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COLORADO SPRINGS, COLORADO

MARCH, 1987 ADDENDUM
ENGINEERING STUDY AND REVISION OF
THE NORTH SHOOK'S RUN - TEMPLETON GAP DRAINAGE BASIN
COLORADO SPRINGS, COLORADO

Prepared for
Conreal Growth Corporation

Prepared by

Claycomb Engineering Assoc., Inc.
1425 N. Union Blvd., Suite 201
Colorado Springs, Colorado 80909

March 1987
(Revised ~~April~~ 17, 1987)

Job No. 3034.002

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STATEMENTS PAGE

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ENGINEERING STUDY AND REVISION OF
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ENGINEER'S STATEMENT:

The attached Addendum to the Engineering Study and Revision of the North Shook's Run - Templeton Gap Drainage Basin was prepared under my direct supervision and are correct to the best of my knowledge and belief. Said Addendum has been prepared according to the criteria established by the City for drainage reports and said Addendum modifies the master plan of the drainage basin. I accept responsibility for any liability caused by the negligent acts, errors or omissions on my part in preparing this report.

ELMER L. CLAYCOMB
REGISTERED
9549
Seal
PROFESSIONAL ENGINEER
STATE OF COLORADO

Elmer L. Claycomb

Signature and PE # 9549

DEVELOPER'S STATEMENT

The Developer and/or his representative has read and will comply with all the requirements specified in this Addendum.

Clint W Wilson Vice Pres.

Signature

Filed in accordance with Section 15-3-906 of the Code of the City of Colorado Springs, 1980, as amended.

4/30/87

City Engineer Date

MARCH, 1987 ADDENDUM

ENGINEERING STUDY AND REVISION OF

THE NORTH SHOOK'S RUN - TEMPLETON GAP DRAINAGE BASIN

COLORADO SPRINGS, COLORADO

I. INTRODUCTION

This document is an Addendum to the Engineering Study and Revision of the North Shook's Run - Templeton Gap Drainage Basin, Colorado Springs, Colorado, dated September, 1977. The referenced study was prepared by Lincoln DeVore Testing Laboratory under contract to the City of Colorado Springs and the Colorado Springs Drainage Board. The Addendum has been prepared by Claycomb Engineering Associates, Inc. under contract to Conreal Growth, owners of a parcel of land northeast of the corner of North Nevada Avenue and Austin Bluffs Parkway.

The Addendum has been prepared to provide more specific detail regarding Line Point 59 in sub basin G2 as delineated in the referenced study. Specifically, the Addendum deals with the existing ponding area located east of the old Santa Fe Railroad embankment and the structure extending from the old railroad embankment to Monument Creek. A brief analysis was also made of Line Point 60 which is located just to the south of Line Point 59.

This Addendum will discuss the effect of the ponding area on the 100-year peak discharge at and downstream of Line Point 59. Due to the attenuation effect of the ponding area at Line Point 59, the 100-year discharge downstream from Line Point 59 will be significantly reduced. Any proposed facility downstream of Line Point 59 designed for the unattenuated discharge can be decreased in size or omitted if existing facilities are adequate to convey the attenuated discharge. The inundation within the ponding area will have no significant effect on proposed facilities upstream of the pond. Future channel improvements upstream of the detention area could be constructed as proposed by the LDV base report.

The City of Colorado Springs Engineering Department has reviewed this document and formally adopted it as an Addendum to the base study.

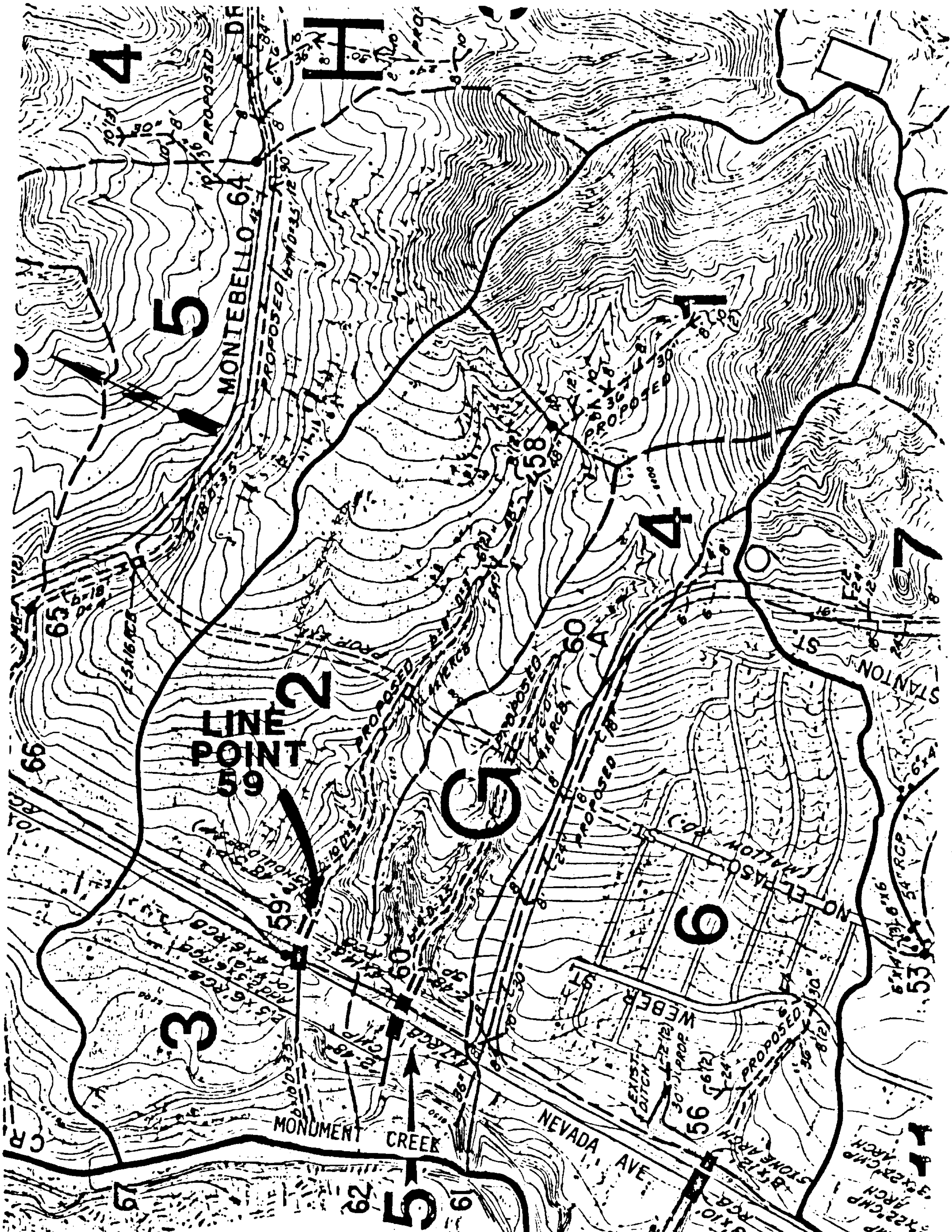
Lincoln DeVore (LDV) in 1977 determined the 5- and 100-year peak discharges within the study area at designated points. The designated points are either the mouths of a sub basin or at major points within a channel, also referred to by LDV as Line

Points. For each Line Point, the Master Drainage Study included in addition to peak discharges, hydrologic data such as basin area, SCS soil curve numbers, and time of concentration. Also, qualitative descriptions are given for major drainage structures. No hydraulic data or analysis was presented for major drainage structures, such as capacity, depth of flow, velocity and water surface elevations. Since the 5- and 100-year peak flows were determined by LDV for fully developed conditions, and therefore believed to be applicable in this study, the same hydrologic parameters and the peak discharges were used in this Addendum.

Line Point 59, is located within "Sub Basin G2" on an unnamed intermittent stream north of Austin Bluffs Parkway, as shown in Figure 1. The intermittent stream, referred to in this report as the north drainageway, flows in a westerly direction and drains into Monument Creek. Line Point 59, with a drainage area of 212 acres, is located at the inlet of a culvert system consisting of two 48-inch CIP (Cast Iron Pipe) culverts underneath the abandoned Santa Fe Railroad embankment; a 4.3-foot by 6-foot box culvert underneath North Nevada Avenue; and a 2.5-foot by 6-foot extension underneath landfill west of North Nevada Avenue. LDV reported that the culvert underneath the landfill consisted of a 2.5-foot by 6-foot box culvert. However, a recent field survey confirmed the 4-foot by 6-foot dimensions. Furthermore, LDV recommended that an additional 2.5-foot by 6-foot box culvert should be placed parallel to the culvert underneath the landfill. The embankment of the abandoned railroad grade crosses a steep-walled, well-vegetated gully forming a relatively large ponding area drained only by the described culvert system.

Due to the hydraulics of the existing culvert system, a large headwater upstream of the railroad embankment would be required to pass major runoff events directly through the system. However, the ponding area upstream of the inlets and east of the embankment behaves as a detention pond, thereby attenuating peak flood discharges from the fully developed basin. It is the intent of this Addendum to demonstrate that the attenuation effect of the ponding area eliminates the necessity of upgrading the culvert crossing underneath North Nevada.

A proposed retail development of the abandoned Santa Fe Railroad Right-of-Way would cause minor modification of the culvert system at Line Point 59. The effect of this existing detention was not analyzed by the original Lincoln Devore study. A detailed hydraulic analysis was therefore performed to determine the capacity of the culvert system, the attenuation effect of the natural depression, and the area of inundation. This Addendum conveys the results of the detailed analysis.



