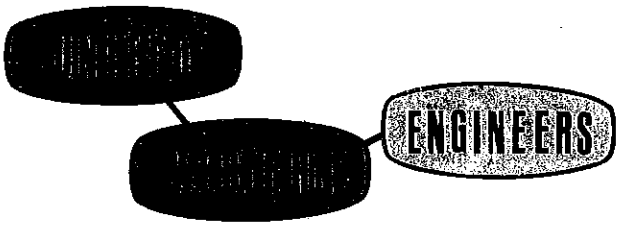

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DRAINAGE REPORT
A RESTUDY OF A PORTION OF THE TEMPLETON GAP DRAINAGE BASIN INVOLVING THE
PARK VISTA ESTATES SUBDIVISION
EL PASO COUNTY, COLORADO
JULY 1974

UNITED WESTERN ENGINEERS

planners · consultants · engineers

Suite 200
4525 Northpark Drive
Colorado Springs, Colo. 80907

(303) 598-3222

July 22, 1974

Mr. DeWitt Miller
Director of Public Works
115 N. Nevada
Colorado Springs, Colorado

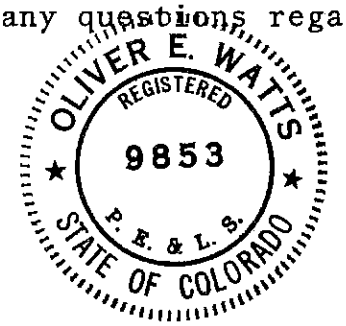
Dear Deke:

We are pleased to transmit herewith the Hydrologic Engineering Study related to the Park Vista Estates Subdivision. This study places particular emphasis on drainage problems within this subdivision, and is a restudy of that portion of the Templeton Gap Drainage Basin of which Park Vista Estates is a part. In accordance with your instructions, the primary greenbelt network is designed and examined for the storm of a return period of 100 years.

Only those costs directly associated with drainage work are included herein. Other costs associated with the eventual street improvements, utility extensions and similar costs are not included.

This report has been prepared by me and under my direct supervision and is certified to be correct and in compliance with all applicable ordinances, criteria and specifications of the City of Colorado Springs.

Please do not hesitate to call me if I may answer any questions regarding this study.



Respectfully submitted,

UNITED WESTERN ENGINEERS

Oliver E. Watts
PE - LS 9853
Engineering Director

Enclosure

OEW/jc

INDEX

List of Figures

SECTION I

I N T R O D U C T I O N

A. Description of Basin

The total drainage basin is shown on plates one and two in the appendix. It is terminated on the West boundary by Academy Boulevard, where a concrete box culvert, constructed in 1971 by the State Highway Department, is the outfall point. The remainder of the western boundary parallels Templeton Gap Road to Barnes Road and then follows a natural ridge, separating it from another portion of the Templeton Gap Basin. The Northern boundary is the ridge dividing the Templeton Gap Basin from the Cottonwood Creek and Sand Creek Basins. The Sand Creek Basin abuts the Eastern and Southern boundaries. The southwestern boundary is a natural division from a separate portion of the Templeton Gap Basin.

The basin comprises 1573 acres (2.46 square miles) and occupies portions of Sections 13, 14, 23, 24, 25 and 26 of Township 13 South, Range 66 West of the 6th P.M. in El Paso County. The basin is typically tear-drop in shape and ranges in elevation from 6408 to 6882, USGS datum. The length of the longest water course is approximately 12,400 feet. As shown on plate number one, over half of the basin is within the City Limits of Colorado Springs.

B. Background Information

This basin is a portion of the 9.6 square mile North Shooks Run - Templeton Gap Floodway Drainage Basin, for which a drainage study was completed on September 10, 1963 by United Western Engineers, under the direction of George D. Morris, PE 2051. The study was accepted by the City of Colorado Springs and has been the basis of all drainage design within the basin since that time.

This study was based on a storm of a return period of 50 years, and defines a general alignment of improved storm sewer systems to route the runoff, designating the design flows for the detailed design of the major systems. The existing outfall point at Academy Boulevard was designated as hydrology point number V and was to accommodate 2976 CFS of runoff from 1841 acres. An open channel was specified through the Park Vista Estates Area on the same alignment as recommended herein, with culverts to be provided across Siferd Road and Siferd Boulevard. This system was obviously necessary because, as shown on the enclosed plates, the Park Vista Estates subdivision was platted (in 1955) in the County without regard to the major natural drainage path traversing it. As shown on plates 3 and 4 this drainage path was a very wide native grassed area with no defined channel.

The basin is now largely developed within the City Limits and no diversion of runoff into the natural basin has been made, nor are any proposed. However this development has increased the runoff and channelized it into the subdivision. This was approved by the City Drainage Board on February 9, 1970 when the drainage plan for Northridge Filing Number Two by the developers of Village Seven was under consideration.

Since the completion of the construction of the channel system. into Park Vista Estates, several intense rainstorms have created siginificant storm drainage to roads and dwellings within the subdivision, which has resulted in considerable news coverage by the local media. This drainage will increase in severity as the basin continues to develop and the drainage system is extended into now undeveloped land.

Because of this situation, and because the Park Vista Estates Subdivision is under consideration for annexation to the City, this study has been commissioned by the City Council and the County Commissioners, the cost of which is divided equally.

Three significant changes have been made which effect the Templeton Gap Basin report, and therefore this study, as follows:

1. In January 1972, the plans for the Shoppers World storm sewer were approved, whereby the runoff from approximately 268 acres lying west of Templeton Gap Road was diverted from this basin. This storm sewer crosses Academy Boulevard at Montebello Drive, traverses the Mall of the Bluffs Shopping Center and enters the primary greenbelt well below Academy Boulevard.

2. Revisions to City Criteria required storm sewer in Templeton Gap Road where none was previously required or specified in approved drainage reports. This storm sewer and the West half of Templeton Gap Road has and will continue to be installed by City Developers, while the City is participating in one-half the cost of the storm sewer. The remaining half of the cost is attributable to future improvement on the East side of Templeton Gap Road and is therefore included in the costs of this report.

3. Recent requirements of the Federal Flood Insurance Act and subdivision standards of FHA-VA-HUD require the primary greenbelt system to be designed for the 100 year storm. This is the basis for this study and, in addition, all existing facilities within the basin, which were constructed under the 50 year criteria, are analyzed for carrying capacity.

C. Basin Development

1. Existing Developments

As shown on plates one and two a considerable portion of the basin has been platted, although not completely developed at this time.

Three County Subdivisions exist. Park Vista Estates occupies the Westerly extremity of the Basin in Section 26. Park Vista occupies that portion of Section 23 lying North of Templeton Gap Road. The Templeton Heights Subdivision occupies a small portion of crest of the basin East of Templeton Gap Road. Each of these subdivisions is only partially occupied and are served by Wells and Septic Tanks. They are typically in gross non conformance to regulations now in effect due to recent land use legislation. In particular the provisions for drainage are severly inadequate.

Because of current regulations regarding well development and sewage disposal it is very doubtful that full development will be realized prior to annexation of these areas to the City.

