City of Albuquerque
P.O. BOX 1293 ALBUQUERQUE, NEW MEXICO 87103

December 15, 1995

Thomas J. Bellon
Community Science Corp
P.O. Box 1328
Corrales, NM 87048

RE: DRAINAGE REPORT FOR TUSCANY SUBD, UNITS 1 & 2 & 3 (A-11/D1)
RECEIVED NOVEMBER 22, 1995 FOR R GRADING, W.O. & FINAL PLAT
ENGINEER'S STAMP DATED 11-10-95

Dear Mr. Bellon:

Based on the information included in the submittal referenced above, City Hydrology accepts the Drainage Report for Rough Grading, Work Order & Final Plat.

The analysis is adequate for the Master Plan of Units 1, 2 & 3 and the specific design of Unit 1. Any use of this Report by Units 2 & 3 or Off-site areas will be dependent on demonstrating the applicability of the data to site specific conditions at the time of development. It is understood that the Consultant has checked the practicality of the two diversions and that the Owners of these off-site properties agree to the diversions.

If you have any questions about this project, You may contact me at 768-2727.

Sincerely,

John P. Curtin, P.E.
Civil Engineer/Hydrology

c: Andrew Garcia
Fred Aguirre, DRB 95-41
Kurt Browning, AMAFCA
Stan Strickman, Curb West Inc, 6301 Indian School NE #680, 87110

City of Albuquerque
P.O. BOX 1293 ALBUQUERQUE, NEW MEXICO 87103

August 21, 1995

Pat Conley, PE
Smith Engineering
6400 Uptown NE Suite 500E
Albuquerque, NM 87110

RE: DRAINAGE REPORT FOR McMAHON BLVD (K-11/D1)
RECEIVED AUGUST 18, 1995 FOR WORK ORDER
ENGINEER'S STAMP DATED 8/25/95

Dear Mr. Conley:

Based on the information included in the submittal referenced above, City Hydrology accepts the Drainage Report for Work Order.

If you have any questions about this project, You may contact me at 768-2727.

Sincerely,

John P. Curtin

John P. Curtin, P.E.
Civil Engineer/Hydrology

c: Andrew Garcia
Kurt Browning, AMAFCA
Doug Hughes, Community Science Corp, PO Box 1328, Corrales NM 87048
Stan Strickman; Curb West, Inc.; 6301 Indian School NE #680; 87110

REVISED/AMENDED

DRAINAGE REPORT

FOR

THE TUSCANY SUBDIVISION,
UNITS I, II, & III (MAP # A11 & A12)
(FINAL DRAINAGE REPORT FOR TUSCANY UNIT I)

PREPARED FOR

CURB WEST, INC.
6301 INDIAN SCHOOL NE, # 680
ALBUQUERQUE, NEW MEXICO 87109

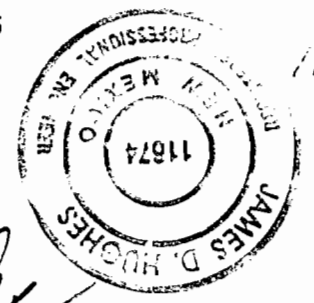
PREPARED BY

COMMUNITY SCIENCES CORPORATION
P. O. BOX 1328
CORRALES, NEW MEXICO 87048

THOMAS J. BELLON, JR.
PROJECT MANAGER

NOVEMBER 10, 1995

11-10-95



James D. Hughes, P.E.

P.O. Box 1328
Corrales, New Mexico 87048
Tel (505) 897-0000 Fax (505) 898-5195

CSC#291-07-695/291-07BA.REP

SURVEYING
LAND PLANNING
CIVIL ENGINEERING
DEVELOPMENT CONSULTANTS

TABLE OF CONTENTS

I. PURPOSE AND SCOPE

Curb West, Inc. is currently planning for the development of The Tuscany Subdivision, Units 1, 2, and 3. The proposed development consists of approximately 50.9 acres and is to be subdivided into 200 single family residential lots and a park site.

The original report presented a Preliminary Drainage Management and Conceptual Grading Plan for approval by the City of Albuquerque in order that subsequent subdivision and development may commence.

This Revised Report presents a Drainage Management concept for the area between future Unser Boulevard and Bandelier Drive and from McMahon Boulevard to the Calabacillas Arroyo. It also presents the final design for drainage purposes for Unit #1 of the Tuscany Subdivision. As a part of this report the Final Grading Plans and Drainage Calculations for Tuscany Unit #1 are supplied.

II. SITE DESCRIPTION AND HISTORY

The project site is located on the north side of the Calabacillas Arroyo, between the Arroyo and McMahon Blvd. (to 500' ± South of the McMahon right-of-way) and from Bandelier Drive, on the east, to Hillside St., NW in the Paradise Heights Unit 2 Subdivision on the west (see Plate 1 - Vicinity Map).

The Tuscany Subdivision, Units 1, 2, and 3 is comprised of Tracts 6A, 8A, and 9A of Paradise North and a portion of Bandelier Dr., NW, situated within the town of Alameda Grant, "projected" sections 11 and 12, T11N, R2E, N.M.P.M.,City of Albuquerque, Bernalillo County, New Mexico.

The existing terrain slopes from the ridgeline, at about the McMahon Blvd. right-of-way to the Calabacillas Arroyo. The site shows several terraced areas at progressively higher elevations moving North away from the Calabacillas Arroyo, they are coincident with the Rio Grande Rift uplift periods. The lower half of site, south of Bandelier Dr., is flat and above the 100yr. floodplain, the upper portion, north of Bandelier Dr., is steeper (15% ± avg.). The Calabacillas Arroyo is at or near the southerly boundary of the project. The prudent line, addressed in another report, for this property is the Simons and Li line established about 1983.

The site has previously been subdivided for single family homes, but not built. The site was restored to bulk land in the current tracts 6A, 8A and 9A of Paradise North. The zoning for the site is R1 for Tracts 6A and 8A and RT for Tract 9A.

Drainage for this site has always been toward the Calabacillas. The Natural Ridge Line that runs approximately along the McMahon Blvd. right-of-way, from Bandelier Dr. to about 1000' west of Bandelier Dr., then the ridgeline moves south of McMahon Blvd. crossing through the Lands of Lincoln, Paradise Heights, Unit 2 and Tracts 13A-1 and 15A-1 of Paradise North (and several other smaller properties). This ridgeline demarks the split between the Calabacillas Arroyo Drainage Basin and the Black's Arroyo Drainage Basin. Research of the Black's Arroyo drainage plan show that the ridgeline is the dividing point. Ultimately from Unser Blvd. to Bandelier Dr., McMahon Blvd. will likely be the future separation between the Calabacillas' and Black's Drainage Basins, at least that is the logical premise we have taken in this report. Earlier subdivision plans for this area offered alternate, but similar drainage solutions.

III. DESIGN CRITERIA

A. Flood Control Regulations

The drainage plan presented in this report has been designed to comply with AMAFCA resolution 80-15, which requires that proposed land development projects be designed such that no flooding of private properties will occur during any storm up to and including the 100-year frequency event. Additionally, this drainage plan has been designed to comply with current "City of Albuquerque Drainage Ordinance" and Chapter 22 of the Development Process Manual (DPM), and subsequently adopted general policies of the City of Albuquerque.

1. 100-year storm:

   a. Stormwater flow depth not to exceed the top of curb in any street.

   b. Jump depth to be contained within right-of-way.

2. 10-year storm:

   a. Local street - velocity times depth less than 6.5.

   b. Arterial streets:

      i. Flow not to exceed a depth of 0.50.

      ii. Velocity times depth less than 6.5.
      iii. One driving lane in each direction free of stormwater.

B. Engineering Parameters

In accordance with AMAFCA criteria, all hydrological analysis is based on the 100-year frequency, 6-hour duration storm, as represented in Section 22.2, Hydrology, of the "Development Process Manual, Volume 2, Design Criteria for the City of Albuquerque, New Mexico, January 1993.

Ten-year, 6-hour values were also used for subcatchments, in accordance with City drainage policies regarding street flow.

The four rainfalls pertinent to the study are as follows:

	10-Year	100-Year
One-Hour	1.27 "	1.90 "
Six-Hour	1.47 "	2.20 "

IV. COMPUTATIONAL PROCEDURES

The analysis approach follows standard engineering practice. Key points of confluence were selected and the associated individual and aggregate contributing basins were subsequently defined.

Hydrological computations were accomplished by means of the January 1994 version of AHYMO Computer Program as developed by AMAFCA. The input parameters and resulting flows for the basins are summarized on Table 1. Summary and detailed input AHYMO printouts are contained in Appendix A. (Detailed AHYMO output provided on request).

Times of concentration were estimated using the Upland Method and then converted to times to peak (Lg), in accordance with the above referenced Section 22.2 which also establishes the minimum time of concentration as 12 minutes.

Flow characteristics for conveyance swales, channels, and streets were analyzed based on the Manning equation for uniform flow. Streets are assumed to have a 2% cross slope from lip of gutter to crown and curb and gutter per City of Albuquerque Standard details. Finished grade at the right-of-way is 0.33' above top of curb.

V. OFF-SITE DRAINAGE

Off-site Drainage was not revised very much, but due to some changes (i.e. street name changes), a revision will follow.

Off-site drainage for this project takes on 3 forms: (1) off-site areas actually tributary to the project site under natural conditions; (2) off-site areas that will or may become tributary to the project site as a result of future development, by this developer or by others; and (3) off-site areas tributary to the off-site infrastructure required for this project, specifically McMahon Blvd. from Bandelier Dr. to Golf Course Rd.