NOTE:
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AND REVISION OF N. SHOOKS RUN -
TEMPLETON GAP BY LINCOLN DEVORE,
1977

FIGURE 1
OLD SANTE FE CENTER
LINE POINT 59 LOCATION MAP

3034.005

II. DESCRIPTION OF EXISTING DRAINAGE CONDITION

Lincoln DeVore (1977) determined peak discharges for the 5- and 100-year events at the inlets of the north and southern culverts as follows:

Line Point	Total Drainage Area	5-year Event	100-Year Event
59	212 acres	277 cfs	677 cfs
60	59 acres	71 cfs	175 cfs

Apparently, Lincoln DeVore determined the peak discharges based on developed conditions, since they used a SCS curve number of 83, which, approximately corresponds to residential areas with 65 percent imperviousness for sandy soils with hydrologic soil group "B" classification (TR-55 "Urban Hydrology for Small Watersheds" Table 2-2 SCS, 1975)

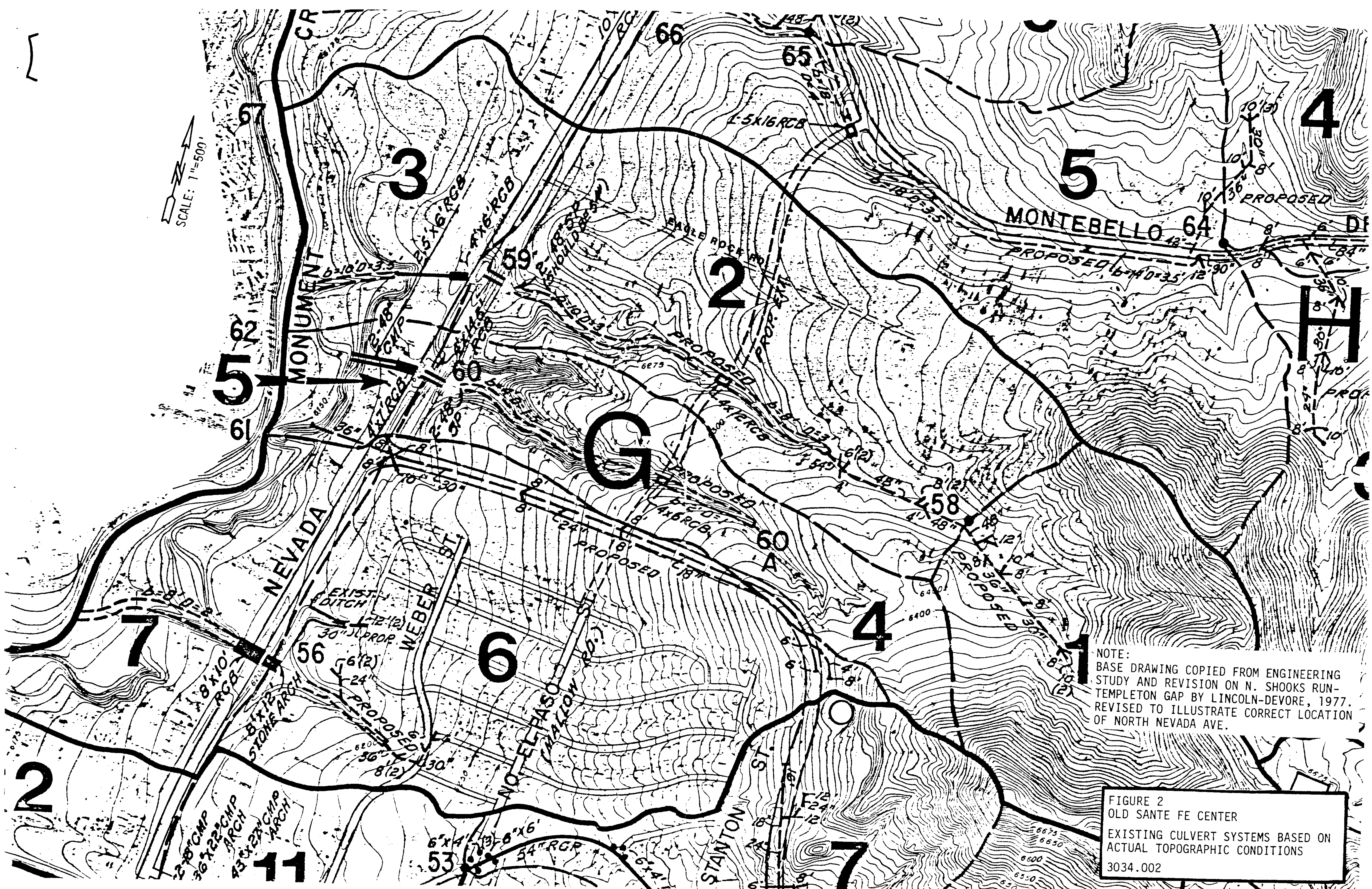
Presently, any on-site runoff flows either towards Eagle Rock Road, North Nevada Avenue, or the north and south culvert systems. The runoff along Eagle Rock Road is intercepted by the roadside ditches along North Nevada Avenue and conveyed to the northern culvert system or a 2-foot by 3-foot box culvert beneath North Nevada Avenue.

The culvert system as illustrated on the Lincoln DeVore Drainage Plan (Figure 1) does not conform to the topography on the same map. The actual location of the culvert system has been more correctly illustrated on Figure 2. Regarding the adequacy of the culvert system, LDV stated the following:

> "The boxes under Nevada Avenue are sufficiently sized, as are the arches and culverts beneath the railroad. For the most part, no major problems will be found in this area until the water attempts to enter and pass through these private culverts west of Nevada. These culverts should definitely be enlarged, both for the safety of the commercial buildings involved and for the safety of Nevada Avenue and the railroad."

The report did not state for which flood event the structures were adequate and how much headwater was required at the inlet of the two 48 inch CIP culverts.

A conceptual schematic of the existing culvert system, together with the proposed modifications, is shown in Figure 3. The culverts drain from the east towards the west. As explained earlier, the two 48-inch CIP culverts convey runoff underneath



NOTE:
BASE DRAWING COPIED FROM ENGINEERING STUDY AND REVISION ON N. SHOOKS RUN-TEMPLETON GAP BY LINCOLN-DEVORE, 1977. REVISED TO ILLUSTRATE CORRECT LOCATION OF NORTH NEVADA AVE.

