CITY OF ALBUQUERQUE

April 16, 2005

Gordon Mossberg, PE
Resource Technology, Inc.
1720-B Randolph SE
Albuquerque, NM 87106

Re: Tres Lagunas Subdivision Engineering Certification
Engineer's Stamp dated 12-27-04, (E16/D16)

Dear Mr. Mossberg,

Based on your information contained in your submittal dated 3-23-05, the above referenced certification is approved for Release of Financial Guarantee and SIA.

If you have any questions, please contact me at 924-3986.

Sincerely,

Bradley L. Bingham, PE
Principal Engineer, Planning Dept.
Development and Building Services

C: Marilyn Maldonado, CPN 659481
file

P.O. Box 1293

Albuquerque

New Mexico 87103

www.cabq.gov

CITY OF ALBUQUERQUE

PUBLIC WORKS DEPARTMENT
CONSTRUCTION MANAGEMENT DIVISION
SURVEY SECTION

August 3, 2004

Russ P. Hugg, PS
Surv-Tek
5643 Paradise Blvd NW
Albuquerque, NM 87103

City Project No. 659481

NOTICE OF ACCEPTANCE

RE: CENTERLINE MONUMENTATION – Tres Lagunas Subdivision

P.O. Box 1293

Albuquerque

New Mexico 87103

www.cabq.gov

Dear Mr. Hugg:

This letter is to serve as notice of acceptance of the Centerline Monumentation for the above referenced subdivision.

Field inspection performed by this office shows that centerline monuments have been placed as per approved plan satisfying your commitment to monument the centerline of the streets within the above referenced subdivision.

Thank you for your cooperation in this matter.

Sincerely,

Glen B. Haikin, P.S.
City Surveyor

GBH:et

cc: Mary Sandoval, Engineering Technician
Kathy Jaramillo, DRC
File Centerline Monumentation # 020501

RECEIVED
MAR 23 2005
HYDROLOGY SECTION

MIDDLE RIO GRANDE CONSERVANCY DISTRICT

May 4, 2000

Resource Technology, Inc.
1720-B Randolph Road SE
Albuquerque, NM 87106
Attn.: Mr. Elvidio Diniz

REF.: TRES LAGUNAS SUBDIVISION

Dear Mr. Diniz:

We have reviewed your grading and drainage plan for the above referenced. The District concurs with your grading and drainage plan per your submittal dated April 6, 2000. The District understands storm water from the subdivision will be stored in retention ponds sized for a 100 year storm event located within the subdivision site. Storm flows will overflow the subdivision site to the Alameda Lateral via a grouted rip-rap spillway during emergencies when rain storms exceed the 100 year storm event.

No license or permit is required from the District because no work or structures are to be constructed within the Alameda Lateral Right-of-Way.

If you have any questions please feel free to contact Mr. Ray Gomez for more information at (505) 247-0234.

Sincerely,

Subhas K. Shah
Chief Executive Officer

P.O. Box 581

87103-0581

1931 Second St. SW

Albuquerque, NM

87102-4515

505-247-0234

Fax # 505-243-7308

xc: Leonard Utter, Assistant Engineer
Ray Gomez, Engineer 1
File

TRES LAGUNAS SUBDIVISION

DRAINAGE REPORT

Prepared For:

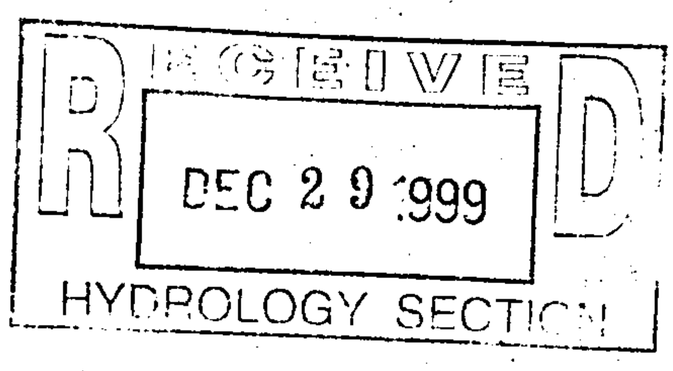
D. Stuart Harroun
7020 Edith Blvd. NE
Albuquerque, NM 87113

Prepared By:

Resource Technology, Inc.
ENGINEERS AND ENVIRONMENTAL SCIENTISTS
1720-B Randolph Road SE, Albuquerque, NM 87106
(505) 243-7300 - (505) 243-7400 (fax)

December, 1999

RTI Project No. 99-100



INTRODUCTION

The site is a 14.9 acre tract identified as Tracts A1 and B2, Lands of Lloyd Lozes Goff, as shown on the Vicinity Map (Figure 1). The site is to be subdivided into eleven lots. It is proposed that seven lots be developed for single dwelling units, and four lots be used as recreation areas (common areas) for the enjoyment of the residents.

A landscaping feature of the subdivision is a series of ponds connected in a recirculatory system (see plan sheet) where water is continuously pumped (at approximately 150 gpm) from the low pond (Pond 3B) on Lot #11 to the high pond (Pond 1) on Lot #5. The water then gravity drains back through Ponds 1, 2, and 3A. A private well provides make-up water for evaporation/infiltration losses.

To better serve the proposed lot layout, the existing landscape pond system Ponds 3A and 3B will be enlarged, as shown on the plan. Dip sections in driveways and maintenance roads will control overflows from Ponds 2 and 3A. The existing pump located near Pond 3B will remain, as will the piping system back to Pond 1.

The site is shown as Zone X on the current FIRM map, outside of any major flooding hazard (Figure 2).

OFF-SITE DRAINAGE

There is no off-site drainage onto this study site. The site is located in a portion of the historic Bear Canyon Arroyo; however, upstream flood waters no longer reach the site due to the construction of the North Diversion Channel, located approximately 3200 ft east of the site, and excavation of a gravel/sand quarry immediately upstream of the site. The former quarry area is the location of the Vista del Norte Subdivision (Figure 3). The approved plans for this upstream subdivision call for all drainage to be directed to the south, so that no runoff will be discharged to the study site. The existing tracts located north and south of the site also do not drain onto the site.

ON-SITE DRAINAGE - EXISTING CONDITION

The site is sloped so that most on-site runoff drains to the path of the historic Bear Canyon Arroyo which runs from the east lot line to the northwest corner of the site. The existing landscape pond system is designed to follow this natural drainage path, culminating at Pond 3B which also acts as a ponding area for on-site runoff. A storm of sufficient magnitude to fill this pond would overflow partially onto the tract to the north and into the Alameda Lateral. The Alameda Lateral is the natural discharge point for runoff from the site, and this ditch previously accepted and currently accepts all drainage from the site. The existing condition hydrology is as follows:

Table 1
Existing Condition Hydrology

Direction of Drainage	Area (ac)	Land Treatment				100-yr Qp (cfs)	100-yr Volume (ac-ft)
		%A	%B	%C	%D		
NW Corner	13.12	80	5	10	5	24.7	0.85
SW Corner	1.78	80	5	10	5	3.4	0.11

The south side of the site drains to two on-site detention ponds, 4A and 4B, located just east of Edith Boulevard. An existing stormwater force main pumps this runoff back to Pond 3A as shown on the plan. This pumping system discharges approximately 35 gpm, and slowly drains these small ponds located near the entrance to the site, on Lot #1.