Under natural/existing conditions off-site basins # 130, # 155, # 110, and # 115 partially flow to Black's Arroyo, due to the existing ridgeline, and the balance flows to the Calabacillas Arroyo. The portions of the above basins that drain toward the Calabacillas do so in a southeasterly direction, due to the existing terrain some of this flow ultimately crosses into the project site.

Under existing conditions, off-site basins # 140, # 135, # 145, # 150, # 165, # 175 and # 292combine with the partial flows from basins mentioned in previous paragraph and flow toward the project site and the arroyo. Provision for an earthen dike to direct water to a temporary pond with overflow to the Calabacillas in basin # 152 will take most of these flows to the arroyo before entering the project site. Basins # 155, # 165 and #175 will be trapped in a temporary pond near the Sorrento Drive (future) and Bandelier Drive intersection, on the south side.

When the east segment of Bandelier is developed basins # 130 and # 140 flows will combine with basins # 135 and # 145 in Bandelier Drive and those flows will be conveyed, in inlets via future storm drain systems through basins # 292 and # 152, combining with their flows, to the Calabacillas Arroyo. An outfall will be provided upstream of existing Grade Control Structure #1. Half of basin # 155 will be picked up in future inlets and connected to future storm drain system through basin # 152. **

Off-site basins, when developed, half of # 155, # 165, # 175, and # 150 will combine flows with basins # 160, # 170, and # 180 in Bandelier Dr. and capacity, to be picked up by a proposed storm drain constructed with this project, will be provided. In the interim natural condition a temporary ponds will be provided and possibly temporary pipes will be provided to pick-up runoff.

Off-site basins # 110 and # 115 are located in the grading borrow site for this project. Drainage from these basins will surface flow to retention pond and a temporary pipe will be provided to intercept drainage (with Unit 3), this pipe will connect to storm drain system in basin # 125 and convey drainage to the Calabacillas Arroyo. When basins # 110 and # 115 are developed, the storm drain system, proposed with this project, will provide capacity for these areas. (Developed Q's).

Off-site basins # 225, # 210, and # 205 slope toward the project site and the existing runoff flows through the project site to the Calabacillas Arroyo with this project's development will be provision to intercept these flows in a drainage bench and surface drain to inlets that would pick up the flows and convey them via pipes and outlet to the proposed streets, and thence the on-site storm drain system will pick-up these flows.

Off-site drainage from basins # 300, # 305, # 190 and # 310 will be collected in McMahon Blvd. and conveyed to Bandelier Dr., and then taken into Bandelier Dr. and ultimately picked up by inlets of the proposed on-site storm drain system. (As shown on Proj. No. 5208.90) (See comparison letter in Appendix)

Off-site basins # 315, # 325, # 335, # 295, # 330, # 350, and # 320 will flow to sump points along McMahon Blvd. and be collected in inlets and conveyed via pipe under McMahon Blvd. and re-outletted in basin # 360, thence to flow to the Calabacillas Arroyo. Future development of basin # 360 will provide storm drain system to intercept the re-outletted flows, previously discussed above, and convey them to the arroyo. (As per Prog. No. 5208.90) (See comparison letter in Appendix)

Off-site basins # 340, # 345, and # 355 will surface flow in McMahon Blvd. right-of-way to Golf Course Rd. Existing flows for these basins follow this pattern and future flows will continue in same pattern.

Off-site basins # 105, # 100, 100.1, # 185 and 185.1, under existing conditions, flow to Black's Arroyo. When McMahon Blvd. is ultimately developed the flows from these basins will be conveyed on the surface to off-site basin # 115 and combine flows. These flows will be collected in a future storm drain and connected to the proposed storm drain system of this project. This project is providing capacity for pick-up of these off-site flows in the design of the on-site storm drain system. Approximately 40cfs will be allowed to bypass future inlets in McMahon Blvd. and flow to East to join with flows in McMahon at Bandelier Drive. Future development of these sites may require some on-site ponds. (to detain/retain Q's above the existing conditions). Basins # 100.1 and # 185.1 were added to match Smith Engineering Company Report for Proj. No. 5208.90. (See attached letter comparing reports) ***

VI. ON-SITE DRAINAGE

On-site Drainage was changed in several ways (i.e. relocation of low point in Bandelier Drive). This revision will reflect both the ultimate drainage management design for Tuscany Units 1, 2, and 3 and the interim design for Drainage Management with the construction of Tuscany Unit #1. With subsequent units of Tuscany (Units #2 and #3), an addendum to this drainage report will be submitted.

On-site basins # 270, # 275, and # 280 will combine flows and be collected in a storm drain system located in basin # 280 and conveyed via pipe to the outfall structure near AMAFCA's existing Grade Control Structure #2. An overflow will be provided in case of any inlet blockage.

On-site Basin # 215, combined with flows from off-site Basins # 210 and # 205, will surface drain to Bandelier Drive, via street improvements, to combine with on-site Basin #195.

On-site Basin # 230, combined with portion of flows from off-site Basins # 225 and # 115, will surface drain via street improvements, to Bandelier Drive. There that flow will combine with Basin # 220 and the previously mentioned Basin # 195. IN Basin # 220, the first inlets in Bandelier Drive will be constructed. Basin # 200 (park site) will drain to a collection point in the park and be picked up by an inlet and conveyed via pipe to the storm drain system in Bandelier Drive in the Basin # 220 reach. Basin # 220 then combines with Basin # 235, the sump reach for Bandelier and the location of outfall #1 to the Calabacillas Arroyo.

On-site Basin # 125 (including flows from future off-site Basins # 100, # 100.1, # 185.1, # 185 # 105, and immediate off-site Basins # 110 and # 115) will be conveyed via a combination of surface flow in the street improvements and a storm drain system to be constructed with Unit #3. Storm drain system will be sized to include the off-site Basin flows.

On-site Basins # 160, # 170 and # 180 (combined with flows from off-site Basins # 155, # 165 and # 175) will be intercepted storm drain system to be constructed in Bandelier Drive and conveyed to arroyo at outfall #1.

On-site Basin # 265 combines with Basin # 180 and # 125, described above, at Bandelier Drive. Flows will be either in form of surface flow in street or captured flows in storm drain system. Flows then join Basin # 260 flowing east.

On-site Basin # 252 (including portion of lfows from off-site Basin # 115) will surface flow via street improvements to Bandelier Drive. There the flow from on-stie Basin # 252 will combine with on-site Basins # 255, # 240 (as described above) and # 250. Inlets in Bandelier drive will pick up a portion of this flow.

On-site Basin # 240 (including portion of flows from off-site Basins # 225 and # 115 (not picked up by on-site Basins # 230 or # 252)) will surface flow via street improvements to Bandelier Drive.

At on-site Basin # 235, flows from on-stie Basins # 250, # 240 and # 220 (all previously described in this report) will combine. These flows will be conveyed either in the storm drain system or as surface flow in Bandelier Drive. All flows culminate at the sump point located in Bandelier Drive between Sorrento Drive (Basin # 240) and Vecchio Drive (Basin # 230), and will be picked up by inlets to storm drain system and conveyed, via pipe, to the Calabacillas Arroyo. An overflow will be provided in case of any blockage to the sump inlets.

During construction of Tuscany Unit #1, and until Tuscany Units #2 and #3 are constructed, several temporary retention ponds will be provided. Two temporary retention ponds will be provided in Tract 10A-1 (off-site Basin # 152); one will collect flows from off-site Basins #130, # 140, # 135, # 145, # 292 and most of # 152; the other retention pond will take balance of off-site Basin # 152 and # 150. Another temporary retention pond will be provided for off-site Basins # 110 and # 115. Another temporary retention pond will be provided for off-site Basins # 300, # 310 and # 315. The only temporary retention pond to be constructed on-site will be placed near Sorrento Drive, on north side of Bandelier Drive, and will collect flows from off-site Basins # 155, # 165, # 175, # 225, # 210 and # 205 along with flows from on-site basins # 125, # 252, # 240 # 230 and # 215. All of these temporary retention ponds will be designed per DPM criteria. (Calculations and descriptions in this report).

When Unit #2 of Tuscany is constructed, the on-site temporary retention pond wil be resized and relocated on to Unit #3. And when Unit #3 of Tuscany is constructed this remporary retention pond will once again be resized and relocated onto Lot #10 of Unit #3.

The off-site temporary retention ponds will be relocated and/or redesigned when those properties are developed. When the development design is underway, this information and supporting calculations will be part of the drainage report addendums for Tuscany Units 2 and 3.

Sedimentation, erosion, and "Prudent line" set-backs are addressed in several separate reports prepared by CSC (April, 1995 and addendum dated August, 1995), and Smith Engineers (dated 9/2/94 & 10/5/95). The recommendations of these reports has been and will be taken into account when final design plans are prepared. The prudent line concerns have been taken into account in the layout and preliminary grading of the proposed subdivision.

VII. EROSION CONTROL

Control of excessive soil erosion into City streets and drainage improvements during construction will be accomplished by use of temporary lot line, water-trap berms. These will be windrowed into place following mass grading operations and left in place until each home is constructed and sold. Plate 3 illistrates the dimensions of these berms, and they will be located along those boundaries of each lot which are common to City rights-of-way or public easements.