FIGURE 2
OLD SANTE FE CENTER

EXISTING CULVERT SYSTEMS BASED ON ACTUAL TOPOGRAPHIC CONDITIONS

3034.002

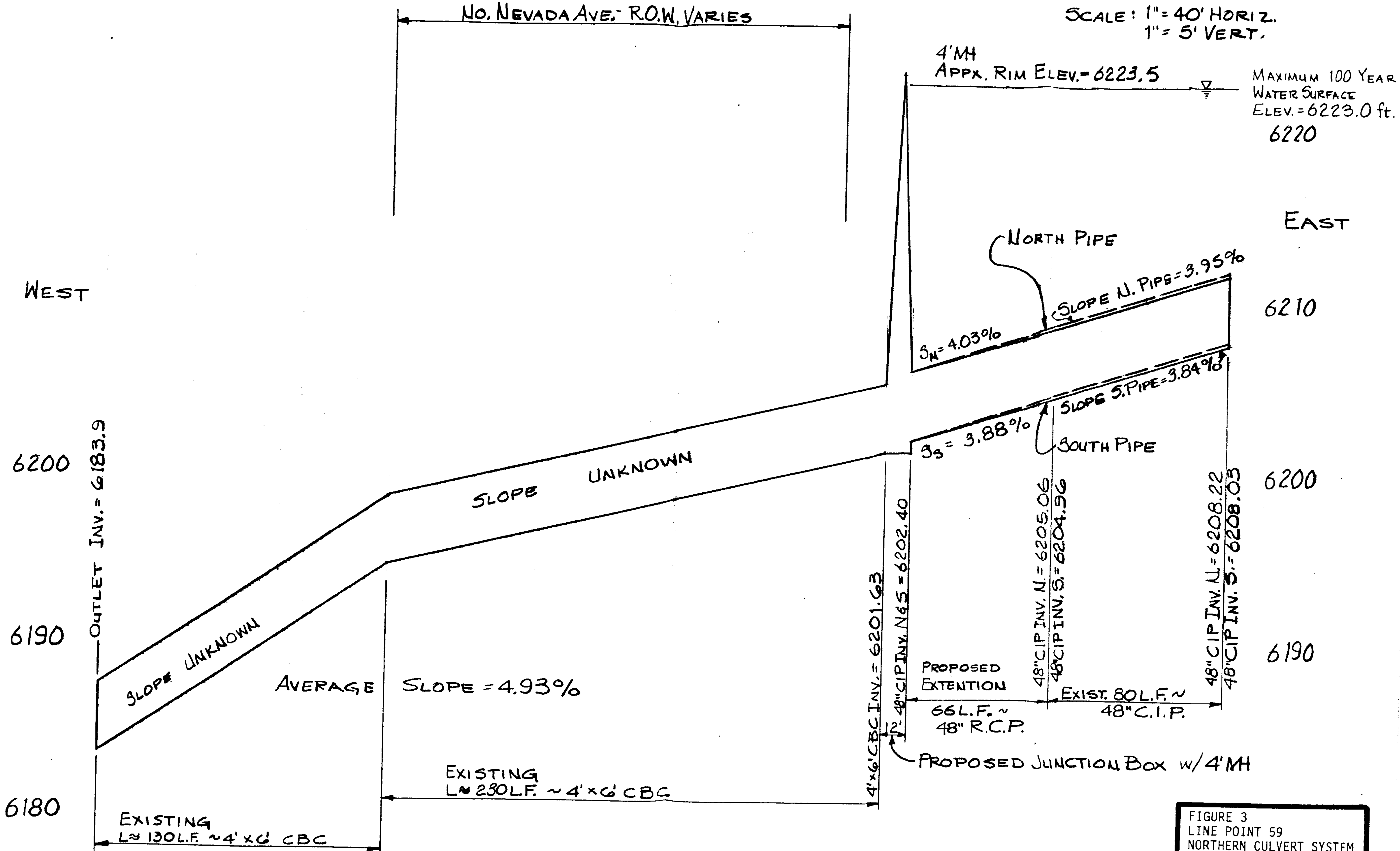


FIGURE 3
LINE POINT 59
NORTHERN CULVERT SYSTEM
CROSS SECTIONAL VIEW

OLD SANTE FE- -3034.002

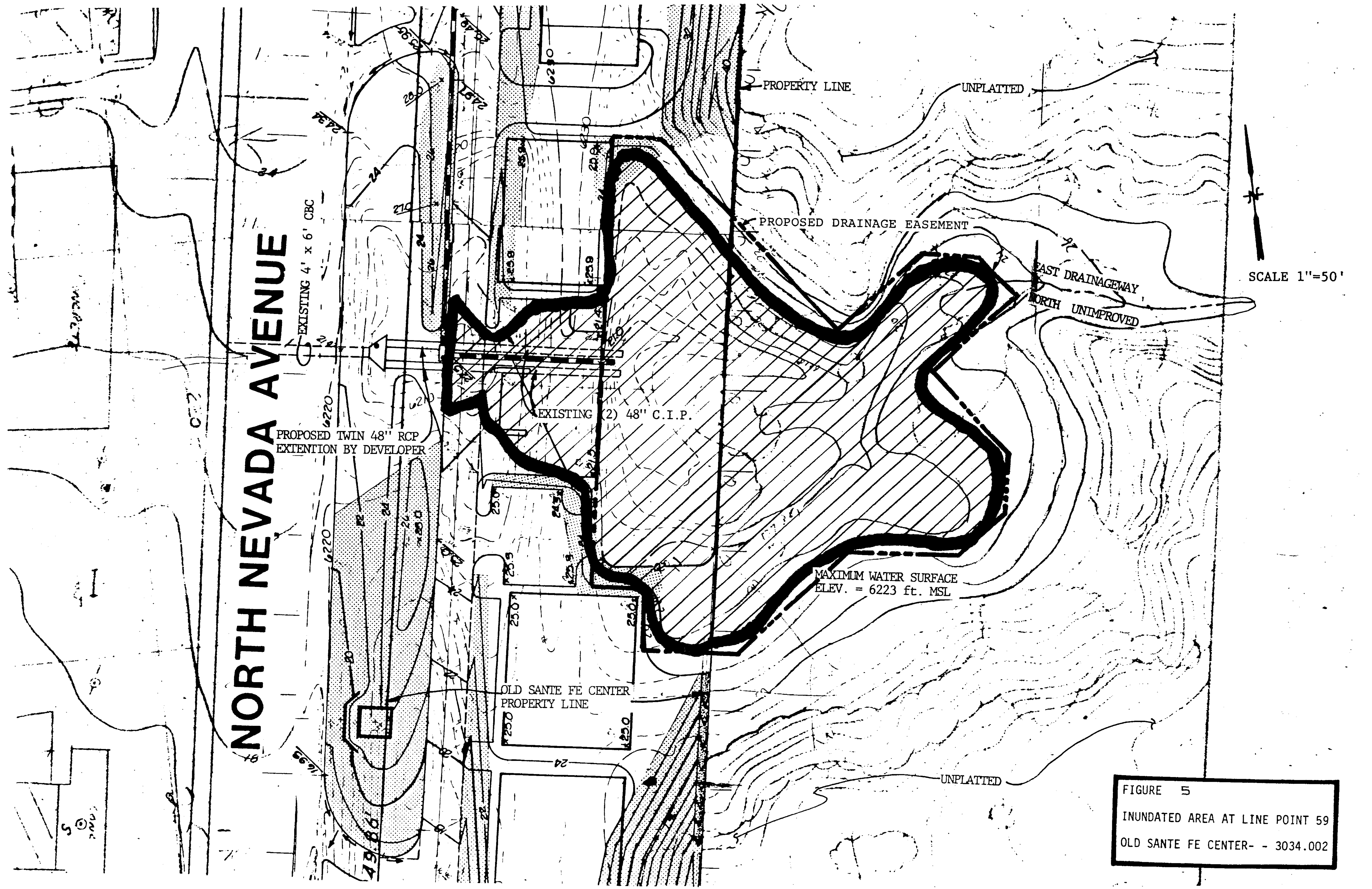


FIGURE 5

INUNDATED AREA AT LINE POINT 59

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the railroad embankment and release the runoff in a depression between the railroad embankment and North Nevada Avenue. Thereafter, a 4.3-foot by 6-foot box culvert conveys runoff underneath North Nevada. On the west side of Nevada a 4-foot by 6-foot box culvert, which is connected to the highway box culvert, conveys runoff underneath fill placed for commercial buildings. The runoff discharges into a steep side canyon of Monument Creek.

The proposed development of the previous Santa Fe Right-of-Way would cause only minor modifications to the culvert system. The modifications would consist of extending the existing 48-inch CIP culverts towards the 4.3-foot by 6-foot box culvert and constructing a junction box with a manhole between the two culverts.

III. HYDRAULIC ANALYSIS

The capacity of the culvert system, required headwater and area of inundation was determined for the 100-year storm event. In the hydraulic analysis of the north drainage culvert system, the following assumptions were used:

1) Negligible tailwater effects exist at the outlet on the western end of the fill since the culvert discharges into a steep tributary canyon of Monument Creek.

2) Approach velocity at the east entrance and within the junction box is negligible. This is a conservative assumption since an approach velocity reduces the required head for inlet control at the junction box and east entrance.

The hydraulic analysis was performed using the Bureau of Public Roads "Hydraulic Charts for the Selection of Highway Culverts" and "Capacity Charts for the Hydraulic Design of Highway Culverts". A rating table was first developed for inlet control at the entrance of the two 48-inch CIP culverts. Second, a rating table was developed for inlet control at the entrance of the 4.3 foot by 6 foot box culvert which set the required depth for each discharge inside the junction box. Third, based on the depth of water within the junction box, or outlet control, a rating table was developed of the required headwater at the entrance of the two 48-inch CIP culverts. In outlet control, the required headwater is a function of the water depth inside of junction box, elevation drop of the culvert, friction losses and entrance losses.

A rating table of water surface elevation versus discharges was determined for the eastern inlet of the two 48-inch CIP culverts.

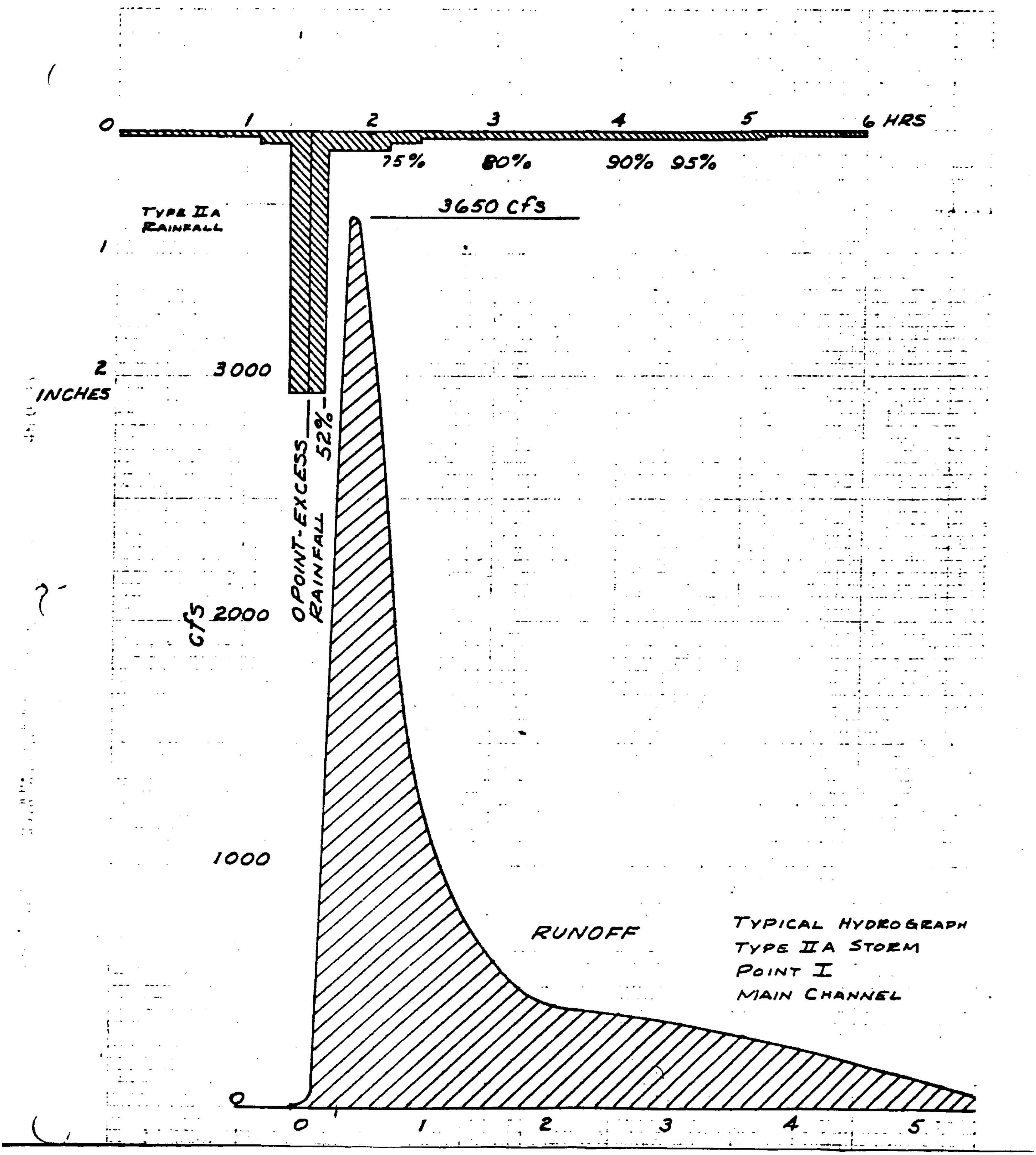
The invert elevations of the two CIP culverts are 6208.3 and 6208.0 feet. Inlet conditions at the two 48-inch CIP culverts control for water surface elevations up to 6216 feet. For water surface elevations greater than 6216 feet, inlet conditions within the junction box control.

In the Master Drainage Study for the area, Lincoln Devore determined a typical hydrograph shape as shown in Figure 4. Based on the typical hydrograph shape, the peak discharge of 677 cfs and time to peak, an inflow hydrograph was determined. The time to peak, which is a function of the time of concentration, was determined using procedures outlined in the National Engineering Handbook No. 4 (SCS 1972). The time of concentration, using a nomograph within the Colorado Springs Drainage Criteria, is a function of the hydraulic length and basin relief. The inflow hydrograph is determined based on the ratio of peak discharge and time to peak at Line Point 59 to that of the typical hydrograph. Based on the topography of the depression and some fill of the development within the pond, a stage storage relationship was determined. The inflow hydrograph was subsequently routed through the detention pond, using the U.S. Army Corps of Engineers HEC-1 computer model.