Three major City developments exist, of roughly equal size. Village Seven occupies all of Section 26 excepting Park Vista Estates and the Vista Grande development lying North of Templeton Gap Road. Village Seven is nearing complete development, excepting portions of the Multi-family zoning. The major greenbelt is complete to Barnes Road and all interior drainage systems are in place.

The Homestead Development by Col-Terra occupies all of Section 25 within the basin and South of Barnes Road. It is about one-third developed and drainage facilities are about 90% complete.

The Ridge Subdivision by Otero Savings and Loan occupies part of Sections 24 and 25 North of Barnes Road. This development is in the early stages of construction, however the principal drainage structures are complete.

All approved drainage plans for the existing developments have been reviewed and are analyzed in Sections II B6 and II D.

2. Anticipated Future Development

The entire basin is analyzed on the anticipated future development to the highest and best use of the land. Existing or proposed subdivisions are considered to be fully developed. Drainage facilities are shown on plate two to upgrade the County Subdivisions to the City Standards as later described. Unplatted terrain is predicted, for the purposes of this study, to develop as follows.

The area Northeast of the Templeton Gap Road-Barnes Road intersection in Section 23 is considered to develop to a commercial zoning.

Powers Boulevard is anticipated on its latest proposed alignment shown on plates one and two, which will create a commercial area near its intersection with Templeton Gap Road, where the terrain will permit.

The area immediately North of the Ridge development should develop similar to that of the Ridge, where terrain slopes permit. The steepest slopes will be left in their natural state in the form of a park similar to Rockrimmon, the Ridge or Homestead. The moderately steep terrain near the northern limits of Templeton Gap Road will develop into larger tracts or a cluster system similar to Rockrimmon with a high percentage of open space.

SECTION II

H Y D R O L O G I C D A T A

A. Hydrology

All computations have been performed in accordance with the U.S. Department of Agriculture Soil Conservation Service Synthetic Hydrograph Method. This method has been used throughout the City of Colorado Springs since the early 1960's and is the method adopted by most counties, including El Paso County, in accordance with recent land use legislation. The SCS has recently clarified and interpreted the many variations of this method into a series of methods prescribed to be used in the State of Colorado. The prescribed method, as described herein, is felt to be fully adequate for this basin and compares favorably with the criteria of other Major Cities in the area. The City is now revising its drainage criteria to bring it up to date and standardize its use among the various Engineering Consultants practicing within the City.

1. General Method

Within this climatic region, storm runoff is computed to result from high intensity rainstorms that have occurred following a five day period of near average conditions, or an accumulated five day rainfall of between 1.4 and 2.1 inches, preceeding the storm of six hours in duration. The runoff is computed from the formula

$$q_p = q\ A\ Q$$

where:

q_p = design discharge in cubic feet per second.

q = Runoff per square mile per inch if rainfall, as determined from the enclosed figure number one.

A = Area of the Basin in Square Miles.

Q = Runoff, as later described.

2. Rainfall

The rainfall for a particular design storm of six hour duration was developed by the U.S. Weather Bureau for the SCS in October, 1967, accounting for the 1965 flood. A plot of the rainfall amounts, in inches, versus the return period in years for the Park Vista Estates area is shown on figure 2, as taken from the USWB isopluvial maps. It may be seen that the rainfall varies from 1.7 inches for the two year storm to 3.6 inches for the 100 year storm.

3. Time of Concentration (T_c)

Time of Concentration is defined as the time required for water to travel from the most remote point in a particular basin to the outfall point, and is critical in assessing the total runoff of the basin. As shown on figure one, a shorter T_c results in

greater runoff. This is indicative of the distribution of the rainfall over the 6-hour period, so that the runoff must be computed by the most intense period of the storm corresponding to the time required to drain the basin.

For overland flow conditions (all flow to the first inlet), T_c is computed from

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

where:

T_c in hours.

L = Length of runoff path from the most remote part of the basin, in miles.

H = Elevation difference along the runoff path, in feet.

When water is contained in an enclosed storm sewer the full bore velocity is used in computing T_c. When in an open channel, the actual design velocity is used, resulting in a trial and error procedure until the estimated T_c and the actual T_c closely agree.

4. Soils

By far the most critical part of any drainage study is in assessing the soils into one of the four following major groups.

Group A: Results in low runoff and consists of deep, well drained soils having high infiltration even when thoroughly wetted.

Group B: Consists of moderately fine to coarse textured soils having moderate infiltration when thoroughly wetted. The most common in the Colorado Springs area.

Group C: Has slow infiltration rates when thoroughly wetted, usually having a layer of clay or fine silt impeding the water transmission.

Group D: Results in a high runoff potential, chiefly clays with high swelling potential, permanently high water table or shallow cover to impervious material. Very common along the steeper portions of the City.

All four soil groups are encountered within the basin as shown on plate number one. The mapping and interpretations are that of the local SCS office, and is available at no cost to interested parties.

5. Runoff

The characteristics of runoff are assessed using the above soil groupings in conjunction with the percentage of impervious cover

(pavement, roofs, etc) within the drainage basin. Based on detailed studies of aerial photography of the Colorado Springs region, the following percentages of impervious cover were used:

Type of Development	% Impervious Cover
County Single Family Residential	25% - 30%
City Single Family Residential	40%
Commercial, PUD	70% - 80%
R5	40% - 80%
Road Rights of Way	90%

Parks are assessed as good range land, except where they are to remain natural, when they are assessed as fair. The corresponding complex curve numbers are taken from the enclosed figure number three. The runoff corresponding to these curve numbers is available from various SCS publications.

6. Criteria

The City requires all drainage to be based on the 50 year return period, except for major greenbelts in which the runoff exceeds 500 cubic feet per second (CFS). A detailed analysis of previously approved design flows in the basin was made, in which the prescribed hydrologic method was compared with the method in use at that time. The former method in general gave reasonable results when the basins were quite large, however in smaller basins the flows were much less than expected. The following comparisons are presented:

Basin	Previously Approved Runoff - CFS	Actual Return Period - Years
B7 + B8	205	15
D2 Through D8	367	11
E1	56.2	7
F1	84	18
Hydrograph Pt. #1	327.5	120
#2	415.5	10
#4	847	12
#5	886	9
#6	1285	10
#7	1300	9
#8	1900	14
#11	2976	25

Because of the nature of the previous results, an attempt is made to provide uniformity of design and the following return periods are used in this report.

Location of Structures	Design Return Period - Years
Interior Storm Sewers	10 Years
Minor Greenbelts	50 Years
Major Greenbelts	100 Years

This criteria compares very favorably with that of other major cities and is almost universally more severe than that used in the past in Colorado Springs.

The following is a summary of hydrologic computations.