VIII. SCOUR

Scour was considered and calcualted per the "Sediment and Erosion Design Guide". Scour consideration was taken at the outfall and rip-rap structures proposed adjacent to the Calabacillas Arroyo. The design of final plans take into effect scour potential. (see calculations)

IX. TEMPORARY PONDS

Temporary Ponds were designed using DPM criteria. The Q's used for the sizing of these temporary ponds were for undeveloped conditions, no impervious areas, therefore the capacity is the volume for the 6 hour 100-year storm. (Calculations provided) The following is a summary of the calculated capacities needed:

POND #	Required Volume (cf)(calc)	DESIGN VOL (cf)
1	96,175 CF	120,000 cf
2	7,520 CF	30,000 cf
3	42,702 CF	76,000 cf
4	145,465 CF	150,000 cf
5	16,570 CF	20,000 cf

TABLE 1 (Revised)
ULTIMATE DEVELOPED CONDITION
(If all properties, both on-site and off-site, developed) TP=0.1330

					LAND TREATMENT				INCREMENTAL		FUTURE TOTAL	
Basin I.D.	Area (Sq.Mi.)	Contr. Basin	Sum Area (Sq.Mi.)	Tc (Min.)	A	B	C	D	Q_{100} (cfs)	Q_{10} (cfs)	Q_{100} (cfs	Q_{10} (cfs)
Future McMahon Boulevard (Unser to West Mesa Medical)												
100	0.0035	--------	0.0035	12	---	5	5	90	9.52		9.52	
100.1	0.0035	100	0.0070	12	---	5	5	90	9.52		19.05	
185	0.0024	--------	0.0024	12	---	5	5	90	6.53		6.53	
185.1	0.0024	185 & 100.1	0.0118	12	---	5	5	90	6.53		32.12	
105	0.0366	185.1	0.0484	12	---	21	22	57	84.33		116.45	
Off-site Q at McMahon Boulevard (105) Q_{100} = 116.5 cfs (future inlets to be designed for 76.5 cfs) *NOTE #1: 40 cfs to be allowed to bypass future inlets in future McMahon to join flows at Bandelier to east.												
115	0.0146	105	0.063	12	---	21	22	57	33.65		148.32	
110	0.0171	115	0.0801	12	---	21	22	57	39.41		186.38	
Off-site Q to Tuscany Drive (110) Q_{100} = 186.4 cfs (future inlets to be designed for 146.4 cfs)* AP#9												
125	0.0090	110	0.0891	12	---	29	29	42	19.03		208.08	
Q in Tuscany Dr at Bandelier Dr (125) Q_{100} = 208.1 cfs (future inlets provided with Unit #3 for 168.1 cfs)* AP#5												
155	0.0324	--------	0.0324	12	---	25	25	50	71.72		71.72	
Off-site Basins (future inlets to be designed for 35.9 cfs) 1/2 of flows 155												
160	0.0012	155	0.0336	12	---	5	5	90	3.28		74.99	
165	0.0086	160	0.0422	12	---	25	25	50	19.05		94.04	
175	0.0085	--------	0.0085	12	---	25	25	50	18.83		18.83 1/2.87	
NOTE #2: 1/2 of Basin #155 to join with Basin #150.												
170	0.0009	175 & 165	0.0516	12	---	5	5	90	2.46		114.38	AP#4
265	0.0025	170 & 125	0.1432	12	---	29	29	42	5.30		329.21	
180	0.0006	265	0.1438	12	---	5	5	90	1.65		330.79	
260	0.0005	180	0.1443	12	---	5	5	90	1.37		333.47	
255	0.0007	250	0.1550	12	---	29	29	42	1.49		334.93	
252	0.0090	250	0.1538	12	---	29	29	42	19.03		350.90	
250	0.0010	252	0.1543	12	---	5	5	90	2.73		353.96	
Q in Bandelier Drive at Sorrento Drive (255) Q_{100} = 350.94cfs (353.16)												
205	0.0006	--------	0.0006	12	---	15	15	70	1.50		1.50	
210	0.0027	205	0.0033	12	---	15	15	70	6.67		8.17	
215	0.0095	210	0.0128	12	---	29	29	42	20.08		27.24	
195	0.0022	215	0.0150	12	---	5	5	90	5.99		33.02	
220	0.0007	195	0.0157	12	---	5	5	90	1.92		35.23	
200	0.0013	--------	--------	--------	---	46	47	7	2.19		37.40	
Park Site - Q100 = 2.2 cfs to inlet in park												
1/2 Basin # 225 joins #230 (partial flow from #225 to Vecchio Drive)												
225	0.0038	--------	0.0038	12	---	5	5	90	10.34		10.34	
230	0.0063	225 & 220	0.0258	12	---	29	29	42	13.32		60.46	
1/2 Basin # 225 joins #240 (partial flow from #355 to Sorrento Drive)												
225	0.0038	--------	0.0038	12	---	5	5	90	10.34		10.34	
240	0.0077	225	0.0115	12	---	29	29	42	16.28		25.94	

Rev. 12-13-95 TP=0.1330

TABLE 1 (Revised)
ULTIMATE DEVELOPED CONDITION
(If all properties, both on-site and off-site, developed) TP=0.1330

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Future McMahon Boulevard (Unser to West Mesa Medical)												
100	0.0035	--------	0.0035	12	---	5	5	90	9.52		9.52	
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155	0.0324	--------	0.0324	12	---	25	25	50	71.72		71.72	
160	0.0012	155	0.0336	12	---	5	5	90	3.28		74.99	
165	0.0086	160	0.0422	12	---	25	25	50	19.05		94.04	
175	0.0085	--------	0.0085	12	---	25	25	50	18.83		18.83	
Off-site Basins (future inlets to be designed for 35.9 cfs)? From Basin 155 to 150												
170	0.0009	175 & 165	0.0516	12	---	5	5	90	2.46		114.38	
265	0.0025	170 & 125	0.1432	12	---	29	29	42	5.30		329.21	
180	0.0006	265	0.1438	12	---	5	5	90	1.65		330.79	
260	0.0005	180	0.1443	12	---	5	5	90	1.37		333.47	
255	0.0007	250	0.1550	12	---	29	29	42	1.49		334.93	
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Q in Bandelier Drive at Sorrento Drive (255) Q_{100} = 350.94cfs (353.16)?												
205	0.0006	--------	0.0006	12	---	15	15	70	1.50		1.50	
210	0.0027	205	0.0033	12	---	15	15	70	6.67		8.17	
215	0.0095	210	0.0128	12	---	29	29	42	20.08		27.24	
195	0.0022	215	0.0150	12	---	5	5	90	5.99		33.02	
220	0.0007	195	0.0157	12	---	5	5	90	1.92		35.23	
200	0.0013	--------	--------	--------	---	46	47	7	2.19		37.40	
Park Site - Q100 = 2.2 cfs to inlet in park												
1/2 Basin # 225 joins #230 (partial flow from #225 to Vecchio Drive)												
225	0.0038	--------	0.0038	12	---	5	5	90	10.34		10.34	
230	0.0063	225 & 220	0.0258	12	---	29	29	42	13.32		60.46	
1/2 Basin # 225 joins #240 (partial flow from #355 to Sorrento Drive)												
225	0.0038	--------	0.0038	12	---	5	5	90	10.34		10.34	
240	0.0077	225	0.0115	12	---	29	29	42	16.28		25.94	

TABLE 1 (Revised)
ULTIMATE DEVELOPED CONDITION (continued)
(If all properties, both on-site and off-site, developed) TP=0.1330

					LAND TREATMENT				INCREMENTAL		FUTURE TOTAL	
Basin I.D.	Area (Sq.Mi.)	Contr. Basin	Sum Area (Sq.Mi.)	Tc (Min.)	A	B	C	D	Q_{100} (cfs)	Q_{10} (cfs)	Q_{100} (cfs	Q_{10} (cfs)
240T	--------	230	0.1923	12	---	---	---	---	--------		86.95	
235	0.0007	255 & 240T	0.1943	12	---	5	5	90	1.92		435.09	
Total Q in Bandelier Drive (235) SUMP Q_{100} = 435.1 cfs: (359.2 cfs to outfall #1) (Reduction in Q_{100} to outfall #1 is result of Note #1 & Note #2 flow diversions)												
270	0.0096	--------	0.0096	12	---	27	27	46	20.78		20.78	
275	0.0076	270	0.0172	12	---	27	27	46	16.45		34.81	
280	0.0122	275	0.0294	12	---	27	27	46	26.40		57.55	
Total Q at Sump in Sicily Place Q_{100} = 57.6: to outfall # 2												
130	0.0277	--------	0.0277	12	---	25	25	50	61.62		61.62	
135	0.0017	130	0.0294	12	---	5	5	90	4.63		65.95	
140	0.0115	--------	0.0115	12	---	25	25	50	25.47		88.69	
145	0.0018	135 & 140	0.0427	12	---	5	5	90	4.91		93.38	AP#1
150	0.0007	145	0.0434	12	---	5	5	90	1.92		95.21	
152	0.0148	150	0.0582	12	---	25	25	50	32.77		116.70	
292	0.027	152	0.0852	12	---	25	25	50	59.77		175.99	
Total Q at temp. Off-site Pond #1 and #2 (292T) Q_{100} = 176.10 cfs (ultimate flows)(211.9 cfs to future outfall) The increase in Q at future outfall is a result of Note #2 diversion.												
305	0.0015	--------	0.0015	12	---	5	5	90	4.09		4.09	
190	0.0014	~~305~~	0.0029	12	---	5	5	90	3.82		3.82	
Q in McMahon Boulevard at Bandelier Drive (190)(N/S=24.1 cfs & S/S=23.8 cfs)*												
300	0.0018	~~305~~ 300	0.0018	12	---	5	5	90	4.91		9.00	
310	0.0012	300	0.0030	12	---	5	5	90	3.28		12.27	
315	0.0093	310	0.0123	12	---	25	25	50	20.60		32.87	
Q on Borrow Site - North of McMahon Boulevard (315) Temporary pond #5 Q_{100} = 32.9 cfs (ultimate)												
320	0.0021	315	0.0159	12	---	5	5	90	5.72		32.16	
295	0.0021	190	0.0035	12	---	5	5	90	5.72		7.97	
325	0.0030	320	0.0224	12	---	5	5	90	8.17		39.57	
Total Q at Inlets near Dover Street (325) Q_{100} = 39.6 cfs (ultimate) (N/S Q = 59.6; S/S Q = 28.0 cfs) Due to addition of future 40 cfs (see Note #1), the Q_{100} to inlets west of Dover is 87.6.												
335	0.0259	325	0.0483	12	---	25	25	50	57.33		88.92	
330	0.0012	335	0.0495	12	---	5	5	90	3.28		92.05	
350	0.0012	330 & 295	0.0507	12	---	5	5	90	3.28		102.11	
Total Q to Calabacillas in S.D. - McMahon Boulevard (350) Q_{100} = 102.1 cfs (ultimate) (only Q_{100} = 47.2 cfs at sump inlets) See Note #1 about 40 cfs bypass. (Total future flow to Calabacillas is 142.1 cfs) (2 inlets N/S west of Dover Q = 51.5 & S/S = 15.0 cfs) N/S inlet east of Dover = 13.0 cfs (N/S inlet of west Redbud = 13.0 cfs; N/S inlet east of Redbud = 25.5 cfs)												
340	0.0034	--------	0.0034	12	---	5	5	90	9.25		9.25	
355	0.0033	340	0.0067	12	---	5	5	90	8.98		18.23	
345	0.0032	355	0.0099	12	---	5	5	90	5.68		23.91	
Q from McMahon Boulevard to Golf Course Road (345) Q_{100} = 23.9 cfs (ultimate)												

