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DRAINAGE REPORT

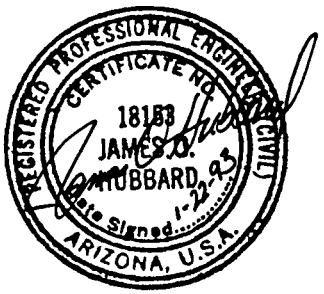
FOR

TATUM BOULEVARD

CAIDA DEL SOL TO SHEA BOULEVARD

City of Phoenix

Project No. P-886515



By:

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January 22, 1993

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REGISTERED PROFESSIONAL ENGINEER (CIVIL)
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Date Signed 1-22-93
ARIZONA, U.S.A.

INTRODUCTION

This report presents a storm water management plan to alleviate drainage problems associated with offsite flows contributing to Tatum Boulevard between Caida Del Sol and Shea Boulevard. This management plan will also address pavement drainage resulting from the widening of Tatum Boulevard for a 2-year storm event. The existing hydrologic conditions for Tatum Boulevard were previously addressed in a report to the City of Phoenix by Kaminski-Hubbard Engineering, Inc. titled, Final Hydrology Report for Tatum Boulevard, Caida Del Sol to Shea Boulevard. The offsite flow results from this study were used to size the cross drainage structures under Tatum Boulevard.

Tatum Boulevard is a four-lane arterial street that will be upgraded to a six-lane arterial street (three lanes in each direction) with a raised, landscaped median. Numerous roadway dips along Tatum Boulevard will be eliminated to prevent the surface flow of storm runoff across 88 feet of roadway pavement. Three proposed storm drain systems will collect storm runoff on the western half of Tatum Boulevard and convey the runoff to adequate outfall locations east of Tatum Boulevard. Every effort was made to mitigate the drainage impacts east of Tatum Boulevard within the Town of Paradise Valley. The Town of Paradise Valley has its boundary 40 feet east of the Tatum Boulevard monument line.

This is the final report prepared by Kaminski-Hubbard Engineering, Inc. for the Tatum Boulevard Drainage Study. The first report, Preliminary Hydrology Report for Tatum Boulevard, Mockingbird Lane to Shea Boulevard, dated March 30, 1990, addressed the existing hydrologic conditions and drainage facilities within the study area. The hydrology study was limited to offsite runoff (outside of the Tatum Boulevard right-of-way) contributing to Tatum Boulevard. Peak discharges were determined using the HEC-1 computer model. Subsequently, this preliminary report was reviewed and certain changes made to the hydrology which are addressed in the report, Final Hydrology Report for Tatum Boulevard, Caida Del Sol to Shea Boulevard.

STUDY AREA

The study area (See Figure 1) is located in northeast Phoenix near the Town of Paradise Valley. The study area is bound to the west and south by the Phoenix Mountain Preserve, to the east by Tatum Boulevard, and to the north by Shea Boulevard. A large percentage of storm runoff originates in the mountains west of Tatum Boulevard which naturally drains easterly to the Indian Bend Wash. Much of the study area is impervious because of rock outcrops in the mountains and the high degree of development throughout the area.