UNITED WESTERN ENGINEERS

Project Park Vista Estates

Calc. by O.E. Watts date July 1, 1974

Checked by _____ date _____

HYDROLOGY

Basin	A-SM	CN	Tc -hrs-	q	10 Year		50 Year		100 Year		Design Flow
					Q	q_p	Q	q_p	Q	q_p	
A1	0.0164	90	0.160	980	1.44	23.1					23.1
A2	0.0179	90	0.283	905	1.44	23.3					23.3
A3	0.112	87	0.246	930	1.23	128					128
A4	0.0618	87	0.336	875	1.23	66.5					66.5
A3 + 4	0.174	87	0.336	875	1.23	187					187
A2 - 4	0.191	87	0.387	840	1.23	197					197
EA	0-208	88	0.392	840	1.30	227					227
B1	0.0395	85	0.309	900	1.10	39.1					39.1
2	0.0251	85	0.229	940	1.10	26.0					26.0
3	0.0114	85	0.148	990	1.10	12.4					12.4
4	0.0162	75	0.132	1000	0.59	9.6					9.6
5	0.0234	87	0.172	970	1.23	27.9					27.9
6	0.0191	85	0.192	960	1.10	20.2					20.2
7	0.0181	76	0.168	980	0.63	11.1					11.1
8	0.223	78	0.204	960	0.72	154					154
5+6	0.0425	86	0.359	860	1.16	42.4					42.4
7+8	0.241	78	0.204	960	0.72	167	1.27	294			294
C1	0.656	81									N/A
D1	0.0271	87									N/A
2	0.0375	87	0.134	1000	1.23	46.1					46.1
3	0.0220	86	0.117	1000	1.16	25.5					25.5
4	0.105	87	0.234	940	1.23	121					121
5	0.0700	85	0.252	930	1.10	71.6					71.6
6	0.0620	87	0.142	990	1.23	75.5					75.5
7	0.00574	85	0.228	940	1.10	5.9					5.9
8	0.0283	84	0.368	850	1.04	25.0					25.0
D7 + 8	0.0340	84	0.368	850	1.04	30.1					30.1
4 - 6	0.237	86	0.252	930	1.16	256					256
4 - 8	0.272	86	0.253	930	1.16	293					293
3 - 8	0.294	86	0.260	925	1.16	315	1.83	498	2.19	596	596
2 - 8	0.331	86	0.284	905			1.83	548	2.19	656	656
E1	0.0864	81	0.415	830	0.87	62.4					62.4
2	0.0841	81	0.180	980	0.87	71.7					71.7
3	0.113	84	0.244	930	1.04	109					109
F1	0.721	83	0.236	940	0.91	61.7					61.7
2	0.152	84	0.126	995			1.68	254			254
HP #1	0.147	86	0.058	1000			1.83	269	2.19	321	269
HP #2	0.371	85	0.126	995					2.11	779	779
HP #3	0.206	86	0.196	960			1.83	362	2.19	433	362
HP #4N	0.318	85	0.214	960					2.11	644	644

UNITED WESTERN ENGINEERS

Project Park Vista Estates

Calc. by O.E. Watts date July 1, 1974

Checked by _____ date _____

Basin	A-SM	CN	Tc -hrs-	q	10 Year		50 Year		100 Year		De-sign Flow
					Q	q_p	Q	q_p	Q	q_p	
HP#4E	0.455	84	0.144	990					2.03	914	914
HP #4	0.773	84	0.214	960					2.03	1510	1510
HP #5	0.897	84	0.229	940					2.03	1710	1710
HP #6	1.19	85	0.229	940					2.11	2360	2360
HP #7	1.27	85	0.240	940					2.11	2520	2520
HP #8	1.87	83	0.297	900					1.94	3260	3260
HP #9	2.16	83	0.302	900					1.94	3770	3770
HP #10	2.25	83	0.320	890					1.94	3880	3880
HP #11	2.46	83	0.324	890					1.94	4250	4250

Total Basin = 1573 Acres

DATA

$q_p = q\ A\ Q$ 6 hour storm

q from Fig. 1
A = A-SM Measured
Q from SCS Publications

Return Period	Intensity
10 yrs.	2.4"
50 yrs.	3.2"
100 yrs.	3.6"