Rev. 12-13-95

TABLE 1 (Revised)
ULTIMATE DEVELOPED CONDITION (continued)
(If all properties, both on-site and off-site, developed) TP=0.1330

Basin I.D.	Area (Sq.Mi.)	Contr. Basin	Sum Area (Sq.Mi.)	Tc (Min.)	LAND TREATMENT A	B	C	D	INCREMENTAL Q_{100} (cfs)	Q_{10} (cfs)	FUTURE TOTAL Q_{100} (cfs	Q_{10} (cfs)
240T	--------	230	0.1923	12	---	---	---	---	--------		86.95	
235	0.0007	255 & 240T	0.1943	12	---	5	5	90	1.92		435.09	
Total Q in Bandelier Drive (235) SUMP Q_{100} = 435.1 cfs: (359.2 cfs to outfall #1)?												
270	0.0096	--------	0.0096	12	---	27	27	46	20.78		20.78	
275	0.0076	270	0.0172	12	---	27	27	46	16.45		34.81	
280	0.0122	275	0.0294	12	---	27	27	46	26.40		57.55	
Total Q at Sump in Sicily Place Q_{100} = 57.6: to outfall # 2												
130	0.0277	--------	0.0277	12	---	25	25	50	61.62		61.62	
135	0.0017	130	0.0294	12	---	5	5	90	4.63		65.95	
140	0.0115	--------	0.0115	12	---	25	25	50	25.47		88.69	
145	0.0018	135 & 140	0.0427	12	---	5	5	90	4.91		93.38	
150	0.0007	145	0.0434	12	---	5	5	90	1.92		95.21	
152	0.0148	150	0.0582	12	---	25	25	50	32.77		116.70	
292	0.027	152	0.0852	12	---	25	25	50	59.77		175.99	
Total Q at temp. Off-site Pond #1 and #2 (292T) Q_{100} = 176.10 cfs (ultimate flows)(211.9 cfs to future outfall) +36 cfs from 155												
305	0.0015	--------	0.0015	12	---	5	5	90	4.09		4.09	
S 190	0.0014	305	0.0029	12	---	5	5	90	3.82		3.82	
Q in McMahon Boulevard at Bandelier Drive (190)(N/S=24.1 cfs & S/S=23.8 cfs) 8cfs + 40cfs = 48cfs												
300	0.0018	--------	0.0018	12	---	5	5	90	4.91		9.00	
310	0.0012	300	0.0030	12	---	5	5	90	3.28		12.27	
315	0.0093	310	0.0123	12	---	25	25	50	20.60		32.87	
Q on Borrow Site - North of McMahon Boulevard (315) Temporary pond #5 Q_{100} = 32.9 cfs (ultimate)												
320	0.0021	315	0.0159	12	---	5	5	90	5.72		32.16	
295	0.0021	190	0.0035	12	---	5	5	90	5.72		7.97	South
325	0.0030	320	0.0224	12	---	5	5	90	8.17		39.57	
Total Q at Inlets near Dover Street (325) Q_{100} = 39.6 cfs (ultimate) North Side												
335	0.0259	325	0.0483	12	---	25	25	50	57.33		88.92	
330	0.0012	335	0.0495	12	---	5	5	90	3.28		92.05	
S 350	0.0012	330 & 295	0.0507	12	---	5	5	90	3.28		~~102.11~~ 102.11	
Total Q to Calabacillas in S.D. - McMahon Boulevard (350) Q_{100} = 102.1 cfs (ultimate) (only Q_{100} = 47.2 cfs at sump inlets)(Total future flow at sumpis 142.1 cfs)												
340	0.0034	--------	0.0034	12	---	5	5	90	9.25		9.25	
355	0.0033	340	0.0067	12	---	5	5	90	8.98		18.23	
345	0.0032	355	0.0099	12	---	5	5	90	5.68		23.91	
Q from McMahon Boulevard to Golf Course Road (345) Q_{100} = 23.9 cfs (ultimate)												

TABLE 1A

TEMPORARY PONDS (UNIT 1 OF TUSCANY)*

EXISTING AND PROPOSED DEVELOPMENT CONDITIONS*

					LAND TREATMENT				INCREMENTAL		FUTURE TOTAL	
Basin I.D.	Area (Sq.Mi.)	Contr. Basin	Sum Area (Sq.Mi.)	Tc (Min.)	A	B	C	D	Q_{100} (cfs)	Q_{10} (cfs)	Q_{100} (cfs	Q_{10} (cfs)
Ponds #1 and #2 - off-site (West) (Borrow site(#150 and #152))												
130	0.0277	--------	0.0277		80	10	10	0	19.21		19.21	
135	0.0017	130	0.0294	--------	80	10	10	0	1.19		20.40	
140	0.0115	135	0.0409		80	10	10	0	7.98		23.38	
145	0.0018	140	0.4270		80	10	10	0	1.26		29.63	
150	0.0007	145	0.0434		60	20	20	0	0.57		30.20	
152	0.0148	150	0.0582		60	20	20	0	11.89		42.10	
292	0.0270	152	0.0852		80	10	10	0	18.73		60.82	
Pond #3 - off-site (Northwest) Borrow site												
115	0.0146	--------	0.0146		60	20	20	0	11.73		11.73	
110	0.0171	115	0.0317		60	20	20	0	13.74		25.47	
Pond #4 - on-site (at Sorrento and Bandelier)												
125	0.009	--------	0.009		70	15	25	0	6.74		6.74	
155	0.0324	--------	0.0324		80	10	10	0	22.47		22.47	
165	0.0086	155	0.0410		80	10	10	0	5.97		28.44	
175	0.0085	165 & 125	0.0585		80	10	10	0	5.90		41.08	
252	0.009	175	0.0675		80	10	10	0	6.25		47.32	
205	0.0006	--------	0.0006		80	10	10	0	0.42		0.42	
210	0.0027	205	0.0033		80	10	10	0	1.88		2.30	
215	0.0095	210	0.0128		80	10	10	0	6.59		8.90	
225.2	0.0038	--------	0.0038		0	5	5	90	8.52		8.52	
230	0.0063	225.2 & 215	0.0229		80	10	10	0	4.38		21.59	
225.1	0.0038	--------	0.0038		0	5	5	90	8.52		8.52	
240	0.0077	225.1 & 230	0.0344		80	10	10	0	5.35		35.35	
240T	----------	252	0.1019		---	---	---	---	--------		82.57	
Pond #5 - off-site (Northeast) Borrow site												
300	0.0018	--------	0.0018		60	20	20	0	1.45		1.45	
310	0.0012	300	0.0030		60	0	20	0	0.97		2.43	
315	0.0093	310	0.0123		60	20	20	0	7.48		9.90	