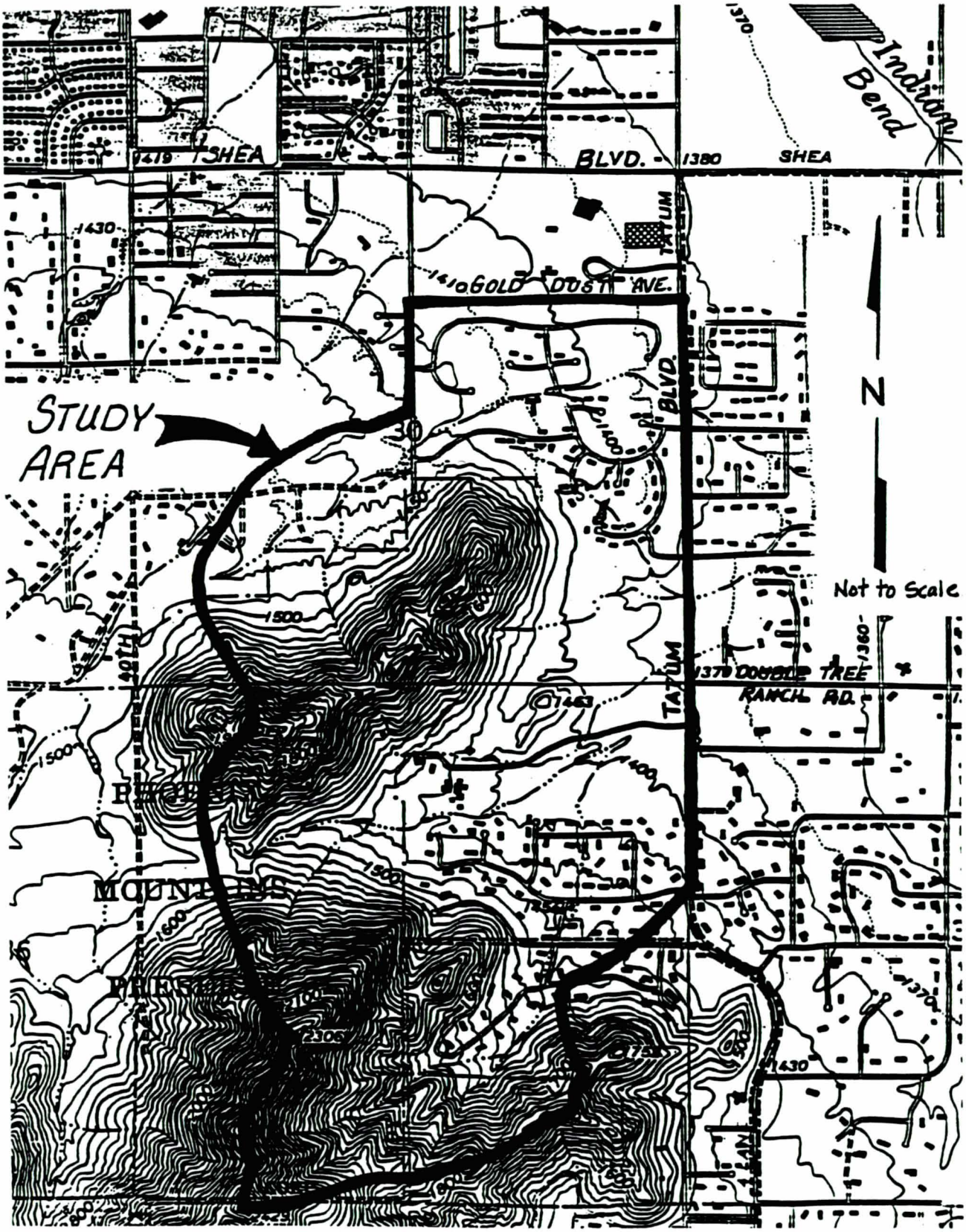
SHEA
BLVD.
SHEA
Indian Bend
TATUM
GOLD DUST AVE.
BLVD.
STUDY AREA
N
Not to Scale
TATUM
DOUBLE TREE RANCH RD.
MOUNTAINS
PRESERVE

FIGURE 1

The storm runoff is conveyed primarily along residential streets and subdivision drainageways. Runoff reaching Tatum Boulevard either "sheet flows" across the roadway via dips or flows under the roadway via pipe culverts. The Tatum Foothills and Rancho Alta Vida subdivision (See Plate 1) have retention basins that collect both onsite and offsite runoff. These retention basins decrease the amount of storm runoff reaching Tatum Boulevard from the Phoenix Mountains.

The study area contains two significant unnamed washes that drain a large portion of the Phoenix Mountain Preserve. The first wash is located in the southern half of the study area and discharges runoff directly onto Tatum Boulevard at Doubletree Ranch Road. Runoff will either flow northerly along Tatum Boulevard or easterly along Doubletree Ranch Road to Indian Bend Wash. The second wash is located in the northern half of the study area and discharges runoff into six 36" x 22" corrugated metal pipe (CMP) arch culverts that convey flow easterly under Tatum Boulevard to a man-made downstream channel.

Runoff along the eastern half of Tatum Boulevard proceeds easterly along residential streets or along subdivision drainageways. The subdivisions east of Tatum Boulevard are located within the Town of Paradise Valley. As storm runoff approaches Indian Bend Wash, the longitudinal slope begins to flatten to approximately 0.5 percent. This flattening of longitudinal slopes creates both a flooding problem and a sedimentation problem. According to our investigations, no plans have been made by the Town of Paradise Valley to implement storm drainage facilities to alleviate these drainage problems. The Town of Paradise Valley has a drainage concept of maintaining the historical drainage patterns regardless of development.