ON-SITE DRAINAGE - FUTURE CONDITION

The proposed drainage system is shown on the plan sheet. In as much as possible the natural drainage patterns will remain, with lots draining to either the landscape pond system or the historic Bear Canyon Arroyo. Road-side swales are also proposed on one or both sides of the road. All drainage swales, both existing or proposed, will drain to Ponds 3A and 3B for most of the site (13.12 acres), while the southwest corner area (1.78 acres) draining to Ponds 4A, 4B and 4C.

For future condition drainage analysis, the total area (eleven lots) was divided into eight sub-basins with sub-basins B-1 through B-6 draining to enlarged Ponds 3A and 3B in the northwest, and sub-basins B-7 and B-8 draining to Ponds 4A, 4B and 4C in the southwest part of the site. The future condition hydrology calculations are shown in Table 2.

Table 2
Future Condition Hydrology

Sub-basin	Area (ac)	Land Treatment				Q peak (cfs) 6-hr		Volume (ac-ft)		
		%A	%B	%C	%D	10-yr	100-yr	24-hr 10-yr	24-hr 100-yr	10-day 100-yr
B-1	2.66	80	5	10	5	1.80	5.10	0.053	0.151	0.169
B-2	0.83	40	30	10	20	1.03	2.16	0.032	0.069	0.092
B-3	2.58	40	30	10	20	3.20	6.63	0.098	0.212	0.281
B-4	2.28	10	20	30	40	4.54	7.85	0.145	0.266	0.388
B-5	1.66	40	30	10	20	2.05	4.28	0.063	0.137	0.181
B-6	3.10	75	5	10	10	2.53	6.40	0.077	0.196	0.238
Total draining to Ponds 3A and 3B (13.12 ac.)						15.15	32.42	.4681	1.03	1.35
B-7	0.54	0	64	0	36	1.06	1.92	0.034	0.064	0.093
B-8	1.24	36	36	15	13	1.42	3.06	0.041	0.095	0.116
Total draining to Ponds 4A, 4B and 4C (1.78 ac.)						2.48	4.98	0.075	0.159	0.21

DETENTION POND DESIGN

Since there is no appropriate off-site drainage system to accept the developed condition runoff from the site, a new detention area, identified as Ponds 3A and 3B will be constructed to allow the existing landscape pond system to surcharge and will serve as a storage basin for the developed condition 100-yr; 10-day runoff volume from all of the site except Lot #1 (sub-basins B-7 and B-8) which drains to the southwest corner of the site.

The ponding system will incorporate the gravel maintenance road between Ponds 3A and 3B as a weir to fix the normal 100-yr water surface elevation in the ponds and to allow for a controlled overflow location for storms in excess of the 100-year storm, while the existing pump system will maintain the normal water surface elevation in Pond 3B at 5002.0 ft. which is the maximum elevation of the recycled landscape water. Stormwater entering the system will accumulate first in Pond 3B, causing the water surface elevation to increase above 5002.0 feet. As the water surface continues to rise above 5002.0, the runoff will begin to fill Ponds 3A and 3B simultaneously, by means of the 24-inch culvert between the two ponds.

This ponding system, capable of storing the 100-yr; 10-day runoff volume from the site, complies with and exceeds City of Albuquerque and Bernalillo County design requirements for both detention and retention ponds. Consequently, the pond will be compatible with any future Edith Boulevard drainage system that will provide an eventual outfall for the pond.

In the interim, runoff will be detained on-site and used in the recirculating system until it evaporates or infiltrates. A temporary 6-inch outlet pipe equipped with a manually operated isolation valve (normally shut) will be installed near the overflow spillway on Pond 3B and will allow the pond to discharge to the Alameda Lateral. This valve will only be opened after approval is granted by the MRGCD. The maximum discharge rate from the pipe will be 0.5 cfs and will be controlled by a 4-inch diameter orifice plate. At this discharge rate the entire 100-year; 10-day volume of 1.31 acre-feet would be drained in approximately 32 hours. The outlet pipe to the Alameda Lateral is temporary. With the construction of a drainage system in Edith, the outlet pipe will be moved to an appropriate outfall.

A berm will be constructed around Pond 3B to allow a maximum water surface elevation of 5005.0 before topping the overflow spillway. The total storage volume at water surface elevation = 5005.0 is 1.39 ac-ft. The total 100-year; 10-day runoff volume is 1.31 ac-ft. Table 3 summarizes the storage capacity of Ponds 3A and 3B.

Table 3
Detention Storage Summary

Pond	Normal Water Surface Elevation (ft)	Flood Water Surface Elevation (ft)	Volume @ Flood Water Surface Elevation* (cf)	Storm Water Storage Capacity* (ac-ft)
3A	5001	5005	34,200	0.787
3B	5002	5005	26,000	0.598
		Total Volume Available		1.385

*for storm water only; does not include storage below normal water surface elevation.

A 5% sediment bulking factor applied to the 100-yr; 24-hr volume results in a total sediment volume of 0.07 ac-ft for the site excluding Lot 1, and 0.01 ac-ft for Lot 1. The 5% bulking factor is a conservative value since average bulking factors for the east mesa at low discharges are typically 3.5% to 4% (Appendix D).

Existing Ponds 1 and 2 will serve as sediment traps for stormwater draining directly into these ponds. However, the majority of the sediment expected from this site (0.07 ac-ft) will accumulate in the remaining ponds, with Pond 3B as the primary sediment trap. The sediment storage available in this pond is listed in Table 4.

Table 4
Sediment Storage Summary

Pond	Bottom Elev. (ft)	Normal Water Surface Elevation (ft)	Volume @ Normal Water Surface Elevation (cf)	Sediment Storage Available (ac-ft)
3B	5001.0	5002.0	5,000	0.11

Any sediment entering the ponds will settle to the bottom and displace volume associated with the landscape water storage. Sediment would not begin to occupy the available stormwater retention volume until the depth of sediment exceeds elevation 5002.0 in Pond 3B. All of the landscape water storage volumes in Ponds 1A and 2A are also available for sediment storage as needed.

where are these?

An additional detention pond (4C) is proposed on Lot #1 as shown on the plan. This additional detention storage is necessary because the existing ponds (4A and 4B) and force main system (as previously described in Existing Conditions) do not have the capacity to control the 100-yr developed condition runoff. The discharge capacity for the force main is about 35 gpm (0.08 cfs) while the peak 100-yr flow rate to this area is approximately 5 cfs.

Pond 4C alone will have a storage capacity in excess of the 100-yr; 24-hour runoff volume (0.21 ac.ft.) with the total available volume in Ponds 4A, 4B and 4C being 0.3 ac.ft., 50% more than is required. A small 4" outlet pipe will allow Pond 4C to be pumped to Pond 3A along with Ponds 4A and 4B. In the event that the force main fails to operate, Ponds 4A and 4B will not overflow to the entrance road and then to Edith Boulevard until their storage volume is exceeded. Only storms greater than the 100-year storm would cause this overflow.

FUTURE EDITH BOULEVARD DRAINAGE IMPROVEMENTS

An off-site drainage area map is shown in Figure 3. The drainage areas shown represent the existing condition. As proposed, drainage from the study site will be retained and will not affect Edith Boulevard flows except for emergency overflows in excess of 100-yr levels from Ponds 4A and 4B. The following discussion of offsite conditions is included at the request of the City of Albuquerque Hydrology staff.

Basins 12A and 12B (see Figure 3) drain to a section of Edith Boulevard that slopes south to Osuna Road. Basin 13 runoff is conveyed across Edith Boulevard in an existing culvert. Future drainage from Basins 12A, 12B, and 13 has been addressed in the "Edith Boulevard Drainage Analysis," Boyle Engineering, Nov. 1990. A copy of selected portions of this document are contained in Appendix B.