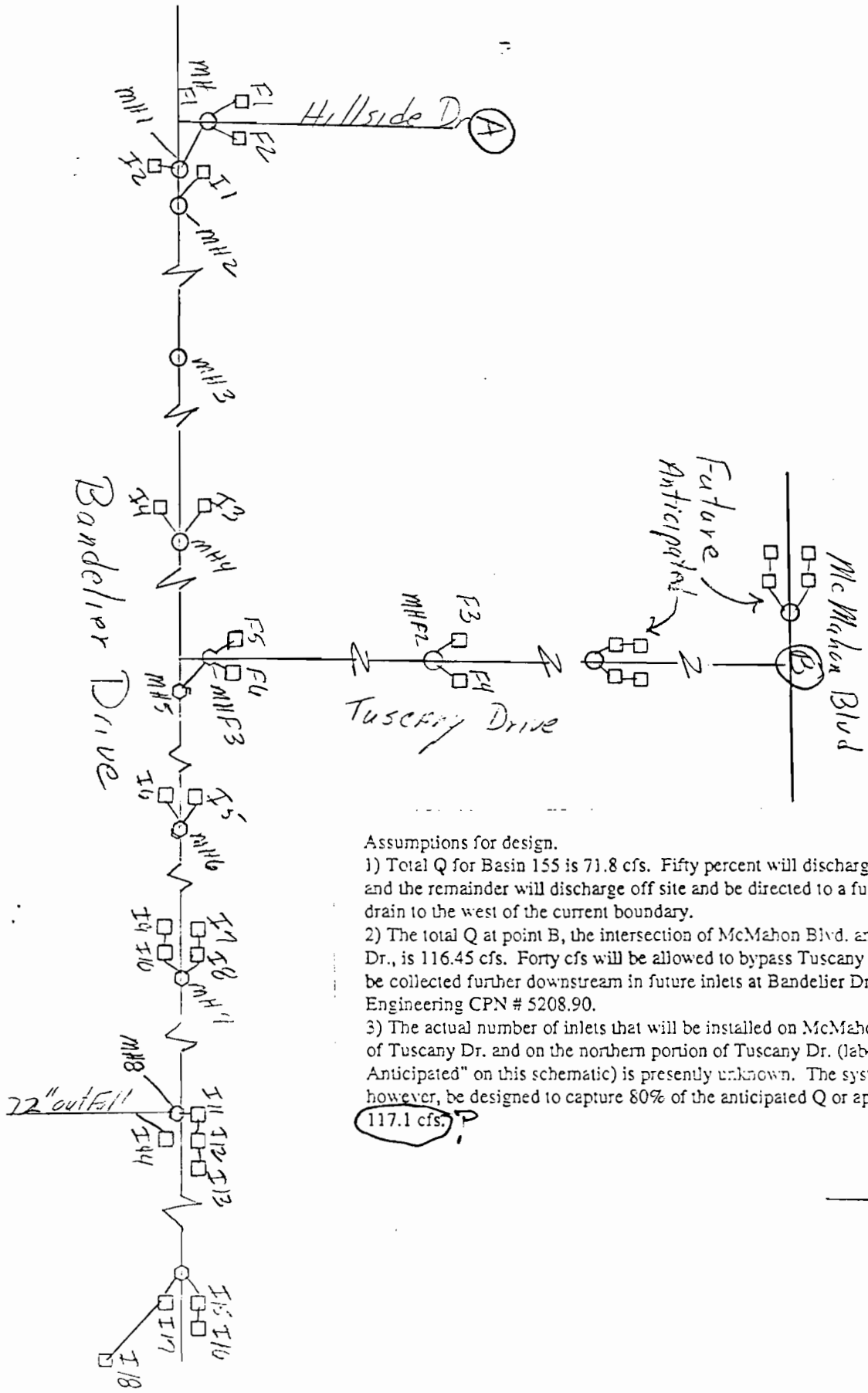
*No impervious areas considered. Tp=0.20 used.

TABLE 2

STREET FLOW CHARACTERISTICS

STREET	WIDTH	CURB TYPE	LOCATION	SLOPE %	Q	Dn	Dc	Vn	Vc	EG	F	*POOL DEPTH
BANDELIER	40 FT	STND	20+40	1.00	31.50	0.51	0.53	3.30	2.98	0.67	1.19	0.70
BANDELIER	40 FT	STND	27+50	2.28	58.10	0.54	0.67	5.40	3.63	0.99	1.84	0.93
BANDELIER	40 FT	STND	32+35	0.60	49.51	0.63	0.63	3.45	3.40	0.81	1.00	0.85
BANDELIER	40 FT	STND	34+95	0.06	57.93	0.66	0.67	3.61	3.63	0.87	1.01	0.93
BANDELIER	40 FT	STND	35+10	0.06	37.93	0.57	0.58	3.06	3.16	0.72	0.97	0.77
BANDELIER	40 FT	STND	40+45	1.93	33.00	0.46	0.54	4.18	3.03	0.68	1.62	0.72
MILANO	25 FT	MNT	14+00	0.50	6.54	0.28	0.30	1.71	2.05	0.35	0.78	0.38
PLACE NAPOLI	27 FT	MNT	16+00	0.50	9.00	0.33	0.31	1.88	2.21	0.39	0.80	0.40
PLACE NAPOLI	25 FT	MNT	16+50	2.75	6.00	0.22	0.27	3.03	2.00	0.37	1.91	0.35
PALERMO	25 FT	MNT	12+00	1.08	6.00	0.26	0.27	2.13	2.00	0.33	1.13	0.35
PALERMO	25 FT	MNT	14+00	3.14	9.00	0.25	0.31	3.51	2.26	0.44	1.95	0.41
SICILY CUL-DE-SAC	28 FT	STND	ALL	0.50	57.60	0.74	0.72	4.04	3.91	0.98	0.96	1.02
SICILY	27 FT	MNT	22+00	0.51	5.10	0.27	0.25	1.67	1.93	0.31	0.89	0.32
SICILY	28 FT	STND	12+00	0.51	18.85	0.48	0.46	2.53	2.80	0.58	0.87	0.61
SICILY	28 FT	STND	15+50	1.88	33.35	0.59	0.57	3.14	3.37	0.75	0.91	0.79
SICILY	28 FT	STND	20+00	0.51	54.50	0.72	0.70	3.83	3.97	0.95	0.95	0.95
SICILY	28 FT	STND	21+56	0.51	56.50	0.73	0.72	3.88	4.01	0.97	0.95	1.03
TUSCANY	27 FT	MNT	ALL	0.53	4.20	0.26	0.24	1.49	1.85	0.30	0.82	0.31

* POOL DEPTH = Dc + 1.25(Vc^2)/(2g)



Assumptions for design.

1) Total Q for Basin 155 is 71.8 cfs. Fifty percent will discharge at point A and the remainder will discharge off site and be directed to a future storm drain to the west of the current boundary.

2) The total Q at point B, the intersection of McMahon Blvd. and Tuscany Dr., is 116.45 cfs. Forty cfs will be allowed to bypass Tuscany Dr. and will be collected further downstream in future inlets at Bandelier Dr. per Smith Engineering CPN # 5208.90.

3) The actual number of inlets that will be installed on McMahon Blvd. west of Tuscany Dr. and on the northern portion of Tuscany Dr. (labeled as "Future Anticipated" on this schematic) is presently unknown. The system must, however, be designed to capture 80% of the anticipated Q or approximately 117.1 cfs.?

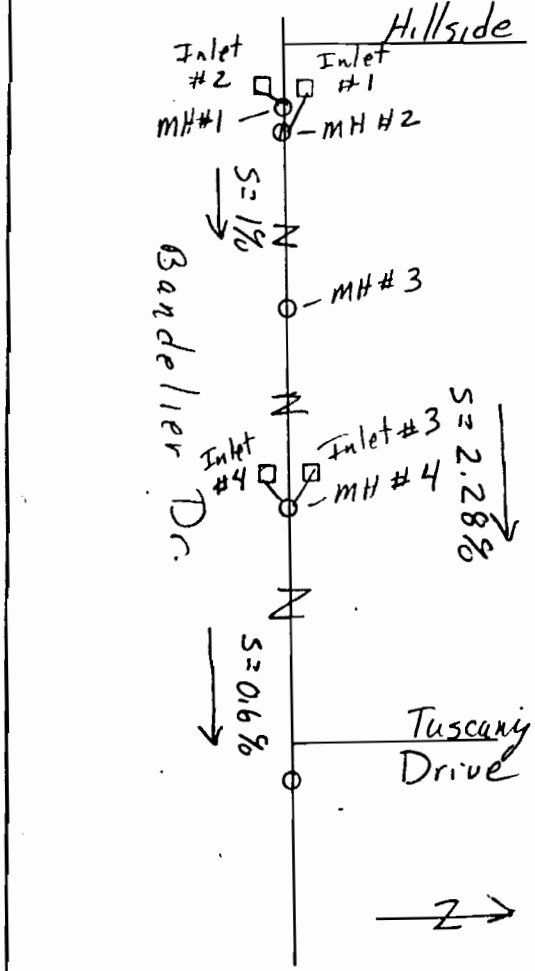
?

CATCH BASIN CALCULATION SHEET

Sht ___ of ___

PROJECT _____ CALCULATED BY _____

DESIGN FREQUENCY _____ DATE _____



Sym.	Drain. Area	Q Total	Q Inter.	Bypass ~~Cap. of~~ ½ Street	Gutter "d"	C.B. No.	C.B. Size	Connector Pipe Head	Connector Pipe L	Connector Pipe Dia.	V Depth
Inlets	1+2	will	take	flows	from	Hillside			Ave		
And	a maximum		of	10 cfs	from	West on	Bandelier.				
1	See	15.75	8	7.75	.51						
2	Above	15.75	8	7.75	.51						
Note:	Surface	flows	from			north		add	42.6 cfs		
	to	15.5	cfs	of	bypass.	Q @	inlets	3+4=	58.1		
3	165, 170, 175, 180	29.05	11.5	17.55	.54						
4		29.05	11.5	17.55	.54						
	Total	bypass	=	35.1	cfs.						