PEAK DISCHARGE IN
CSM PER INCH OF RUNOFF
VERSUS
TIME OF CONCENTRATION, T_C
UNDER 7000'
EAST SLOPE

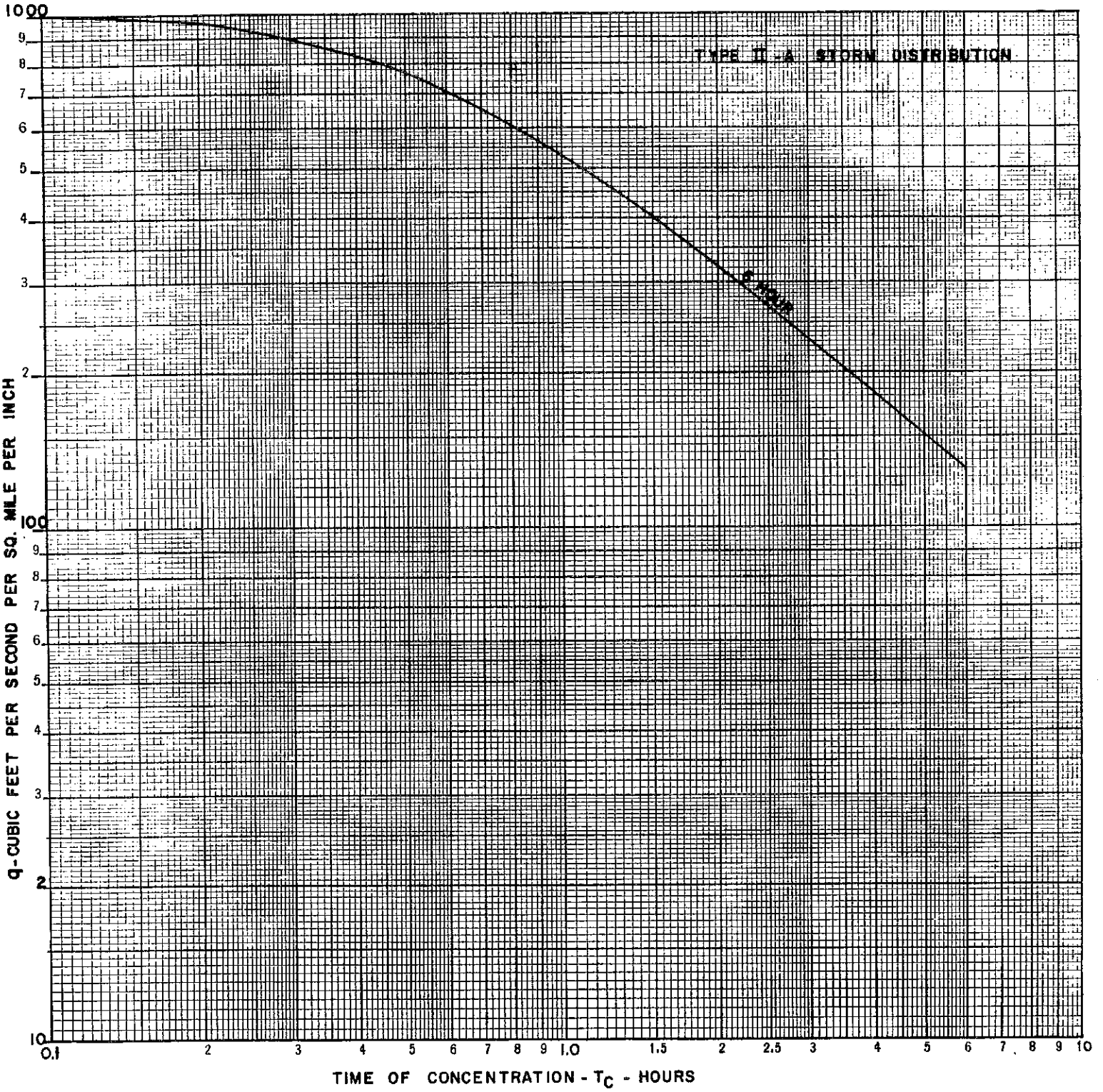
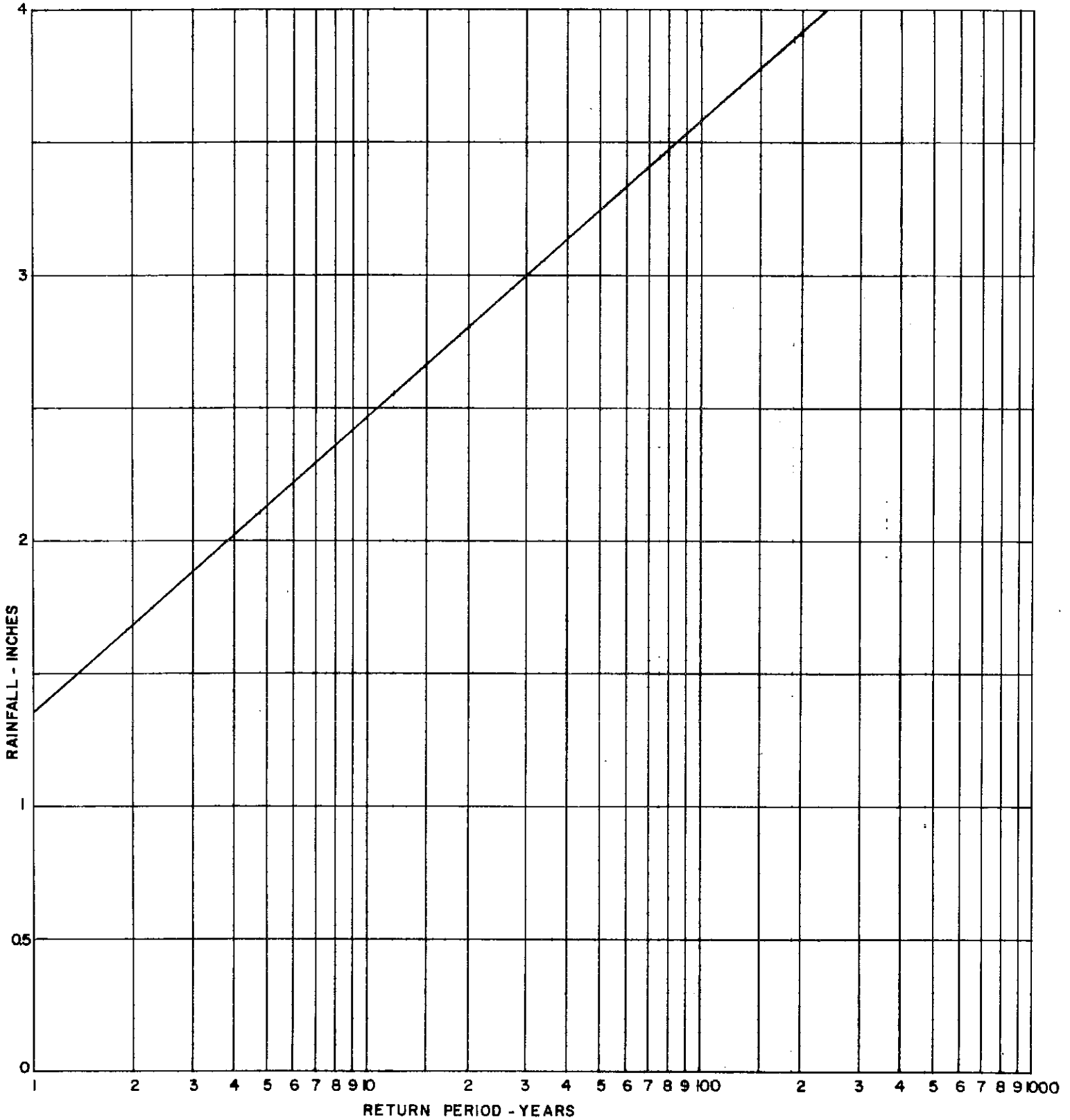