Runoff from Basins 14 and 15 would drain to Edith Boulevard and then flow north to the Alameda Lateral crossing of Edith Boulevard. At this point runoff drains into the Alameda Lateral. However, the northern half of Basin 15 is now a detention pond for Vista del Norte Subdivision (see Figure 5) with a controlled pipe outlet to an existing detention pond west of Edith Boulevard and approximately 1500 feet south of the project site. For this analyses all of Basin 15 was assumed to drain to Edith Boulevard; consequently, the computed flow rates are conservative.

Previously, runoff from the area east of the site (Basin 16, see Figure 4) also drained to this section of Edith Boulevard and into the Alameda Lateral. At present Basin 16 no longer contributes runoff to Edith because it has been mined for gravel and sand and is now a large recessed area. This area is the site of the Vista del Norte Subdivision. A summary of the previous, existing, and future condition drainage to the Alameda Lateral is shown in Table 5. Supporting documentation and AHYMO files are provided in Appendix C.

Table 5
Drainage to Edith Boulevard

Existing Condition

Basin	Area	Land Treatment				100-yr Q peak	100-yr Volume
	(ac)	%A	%B	%C	%D	(cfs)	(ac-ft)
14	30.7	40	20	30	10	37.2	2.23
15	38.4	40	20	30	10	46.5	2.78
Site	14.9	80	5	10	5	28.5	0.85

Future Condition

Basin	Area	Land Treatment*				100-yr Q peak	100-yr Volume
	(ac)	%A	%B	%C	%D	(cfs)	(ac-ft)
14	30.7	0	20	30	50	98.0	4.21
15	38.4	0	20	30	50	122.5	5.26
Site	14.9	See Table 2				37.4	1.52

*Assume 5 D.U./acre

The future condition peak runoff draining to the Alameda Lateral would be approximately 220 cfs for uncontrolled flow from Basins 14 and 15. A runoff peak of this magnitude is beyond the capacity of the lateral. Due to the lack of any other drainage outfall options in this area, future condition stormwater management will require runoff detention with a controlled discharge to the Alameda Lateral. A detention pond capable of reducing the future condition runoff of 220 cfs to a value of 50 cfs would require a storage volume of about 29 ac-ft (see Appendix C). For the purpose of this study 50 cfs is assumed to be the maximum allowable discharge to the Alameda Lateral. A lower discharge will require a larger detention basin.

Three options are presented for future condition stormwater management:

Option 1 -- Detention Basin Located West of Edith Boulevard

Option 1 would include a 60-inch diameter storm drain in Edith Boulevard that outflows to a detention basin located just west of Edith. A reduced discharge from the basin would drain to the Alameda Lateral.

Option 2 -- Detention Basin Located East of Edith Boulevard

Option 2 incorporates a detention basin located just east of Edith Boulevard. The controlled outflow could be discharged to Edith Boulevard as curb and gutter flow. The street would then drain to the existing Alameda Lateral. This option does not require a storm drain, if the controlled release would not exceed the curb and gutter capacity.

Option 3 -- Lot by Lot Ponding Design for Future Development

Option 3 does not utilize a single large detention basin. This option calls for on-site detention for all future development in Basins 14 and 15 so that the total discharge to Edith Boulevard does not exceed the selected outflow rate. This option would result in an allowable discharge of 0.76 cfs/acre in Basins 14 and 15, if the outflow rate is 50 cfs.

DRAINAGE MASTER PLAN
FOR
VISTA DEL NORTE SUBDIVISION

AVID ENGINEERING, INC.

results of the sediment analysis under existing conditions. Peak bulking factors range from 1.04 to 1.13 under existing conditions.

Figure 2 is a reproduction of the SCS soils maps showing the location of the various soils units within the subdivision. The FEMA floodplain maps, shown in Figure 3, do not indicate the presence of any floodplains on or near the site, other than the diversion channel north of the site.

DEVELOPED DRAINAGE CONDITIONS

DRAINAGE BASIN DELINEATION

The site is divided into two major drainage basins, a north basin and a south basin. These major basins are divided into several subbasins that follow the layout of the tracts. Generally each subbasin fronting a major street (Valle Norte Drive and Las Lomitas Drive) includes one-half of the street right-of-way for the basin characteristics. Park "A" and Park "B" basins are the two exceptions. These two park basins comprise 6.5 acres that will be dedicated as neighborhood parks. Plate 1, Grading Plan, displays the major basins and each of their subbasins. The north basin consists of the on-site parcels north of Valle Norte Drive and includes the off-site subbasin of the Way-Cor concrete plant. The south basin includes most of the tracts south of Valle Del Norte Drive and the off-site basin of the Sego-Cox subdivision. There are two basins, basin A and the west basin, where on-site ponding will be allowed. Because of the low density (3 du/ac.) and the existing grade of basin A, the streets and finished floor elevations of the houses in basin A will be raised above the existing grade to allow for ponding of storm water within each lot. Also the west basin, which is the west entrance from Edith Boulevard, is located mostly in Bernalillo County and is much lower in elevation than the rest of the development. Therefore, the west basin will pond the runoff adjacent to the entrance road just east of Edith Boulevard.

HYDROLOGIC ANALYSIS

To determine the peak flows and runoff volumes of each subbasin an AHYMO analysis was performed in accordance to section 22.2 of the Development Process Manual (DPM.) The analysis included the 100-year 6-hour storm and the 100-year 24-hour storm. The 100-year 6-hour storm is the basis for determining peak flows to size the storm sewer collection system. The 100-year 24-hour storm was used to determine the required capacity of the detention and retention ponds. The design storm values are based on Tables C-1, C-2, and C-3 of the DPM's section 22.2. The Vista del Norte subdivision site is contained within sections D-16-Z and E-16-Z of the City of Albuquerque Zone Atlas Map. The location of the site results in the following design storms:

Table 1 Existing Drainage Conditions

BASINS	Area (acres)	100yr-6hr Peak Flow (cfs)	100yr- 24hr Runoff Volume (acre-ft)	Sediment Bulking Factor	Land Treatment
North Basin Subbasin	—	—	—	—	—
C	17.06	56.21	1.67	1.06	100%C
D	42.67	141.78	4.21	1.07	100%C
G	31.63	105.95	3.14	1.08	100%C
H	24.06	80.41	2.42	1.09	100%C
O	7.93	25.86	.77	1.05	100%C
P	23.96	78.73	2.34	1.06	100%C
Q	3.14	10.81	.32	1.11	100%C
R	2.55	8.75	.26	1.10	100%C
S	1.27	4.26	.13	1.07	100%C
Way-Cor site	22.41	99.45	4.09	1.07	20%C, 80%D
South Basin Subbasins	---	---	---	---	---
B	30.63	98.92	2.94	1.04	100%C
E	25.23	82.15	2.44	1.05	100%C
F	16.22	54.77	1.63	1.09	100%C
I	28.66	97.86	2.90	1.10	100%C
J	48.17	169.19	5.01	1.13	100%C
K	25.61	86.58	2.57	1.09	100%C
L	19.16	66.50	1.97	1.09	100%C
M	23.71	80.09	2.38	1.12	100%C
N	6.54	21.28	.63	1.05	100%C
Park "A"	2.50	8.14	.24	1.05	100%C
Park "B"	4.00	13.14	.39	1.05	100%C
Sego-Cox	10.00	46.92	1.96	1.06	8%B, 92%D
Other Basins	---	---	---	---	---
A	19.94	66.30	1.97	1.07	100%C
West	1.60	5.17	.15	1.04	100%C