PLATE 22.3 D-10

22.3

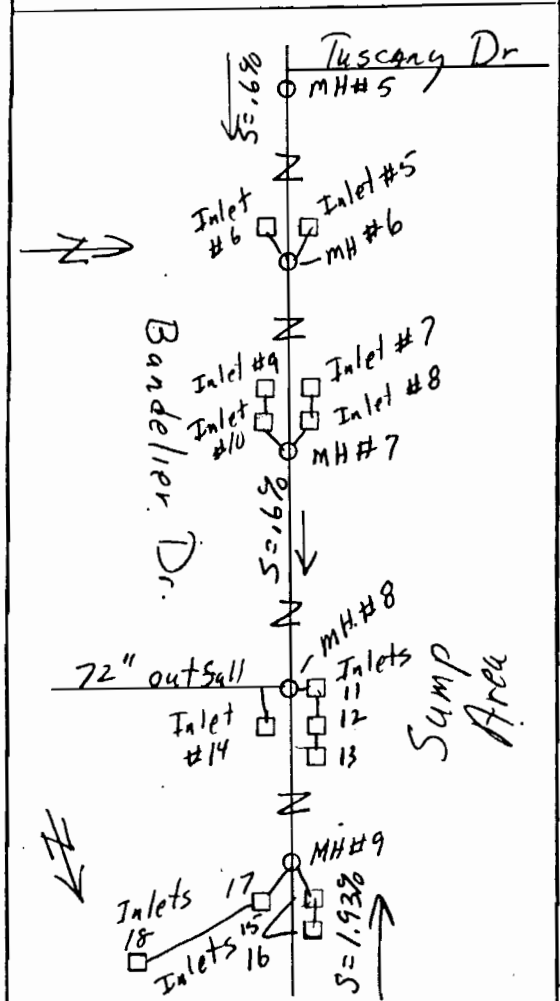
CATCH BASIN CALCULATION SHEET

Sht. ___ of ___

PROJECT _____ CALCULATED BY _____

DESIGN FREQUENCY _____ DATE _____

FLOW DIAGRAM (Indicate street slopes)



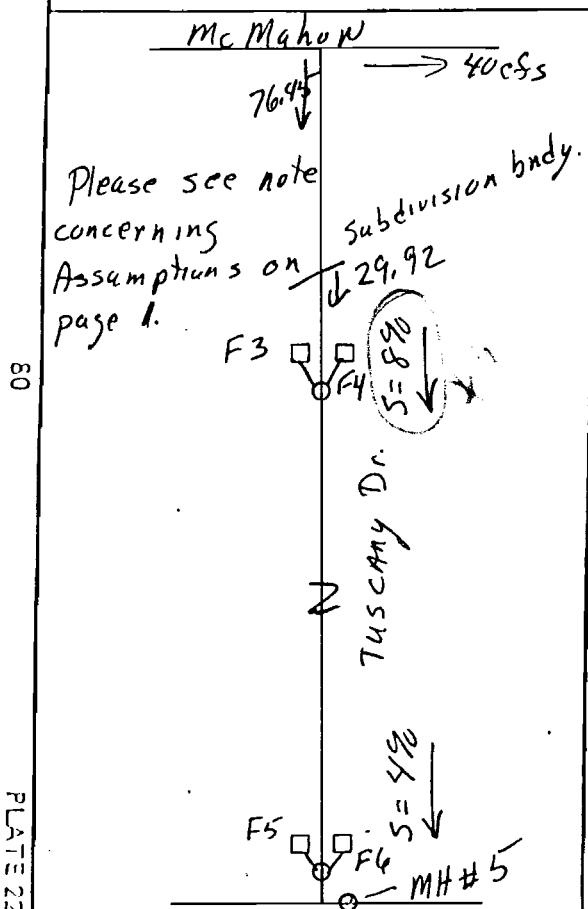
Sym.	Drain. Area	Q Total	Q Inter.	Bypass Cap. of ½ Street	Gutter "d"	C.B. No.	C.B. Size	Connector Pipe Head	Connector Pipe L	Connector Pipe Dia.	V Depth
I5	Bypass from Tuscany + I3+I4	24.75	9.6	15.15	.63						
I6		24.75	9.6	15.15	.63						
I7	Bypass 5+6 + Basins 250, 252	28.96	10	18.96	.66						
I9	255	28.96	10	18.96	.66						
I8	Bypass 7	18.96	8.2	10.76	.57						
I10	Bypass 9	18.96	8.2	10.76	.57						
	Bypass to Sump			= 21.5							

CATCH BASIN CALCULATION SHEET

Sht.__ of__

PROJECT _____ CALCULATED BY _____

DESIGN FREQUENCY _____ DATE _____



Sym.	Drain. Area	Q Total	Q Inter.	Bypass Gap of ½ Street	Gutter "d"	C.B. No.	C.B. Size	Connector Pipe Head	Connector Pipe L	Connector Pipe Dia.	V Depth
F3		14.96	8	6.96	.37						
F4		14.96	8	6.96	.37						
F5	Bypass from F3 + F4 + Basin 125	16.2	9	7.2	.42						
F6		16.2	9	7.2	.42						

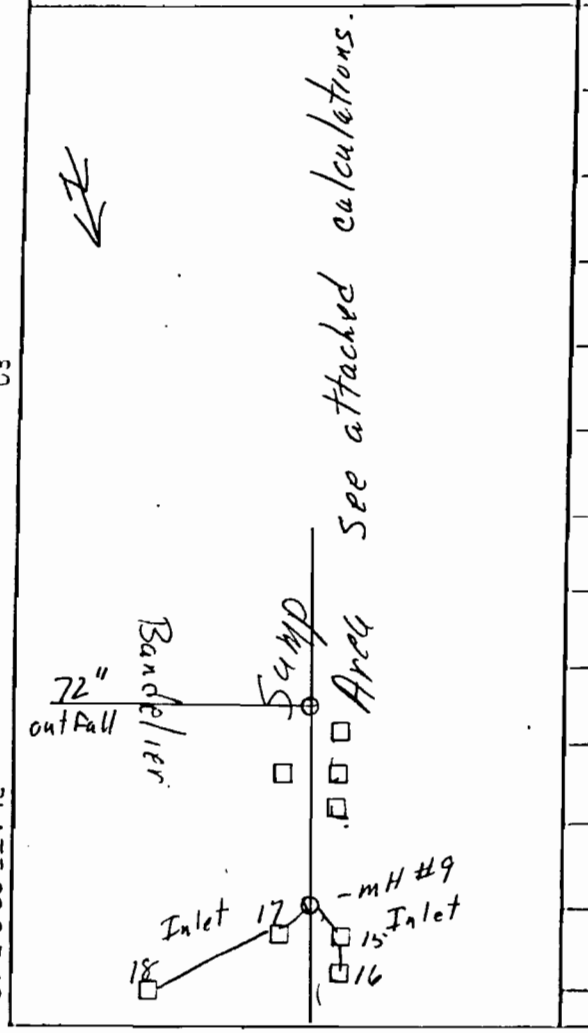
Total bypass from Tuscany Drive onto Bandelier = 14.4 cfs

CATCH BASIN CALCULATION SHEET

Sht.__ of__

PROJECT _____ CALCULATED BY _____

DESIGN FREQUENCY _____ DATE _____



Sym.	Drain. Area	Q Total	Q Inter.	Bypass Cap. of ½ Street	Gutter "d"	C.B. No.	C.B. Size	Connector Pipe Head	Connector Pipe L	Connector Pipe Dia.	V Depth
18	Parksite	2.2	2.2								
17	Basins 215,	16.5	8.8	7.7	.47						
16	210 + 205	16.5	8.8	7.7	.47						
15	Bypass 16	7.7	5.3	2.4	.37						
	Bypass to Sump =				10.1	cfs					

PLATE 22.3 D-10

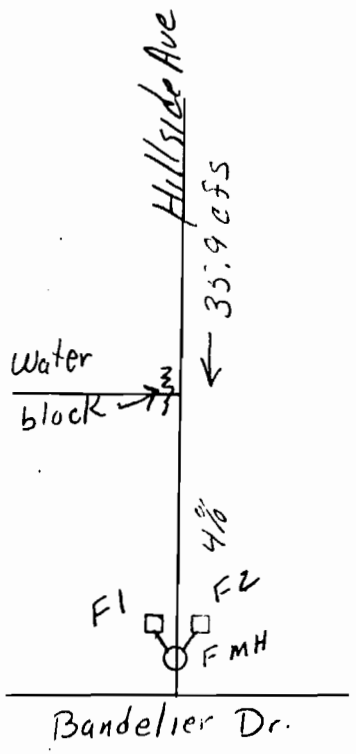
22.3

CATCH BASIN CALCULATION SHEET

Shl.___ of___

PROJECT _____ CALCULATED BY _____

DESIGN FREQUENCY _____ DATE _____



Sym.	Drain. Area	Q Total	Q Inter.	Bypass ~~Cap. of~~ ½ ~~Street~~	Gutter "d"	C.B. No.	C.B. Size	Connector Pipe Head	Connector Pipe L	Connector Pipe Dia.	V Depth
Future inlets F1 and F2 will be placed at end of Hillside Ave to mitigate flows from Basin 155. A water block is also provided to direct ½ of the total basin Q to future SD system. Total surface flow to Inlets F1 and F2 is 35.9 cfs And is assumed to be equally divided.											
F1	155	17.95	7.2	10.75	.37						
F2	155	17.95	7.2	10.75	.37						

Calculations for Sump Basin on Bandelier

Bypass from inlets to North-east 10.1 cfs
Bypass from inlets to West 21.5 cfs
Sky water from Basins 225, 230, 235, 240 50.3 cfs

Total Q @ Sump = 81.9 cfs

Assume equal distribution over 4 inlets ⇒ 20.48 cfs per inlet

Orifice calculation:

Area of orifice = 55% of gross area of grate

$2 \times 7 \times .55 = 7.7$ sq ft

$V = Q/A = 20.48/7.7 = 2.66$ cfs

$H = 1.2 V^2/2g = 1.2(2.66)^2/64.4 = 0.132'$

Assuming loss of 50% of Area ⇒ $V = 20.48/3.85$

$H = 1.2(5.32)^2/64.4 = .527'$ $V = 5.32$ fps

Weir Calculations: $Q = 2.7 L H^{3/2}$

Length of weir = 11'

$20.48 = 2.7(11)(H^{3/2}) \Rightarrow H = .78'$

Assuming loss of 50% of weir length.