Using the maximum 100-year discharge released by the detention pond, a detailed hydraulic analysis was performed from the outlet of the system through the 4.3 foot by 6 .0 foot box culverts. This analysis, using Bernoulli's equation, was performed to verify that inlet conditions within the junction box do control the culvert system. The head within the junction box required to convey the maximum discharge and accommodate friction and entrance losses, was compared to the head required for inlet control. Detailed hydraulic calculations and HEC-1 computer printouts are included within the Appendices.

IV. RESULTS AND CONCLUSIONS

The routing calculations show that the maximum discharge of the 100-year event through the culverts equals 398 cfs., which corresponds to a maximum water surface elevation of 6223 feet, a detention volume of 4 Acre Feet, and a ponding area of 1.1 acres. The inundated area includes a portion of the parking lot. Hydraulic analysis of the culvert system west of the junction box, using a discharge of 398 cfs, verified that inlet conditions at the junction box does control. Due to the attenuation effects of the pond, the 4-foot by 6.0-foot box culvert is adequate to convey the 100-year storm runoff and a 2.5-foot by 6-foot parallel box as recommended by LDV is not necessary. The 4.0-foot by 6.0-foot box culvert can convey the routed maximum discharge of 398 cfs without causing overflow over the road and



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1977

FIGURE 4
OLD SANTE FE CENTER
TYPICAL HYDROGRAPH FOR
N. SHOOKS RUN- TEMPLETON GAP AREA
3034.005

the railway embankment. The inundated area with the corrected culvert location is shown in Figure 5. The natural depression just upstream of the culvert system would inundate whenever a major storm event occurred and should be considered, up to elevation 6223, an official 100-year flood plain. A drainage easement of the pond area (as shown on Figure 5) is required to prevent future encroachment of the ponding area. Even though the maximum depth of inundation is greater than ten feet, any railroad or highway embankment is excluded from the State Engineer's review according to rule 15 of the Rules and Regulations for Dam Safety and Dam Construction.

V. BASIN G4

A hydraulic analysis was completed on the existing culvert system at design point 60. The analysis revealed that the system will pass the 100-year flow without allowing for the attenuation effect of the existing ponding area at design point 60. Therefore, no change to the LDV, 1977 base study is required. The hydraulic calculations are included in the Appendix of this report.

VI. BASIN FEES

In the base study (sheet 2 of 2G, Major Bridge Inventory) LDV stated a cost of $38,000 would be required to construct the additional 2.5-foot by 6-foot box culvert. The drainage basin fees as set forth in October 26, 1977 equaled $1,295 per acre, while the bridge fees equaled $15 per acre. The 1987 drainage basin fees equal $2,686 per acre, or 2.07 times as much as the 1977 drainage basin fees. For bridge fees, the ratio equals 1.933. The additional 2.5-foot by 6-foot box culvert, which is considered a drainage facility, would now cost, based on the ration of 2.07, $78,660.

Eagle Rock, which is also located in Basin G-2, intercepts runoff from 36.6 acres. The base report considers this area tributary to Line Point 59. However, Eagle Rock Road conveys the runoff onto North Nevada Avenue, a State highway, which is against State Highway criteria. It is therefore necessary to route the runoff back to the detention area upstream of Line Point 59 through a "public" storm sewer system which would include several inlets. Since the additional 2.5-foot by 6-foot box culvert is not necessary, the amount allocated for the construction of the box culvert, a portion or its entirety, could be used to construct the storm sewer system. The cost of the storm sewer with inlets has an estimated cost of $70,824, which is $7,836 less than the cost of $78,660 required to construct the box culvert. Since the drainage fees as established by LDV were based on 2,949 acres the drainage basin fees should be reduced $2.66 per acre. This magnitude of change is so small (0.1%) that no change in the basin fee is appropriate.

APPENDIX

Addendum - Hydraulic Calculations

I Introduction

Two major drainages conveys runoff into the site. The drainages are within the master drainage study entitled: "Engineering Study & Revision of the North Shook's Run-Templeton Gap Drainage Basin, Colorado Springs" by Lincoln DeVore in 1977. The two drainage basins are refered to as the "North Basin and "South Basin in this study. Lincoln DeVore (LDV) determined the 5 and 100 year peak discharges and other hydrologic parameters as shown below:

Basin	LDV designation	Basin Area acres	5 year Flow cfs	100 Year Flow cfs
North	G2	212	277	677
South	G4	55	71	175

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SCALE revised 2-12-87

Since for the North Basin $Q_{100} > 500$ cfs, analyze for the 100 year event. The South Basin should be analyzed for the 5 year event.

From the North basin, runoff is drained underneath the abandoned Sante Fe embankment through a pair of 48" C.I.P culverts. Immediately upstream of the two 48" C.I.P culverts, there is a considerable amount of potential storage or detention. On the west side of the Sante Fe embankment, runoff exits the two 48" culvert into a depression which is drained by a 4.3' x 6' Concrete Box Culvert which drains underneath N. Nevada Av and commercial land fill, and daylights into a steep side canyon of Monument creek.

The south basin also consist of two 48" C.I.P culverts underneath the Sante Fe embankment, a 4' x 6' CBC underneath N. Nevada, but is connected to (according to LDV) a pair of 48" CMP placed underneath fill.

The following assumptions were made for the hydraulic analysis.

1. Culverts underneath the abandoned Sante Fe embankment & N. Nevada will be cleaned from silt.
2. Since the culvert of the "North Basin" discharges into a steep side canyon of Monument Creek, assume that no significant tailwater effect will exist.
3) For developed conditions, junction boxes will be build, with manholes, assume that the velocity head is negligible for inlet control.

Hydraulic analysis consists of following steps

1) Rating Table inlet control @ inlet two 48" C.I.P.
2) Rating Table inlet control @ inlet of 4.3'x6' CBC
3) Outlet control based on inlet control of 4.3x6' CBC
4) Rating table of discharge-elevation at inlet of two 48" C.I.P. based on either inlet control or outlet control.
5) Reservoir routing & peak discharge determination
6) Calculation of hydraulic grade line through culvert system

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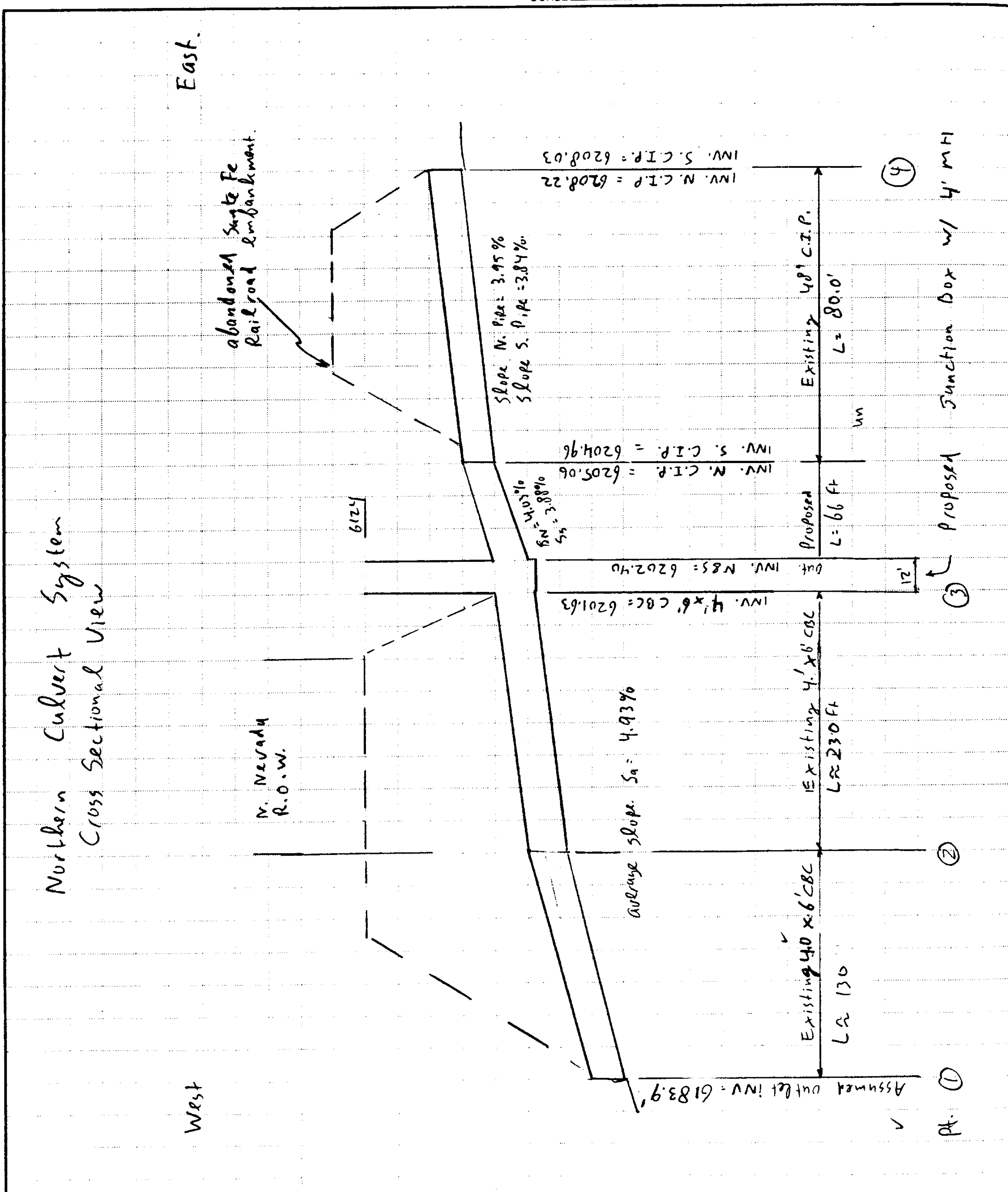

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North Drainage - 2-48" C.I.P.