HYDROLOGY

The HEC-1 computer program was used to determine the rainfall-runoff relationship for offsite areas contributing to Tatum Boulevard. The study area was divided into a number of subareas to better define the runoff process by taking into account the variations in topography and urbanization. The offsite peak discharges for the 2-, 10-, 50- and 100-year storm events were determined using a 6-hour and 24-hour storm duration. The precipitation values recommended by the City of Phoenix in their Storm Drain Design Manual were used for this study.

Plate 1 illustrates the subarea delineations and concentration point locations developed for the HEC-1 model. The peak discharge at all culvert crossings under Tatum Boulevard were computed to determine their hydraulic efficiencies.

A time of concentration for each subarea or drainage area was determined using the method outlined in the manual, Urban Hydrology for Small Watersheds, June 1986, by the U.S. Department of Agriculture. A minimum time of concentration was set for 10 minutes.

The Rational Method of hydrology was used to compute the peak runoff within the study area for offsite drainage areas less than 40 acres and within the Tatum Boulevard right-of-way. The peak discharges were computed for the 2-, 10-, 50-, and 100-year storm event using the City of Phoenix rainfall intensity-duration-frequency curves. A runoff coefficient of 0.50 to 0.60 was used for residential areas and 0.70 for mountainous terrain with slopes greater than 10 percent.

Gutter velocities for paved streets were computed using the "Gutter Flow" chart on page 26 of the City of Phoenix Storm Drain Design Manual. The drainage path length was divided by the gutter velocity to obtain the time of concentration. Where applicable, the overland flow time was determined using the nomograph on page 21 in the City of Phoenix Storm Drain Design Manual.

HYDRAULICS

Catch basins have been located along Tatum Boulevard to insure that one 12-foot wide travel lane remains "dry" in each travel direction during the 2-year storm event. The catch basin inlets were sized using the procedures described in HEC-12, Drainage of Highway Pavements, by the Federal Highway Administration. The clogging factors presented in the City of Phoenix Storm Drain Design Manual were used to determine the inlet capacities. Inlets in a sump have been sized for the 10-year storm event.

Storm drain pipes have been designed for the 2-year storm event. Pipes have been designed to maintain a minimum velocity of 5 feet per second using a Mannings' "n" of 0.012. The pipe capacity was computed using the method outlined by the Arizona Department of Transportation in their Hydrologic and Hydraulic Training Session Manual.

The hydraulic grade line (HGL) elevation at the outlet was assumed to be at the top of pipe. The slope for the HGL was calculated from Table 4-17 in Appendix F. Storm drain pipes have been designed to convey runoff from a two year storm event with the HGL contained inside the pipe or very close to the crown of the pipe. Head loss across manholes was considered negligible.

The roadway culvert conveyance capacities were computed using the procedures outlined in HEC-5, Hydraulic Design of Highway Culverts. Culverts have been designed to not exceed the downstream channel and street capacities.

HYDRAULIC RESULTS

Drainage Area No.1

The topography of Drainage Area No. 1 consists of approximately 68 percent mountainous areas and 32 percent residential areas. The mountainous areas are within the Phoenix Mountain Preserve and characterized by rugged terrain, steep gradients and somewhat "impermeable" soils (Group D soil). Prior to development, this area contained numerous washes that flowed from the mountains into an unnamed wash that flowed northeasterly to Doubletree Ranch Road. This wash has since been channelized through the Doubletree Canyon subdivision as a part of their development.

Plate 1 illustrates the delineation of subareas for Drainage Area No. 1, which encompasses approximately 404 acres. Runoff from Sub-Basin Nos. 100-104, 200-201, 300-302 and 501-503 are collected and conveyed by the above mentioned channel through the Doubletree Canyon subdivision. This gunite-lined channel begins at Horseshoe Road and continues northeasterly through the subdivision to 47th Place. Prior to reaching 47th Place, the channel composition changes from gunite to grass. Also, the channel cross-section gradually widens as it approaches Tatum Boulevard. The channel was designed for the 100-year storm event.