$20.48 = 2.7(5.5)(H^{3/2}) \Rightarrow H = 1.24'$

ROUGH PIPE SIZING CALCULATIONS:

ASSUMES PIPES FULL & $S_o \cong S_f$

#265 @ SUMP (ARIANO CT) $Q_{100} = 266.1$ CFS $S_o = 0.008$

~90% IN PIPE $Q_{100\ pipe} \cong 239.5$ CFS

$K = Q/S^{1/2} = 2677.69$ $> K = 2604$ (60") $< K = 2966$ (63")

USE 60" RCP – WILL BE UNDER PRESS. (OR EQUIV.) @ OUTFALL

#280 @ SUMP (CELANO) $Q_{100} = 50.6$ CFS (100% CAPTURE) $S_o = 0.01$

$K = 50.6/0.1 = 506.0$ $> K = 410.1$ (30") $< K = 528.7$ (33")

USE 33" RCP OR EQUIV. @ OUTFALL

@ #180 $Q_{100} = 101.4$ CFS $S_o = 0.005$

~90% CAPTURED $Q_{100\ pipe} \cong 91.3$ CFS

$K = 91.3/.071 \cong 1291.2$ $< K = 1209$ (45") $> K = 1436$ (48")

USE 45" OR EQUIV.

@ #250 $Q_{100} = 93.6$ CFS $S_o = 0.005$

~90% CAPTURE $\cong 84.3$ CFS

$K = 843/.071 = 1186.5$ $\leq K = 1006$ (42") $> K = 1209$ (45")

USE 42" OR EQUIV.

TUSCANY (UNIT #1) – PONDS – 291-07-030 9-21-95 TJB

TEMPORARY PONDS REQUIRED WITH UNIT #1 CONSTRUCTION. SIZED PER DPM SECTION 22.2.

- PER EQUATION (7-9)

$$V_{10dy} = V_{360} + A_D * (P_{10dy} - P_{360}) / 12^{IN}/_{FT}$$

- PROJECT IN ZONE #1

$P_{10dy} = 3.67$ & $P_{360} = 2.20$ (Tbl A-2)

TEMPORARY PONDS WILL be TAKING ONLY UNDEVELOPED Q_S, NO IMPERVIOUS AREA $\therefore A_D = 0$, SO

$$V_{10dy} = V_{360} + 0 * (3.67 - 2.20) / 12^{IN}/_{FT}$$

$$\therefore V_{10dy} = V_{360}$$

V_{360} IS TAKEN FROM ~~AH4MO~~ RUN FOR TRIBUTARY BASINS (~~*ONLY~~)
(TOSCANY.DAT = ULTIMATE DEVELOPMT.)
*TOSCANP2.DAT = UNDEV. TRIBUTARIES TO PONDS ONLY

The POND VOLUMES ARE AS Follows: (Calculated IN AH4MO)
(HYDROGRAPH # PROVIDED)

POND #1 & #2 (HYD. #152.2)

V_{360} = 2.3805 Ac-Ft = 103,695 CF (REQUIRED CAPACITY)

POND #1 + #2 ON GRADING HAVE 150,000 CF±

POND # 2 (ONLY) – (HYD #150.0 + 1/3 OF HYD #152.0)

$V_{360} \cong$ 0.1726 Ac-Ft = 7,520 CF (REQ. CAP.)

POND #2 ON G.P. = 30,000 CF CAP. – ok

POND #1 (ONLY)

103,695 CF – 7,520 CF = 96,175 CF = RC*

*(RC = REQUIRED CAPACITY)

POND #1 PER G.P. HAS 120,000 C.F. CAP. ok

POND # 3 (HYD. # 115.1)

V_{360} = 0.9803 Ac-Ft = 42,702 CF = R.C.*

POND # 3 PER G.P. HAS 76,000 C.F. CAP. – ok

POND # 4 (HYD # 240.3)

V_{360} = 3.3394 Ac.-Ft. = 145,465 CF = REQ. CAP.

Pond #4 PER G.P. HAS 150,000 CF CAP. – ok

POND #5 (HYD #315.1)

V_{360} = 0.3804 Ac.-Ft = 16,570 CF = REQ. CAP.

POND #5 PER G.P. HAS 29000 CF CAP. – ok

CONCLUSIONS

PONDS PROVIDED PER DESIGNED GRADING PLAN EXCEED REQUIRED STORAGE CAPACITY. THESE PONDS ARE TEMPORARY AND WILL BE ELIMINATED WHEN THE PROPERTIES THEY ARE IN BECOME DEVELOPED. (COPY OF AHYMO FILE TUSCANP.DAT AND .SUM ARE INCLUDED IN REPORT.)

	REQUIRED	CAP. PER. GP.
POND #1	96,175 CF	120,000 CF
POND #2	7,520 CF	30,000 CF
POND #3	42,702 CF	76,000 CF
POND #4	145,465 CF	150,000 CF
POND #5	16,570 CF	20,000 CF

LOCAL SCOUR @ CULVERT OUTLET:

OUTFALL #1

$Q = 312$ CFS $\quad D = 72'' = 6'$

$t = 30$ MIN $\quad t_o = 316$

$q =$ GRAVITY

$$\frac{h_s}{D} = \alpha \left(\frac{Q}{\sqrt{g} D^{5/2}} \right)^{\beta} \left(\frac{t_o}{t_o} \right)^{\theta}$$

$$h_s = \left[2.72 \left(\frac{312}{32.2 (6)^{5/2}} \right)^{0.375} \left(\frac{30}{316} \right)^{0.10} \right] 6'$$

$$h_s = \underline{5.647'}$$

OUTFALL #2

$Q = 51$ CFS $\quad D = 30'' = 2.5'$

$t = 30$ MIN

$$h_s = \left[2.72 \left(\frac{51}{32 (2.5)^{5/2}} \right)^{0.375} \left(\frac{30}{316} \right)^{0.10} \right] 2.5'$$

$$h_s = \underline{2.71'}$$

ADDENDUM TO DRAINAGE REPORTS

Re: Tuscany Drainage Report by Community Sciences Corporation, and McMahon Boulevard Drainage Report by Smith Engineering Company.

This addendum to drainage reports, by both Smith Engineering Company dated May 1995, and Community Sciences Corporation,dated March 15, 1995, is to show the similarities and discuss differences, differences are minor.

The overall Q's presented in both reports agree with one another and only vary in the differences created by using two sets of land treatments and basin division lines. The differences are small enough to ignore on the basis of "Big Picture" overview of project.

The following is a comparison between the two reports:

LAND TREATMENTS: CSC VS. SEC

		A	B	C	D
CSC	Residential	0	25	25	50
CSC	Commercial	0	5	5	90
SEC	"Developed"	10	25	20	45
CSC	McMahon Blvd.	0	5	5	90
SEC	McMahon Blvd.	7	0	0	93

MCMAHON BOULEVARD SUB-BASINS: CSC VS. SEC

COMMUNITY SCIENCES CORPORATION BASINS			SMITH ENGINEERING COMPANY (EQUIVALENT BASIN)		
Basin	**Q_{100} (cfs)**	**Area**	**Basin**	**Q_{100} (cfs)**	**Area**
105	22.91	23.4 ac	F	27.3	8.6 ac
100	7.85	2.2 ac	***E	11.2(5.6/1/2)	4.2 ac/12.6
*100	*7.85				
	** (T=38.6)		AP #1 (FUT)(38.5)(at McMahon)		
185	5.39	1.5 ac	***E	20.4(10.2/1/2)	4.2 ac/12.6
190	3.15	0.9 ac			
305	3.38	0.96 ac			
*185	*5.39				
	(**T=55.9)		AP #2 (FUT)(58.9)(at McMahon)		
300	4.05	1.15 ac	A	35.3	11.1
310	2.70	0.77 ac			
315	16.43	5.95 ac			
325	6.73	1.92 ac			

	(**T=85.8)			(94.2)(at McMahon) AP #3 (FUT)/AP #1 (NOW)	
320	4.72	1.3 ac	***E	5.4(2.7/1/2)	2.1 ac/12.6
295	4.72	1.3 ac			
	(**T=95.2)			(99.6)(at McMahon) @ AP #3 (FUT)/AP #1 (NOW)	
330	2.70	0.77 ac	E	15.8(7.9/1/2)	2.1 ac/12.6
350	2.70	0.77 c			
(**T=100.6)/(**T=38±) after inlets installed				AP #4 (FUT)/AP #2 (NOW) (102.4)(at McMahon) or (37.0) after inlets installed	
335	45.72	16.6 ac	B	13.0	4.1 ac
			C	13.3	4.2 ac
			D	2.5	0.8 ac
(**T=22.5) to inlets at AP #4(FUT)				(**T=28.8) at AP #4 (FUT)	

(* SEC used full width McMahon; CSC used 1/2 width therefore add 2x)
(** Total Q's at points along McMahon Boulevard)
(*** Basin E (SEC) is complete portion of McMahon. From Unser to sump point, breakup at AP's shown per SEC Basin Map)

Community Sciences Corporation was more conservative in some of the areas drained by assuming that high points would extend to Tract Boundary's as opposed to stopping at natural ridge lines (as used by SEC).

At the analysis points (used in SEC report) Community Sciences Corporation's Q's and SEC's Q's match within 5% of one another. Community Sciences Corporation assumed that at AP #1 (FUT) runoff would be channeled via storm drain to arroyo. SEC assumed that flow in McMahon would be straight grade from Unser Boulevard to sump point at easterly end of Tract 1A of Paradise North (AP #4 (FUT)). In the comparison table the Community Sciences Corporation Q's have been tabulated similarly to SEC, and this shows little difference between calculated Q's.