Check Inlet Control Conditions

W.S. Elev. ft M.S.L	$HW_{N. Pipe}$ ft	$HW_{S. Pipe}$ ft	HW/D_N ft/ft	HW/D_S ft/ft	Q_N cfs	Q_S cfs	Q_T cfs
6210.5	2.28	2.47	0.57	0.62	29	35	64
6212	3.78	3.97	0.95	0.99	68	71	139
6214	5.78	5.97	1.45	1.49	105	107	212
6216	7.78	7.97	1.95	1.99	140	142	282
6218	9.78	9.97	2.45	2.49	170	170	340
6220	11.78	11.97	2.95	2.99	190	190	380
6222	13.78	13.97	3.45	3.49	205	205	410
6224	15.78	15.97	3.95	3.99	215	215	430
6226	17.78	17.97	4.45	4.49	222	222	444
6228	19.78	19.97	4.95	4.99	230	230	460

assume: proposed headwall with square edge conditions

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North Drainage 4.3 x 6.0 Box Culvert

assuming inlet control within Junction box

HW = depth + velocity head $\left(\frac{V^2}{2g}\right)$

assume that due to sudden expansion in junction box, $\frac{V^2}{2g} \approx 0$ 2)

Q_T	$\frac{Q_T}{span}$	HW/D 1)	HW	
cfs	cfs/ft	ft/ft	ft	
64	10.7 ✓	0.53 ✓	~~2.12~~	2.28
139	23.2	0.91	3.91	
212	35.3	1.31	5.63	
282	47.0	1.70	7.31	
340	56.7 ✓	2.2 ✓	9.46 ✓	
380	63.3	2.6	11.18	
410	68.3	2.9	12.47	
430	71.7	3.2	13.76	
444	74.0	3.5	15.05	
466	76.7	3.8	16.34	
677	113	6.8	29.24	

1) Wingwall between 30° to 75° entrance conditions; scale (1)

2) assume that velocity head is negligible, most conservative condition, since the velocity head would reduce required water depth for inlet control

North Drainage - 2 - 48" ϕ Cast Iron Pipe, Existing

Assuming Outlet Control, calculate HW @ inlet of 2-48" C.I.P. culverts

$HW_N = H + h_o - S_{N_1}L_{N_1} - S_{N_2}L_{N_2} + 6208.22$

$HW_S = H + h_o - S_{S_1}L_{S_1} - S_{S_2}L_{S_2} + 6208.03$

where H = head in feet from reference #1

h_o = tailwater depth assuming inlet control into 4.3x6' Box culvert, ie depth inside junction box

For h_o less than 48" ϕ pipe

$h_o = (d_c + 4ft)/2$ or h_o whichever is greater

L_{N_1}, L_{S_1} = 66 ft, proposed 48" ϕ connection

S_{N_1}, S_{S_1} = 3.88 %

L_{N_2}, L_{S_2} = 80 ft, existing 48" ϕ cast iron pipe

S_{N_2} = 3.95%

S_{S_2} = 3.84 %

Total L = 146 ft

K_e = 0.5

North pipe controls due to greater inlet elevation.

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SCALE

North Drainage - 2-48" existing C.I.P

for Outlet conditions E.

$S_o L_N = 5.82$

Q_N cfs	Q_S cfs	H_N ft	H_S ft	h_o [1] ft	HW_N ft	Q_T cfs
29	35	.1	.1	2.12 (2.26)	6204	64
68	71	1.0	1.0	3.91	6207	139
105	107	2.3	2.3	5.63	6210.3	212
140	142	4.4	4.4	7.31	6214.1	282
170	170	6.4	6.4	9.46	6218.3	340
190	190	7.7	7.7	11.18	6221.3	380
205	205	9.1	9.1	12.47	6224	410
215	215	10.0	10.0	13.76	6226	430
222	222	10.6	10.6	15.05	6228.1	444
230	230	12.0	12.0	16.34	6230.7	460
339	339	22	22	29.24	6253.6	678

Inlet control per table on page 5

Outlet Control This table

[1] h_o for inlet condition at 4x6 box culvert with no allowance for velocity head

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North Drainage Pond
Stage, Storage, Discharge Capacity Relationship

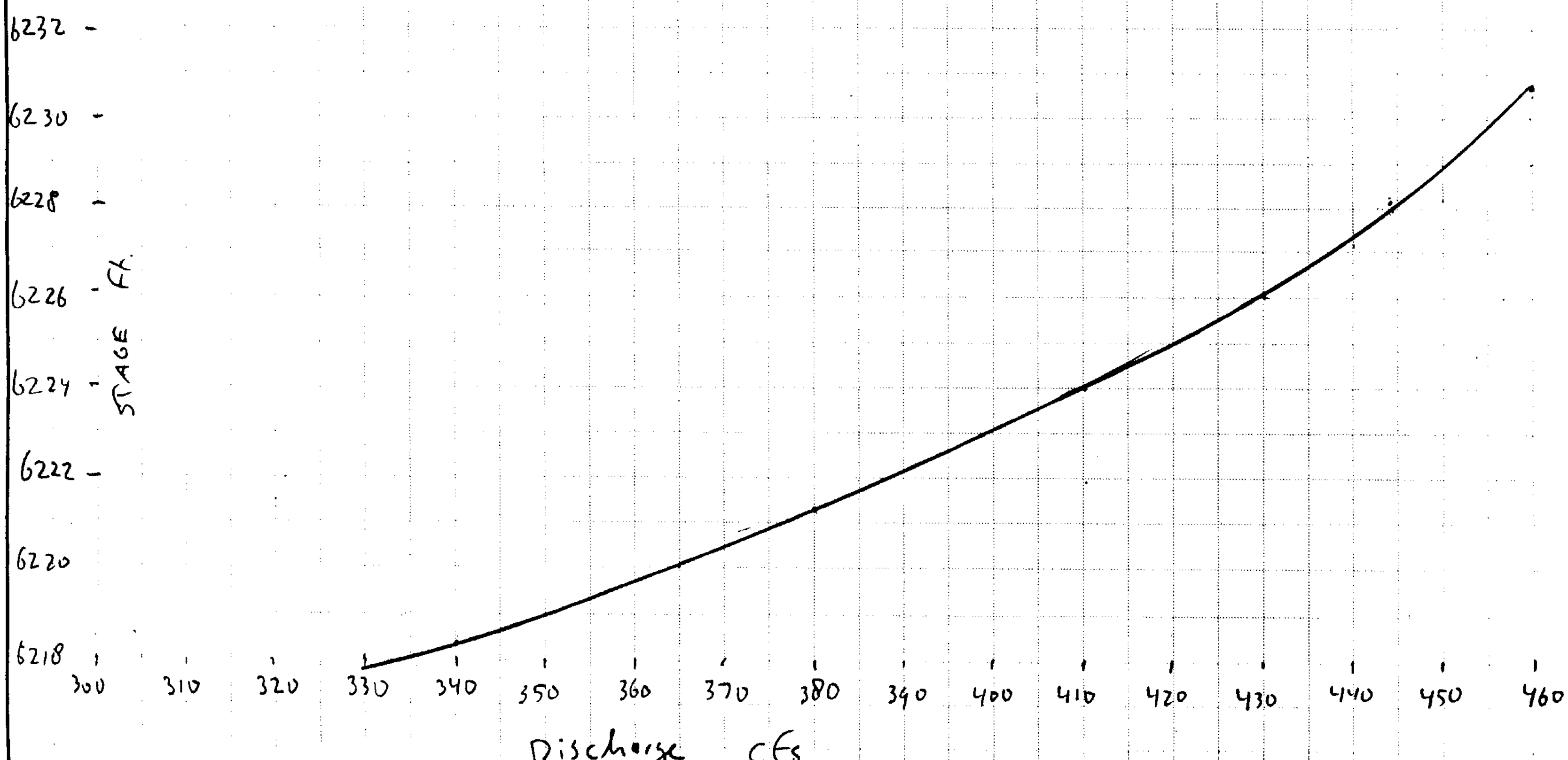
Elevation ft msl	Area acres	Avg Area acres	Δ d ft	Δ Volume ac-ft	Σ Volume ac ft	Q * cfs
6212	0.02	0.01	4	0.04	0.04	139
6214	0.09	0.055	2	0.11	0.15	212
6216	0.23	0.16	2	0.32	0.47	282
6218	0.41	0.32	2	0.64	1.10	335
6220	0.57	0.49	2	0.98	2.09	365
6222	0.83	0.70	2	1.40	3.49	388
6224	1.26	1.045	2	2.09	5.58	410
6226	1.57	1.415	2	2.83	8.41	430
6228	1.92	1.745	2	3.49	11.90	443

* From hydraulic calculations

from elevations 6212 to 6216, inlet controls,

" " 6218 to 6228 outlet, of the 2-48" C.I.P. controls.

Stage Discharge at Inlet of the Two 48" C.I.P. - North Drainage



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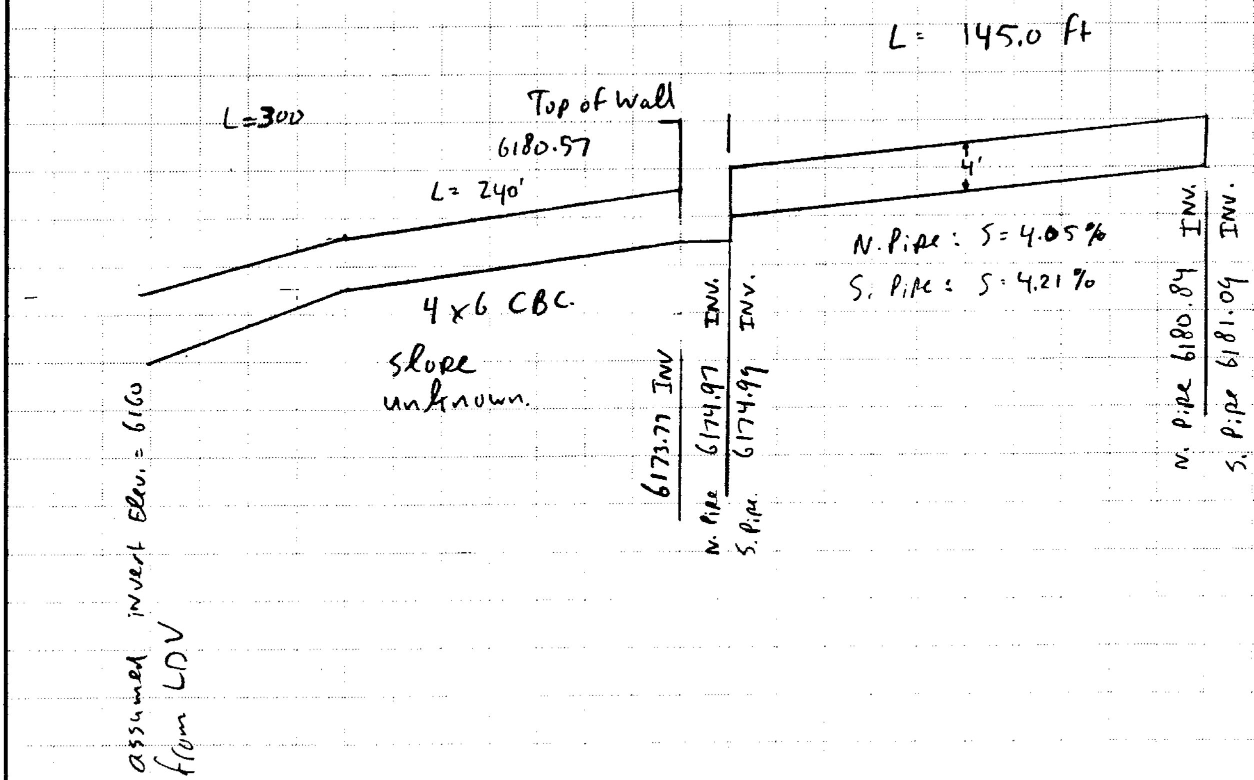
JOB Old Santa Fe 3034.005
SHEET NO. 4 OF 15
CALCULATED BY PGD DATE 12-1-86
CHECKED BY Ifr DATE 3/2/87
SCALE

B. South Drainage

Since the 100 year flow event is less than 500 cfs analyse for 5 year flow event.