FIGURE No. 1



PARK VISTA ESTATES DRAINAGE BASIN

RAINFALL PLOT - 6 HOUR STORM

UWE

FIGURE No. 2

URBAN HYDROLOGIC SOIL COVER COMPLEX
AND ASSOCIATED CURVE NUMBERS

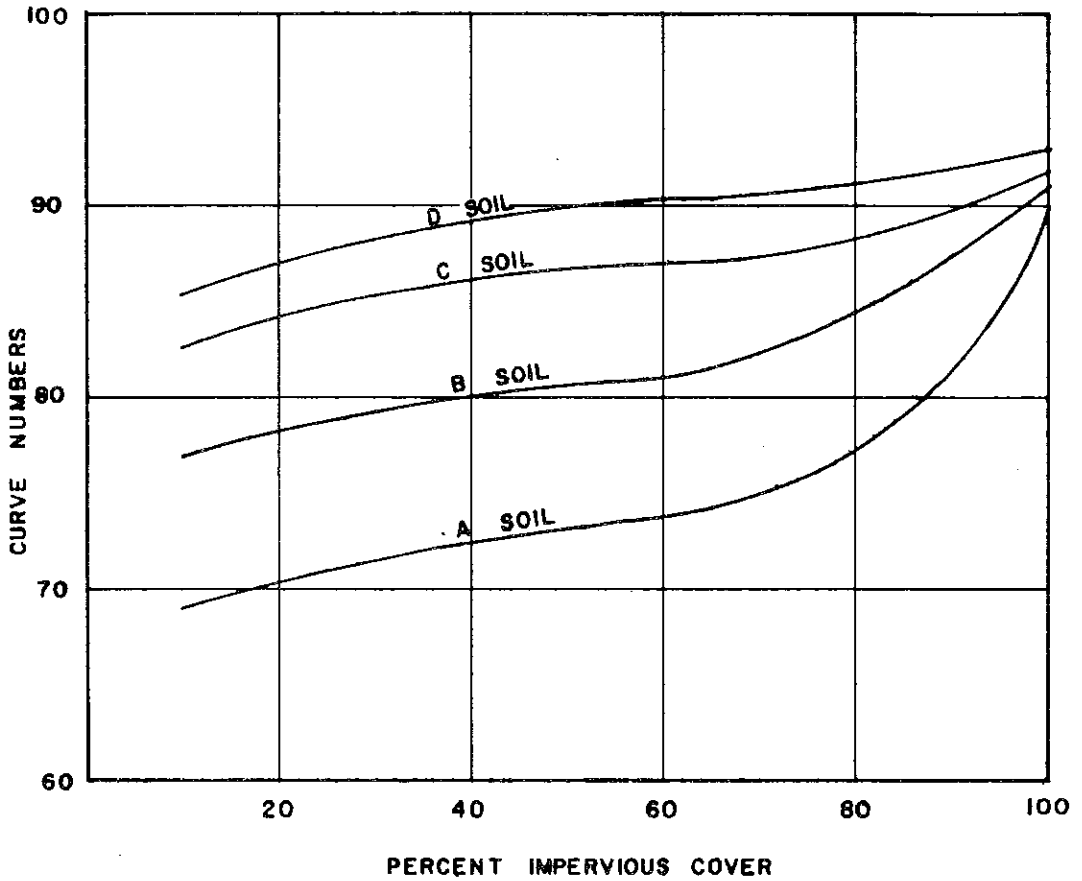


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FIGURE No. 3

B. Hydraulics

Structure sizing is computed in accordance with the criteria of the City of Colorado Springs. The basic formula used is Mannings equation, for which the following roughness coefficients (n) apply.

Type of Structure	n
RCP Storm Sewer	0.013
Concrete Lined Channels	0.015
Formed Concrete Structures	0.013
Natural Earthlined Channels	0.020
Park Vista Estates Flood Plain	0.030

Closed conduits are designed to flow as full as possible under no pressure. The storm sewers shown on the enclosed plates are sized in accordance with the enclosed figure number 4, where the RCP alternative is utilized.

Open channels are proposed to be fully concrete lines, with a minimum freeboard of one foot, including curvature superelevation. These are sized on the most economical ditch sections as shown on figure 5. This design is strongly recommended after detailed observation and examination of many other alternatives throughout the past several years.

Transition sections are of sufficient length to maintain a uniform water surface profile. The tangent of the water surface flare angle is one divided by three times the Froude number.

Box culverts at road crossings are designed as open channels if no piers exist. If piers are present they must be analyzed in detail for water surface profile. In most cases, the slope is steep and the water sruface will rise within the structure. Interested parties are referred to reference numbers 4a and 6 cited in the appendix.

Catch basins are sized in accordance with that of the proposed revised City Criteria. All streets are assumed to have a 36 foot width with vertical curbs, except for major arterials, where the surface runoff is limited to 30 CFS. Other streets are allowed to flow at a depth equal to half the curb height. Various streets are observed to violate this criteria, however where storm sewers exist no modification is proposed. No danger of flooding from these situations is anticipated.

C. Flow Routing

The proposed storm sewer alignment throughout the basin is very nearly exactly what has been previously approved in the Templeton Gap Basin report. Certain modifications based on field observations and proposed development plans have been made. The following describes the primary greenbelt alignments.

1. Upper Basin

North of Barnes Road the channel alignments are constrained by the Ridge development and the proposed extension of Oro Blanco Drive to Templeton Gap Road. The proposed alignment is modified to the south of Oro Blanco and into Barnes Road to provide the necessary radii of curvature at channel junctions and into the culverts at Oro Blanco and Barnes Road.

2. Park Vista Estates

The channel alignment is constrained by four major considerations.
1. The existing channel alignment in Village Seven.
2. Existing roads, which should remain in place and which will eventually be loaded with utilities (existing sewer and proposed water and gas - telephone and electric is overhead).
3. The existing alignment of the Box Culvert on Academy Boulevard.
4. Existing utilities on Siferd Boulevard.

In an effort to maintain as much of the existing subdivision as possible the alignment shown on plates 3 and 4 are recommended for both the open and closed alternatives. Some minor adjustments in alignment are possible, however it is readily apparent that right of way must be obtained in the outright purchase of 29 lots, either totally or in part. Some minor changes in gradient may be made above Siferd Boulevard, however the gradient is fixed at and below Siferd by an existing sewer which cannot be significantly relocated and the existing outfall point. The proposed gradient will require the relocation of the following utilities.

1. 36-inch ADP water line, approximately 5 feet below the surface.
2. 20-inch gas line feeding the north power plant, as well as a 10-inch domestic main and some 2-inch service lines.
3. The Norad telephone "A" cable.
4. 12-inch sewer at Lily Street.

The proposed design is preliminary since it is based on topography, but is considered adequate for cost estimating purposes. Any final design should be based on detailed field surveys.

D. Analysis of Existing Facilities

The following is the analysis of all existing facilities on the primary greenbelt, describing their ability to contain the runoff described in Section II B.

1. Park Vista Subdivision

The existing runoff creates a wide flood path through many of the lots and the existing culverts will not handle a minor design storm. Facilities shown on plate two are prescribed to update the development to existing City Criteria, provided street flow is contained by curb and gutter and pavement. Existing culverts across

Templeton Gap Road are updated in accordance with State Highway Criteria to pass the 100 year storm. A 60" RCP is required where a 36" CMP exists, and a 10' x 6' RCB Culvert is required where a 48" CMP exists.

2. The Ridge Subdivision

The "A" channel is sufficient through hydrograph point number 2, having a capacity of 867 CFS. Into hydrograph point number 4 the runoff will overtop the lining by 0.11 foot on tangent and 0.60 foot on the curve. The "B" channel to hydrograph point number 5 will barely contain the design flow of 1710 CFS, having a capacity of 1760 CFS. One foot should be added for tangent freeboard and two feet should be added on a 200 foot radius curve into Oro Blanco Drive.

3. Village Seven

The culvert on Barnes Road appears to be sufficiently sized, provided adequate design measures are taken with the incoming channel in the future.

The existing greenbelt is wide and shallow with an unpaved bottom and concrete sides. Concrete drop structures are provided at intervals to maintain an approximate one percent slope. Some considerable maintenance may be required to maintain the silt load below the design depths. Design velocities will be in the 15 to 17 fps range, creating a severely erosive condition in the bottom and below the drops. The upper half of the existing channels should contain the flows if well maintained. The lower half of the channels will overtop by 0.3 feet on tangents and 0.4 feet on curves.

The existing bridge at Hopeful Drive is sufficiently large and the channel into Park Vista Estates below the bridge would have 0.50 foot of freeboard, if 2.5 feet of silt were removed to the design depth.

4. Park Vista Estates

The County Road department has spent considerable time installing berms, raising roadways and grading the subdivision in recent months. This work was accomplished after the aerial photography for this study and is not shown on plates 3 and 4. The work will accommodate the nuisance water and runoff from minor storms and snowmelt experienced to date, however it will not be sufficient for the normal severe summer thunderstorms, and will be completely inadequate for the runoff for which facilities are proposed in this report. None of this work will significantly effect the proposed plans. The flood plain is shown on Plate 5.