SEDIMENT ANALYSIS

A detailed sediment analysis was performed for existing conditions following the methods described in Section 3.3 of the "Sediment and Erosion Design Guide" by AMAFCA. First, the sediment wash load was computed for each basin using the Modified Universal Soil Loss Equation (MUSLE) as shown in the AMAFCA Sediment Guide. Input parameters of the MUSLE equation were determined for each basin following the procedure given in Appendix B of the AMAFCA Sediment Guide. Next, the coefficients and exponents for the unit bed load power function equation (equation 3.41 in the AMAFCA Sediment Guide) were determined using Figure 3.10 in the AMAFCA Sediment Guide inputting an average D_{50} for the site. Table 1 shows the

HYDROLOGY SECTION

DEC 2 3 1999

SITE

N00°08'10"E 794.39'

Existing Lot 84

Proposed Lot 84

934.88'

S89°29'05"E

1063.81'

N80°09'03"W

N26°15'18"W

293.13'

BASIN A
19.94 acres

WEST BASIN
0.9017 acres

N08°07'4
205.23'

N47°

PROPOSED
3H:1V SLOPE
MAX.

1607.83'

N39°12'56"W

BASIN B
30.63 acres

Qp_{100}=109 cfs
DIA.=30"

#12

N08°19'01"W
412.43'

SOUTH POND
100-YR W.S.=5029.0
V_{100}=30.8 ac-ft

POND EASEMENT
POND INV.=5014.0

Qp_{100}=60 cfs
DIA.=30"

#10

BASIN F
16.08 acres

Qp_{100}=551 cfs
DIA.=84"

#52

#15

Qp_{100}=119 cfs
DIA.=36"

#50

Qp_{100}=89 cfs
DIA.=30"

L=450'

#2

L=450'
DIA.=24"

#3

L=450'

#4

35" PROPOSED
DRAINAGE EASMENT

#53

BASIN E
25.23 acres

BASIN K
25.61 acres

#20

35' PROPOSED
DRAINAGE EASMENT

Qp_{100}=456 cfs
DIA.=72"

PARK B BASIN
4.00 acres

SOUTH BASINS

R.O.W.

A triplex pump station is located near the southeast corner of Detention Basin No. 3, South. This pump station is equipped with three 15 horsepower pumps having a maximum combined discharge of 2,230 gallons per minute (5 cfs). The number of pumps operating at any given time is determined by the water level in the detention basin. At low water levels, only one pump will be in operation, but as the water level increases, the other pumps will begin operating.

"EDITH BLVD DRAINAGE ANALYSIS" BOYLE ENGINEERING NOV. 1990

The storm water is pumped into a 2000 foot long, 12-inch diameter ductile iron force main. A high point in this line occurs near the intersection of Mission Avenue and Edith Boulevard, at which point a combination air release valve is located. The force main discharges into System 10 near the intersection of El Ensueno Road and Edith Boulevard.

The control for the hydraulic grade line for System 11 is the 10-year water surface elevation in Detention Basin No. 3 (4969.6). It was assumed that the 10-year runoff was at equilibrium (the same water surface elevation in each basin).

E. <u>System 12</u>

System 12 drains the area east of Edith Boulevard from the Vineyard Development north to Arroyo Seco, including overflow from an existing storm drain system on Osuna Road. Runoff from areas between Edith Boulevard and the Alameda Lateral will be drained by combination inlets placed on Edith Boulevard. Sub-basins 12D, 12E, and 12F are to be drained by future drainage facilities on El Paraiso Road, Niagara Road, and Vineyard Road, respectively. Stub-outs are provided for these future facilities. Runoff from sub-basin 12G, The Vineyard Development, is collected by an inlet structure located just east of Edith Blvd. in the old Stott's lateral. Existing facilities direct the runoff to this location.

System 12 has two branches - a northerly branch and a short southerly branch. The southerly branch consists primarily of the inlet structure mentioned above and RCP connecting it to a junction at the intersection of Vineyard Road and Edith Boulevard.

The northerly branch begins about 250 feet north of Bear Canyon Lane, where a median drop inlet is placed in a natural depression on the east side of Edith Boulevard. Two additional median drop inlets are placed on the east side of the intersection of Bear Canyon Lane and Edith Boulevard. From that point, System 12 continues south to where it connects to a short section of existing RCP at Osuna Road, then continues south to the junction at Vineyard Road. The combined flow travels west for 800 feet, where it discharges into Detention Basin No. 3, North. This detention facility is discussed in the description of System 11. See Figure 12 for a schematic of System 12 and Table 9 for a hydrologic summary of Basin 12.

A portion of the runoff generated in basin 12B (Cross hatched area on Plate IA) is drained by existing combination inlets placed on the west side of Edith Boulevard just north of Osuna Road. The flow from this area does not contribute to the Edith Boulevard Storm Drain System. System 12 control for the hydraulic grade line is the 10-year water surface elevation in detention basin No. 3 (4969.6).

because it drains to Osuna System

F. System 13

System 13 drains Sub-basin 13, the area surrounding the Bear Canyon Arroyo. The system basically consists of a small detention facility and several hundred feet of RCP used to route the discharge. Detention Basin No. 6 is located in the Bear Canyon Arroyo site between Edith Boulevard and the AT & SF railroad. The basin accepts runoff directly from the Bear Canyon Arroyo, and releases the discharge at a controlled rate by means of a flow restriction plate. The capacity of Detention Basin No. 6 is 3.5 acre feet with two feet of freeboard. The side slopes are 2 to 1, and the depth is 8.5 feet.

10

Detention Basin No. 6 discharges into a 24-inch RCP that extends west the AT & SF railroad right-of-way. Then south for about 180 feet within the AT & SF right of way where it discharges into an existing drainage channel. The channel travels under the railway and continues in a westerly direction. At Second Street, the flow of 7.5 cfs from the channel enters a 24-inch CMP which discharges into the Alameda Drain. No hydraulic grade line was

computed for the discharge pipe or the existing channel. See Figure 13 for a schematic of System 13 and Table 10 for a hydrologic summary of Basin 13.

G. Gravel Pit Area

Although no drainage system will be provided for the Gravel Pit area under the Edith Boulevard Widening Project, a short description of the detention requirements is presented.

The Gravel Pit Detention Basin must be able to store runoff from basins 18, 19 and 20. A storage capacity of 45.1 acre-feet is required to store runoff from the 100 year, 24-hour rainfall event.

The location of the detention basin should be approximately as shown in Plate III. Discharge should be controlled by a flow restriction plate. Discharge must be able to be shut completely off by means of a sluice gate. Runoff must be able to be retained in the basin in the event of an extended power failure at the Detention Basin No. 3 pump station. Discharge flow is to be routed to the Edith system by means of a drain pipe down Vineyard Road. Maximum discharge is to be 5 cfs. See Table 11 for a hydrologic summary of the contributing basins.

VII. RESIDUAL FLOODING

The purpose of the storm drainage system associated with the Widening of Edith Boulevard Project is to alleviate the flooding problem within the study area. However, some flooding will occur even with the Edith Blvd. storm drainage system in place.