$Q_5 = 71$ cfs.



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JOB Old Santa Fe 3034.005
SHEET NO. 12 OF 15
CALCULATED BY PGD DATE 12-1-86
CHECKED BY [illegible] DATE 3/2/87
SCALE

South Drainage - 2 48" C.I.P.

Assuming Inlet Control Conditions

Invert N. Pipe: 6180.84 , S Pipe: 6181.09

W.S. Elev. ft msl	$HW_{N. Pipe}$ ft	$HW_{S. Pipe}$ ft	HW/D_N ft/ft	HW/D_S ft/ft	Q_N cfs	Q_S cfs	Q_T cfs
6183.59	2.75	2.50	0.69	0.63	42	37	79
6186	5.16	4.91	1.29	1.23	98	92	190
6188	7.16	6.91	1.79	1.73	130	125	255
6190	9.16	8.91	2.29	2.23	160	155	315
6192	11.16	10.91	2.79	2.73	180	178	358

Inlet Control 4' x 6' Box Culvert

Invert E = 6173.77

Q_T cfs	Q/SPAN cfs/ft	HW/Rise ft/ft	HW ft	W.S. Elev ft msl	
79	13.2	0.75	3.0	6176.77	← for Q_5, HW< rise = 4 ft
190	31.2	1.35	5.40	6179.17	no outlet control on
255	42.5	1.90	7.60	6181.37	48" C.I.P.

1) use scale 2) wing wall flare between 90° and 15°

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North Drainage Inflow Hydrograph

Assume that the inflow hydrograph approximates the SCS Dimensionless Unit Hydrograph which is based on Q/Q_{pk} & t/t_{pk}

From Lincoln Devore N. Shook Run & Templeton Gap Drainage Study 1977.

At line point 59 (N. Drainage outlet on Nevada Av.)

Basin Area: 212 acres or 0.331 square miles

Time of Concentration T_c: 0.168 hrs. (assumed unit)

Q_{pk}: 677 cfs

From S.C.S. empirical relationships NEH-4 Chapter 16 1972

$T_p = \frac{\Delta t}{2} + L$ ✓ eq 16.7 NEH-4

$L = 0.6\ T_c$ ✓

$\Delta t = .133\ T_c$ ✓ eq 16.12 NEH-4

$\Delta t = .133(.168 \text{ hrs})$ –

$\Delta t = 0.02 \text{ hrs}$ – $T_p = .6\ t_c + .0665\ t_c$ 36

$T_p = (0.02 \text{ hrs})/2 + 0.6(.168 \text{ hrs})$ $t_p = .6665\ t_c$

$T_p = 0.112 \text{ hrs}$ or 6 minutes unreasonable

PRODUCT 204-1 NEBS Inc. Groton, Mass. 01471

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JOB Old Sante Fe
SHEET NO. 14 OF 15
CALCULATED BY PGD DATE 12-2-86
CHECKED BY fhc DATE 3/2/87
SCALE

North Drainage Inflow Hydrograph

Runoff Volume.

From "Procedure for Determining Peak Flows in Colorado"
SCS 1980

For $P_{100} = 3.50$ inches & CN = 83 (from Lincoln Devore)

$Q_{100} = 1.86$ inches

Time of Concentration.

Using Drainage Criteria: City of Colorado Springs Fig II, p.46

High Elevation = 6675 ft msl   H = 470 ft
Low Elevation 6208 ft msl
Length of water course ≈ 6200 ft

From Fig II $t_c = .3$ hrs.

Time to Peak

$t_p = 0.6665\, t_c = 0.6665(0.3) \times 60$ min/hr $t_p = 12$ minutes

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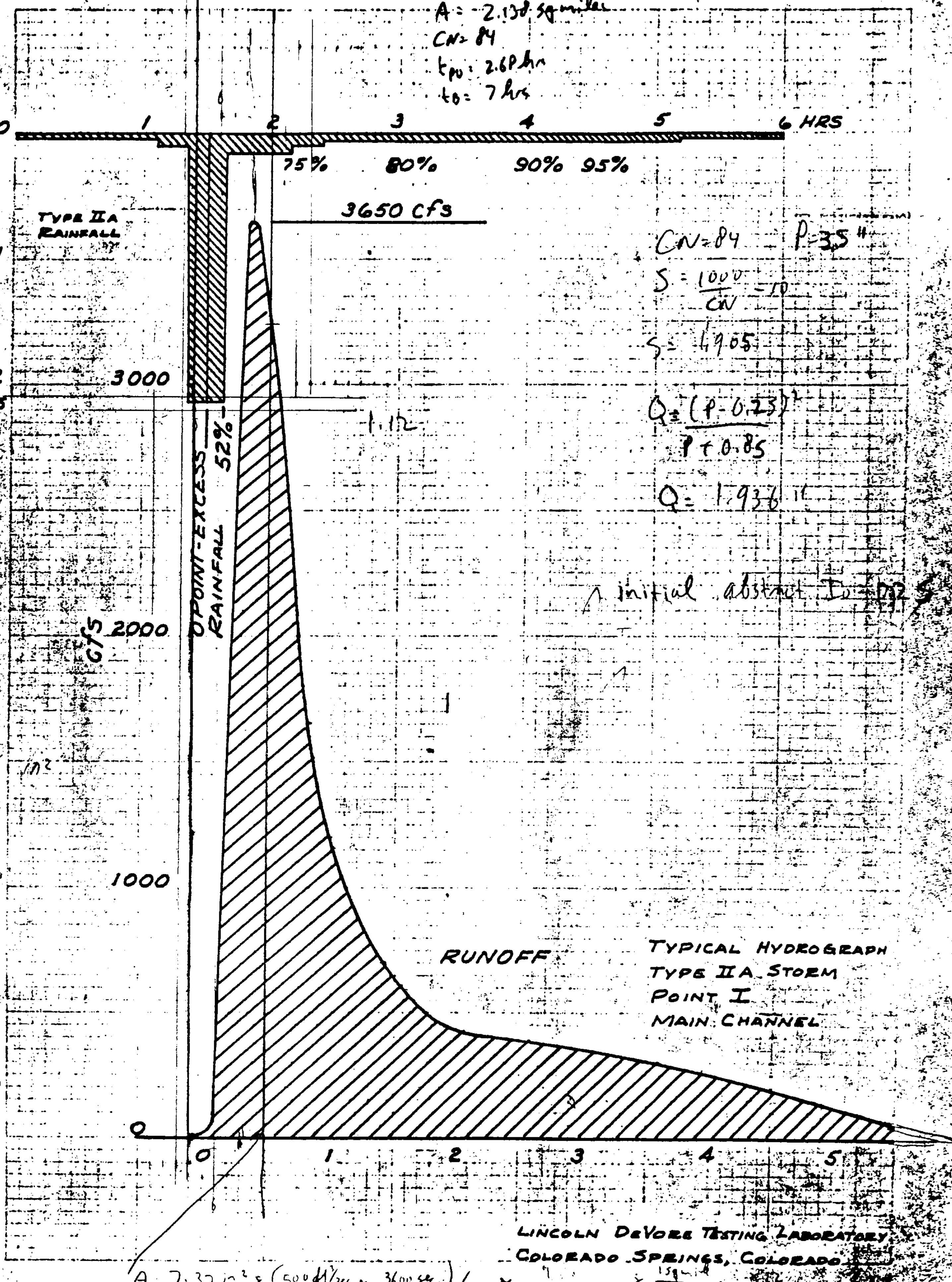
JOB Old Santa Fe
SHEET NO. 15 OF 15
CALCULATED BY PGD DATE 12-2-86
CHECKED BY Eh DATE 3/2/87
SCALE

Time min	Basin I flow cfs	Time N. Drainage flow	Time	Basin I flow cfs	Time N. Drainage flow	Time	Basin I flow cfs	N. Drain. flow
0	0	0						
6	25	3	6	440		6	240	
12	100	6	12	430		12	230	
18	1875	9	18	420		18	220	
24	3650	12	24	410		24	210	
30	3550	15	30	400		30	200	
36	3250	18	36	390		36	190	
42	2800	21	42	380		42	180	
48	2100	24	48	370		48	170	
54	1600	.	54	360		54	160	
1 60	1400	:	3 60	350		5 60	150	
6	1200	:	6	340		6	140	
12	1050	etc.	12	330		12	130	
18	900		18	320		18	120	
24	800		24	310		24	110	
30	700		30	300		30	100	
36	650		36	290		36	80	
42	600		42	280		42	60	
48	550		48	270		48	40	
54	500		54	260		54	20	
2 60	450		4 60	250		60	0	

Qpeak = 3650 cfs
Qpeak N shook basin = 677 cfs

HEC-1 ratio = 677 cfs / 3650 cfs = 0.18548 .18



3.5

7.316 in²

7.42

7.37 in

$A = 7.37\ in^2 \times (500\ ft^3/sec \times 3600\ sec)/in^2 \times \frac{1}{2.138\ sq\ mile} \times \frac{1\ sq\ mile}{(5280\ ft)^2} = 2.0$ in

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Hydraulic Gradeline of North 4'x6' CBC

Verification of inlet-outlet control of Junction Box.