5. Existing Academy Boulevard Culvert

The existing culvert has 3 cells of 13, 16 and 13 feet wide and an inside depth of 8 feet. It is 126 feet long on a slope of 1.4%. It was designed by the Division of Highways under inlet control criteria

for 3000 CFS plus an anticipated 500 CFS in a future storm sewer to enter into the side of the box, which is no longer valid. It will contain the anticipated 4250 CFS under inlet control with a free standing water surface to elevation 6418.5. This will create a substantial lake as shown on plates 3 and 4, however it will not submerge any of the traveled roadways or existing dwellings. When the proposed channel to the culvert is constructed as described in Section II E, the maximum depth of water in the culvert will be about 4.5 feet. An existing 8-inch gas line suspended near the crown will not be endangered in either case.

E. Proposed Structures

All proposed structures are considered tentative with respect to the shown size and exact location, however the flows are considered adequate for detailed design purposes. The general location, type of structure and routing of runoff must be followed as closely as possible.

1. Minor Structures

Minor structures are shown on the enclosed plates and require no detailed explanation.

2. Greenbelts - Upper Basin

The primary and secondary greenbelts shown on plate 2 may be summarized as follows:

Location	Size	Approx. Slope %	Design Flow -CFS-	Normal Depth -ft-	Velocity -fps-
E4	3'x3'	4.0%	153	1.7	20
HP4 North	5'x5'	2.2%	644	3.7	24
HP6 West	5'x5.5'	1.8%	656	4.3	17
HP7	7'x7'	2.3%	2520	1.0	33

3. Park Vista Estates

The street collection network assumes that all streets will eventually be paved and contained by curb and gutter, which will provide the most positive means of collection. The following are the design particulars of the various proposed major greenbelt structures.

a. Open Channel Alternative (see plate number 3)

Station	Approx. Slope	Design Flow -CFS-	Size BxD	Normal Depth -ft-	Velocity -fps-
1+08			29'x5'	4.5'	21.6
	10.4%	3265	Transition, Req'd L=39.6'		
2+00					
	1.3%	3265	10'x8'	6.9' *#1	28.2
6+30					
	1.3%	3770	10'x8.5'	7.4' *#2	29.3
14+65					
	1.7%	3770	10'x8'	6.9' *#3	32.4
21+05					
	1.7%	3770	Transition, Req'd L=25.0'		

Station	Approx. Slope	Design Flow -CFS-	Size BxD	Normal Depth -ft-	Velocity -fps-
21+35					
	0.9%	3770	db1 8'x8'RCB		
22+20					
	0.9%	3880	Transition, Req'd L=30.4'		
22+50.5					
	0.9%	3880	10'x9.5'	8.4' *#4	25.3
25+30					
	0.9%	4250	10'x10'	8.8' *#4	25.9
28+29					
	0.9%	4250	Transition, Req'd L=43.7'		
28+73					
	1.4%	4250	Exist RCB	4.5'	-----
29+99					

*Curve Superelevations:

Curve Number	Radius -ft-	Outside Depth -ft-	Outside Concrete Height -ft-
1	200'	8.33'	9.00'
2	200'	9.05'	10.00'
3	200'	8.33'	10.00'
4	200'	9.69'	11.00'

b. Closed Channel Alternatives (see plate number 4)

The closed channel alternative requires sufficiently large structures that, when compounded with the gradient restrictions, rule out the apparent feasibility of anything but reinforced concrete boxes. To avoid substantial structural problems, these must be double celled, requiring further detailed consideration near storm sewer inlets. The following is a preliminary sizing of the required units.

Station	Approx. Slope	Design Flow -CFS-	Size B xD	Velocity -fps-
1+08			29'x5'Open	21.6
	19%	3265	Transition, Req'd L=40.1'	
1+58				
	1.3%	3265	2-10'x8'	20.4
6+22				
	1.3%	3770	Transition, Req'd L=7.7'	
6+30				
	1.3%	3770	2-12'x8'	19.6
14+61				
	1.3%	3770	Transition, Req'd L=2.8'	
14+65				
	1.7%	3770	2-10'x8'	23.6
21+29				
	1.7%	4250	Transition, Req'd L=7.9'	
21+35				
	0.9%	4250	2-12'x9'	19.7
28+27				

Station	Approx. Slope	Design Flow -CFS-	Size BxD	Velocity -fps-
	0.9%	4250	Transition, Req'd L=45.8'	
28+73				
	1.4%	4250	Exist RCB	-----
29+99				

c. No Structures Alternative (see plate number 5)

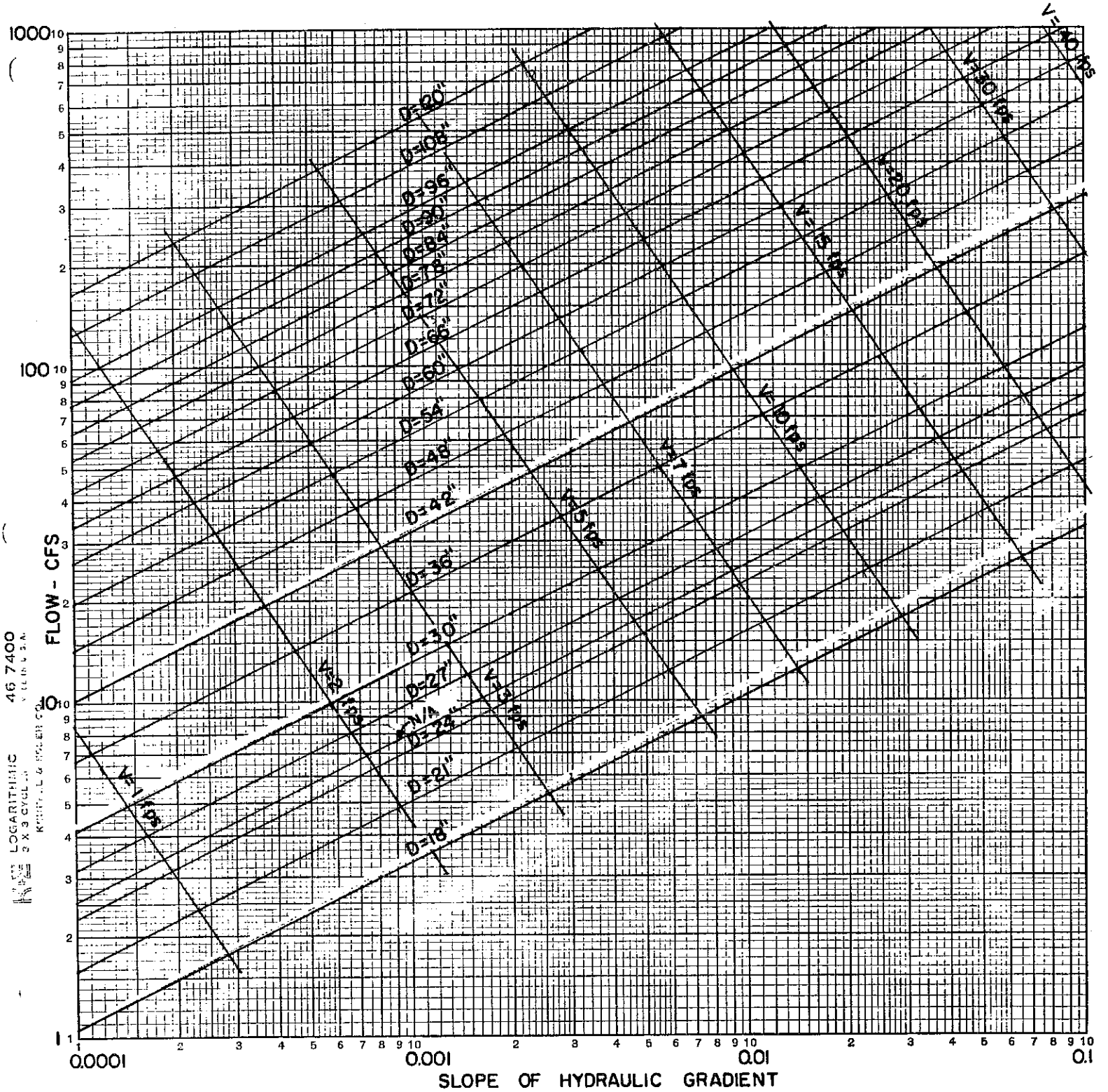
This plate shows the 100 year flood plain through Park Vista Estates if no structures to accommodate the runoff were installed. The flood plain represents the condition of the upper basin being fully developed as described in Section ID, page 5.