Since Edith Boulevard drainage system is only part of the total solution to the local flooding problem, some flooding may occur until the side street storm drain systems, downstream drainage systems, and the upstream detention facilities are operational. Most of the inlet structures to be built with the Edith project are designed to collect runoff from the area between the Alameda Lateral and Edith Boulevard. Runoff generated in areas east of the Alameda Lateral will either flow overland until it reaches Edith

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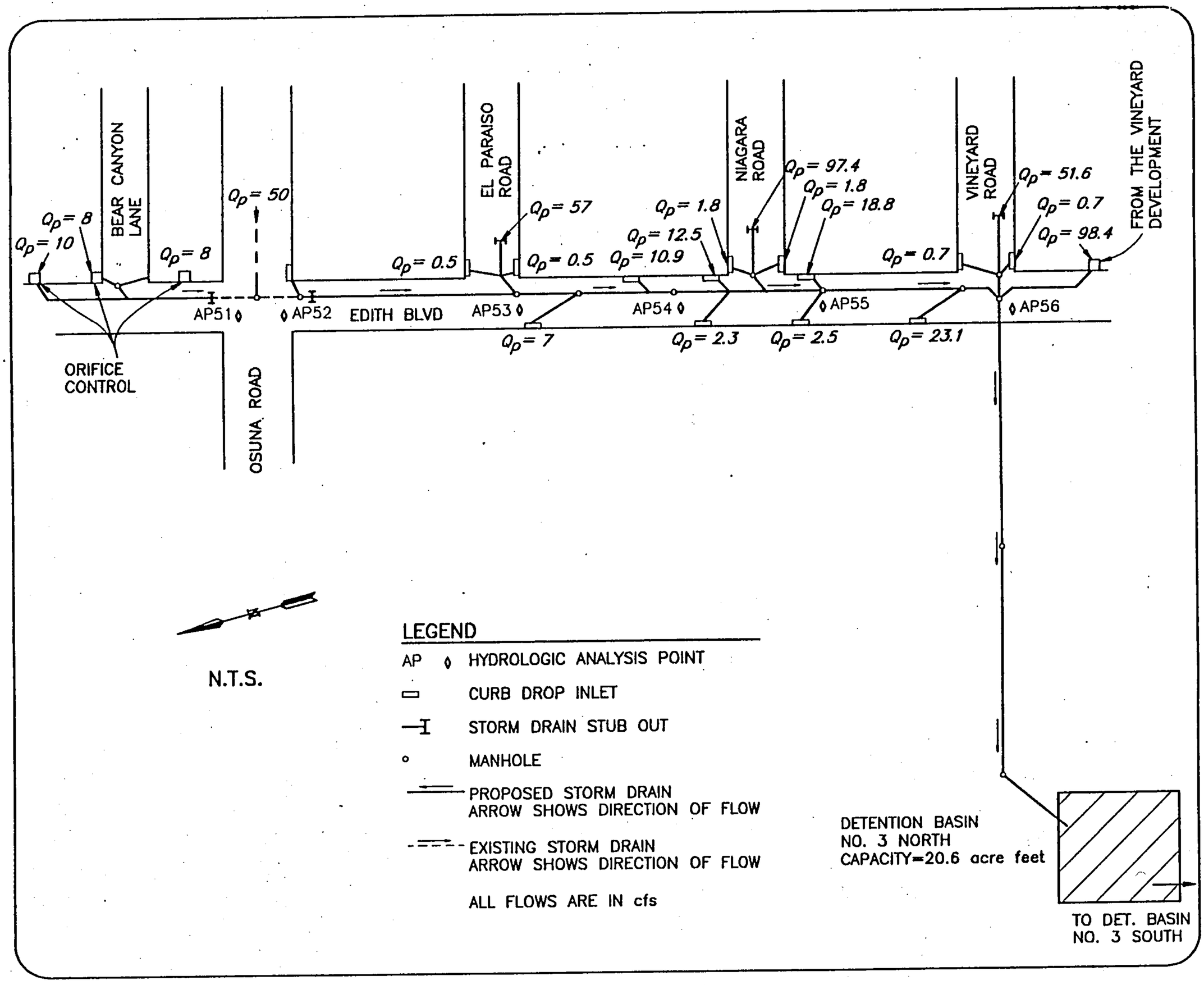
BOYLE

SYSTEM 12

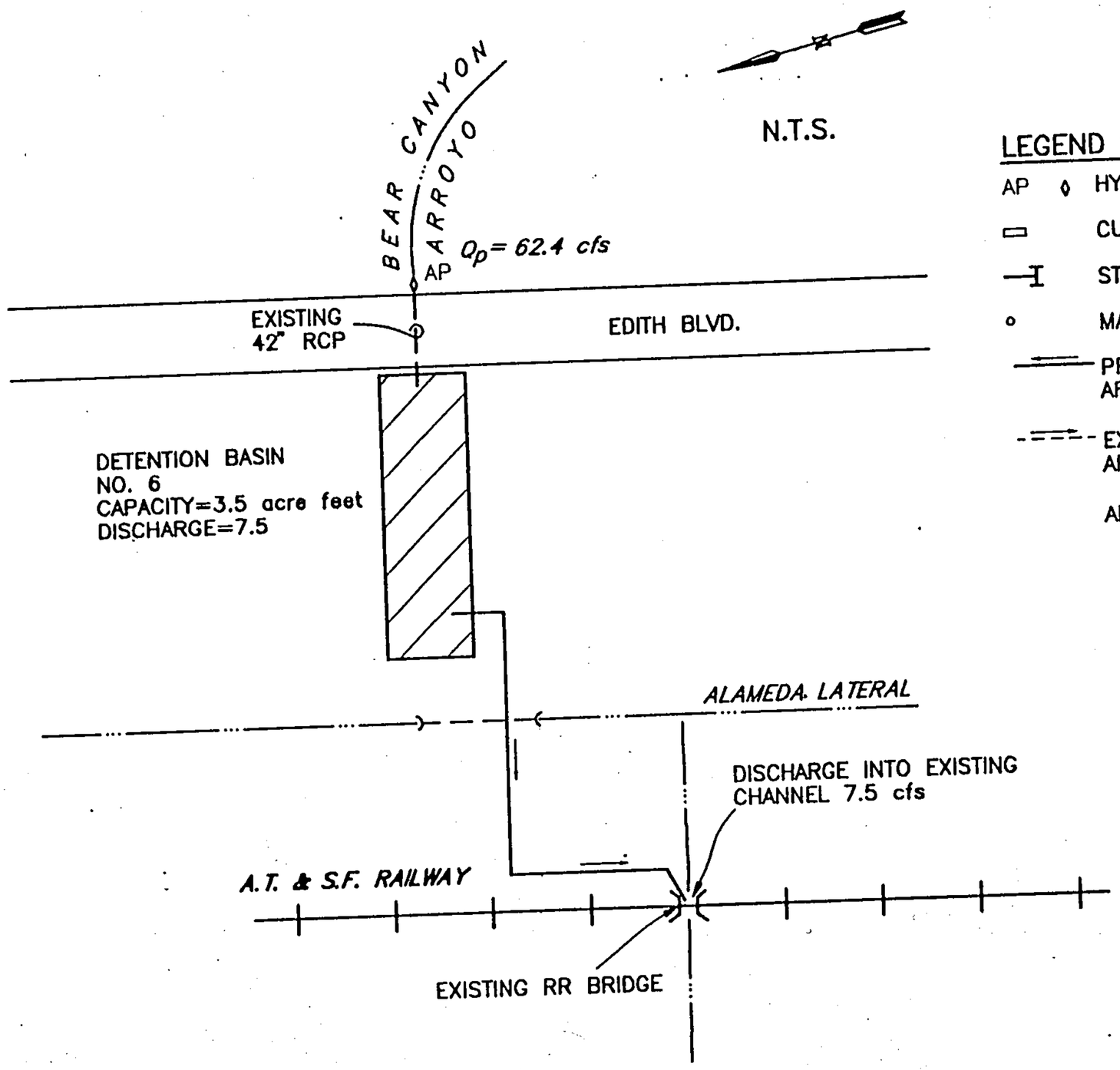
Figure 12



BOYLE ENGINEERING CORPORATION

SYSTEM 13

Figu
13



LEGEND

- AP ◊ HYDROLOGIC ANALYSIS POINT
- ▭ CURB DROP INLET
- —I STORM DRAIN STUB OUT
- ○ MANHOLE
- ═══ PROPOSED STORM DRAIN ARROW SHOWS DIRECTION OF FLOW
- -=-=- EXISTING STORM DRAIN ARROW SHOWS DIRECTION OF FLOW