Using previous assumptions ie)

1) No significant tail water effects at most western outlet, ie depth of water = depth within 4'x6' CBC

2) Negligible approach velocities at east entrance & within junction box.

Analyze for outlet of detention pond peak discharge

$Q_{peak} = 398$ cfs

Refer to schematic sheet 4 of 15

Since there is a possible grade break in the 4'x6' CBC between the culverts underneath the highway, and the fill west of the highway, one segment could be flatter than the average slope of 4.93%,

For a 6' wide rectangular channel $N = 0.015$,

a slope of 1.4% could still convey 398 cfs. with a depth of 3.9 ft, ie. open channel flow.

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JOB Old Santa Fe Cut
SHEET NO. 2 OF 3
CALCULATED BY PGD DATE 1-22-87
CHECKED BY ______ DATE ______
SCALE revised 2-12-87

A slope of 4.93 % would convey the 398 cfs at a depth of 2.5 ft. In fact for Q = 677 cfs & S = 4.93%, the depth would equal 3.7 ft.

However assuming the worst possible case for friction, i.e. pressure flow, calculate required head within junction box using Bernoullis from 1 to 3

$$\frac{V_3^2}{2g} + h_3 + Z_3 = \frac{V_1^2}{2g} + h_1 + Z_1 + Sf_{1-3}(L_{1-3}) + he$$

solve for h_3, required head within junction box

say $V_3 = 0$

$$he = 0.5\frac{V^2}{2g}$$

$V_1 = Q/A$

$V_1 = 398\text{ cfs}/24\text{ ft}^2$ $\qquad V_1 = 16.6$ ft/sec

$\frac{V_1^2}{2g} = \frac{(16.6\text{ ft/sec})^2}{64.4\text{ ft/sec}^2}$ $\qquad V_1^2/2g = 4.27$ ft

$Z_1 = 6183.9$ ft

$Z_3 = 6201.63$ ft

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JOB Old Santa Fe Center

SHEET NO. 3 OF 3

CALCULATED BY PGD DATE 1-22-87

CHECKED BY DATE

SCALE revised 2-12-87

for the 4 × 6' CBC assume $n = 0.015$

Friction loss slope:

$A = 24\ ft^2$

$WP = 20\ ft$

$R = 1.20\ ft$

$$S_f = \left(\frac{Q\,n}{1.49\,A R^{2/3}}\right)^2$$

$$S_f = \left(\frac{(398)(0.015)}{1.49(24)(1.20)^{2/3}}\right)^2 \qquad S_f = 0.0219\ ft/ft$$

$h_f = L \cdot S_f$

$h_f = 360\ ft * .0219$ $\qquad h_f = 7.9\ ft$

Entrance losses

$h_e = 0.5\,(4.27\ ft)$ $\qquad h_e = 2.1\ ft$

h_1, say h_1 = height of culvert

so $h_3 = 4.27\ ft + 4\ ft + 6183.9\ ft + 7.9\ ft + 2.1\ ft - 6201.63\ ft$

$h_3 = 0.54\ ft$ ie inlet controls.

```
1
                                             ****
                    FLOOD HYDROGRAPH PACKAGE HEC-1 (IBM XT 512K VERSION) -FEB 1,1985
      U.S. ARMY CORPS OF ENGINEERS, THE HYDROLOGIC ENGINEERING CENTER, 609 SECOND STREET, DAVIS, CA. 95616
                                             ****



      THIS HEC-1 VERSION CONTAINS ALL OPTIONS EXCEPT ECONOMICS, AND THE NUMBER OF PLANS ARE REDUCED TO 3


1
                                         HEC-1 INPUT                                             PAGE  1

  LINE            ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

     1            ID        OLD SANTE FE - NORTH DRAINAGE BASIN
     2            IT       3                       100

     3            KK    UPST
     4            BA   0.331       0 0.18548
     5            QI     0.0      25     100    1875    3650    3550    3250    2800    2100    1600
     6            QI    1400    1200    1050     900     800     700     650     600     550     500
     7            QI     450     440     430     420     410     400     390     380     370     360
     8            QI     350     340     330     320     310     300     290     280     270     260
     9            QI     250     240     230     220     210     200     190     180     170     160
    10            QI     150     140     130     120     110     100      80      60      40      20
    11            QI     0.0

    12            KK   NORTH
    13            KM            RESERVOIR ROUTING
    14            RS       1    ELEV  6108.2
    15            SV     0.0    0.04    0.15    0.47    1.10    2.09    3.49    5.58    8.41   11.90
    16            SQ     0.0     139     212     282     335     365     388     410     430     443
    17            SE  6208.2    6212    6214    6216    6218    6220    6222    6224    6226    6228
    18            ST    6228     100     2.4     1.5
    19            ZZ
1
                                             ****
                    FLOOD HYDROGRAPH PACKAGE HEC-1 (IBM XT 512K VERSION) -FEB 1,1985
      U.S. ARMY CORPS OF ENGINEERS, THE HYDROLOGIC ENGINEERING CENTER, 609 SECOND STREET, DAVIS, CA. 95616
                                             ****



                         OLD SANTE FE - NORTH DRAINAGE BASIN

   IT          HYDROGRAPH TIME DATA
                    NMIN          3  MINUTES IN COMPUTATION INTERVAL
                   IDATE      1   0  STARTING DATE
                   ITIME       0000  STARTING TIME
                      NQ        100  NUMBER OF HYDROGRAPH ORDINATES
                  NDDATE      1   0  ENDING DATE
                  NDTIME       0457  ENDING TIME

             COMPUTATION INTERVAL     .05 HOURS
                  TOTAL TIME BASE    4.95 HOURS

        ENGLISH UNITS
```