It may be seen that 81 lots are not usable due to the flooding hazard, which involves 3 existing dwellings, endangering a fourth, and one unoccupied garage structure.

The velocity of the runoff across the subdivision varies from 5 to 11 feet per second. The natural grassland should sustain this flow with little damage, however the existing unpaved roads will be severely eroded, resulting in a perpetual maintenance expense of considerable magnitude.

Not shown on plate 5 is the flood plain that would result along Templeton Gap Road if the road is not improved, nor the structures shown on plates 3 and 4 installed. This flood plain is nearly impossible to predict, however it is quite evident that culverts across driveways and roads south of Templeton Gap are insufficient. The existing dwellings along Templeton Gap will therefore sustain substantial damage, however the extent of this damage is impossible to predict.

The cost for this alternative presented in the next section is that associated with purchasing the lots and structures rendered unusable by the flood plain and leaving this part of the subdivision in its existing state.

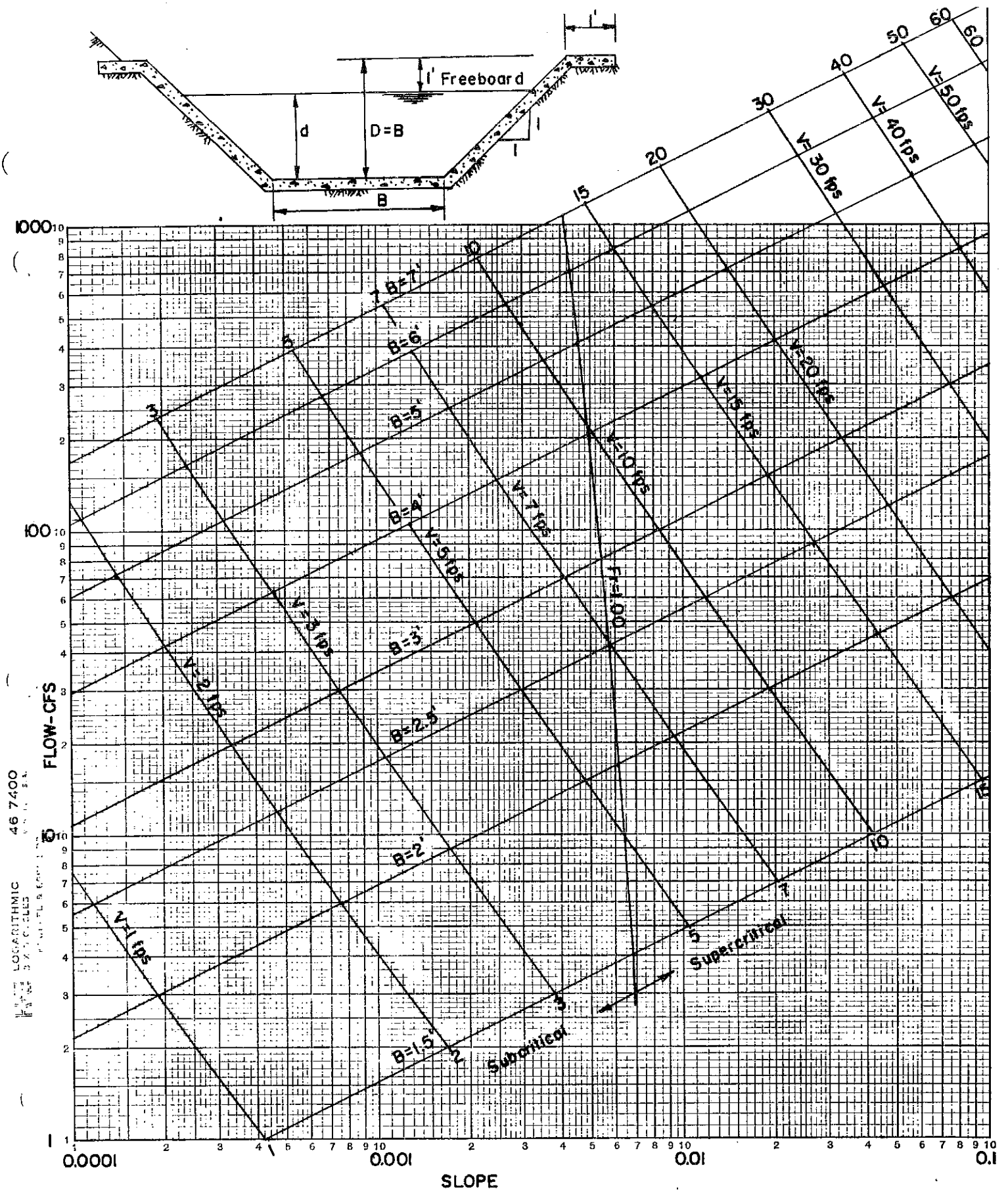


CAPACITY OF CONCRETE PIPE FLOWING FULL

$$Q = \frac{0.463}{n} D^{8/3} S^{1/2} \quad \text{WHERE} \quad n = 0.013$$

U.W.E.

Figure NO. 4



MOST ECONOMICAL CONCRETE DITCH SECTION

$$Q=\frac{1.486}{n} A R^{2/3} S^{1/2}$$ WHERE D=B Z=1 n=0.015 FREEBOARD = 1' d = D − 1

U.W.E

Figure No. 5

SECTION III

COST ESTIMATES
PARK VISTA ESTATES

A. Minor Storm Sewers

1. Interior Facilities - These are the collection facilities within the limits of the subdivision as shown on plates 3 and 4, common to both the open channel and closed alternatives.

Item	Quantity	Unit Cost	Cost
18-inch RCP	155 LF	$ 18.00	$ 2,790.00
21-inch RCP	219 LF	20.00	4,380.00
24-inch RCP	144 LF	22.00	3,168.00
27-inch RCP	78 LF	23.50	1,833.00
30-inch RCP	32 LF	25.00	800.00
36-inch RCP	25 LF	28.00	700.00
54-inch RCP	540 LF	40.00	21,600.00
4-foot Curb Inlet	2 ea	800.00	1,600.00
16-foot Curb Inlet	2 ea	1,900.00	3,800.00
22-foot Curb Inlet	4 ea	2,300.00	9,200.00
Subtotal			$49,871.00
Engineering and Contingencies (10%)			4,987.10
Total			$54,858.10

2. Templeton Gap Storm Sewer - These are the existing and proposed structures along Templeton Gap Road for which the City is participating in half the cost as described in Section I B.