ALL FLOWS ARE IN cfs

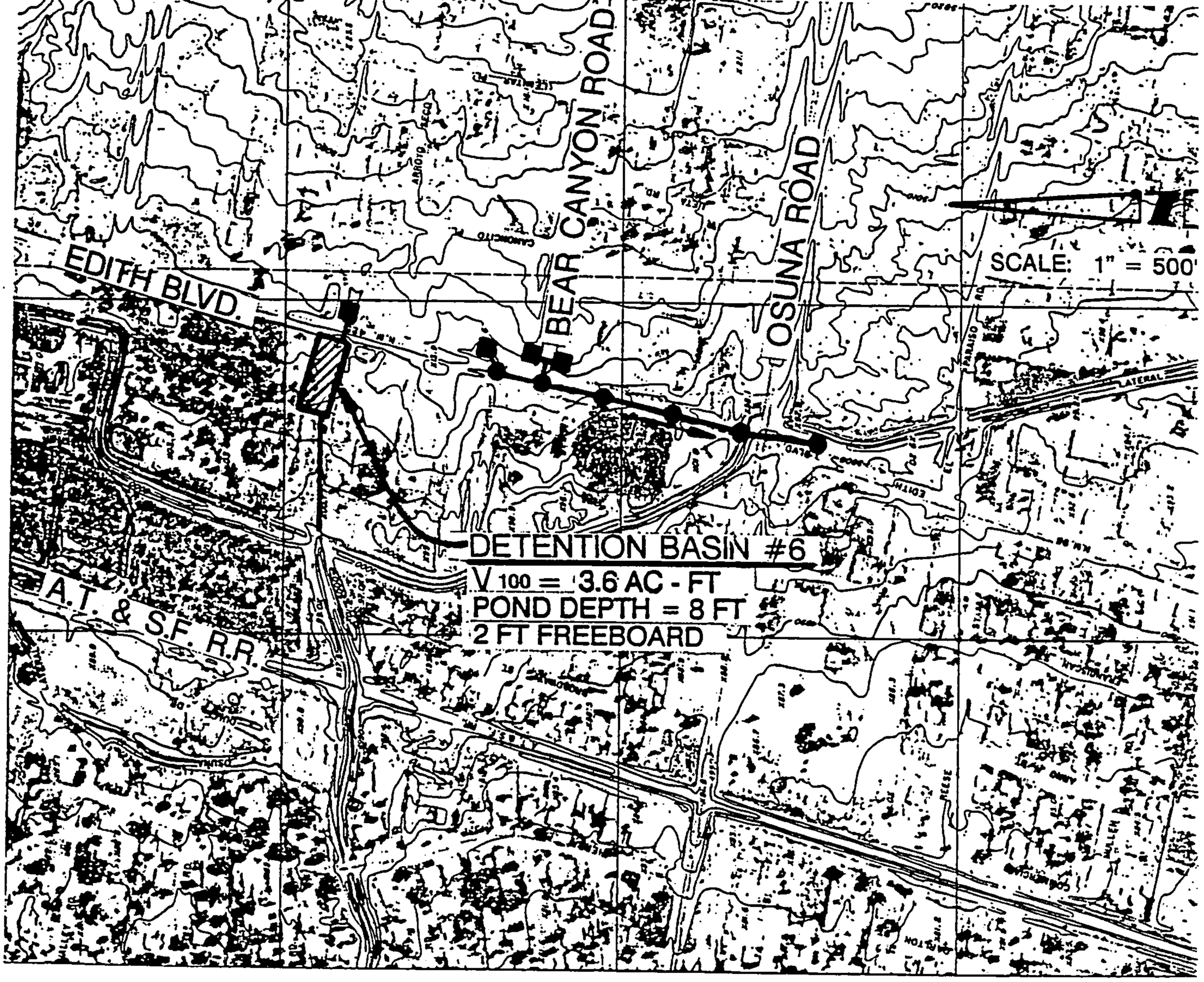


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PLATE IIIA

EDITH BLVD.
DRAINAGE STUDY
PROPOSED DRAINAGE PL

SCALE: 1" = 500'

DRAINAGE STUDY BOUNDARY

EXISTING GRAVEL PIT

GRAVEL PIT

SUBBASIN BOUNDARY

13

12B

12CS

12A

OSUNA ROAD

EDITH BLVD

DOES NOT CONTRIBUTE TO EDITH BLVD. SYSTEM

Boyle Engine Corpora

PLATE IA

EDITH BLVD DRAINAGE STU

SUB-BASIN MA

OFFSITE DRAINAGE AREAS

Basin 14 Area = 30.7 ac (.048 sq mi)

	A	B	C	D
Existing Land Treatment	40	20	30	10
Future Land Treatment (@ 5 DU/ac)	0	20	30	50

Length = 2400' Existing V = 1.4 fps Existing Tp = .32

Slope = .019 ft/ft Future V = 2.8 fps Future Tp = .16

	Qpeak	Volume
Existing Condition	37 (cfs)	2.23 (ac-ft)
Future Condition	98 (cfs)	4.21 (ac-ft)

Basin 15 Area = 38.4 ac (.060 sq mi)

	A	B	C	D
Existing Land Treatment	40	20	30	10
Future Land Treatment (@ 5 DU/ac)	0	20	30	50

	Qpeak	Volume
Existing Condition	47 (cfs)	2.78 (ac-ft)
Future Condition	123 (cfs)	5.26 (ac-ft)

EDITH BOULEVARD DRAINAGE

S = .0053 ft/ft

Find max street flow assume 48' FF street
From DPM Plate 22.3 D-3 Q = 2 x 26 cfs
Q = 52 cfs

Find storm drain diameter required to intercept additional flow.