Conclusions:

The Smith Engineering and CSC reports differ slightly. The proposed sizing of inlets by SEC are conservative. The proposed storm drain system by CSC is conservative. The combination of the drainage systems proposed offer storm protection that surpasses DPM requirements in ultimate condition and meets DPM criteria in the interim/construction condition. The proposed storm drain system presented in the SEC and CSC reports offers the City, for future development, along the McMahon Boulevard corridor greater flexibility to deal with drainage.

PER SECTION 3.5.4 - "SCOUR ALONG A FLOODWALL", IN THE "SEDIMENT AND EROSION DESIGN GUIDE", THE FOLLOWING FORMULAE WERE TO BE USED TO CALCULATE SCOUR DEPTH (Y_S) DUE TO DIFFERENT CRITERIA.

- FOR VERTICAL WALL(ONLY):

$(0.73Y) = Y_S$: (SxN 3.5.4)(⊥ PARALLEL)

- FOR PARALLEL FLOWS:

$\frac{Y_S}{Y} = 0.73 + 0.14 \Pi F_r^2$: (EQ. 3.89)

- FOR "ATTACK" AT AN ANGLE(θ):

$\frac{Y_S}{Y} = (0.73 + 0.14 \Pi F_r^2)\cos\theta + 4 F_r^{0.33} \sin\theta$: (EQ. 3.90)

TWO REACHES, WHERE WE PROPOSE TO CONSTRUCT THE OUTFALL STRUCTURES ARE OF CONCERN REACH # (ESB REPT. DATED) @ SECTION STA. 22+98 AND REACH # (ESB REPT. DATED) @ SECTION STA. 35+95 (AT "HEAD" OF EX. G.C.S. #2)

THESE TWO OUTFALL REACHES ARE ANALYZED FOR THE DOMINANT STORM, Q_0=2500 CFS, AND FOR THE

100yr STORM, $Q_{100} = 12,500$ c.f.s.

@ OUTFALL #1, (STA. 22+98 Ex.)

$Q_D = 2,500$ CFS
$F_r = 1.08$
$Y = 3.75$
SUBCRITICAL FLOW

$Q_{100} = 12,500$ CFS
* $F_r = 1.06$ (HEC-2 Comp.)
* $Y = 5.96'$ (" " ")
SUPERCRITICAL FLOW
* for 100yr-FLOW NEAR TRANSITION ($Y = 6.46$; $F_r = 0.83$-SUB.F

FOR $Q_D = 2500$ CFS & $Q_{100} = 12,500$ CFS

• VERT. WALL:

for Q_D: $Y_s = 0.73 \times 3.75 = 2.74'$

for Q_{100}: $Y_s = 0.73 \times 5.96' = 4.35'$

• PARALLEL FLOW:

for Q_D: $Y_s = [(0.73 + 0.14(3.142)(1.08)^2] \times 3.75' = 4.66'$

for Q_{100}: $Y_s = [0.73 + 0.14 \pi (1.06)^2] \times 5.96' = 7.30'$

• ANGLED ATTACK; $\theta = 45°$ & $30°$ ASSUMED

for Q_D; 45°: COS 45 = 0.71; SIN 45 = 0.71

$Y_s = [[0.73 + 0.14(3.142)(1.08)^2] \cos 45° + 4(1.08)^{0.33} \sin 45°] 3.75'$

$Y_s = [(1.243).71 + 4(1.026).71] 3.75' = 14.23'$

for Q_D; 30°: SIN 30 = 0.50; COS 30 = 0.87

$Y_s = [[0.73 + 0.14(3.142)(1.08)^2] 0.87 + 4(1.08)^{0.33} 0.50] 3.75'$

$Y_s = [(1.243).87 + 4(1.026).50] 3.75' = 11.75'$

for Q_{100}; 45°:

$$Y_s = [[0.73 + 0.14(\pi)(1.06)^2]0.71 + 4(1.06)^{0.33}0.71]5.96'$$

$$Y_s = [(1.224).71 + 4(1.019).71]5.96' = 22.43'$$

for Q_{100}; 30°:

$$Y_s = [1.224(.87) + 4(1.019).50]5.96 = 18.49'$$

for Q_{100}* ; 45°: $Y = 6.46'$ $F_r = 0.83$

$$Y_s = [(0.73 + 0.14\pi(.83)^2).71 + 4(.83)^{0.33}.71]6.46' =$$

$$Y_s = [(1.033).71 + 4(0.94).71]6.46 =$$

$$Y_s = 21.98'$$

for Q_{100}* ; 30°:

$$Y_s = [(1.033)0.87 + 4(0.94)0.50]6.46' = 17.95'$$

CONCLUSION:

BANK RIP-RAP @ OUTFALL #1 IS OK FOR DOMINANT STORM EVENTS, but NOT FOR ANGULAR ATTACK 100 YR EVENT. PARALLEL FLOWS ARE ANTICIPATED IN THIS REACH DUE TO STEEP BANKS (2:1±) ON SOUTH SIDE OF ARROYO. Therefore ANGULAR ATTACK RISK IS MINIMIZED. THE PROPOSED 6' OF STRUCTURE BELOW EX. FLOWLINE SHOULD BE ADEQUATE

@ OUTFALL #2, (STA. 35+95 EX.)

$Q_D = 2500$ CFS : $Q_{100} = 12,500$ CFS

$F_r = 0.63$: $F_r = 1.19$ (HEC-2 RUN)

$Y = 2.68'$: $Y = 4.90'$ (" " ")

SUBCRITICAL FLOW : SUPERCRITICAL FLOW

FOR $Q_D = 2500$ CFS & $Q_{100} = 12,500$ CFS

• VERT. WALL

for Q_D: $Y_S = 0.73(2.68') = 1.96'$

for Q_{100}: $Y_S = 0.73(4.90') = 3.58'$

• PARALLEL FLOW.

for Q_D: $Y_S = (0.73 + 0.14\pi(0.63)^2)2.68' = 2.42'$

for Q_{100}: $Y_S = (0.73 + 0.14\pi(1.19)^2)4.90' = 6.63'$

• ANGLED ATTACK, $\theta = 45°$ & $30°$ ASSUMED

for Q_D: $\measuredangle 45°$:

$Y_S = [(0.73 + 0.14\pi(.63)^2).71 + 4(.63)^{0.33}.71]2.68' =$

$Y_S = [0.905(.71) + 4(0.86).71]2.68' = 5.76'$

for Q_D: $\measuredangle 30°$:

$Y_S = [0.905(.87) + 4(.86).50]2.68' = 6.72'$

Q_{100} & $\phi = 45°$:

$$Y_s = [(0.73 + .14\pi(1.19)^2) .71 + 4(1.19)^{0.33} .71] 4.90' =$$

$$Y_s = [(1.3528) .71 + 4(1.06) .71] 4.90' = 19.44'$$

Q_{100} & $\phi = 30°$:

$$Y_s = [(1.3528) .87 + 4(1.06) .50] 4.90' = 16.16'$$

CONCLUSIONS:

OUTFALL #2 IS UPSTREAM OF EX. G.C.S. #2, AND BANK PROTECTION ON THE SOUTH SIDE OF ARROYO IS TO BE IN PLACE. THESE CONDITIONS ALMOST ASSURE THAT PARALLEL FLOWS WILL OCCUR FOR HIGHER FLOWS (IN EXCESS OF 1000CFS), THEREFORE ANGULAR ATTACK IS MINIMIZED. THE PROPOSED 6' OF BANK PROTECTION & RIP-RAP BELOW EX. SURFACE SHOULD BE ADEQUATE.

SOIL CEMENT @ OUTFALL #1 (LOWER) SPILLWAY

$Q_{100} = 312 \text{ CFS} \pm$ $\quad n = 0.02$, $W = 20'$ WIDE CHNL

1' DEPTH - @ $S = 0.009$

$$Q = \frac{1.486}{.02}(20.0\text{SF})(.94)(.094) = 131.3\text{CFS}$$

2' DEPTH @ $S = 0.009$

$$Q = \frac{1.486}{.02}(42\text{SF})(1.45)(0.094) = 425.3\text{ CFS}$$

425CFS > 312CFS OK

∴ USE 3' DEEP DESIGN (1' MIN. FREEBOARD)

SOIL CEMENT @ OUTFALL #1 (UPPER) SPILLWAY

1' DEPTH @ $S = 0.125$

$$Q = \frac{1.486}{0.02}(20\text{SF})(0.94)(.354) \simeq 494.5\text{ CFS}$$

495 CFS > 312CFS OK

∴ USE 2' DEEP DESIGN (1' MIN FREEBOARD)

SOIL CEMENT @ OUTFALL #2 SPILLWAY

$Q_{100} = 50.6$ CFS $S \cong 0.012$

FOR 1' DEPTH & 15' W CHANNEL

$$Q = \frac{1.486}{.02}(15.0)(.92)(.11) \cong 112.8 \text{ CFS}$$

113 CFS > 51 ± CFS ok

∴ USE 2' DEPTH DESIGN (1' MIN FREEBOARD)

* FOR 1' DEPTH & 10' W CHANNEL

$$Q = \frac{1.486}{.02}(10\text{SF})(0.886)(.11) \cong 72.4 \text{ CFS}$$

72.4 CFS > 51 CFS ± ok

∴ USE 10' WIDE & 2' DEEP DESIGN FOR OUTFALL SPILLWAY (1' MIN FREEBOARD)