Item	Quantity	Unit Cost	Cost
Added Facilities:			
18-inch RCP	60 LF	$ 18.00	$ 1,080.00
36-inch RCP	762 LF	28.00	21,336.00
42-inch RCP	844 LF	33.00	27,852.00
4-foot Curb Inlet	4 ea	800.00	3,200.00
16-foot Curb Inlet	1 ea	1,900.00	1,900.00
Subtotal			$55,368.00
10% Engineering and Contingencies			5,536.80
Total			$60,904.80
50% of Total			30,452.40
Previously Credited to Col-Terra Investments			19,306.72
Previously Credited to Calvary United Methodist Church			6,169.80
Total			$55,928.92

B. Primary Greenbelt

The costs associated with this are estimated through the limits of the subdivision only, plus that amount required to transition into the Village Seven channel.

Associated land costs are those estimated by Mr. Bob Baldridge, City Land Office. Utility relocations are as estimated by the respective departments in January of this year plus a 10% inflationary increase, except for the Lily Drive sewer relocation.

(1) Open Channel Alternative (see plate number 3)

Item	Quantity	Unit Cost	Cost
Channel Excavation	17,820 CY	$ 3.00	$ 53,460.00
Compacted Embankment	9,030 CY	2.00	18,060.00
Compacted Backfill	1,281 CY	4.00	5,124.00
Concrete Lining	9,127.6 SY	11.00	100,403.60
Structural Concrete	196.63 CY	200.00	39,326.00
Structural Steel	25,653 Lbs	0.50	12,826.50
Subtotal			$ 229,200.10
10% Engineering and Contingencies			22,920.01
Total			$ 252,120.11
Utility Relocations:			
Sewer			6,400.00
Water			47,380.00
Gas			45,260.00
Telephone			400.00
Land Aquisition:			
29 Lots @ $1650.00			47,850.00
Interior Structures (Section A1)			54,858.10
Templeton Gap Storm Sewer (Section A2)			55,928.92
Total Drainage Cost			$ 510,197.13

(2) Closed Channel Alternative (see plate number 4)

Item	Quantity	Unit Cost	Cost
Excavation	46,519 CY	$ 3.00	$ 139,557.00
Backfill	34,443 CY	4.00	137,772.00
Concrete	3,973.85 CY	200.00	794,770.00
Steel	601,603 Lbs	0.50	300,801.50
Subtotal			$1,372,900.50
10% Engineering and Contingencies			137,290.05
Total			$1,510,190.55
Utility Relocations:			
Sewer			6,400.00
Water			47,380.00
Gas			45,260.00
Telephone			400.00
Land Aquisition:			
29 Lots @ $1650.00			47,850.00
Interior Structures			54,858.10
Templeton Gap Storm Sewer			55,928.92
Total Drainage Cost			$1,768,267.57

(3) Alternative of No Structures (see plate number 5)

This is the cost of purchasing the lots and structures shown on plate number 5 which are rendered unusable by the 100 year Flood Plain. This area would then be left in its natural state, or could be used for parks, parking lots, agriculture, or other uses compatible with Flood Plain Zoning.

Item	Quantity	Unit Cost	Cost
Purchase of Lots	81 ea	$ 1,650.00	$133,650.00
Purchase of Dwellings	3 ea	30,000.00	90,000.00
Purchase of Structure	1 ea	10,000.00	10,000.00
Templeton Gap Storm Sewer	50 %	55,928.92	55,928.92
Subtotal			$289,578.92
Overhead Expenses @ 10%			28,957.89
Total Cost (Minimum)			$318,536.81

Total Area Affected (excluding streets): Approximately 14.05 acres
Equivalent Cost Per Acre: $22,670

Not included in the above cost:

(1) Maintenance of storm damage
(2) Other normal maintenance costs
(3) Increased expense of structures required below Park Vista Estates to accept a much wider flood plain, compared to previous alternatives.

The above costs are considered the minimum required. The following additional expenditures are recommended to alleviate substantial problems associated with this alternative.

Item	Quantity	Unit Cost	Cost
Purchase of Dwellings	4 ea	$30,000.00	$120,000.00
Major Utility Protection	LS	LS	10,000.00
Regrade Roadways	LS	LS	25,000.00
Road Closures & Cul-de-Sacs	6 ea	1,000.00	6,000.00
Park Development Costs	14.05 Ac	700.00	9,835.00
Subtotal			$170,835.00
Total Cost of this Alternative			$489,371.81

It should be noted that the use of this alternative would render Siferd Boulevard unusable, requiring the re-routing of substantial traffic from Village Seven now utilizing this street for access.

Equivalent Cost for 14.05 Acres: $34,831/Acre

C. Cost Summary

(1) Open Channel Alternative

Item	Cost
Drainage Structures	$ 229,200.10
Engineering and Contingencies	22,920.01
Utility Relocations	99,440.00
Property Aquisition	158,637.02
Total Drainage Cost	$ 510,197.13

(2) Closed Channel Alternative

Item	Cost
Drainage Structures	$1,372,900.50
Engineering and Contingencies	137,290.05
Utility Relocations	99,440.00
Property Aquisition	158,637.02
Total Drainage Cost	$1,768,267.57

(3) No Structure Alternative

Item	Cost
Property Aquisition	$ 289,578.92
Overhead	28,957.89
Total Cost (minimum)	$ 318,536.81
Added Expenditures recommended (see page 28)	170,835.00
Total Cost	$ 489,371.81

Alternative number one above for the open channel facilities is recommended for acceptance. This alternative will prove to be the most economical from a maintenance standpoint and will provide the most workable long range solution.

SECTION IV

A P P E N D I X

A. References

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a. Existing Ordinances, Rules, Regulations and Criteria.

b. North Shooks Run - Templeton Gap Drainage Study, United Western Engineers, September 10, 1963.

c. All approved Drainage Reports within the basin.

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d. Precipitation-Frequency Maps for Colorado, prepared by the U.S. Weather Bureau, Ocotber, 1967.

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3. USDI - Bureau of Reclamation:

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b. Design Standards Number 3, Canals and Related Structures.

c. Hydraulic and Excavation Tables, 1957.

4. USA - Corps of Engineers:

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5. Denver Regional Council of Governments, Drainage Criteria Manual, March, 1969.

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7. State of Colorado, Division of Highways, Roadway Design Manual, May, 1972.

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a. Planning Manual, Part 7, Design.

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12. King & Brater, Handbook of Hydraulics, McGraw-Hill, 1963.

13. Handbook of Steel Drainage and Highway Construction Products, American Iron and Steel Institute, 1971.

14. Handbook of Concrete Culvert Pipe Hydraulics, Portland Cement Association, 1964.

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