Existing condition pipe flow = 84 - 52 = 32 cfs
Future condition pipe flow = 220 - 52 = 168 cfs

Dia = 36" $Q = \frac{1.49}{.015}\left(\frac{3}{4}\right)^{2/3}\frac{\pi}{4}3^2\sqrt{.005}$

Existing Condition
Q_{36} = 41 cfs

Dia = 42" $Q = \frac{1.49}{.015}\left(\frac{3.5}{4}\right)^{2.3}\frac{\pi}{4}3.5^2\sqrt{.005}$

Q_{42} = 62 cfs

Dia = 60" $Q = \frac{1.49}{.015}\left(\frac{5}{4}\right)^{2/3}\frac{\pi}{4}5^2\sqrt{.005}$

Future Condition
Q_{60} = 160 cfs

Dia = 66" $Q = \frac{1.49}{.015}\left(\frac{5.5}{4}\right)^{2/3}\frac{\pi}{4}5.5^2\sqrt{.005}$

Q_{60} = 206 cfs

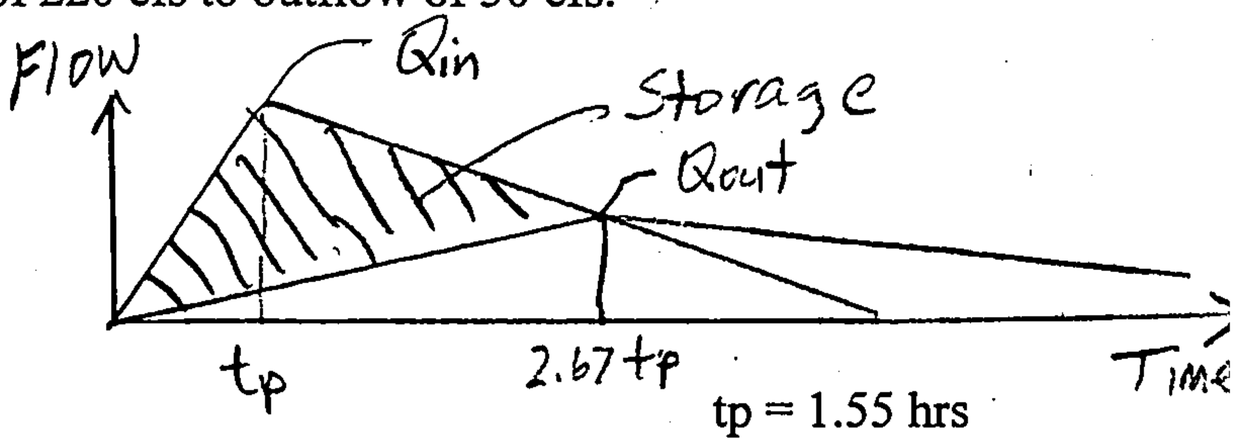
Find required detention volume to reduce inflow of 220 cfs to outflow of 50 cfs.

Storage (s) = ½ (2.67 tp) [Q_{In} - Q_{out}]

S = ½ (2.67) (5580) [220-50]
S = 1266381 ft^3
S = 29 ac-ft

assume average depth = 10'

⇨ area = 2.9 acre



tp = 1.55 hrs
= 5580 sec
(from AHYMO Results)

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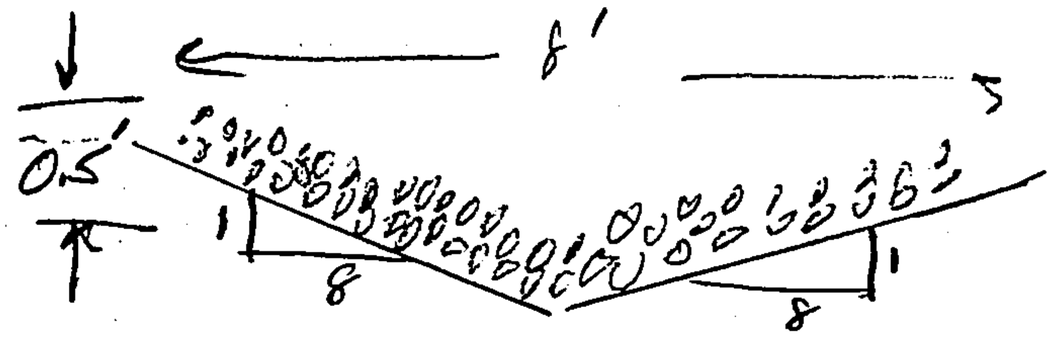
SHEET _____ OF _____
BY JD DATE 12/27

Design of Roadside Swale (Section A-A)



Assumptions:

Max Slope = 2%

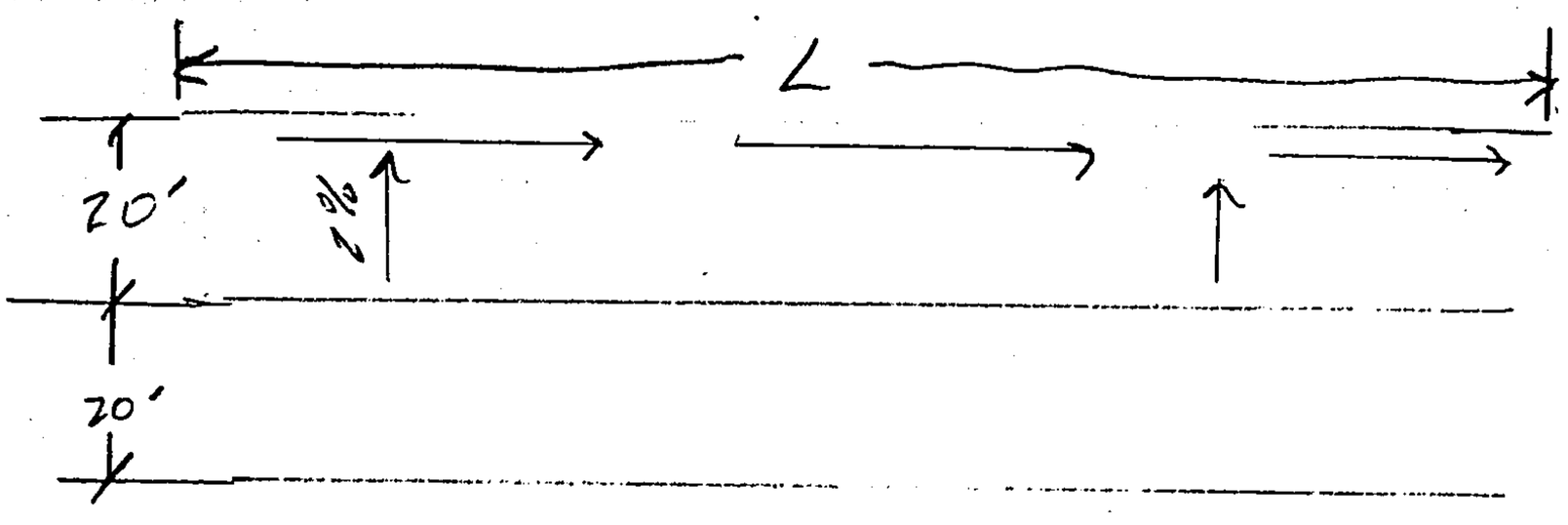
Q max = 1.5 cfs

1.5" Rock Lining

HEC-15 Run Results:

Q = 1.5 cfs
s = .02 ft/ft
n = .06
V = 1.2 fps
shear = 0.49 lb/sf
allowable shear = 0.50 lb/sf → STABLE

depth = 0.39'



At 100-yr; 6-hr storm, find length of roadway (L) that contributes 1.5 cfs

Assume: 100% "D" 20' wide street

use 4.7 cfs/acre (DPM Table A-9)

$$\frac{1.5 \text{ cfs}}{4.7 \text{ cfs/acre}} = 0.319 \text{ acre} = 13902 \text{ ft}^2$$

$$\frac{13902 \text{ ft}^2}{20 \text{ ft}} = 695 \text{ ft}$$

USE L = 700 ft

this will increase > 1.5 cfs

Road side Swale Analysis (section A-A)

CHANNEL LINING ANALYSIS

HEC-15 OUTPUT

Channel Bottom Width (ft)	Side Slope Lt. (Horz. to 1)	Side Slope Rt. (Horz. to 1)	Channel Slope (ft/ft)
0.00	8.0	8.0	0.020

Lining	Permissible Shear (lb/sf)	Discharge (cfs)	Hydraulic Radius (ft)	Manning Coefficient
1.5" Riprap	0.50	1.5	0.19	0.060

