Hydraulic grade line control = 137.25
 23 15.3 27 0 1.12 1.36 Part .00244 7.4 6.1 143.23 143.50 144.39 144.86

Project : STORM WATER SEWER NO. 145

Date:

STORM DRAIN ANALYSIS RESULTS (cont)

Line No	Q (cfs)	D (in)	W (in)	Dn (ft)	Dc (ft)	Flow Type	St-full (ft/ft)	V 1 (fps)	V 2 (fps)	FL 1 (ft)	FL 2 (ft)	HG 1 Calc	HG 2 Calc
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4						Hydraulic grade line control =	137.54						
24	15.4	18	0	1.50	1.41	Seal	.02149	8.9	8.7	141.86	143.50	143.27	146.85
	X =			3.29	X(N) =		.00						

7						Hydraulic grade line control =	138.16						
25	75.3	54	0	1.71	2.53	Part	.00147	13.4	8.2	140.00	143.92	141.73	146.45

LIST OF ABBREVIATIONS

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
 X(N) - Distance in feet from downstream end to point where water surface reaches normal
 X(J) - Distance in feet from downstream end to point where hydraulic jump occurs in fir
 F(J) - The computed force at the hydraulic jump
 O(BJ) - Depth of water before the hydraulic jump (upstream side)
 O(AJ) - Depth of water after the hydraulic jump (downstream side)
 SEAL indicates flow changes from part to full or from full to part
 HJ indicates that flow changes from supercritical to subcritical through a hydraulic
 HJU indicates that hydraulic jump occurs at the junction at the upstream end of the li
 HUD indicates that hydraulic jump occurs at the junction at the downstream end of the li