*** ***

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             **************
 3 KK        *             *
             *    UPST     *
             *             *
             **************
```

SUBBASIN RUNOFF DATA

4 BA SUBBASIN CHARACTERISTICS

TAREA	.33	SUBBASIN AREA
SNAP	.00	NORMAL ANNUAL PRECIPITATION
RATIO	.19	RATIO OF HYDROGRAPH

***

****

HYDROGRAPH AT STATION UPST

****

DA	MON	HRMN	ORD	FLOW	*	DA	MON	HRMN	ORD	FLOW	*	DA	MON	HRMN	ORD	FLOW	*	DA	MON	HRMN	ORD	FLOW
1		0000	1	0.	*	1		0115	26	400.	*	1		0230	51	150.	*	1		0345	76	0.
1		0003	2	25.	*	1		0118	27	390.	*	1		0233	52	140.	*	1		0348	77	0.
1		0006	3	100.	*	1		0121	28	380.	*	1		0236	53	130.	*	1		0351	78	0.
1		0009	4	1875.	*	1		0124	29	370.	*	1		0239	54	120.	*	1		0354	79	0.
1		0012	5	3650.	*	1		0127	30	360.	*	1		0242	55	110.	*	1		0357	80	0.
1		0015	6	3550.	*	1		0130	31	350.	*	1		0245	56	100.	*	1		0400	81	0.
1		0018	7	3250.	*	1		0133	32	340.	*	1		0248	57	80.	*	1		0403	82	0.
1		0021	8	2800.	*	1		0136	33	330.	*	1		0251	58	60.	*	1		0406	83	0.
1		0024	9	2100.	*	1		0139	34	320.	*	1		0254	59	40.	*	1		0409	84	0.
1		0027	10	1600.	*	1		0142	35	310.	*	1		0257	60	20.	*	1		0412	85	0.
1		0030	11	1400.	*	1		0145	36	300.	*	1		0300	61	0.	*	1		0415	86	0.
1		0033	12	1200.	*	1		0148	37	290.	*	1		0303	62	0.	*	1		0418	87	0.
1		0036	13	1050.	*	1		0151	38	280.	*	1		0306	63	0.	*	1		0421	88	0.
1		0039	14	900.	*	1		0154	39	270.	*	1		0309	64	0.	*	1		0424	89	0.
1		0042	15	800.	*	1		0157	40	260.	*	1		0312	65	0.	*	1		0427	90	0.
1		0045	16	700.	*	1		0200	41	250.	*	1		0315	66	0.	*	1		0430	91	0.
1		0048	17	650.	*	1		0203	42	240.	*	1		0318	67	0.	*	1		0433	92	0.
1		0051	18	600.	*	1		0206	43	230.	*	1		0321	68	0.	*	1		0436	93	0.
1		0054	19	550.	*	1		0209	44	220.	*	1		0324	69	0.	*	1		0439	94	0.
1		0057	20	500.	*	1		0212	45	210.	*	1		0327	70	0.	*	1		0442	95	0.
1		0100	21	450.	*	1		0215	46	200.	*	1		0330	71	0.	*	1		0445	96	0.
1		0103	22	440.	*	1		0218	47	190.	*	1		0333	72	0.	*	1		0448	97	0.
1		0106	23	430.	*	1		0221	48	180.	*	1		0336	73	0.	*	1		0451	98	0.
1		0109	24	420.	*	1		0224	49	170.	*	1		0339	74	0.	*	1		0454	99	0.
1		0112	25	410.	*	1		0227	50	160.	*	1		0342	75	0.	*	1		0457	100	0.

****

	PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
				6-HR	24-HR	72-HR	4.95-HR
+	(CFS)	(HR)					
			(CFS)				
+	3650.	.20		378.	378.	378.	378.
			(INCHES)	8.754	8.754	8.754	8.754
			(AC-FT)	155.	155.	155.	155.

CUMULATIVE AREA = .33 SQ MI

HYDROGRAPH MULTIPLIED BY .19

****

HYDROGRAPH AT STATION UPST

****

DA	MON	HRMN	ORD	FLOW	*	DA	MON	HRMN	ORD	FLOW	*	DA	MON	HRMN	ORD	FLOW	*	DA	MON	HRMN	ORD	FLOW
1		0000	1	0.	*	1		0115	26	74.	*	1		0230	51	28.	*	1		0345	76	0.
1		0003	2	5.	*	1		0118	27	72.	*	1		0233	52	26.	*	1		0348	77	0.
1		0006	3	19.	*	1		0121	28	70.	*	1		0236	53	24.	*	1		0351	78	0.
1		0009	4	348.	*	1		0124	29	69.	*	1		0239	54	22.	*	1		0354	79	0.
1		0012	5	677.	*	1		0127	30	67.	*	1		0242	55	20.	*	1		0357	80	0.
1		0015	6	658.	*	1		0130	31	65.	*	1		0245	56	19.	*	1		0400	81	0.
1		0018	7	603.	*	1		0133	32	63.	*	1		0248	57	15.	*	1		0403	82	0.
1		0021	8	519.	*	1		0136	33	61.	*	1		0251	58	11.	*	1		0406	83	0.
1		0024	9	390.	*	1		0139	34	59.	*	1		0254	59	7.	*	1		0409	84	0.
1		0027	10	297.	*	1		0142	35	57.	*	1		0257	60	4.	*	1		0412	85	0.
1		0030	11	260.	*	1		0145	36	56.	*	1		0300	61	0.	*	1		0415	86	0.
1		0033	12	223.	*	1		0148	37	54.	*	1		0303	62	0.	*	1		0418	87	0.
1		0036	13	195.	*	1		0151	38	52.	*	1		0306	63	0.	*	1		0421	88	0.
1		0039	14	167.	*	1		0154	39	50.	*	1		0309	64	0.	*	1		0424	89	0.
1		0042	15	148.	*	1		0157	40	48.	*	1		0312	65	0.	*	1		0427	90	0.
1		0045	16	130.	*	1		0200	41	46.	*	1		0315	66	0.	*	1		0430	91	0.
1		0048	17	121.	*	1		0203	42	45.	*	1		0318	67	0.	*	1		0433	92	0.
1		0051	18	111.	*	1		0206	43	43.	*	1		0321	68	0.	*	1		0436	93	0.
1		0054	19	102.	*	1		0209	44	41.	*	1		0324	69	0.	*	1		0439	94	0.
1		0057	20	93.	*	1		0212	45	39.	*	1		0327	70	0.	*	1		0442	95	0.
1		0100	21	83.	*	1		0215	46	37.	*	1		0330	71	0.	*	1		0445	96	0.
1		0103	22	82.	*	1		0218	47	35.	*	1		0333	72	0.	*	1		0448	97	0.
1		0106	23	80.	*	1		0221	48	33.	*	1		0336	73	0.	*	1		0451	98	0.
1		0109	24	78.	*	1		0224	49	32.	*	1		0339	74	0.	*	1		0454	99	0.
1		0112	25	76.	*	1		0227	50	30.	*	1		0342	75	0.	*	1		0457	100	0.

****

	PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
				6-HR	24-HR	72-HR	4.95-HR
+	(CFS)	(HR)					
			(CFS)				
+	677.	.20		70.	70.	70.	70.
			(INCHES)	1.624	1.624	1.624	1.624
			(AC-FT)	29.	29.	29.	29.

CUMULATIVE AREA = .33 SQ MI

*** ***

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            **************
            *            *
 12 KK      *   NORTH    *
            *            *
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RESERVOIR ROUTING

HYDROGRAPH ROUTING DATA

14 RS STORAGE ROUTING

ITYP	ELEV	TYPE OF INITIAL CONDITION
RSVRIC	6108.20	INITIAL CONDITION
X	.00	WORKING R AND D COEFFICIENT

15 SV	STORAGE	.0	.0	.2	.5	1.1	2.1	3.5	5.6	8.4	11.9
16 SQ	DISCHARGE	0.	139.	212.	282.	335.	365.	388.	410.	430.	443.
17 SE	ELEVATION	6208.20	6212.00	6214.00	6216.00	6218.00	6220.00	6222.00	6224.00	6226.00	6228.00

18 ST TOP OF DAM

TOPEL	6228.00	ELEVATION AT TOP OF DAM
DAMWID	100.00	DAM WIDTH
COQD	2.40	WEIR COEFFICIENT
EXPD	1.50	EXPONENT OF HEAD

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HYDROGRAPH AT STATION NORTH

****

DA	MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE	*	DA	MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE	*	DA	MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE
1		0000	1	0.	.0	6108.2	*	1		0142	35	58.	.0	6209.8	*	1		0324	69	0.	.0	6208.2
1		0003	2	0.	.0	6208.2	*	1		0145	36	56.	.0	6209.7	*	1		0327	70	0.	.0	6208.2
1		0006	3	20.	.0	6208.8	*	1		0148	37	54.	.0	6209.7	*	1		0330	71	0.	.0	6208.2
1		0009	4	232.	.2	6214.6	*	1		0151	38	52.	.0	6209.6	*	1		0333	72	0.	.0	6208.2
1		0012	5	337.	1.2	6218.2	*	1		0154	39	50.	.0	6209.6	*	1		0336	73	0.	.0	6208.2
1		0015	6	371.	2.5	6220.6	*	1		0157	40	48.	.0	6209.5	*	1		0339	74	0.	.0	6208.2
1		0018	7	388.	3.5	6222.0	*	1		0200	41	47.	.0	6209.5	*	1		0342	75	0.	.0	6208.2
1		0021	8	396.	4.2	6222.7	*	1		0203	42	45.	.0	6209.4	*	1		0345	76	0.	.0	6208.2
1		0024	9	398.	4.5	6222.9	*	1		0206	43	43.	.0	6209.4	*	1		0348	77	0.	.0	6208.2
1		0027	10	396.	4.2	6222.7	*	1		0209	44	41.	.0	6209.3	*	1		0351	78	0.	.0	6208.2
1		0030	11	391.	3.8	6222.3	*	1		0212	45	39.	.0	6209.3	*	1		0354	79	0.	.0	6208.2
1		0033	12	383.	3.2	6221.5	*	1		0215	46	37.	.0	6209.2	*	1		0357	80	0.	.0	6208.2
1		0036	13	371.	2.5	6220.5	*	1		0218	47	35.	.0	6209.2	*	1		0400	81	0.	.0	6208.2
1		0039	14	354.	1.7	6219.2	*	1		0221	48	34.	.0	6209.1	*	1		0403	82	0.	.0	6208.2
1		0042	15	324.	1.0	6217.6	*	1		0224	49	32.	.0	6209.1	*	1		0406	83	0.	.0	6208.2
1		0045	16	255.	.3	6215.2	*	1		0227	50	30.	.0	6209.0	*	1		0409	84	0.	.0	6208.2
1		0048	17	142.	.0	6212.1	*	1		0230	51	28.	.0	6209.0	*	1		0412	85	0.	.0	6208.2
1		0051	18	98.	.0	6210.9	*	1		0233	52	26.	.0	6208.9	*	1		0415	86	0.	.0	6208.2
1		0054	19	113.	.0	6211.3	*	1		0236	53	24.	.0	6208.9	*	1		0418	87	0.	.0	6208.2
1		0057	20	86.	.0	6210.5	*	1		0239	54	22.	.0	6208.8	*	1		0421	88	0.	.0	6208.2
1		0100	21	90.	.0	6210.7	*	1		0242	55	21.	.0	6208.8	*	1		0424	89	0.	.0	6208.2
1		0103	22	77.	.0	6210.3	*	1		0245	56	19.	.0	6208.7	*	1		0427	90	0.	.0	6208.2
1		0106	23	83.	.0	6210.5	*	1		0248	57	15.	.0	6208.6	*	1		0430	91	0.	.0	6208.2
1		0109	24	75.	.0	6210.3	*	1		0251	58	11.	.0	6208.5	*	1		0433	92	0.	.0	6208.2
1		0112	25	78.	.0	6210.3	*	1		0254	59	8.	.0	6208.4	*	1		0436	93	0.	.0	6208.2
1		0115	26	73.	.0	6210.2	*	1		0257	60	4.	.0	6208.3	*	1		0439	94	0.	.0	6208.2
1		0118	27	74.	.0	6210.2	*	1		0300	61	0.	.0	6208.2	*	1		0442	95	0.	.0	6208.2
1		0121	28	70.	.0	6210.1	*	1		0303	62	0.	.0	6208.2	*	1		0445	96	0.	.0	6208.2
1		0124	29	69.	.0	6210.1	*	1		0306	63	0.	.0	6208.2	*	1		0448	97	0.	.0	6208.2
1		0127	30	66.	.0	6210.0	*	1		0309	64	0.	.0	6208.2	*	1		0451	98	0.	.0	6208.2
1		0130	31	65.	.0	6210.0	*	1		0312	65	0.	.0	6208.2	*	1		0454	99	0.	.0	6208.2
1		0133	32	63.	.0	6209.9	*	1		0315	66	0.	.0	6208.2	*	1		0457	100	0.	.0	6208.2
1		0136	33	62.	.0	6209.9	*	1		0318	67	0.	.0	6208.2	*							
1		0139	34	59.	.0	6209.8	*	1		0321	68	0.	.0	6208.2	*							

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PEAK OUTFLOW IS 398. AT TIME .40 HOURS

PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
			6-HR	24-HR	72-HR	4.95-HR
+ (CFS)	(HR)					
		(CFS)				
+ 398.	.40		70.	70.	70.	70.
		(INCHES)	1.624	1.624	1.624	1.624
		(AC-FT)	29.	29.	29.	29.

PEAK STORAGE	TIME	MAXIMUM AVERAGE STORAGE			
		6-HR	24-HR	72-HR	4.95-HR
+ (AC-FT)	(HR)				
4.	.40	0.	0.	0.	0.

PEAK STAGE	TIME	MAXIMUM AVERAGE STAGE			
		6-HR	24-HR	72-HR	4.95-HR
+ (FEET)	(HR)				
6222.93	.40	6209.89	6209.89	6209.89	6209.89

CUMULATIVE AREA = .33 SQ MI

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RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD			BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
+					6-HOUR	24-HOUR	72-HOUR			
	HYDROGRAPH AT									
+		UPST	677.	.20	70.	70.	70.	.33		
	ROUTED TO									
+		NORTH	398.	.40	70.	70.	70.	.33		
+									6222.93	.40

1 SUMMARY OF DAM OVERTOPPING/BREACH ANALYSIS FOR STATION NORTH

PLAN 1		INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
	ELEVATION	6108.20	6228.00	6228.00
	STORAGE	0.	12.	12.
	OUTFLOW	0.	443.	443.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
1.00	6222.93	.00	4.	398.	.00	.40	.00

Maximum pond elevation used for setting building elevations.

*** NORMAL END OF HEC-1 ***

Surface Area of ponding within detention area ≈ 1.1 ac