Normal Depth (ft)	Area (sf)	Velocity (ft/sec)	Calculated Shear (lb/sf)	Remarks
0.39	1.27	1.17	0.49	Stable

****

TYPICAL CREEK Section B-B

DESIGNER: DBD

DATE: 10-10-1996

PROJECT: HARROUN

PROJECT NO.: 96-200

STATION:

TC STATION:

DRAINAGE AREA: 0.0 Acres

DESIGN FREQUENCY: 0 Years

CHANNEL DESCRIPTION:

CHANNEL SLOPE: 0.018 ft/ft

Bottom Width: 1.00 ft

Left Side Slope: 4.0

Right Side Slope: 4.0

Lining	Permissible Shear (lb/sf)	Discharge (cfs)	Hydraulic Radius (ft)	Manning Coefficient
[redacted]	[redacted]	27.0	0.75	0.060

Normal Depth (ft)	Area (sf)	Velocity (ft/sec)	Calculated Shear (lb/sf)	Remarks
1.44	9.72	2.76	1.62	Stable

(1) Check for channel bends

CHANNEL LINING ANALYSIS

Rundown for Emergency Spillway Section H-H

Channel Bottom Width (ft)	Side Slope Lt. (Horz. to 1)	Side Slope Rt. (Horz. to 1)	Channel Slope (ft/ft)
0.00	10.0	10.0	0.250

Lining	Permissible Shear (lb/sf)	Discharge (cfs)	Hydraulic Radius (ft)	Manning Coefficient
6" Riprap	2.00	38.0	0.47	0.108

Normal Depth (ft)	Area (sf)	Velocity (ft/sec)	Calculated Shear (lb/sf)	Remarks
0.95	9.11	7.17	14.80	Not Stable

Wire Enclose

COMPUTING APPROXIMATE SEDIMENT BULKING FACTORS FOR HYDROLOGIC USE IN NORTH ALBUQUERQUE ACRES CITY OF ALBUQUERQUE, NEW MEXICO

PREPARED FOR:

ALBUQUERQUE METROPOLITAN ARROYO FLOOD CONTROL AUTHORITY

PREPARED BY:

Resource Technology, Inc.
ENGINEERS AND ENVIRONMENTAL SCIENTISTS
2129 Osuna Rd. NE, Suite 200, Albuquerque, NM 87113
Telephone (505) 345-3115
Fax (505) 345-4132

RTI PROJECT NO. 95-190

AUGUST 1996

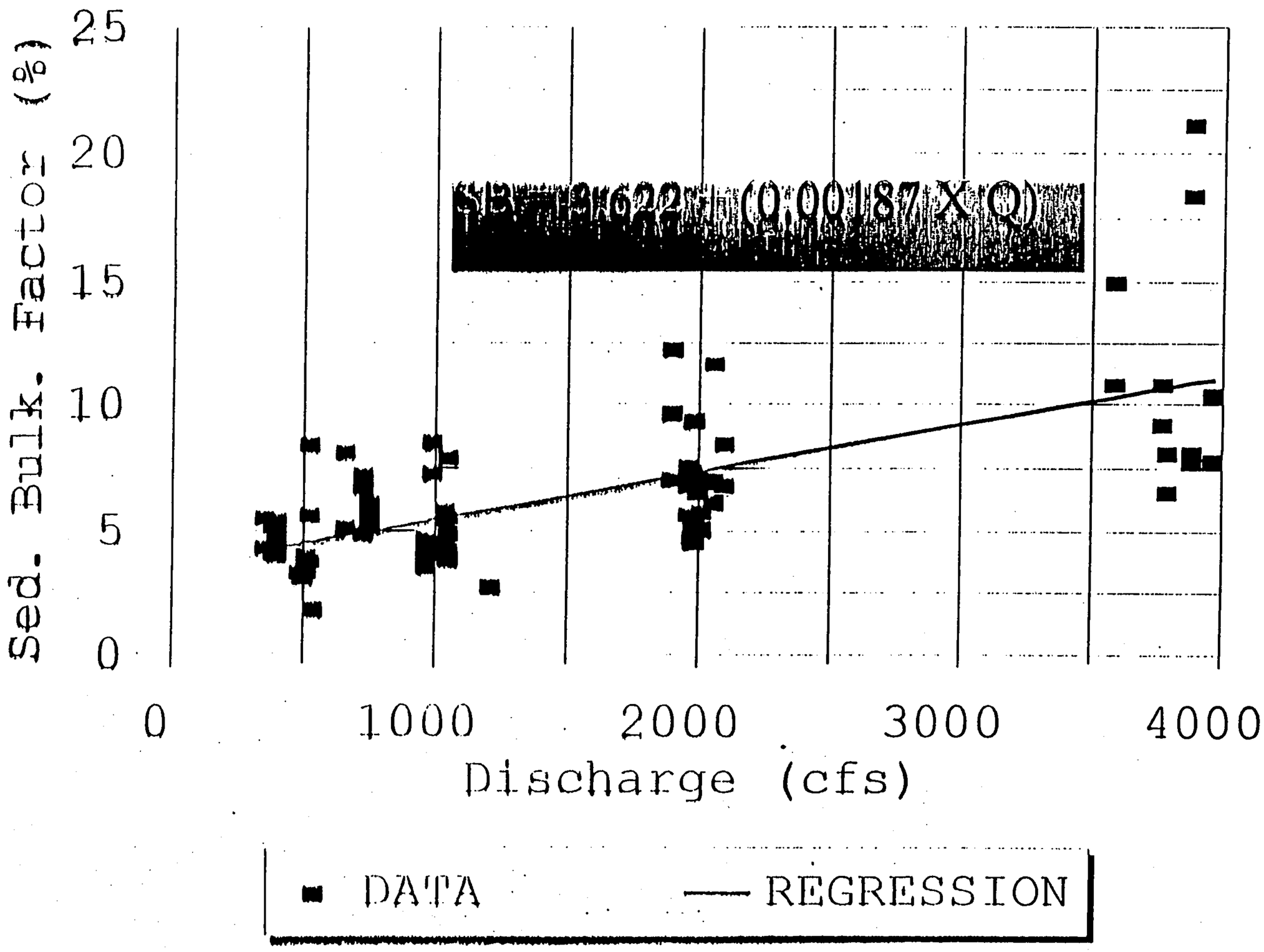


Figure 1. Discharge vs. Average Sediment Bulking Factors From Final Regression Equation

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SHEET _____ OF _____
BY _____ DATE 1/9/97

On-site Sediment Balting Estmate

$$SB = 3.622 + [0.00187\ Q]$$

Assume $Q = 20$ cfs

$$SB = 3.622 + [0.00187\ (20)]$$

$$SB = 3.66\ \%$$

use SB. = 5% as a conservatie estmate

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BY MD DATE _____

1.854 ac-ft = Ten Day ; 100-YR Volume

$1.854 \times 43560 = \underline{80760\ ft^3}$

$24\text{-hrs} \times 3600 = \underline{86400\ seconds}$

$$\text{Average Discharge} = \frac{80760}{86400} = \boxed{0.93\ cfs \sim 1\ cfs}$$

Orifice Analysis $Q = C_d A \sqrt{2gh}$

Assume Average $h = 2.0$ ft, $A = \pi/4\ D_h^2$

$Q = 0.6\ \pi/4\ D^2 \sqrt{2(32.)2}$

Q (cfs)	Dia (inches)
0.33	3
0.58	4
0.93	5
1.34	6