Runoff discharging from the Doubletree Canyon subdivision channel has been a source of flooding at the Tatum Boulevard - Doubletree Ranch Road intersection. Two 8' x 2' concrete box culverts (CBC) are proposed at Station 20+98 to convey this runoff under Tatum Boulevard. A drop inlet structure at the upstream side of the culverts is proposed to collect runoff from this drainage channel. These culverts will discharge into a special underground structure located under the southeast curb return of Tatum Boulevard and Doubletree Ranch Road. This structure will also serve as an outfall point for Storm Drain No. 1 which will later be discussed. Also, the structure will have a 24-inch "bleed off" pipe for discharging "nuisance water" or runoff from storms smaller than the 2-year event into a drainage ditch along the south edge of Doubletree Ranch Road. For larger storm events, the runoff will "bubble" out of the structure through a grated inlet and sheet flow easterly along Doubletree Ranch Road. Doubletree Ranch Road and a parallel side channel have the capacity to convey approximately 140 CFS.

A two or three foot high landscaped berm along the west right-of-way of Tatum Boulevard between Horseshoe Road and Doubletree Ranch Road is proposed to prevent the flow of runoff onto Tatum Boulevard. This berm would also allow runoff from the Doubletree Canyon subdivision channel to enter the above mentioned drop structure. This system will improve the drainage conditions at the Doubletree Ranch Road intersection for the 2-year storm event and maintain the "sheet flow" drainage concept for the larger storms. The pipe culvert design sheet and downstream channel capacity calculations are contained in Appendix D.

The analysis presented in the Preliminary Hydrology Report for Tatum Boulevard for Drainage Area No. 1 has been revised to reflect drainage delineation changes made for Sub-Basin No. 500. For the new analysis, Sub-Basin No. 500 has been subdivided into Sub-Basin Nos. 500-1, 501, 502 and 503 as shown on Plate 1. The 6-hour duration peak discharge results are greater than the 24-hour results for the 100-year storm event. The new peak discharge results for a point southwest of the Tatum Boulevard and Doubletree Ranch Road intersection are presented in Table 1.

Offsite runoff from Sub-Basin No. 510-1 presently drains to Tatum Boulevard and then continues to flow north along the west pavement edge to an existing 24-inch pipe at Station 6+67. This pipe also collects offsite runoff from Sub-Basin No. 510-2. This pipe conveys runoff under Tatum Boulevard and outlets into a natural drainage channel. Drop inlet headwall structures are proposed at each concentration point in order to intercept offsite flows prior to reaching Tatum Boulevard. An 18-inch concrete pipe will be used to connect the two drop inlet structures. The existing 24-inch pipe at Station 6+67 will be lengthened to match into the proposed drop inlet structure.

TABLE 1

PEAK DISCHARGE SOUTHWEST OF TATUM BOULEVARD AND DOUBLETREE RANCH ROAD

STORM FREQUENCY (YR)	PEAK DISCHARGE (CFS) 6-HR STORM DURATION
2	363
10	858
25	1,215
50	1,438
100	1,466

Runoff from Sub-Basin Nos. 500-1 to 500-4, 510-3 and 510-4 presently drains along the western half of Tatum Boulevard to Doubletree Ranch Road. Runoff crosses Tatum Boulevard via a dip in the roadway and continues easterly along Doubletree Ranch Road. A connector pipe system (Storm Drain No. 1) is proposed to collect this storm runoff and convey this runoff to an underground outflow structure proposed at the southeast corner of Tatum Boulevard and Doubletree Ranch Road.

Runoff from Sub-Basin No. 500-1 approaches Tatum Boulevard along Horseshoe Road at a longitudinal slope of approximately 2.0 percent. A combination curb opening and grate inlet with slotted vane drain (Inlet No. 500-1) is proposed along Horseshoe Road to intercept a portion of storm runoff from Sub-Basin No. 500-1. This combination should intercept more runoff prior to reaching Tatum Boulevard than a conventional curb opening or combination inlet catch basin. The hydrology and inlet design calculations are presented in Appendix A and Appendix B, respectively. The storm drain pipe design and hydraulic grade line calculations are presented in Appendix C.

Drainage Area No. 2

This region consists of approximately 31 percent mountainous areas and 69 percent residential areas. The area encompasses a portion of the Phoenix Mountain Preserve and the following subdivisions: a portion of Doubletree Canyon; Tatum Foothills; and a portion of Rancho Alta Vida. Prior to development, this region contained a network of numerous washes flowing from west to east that collected at a low point west of Tatum Boulevard between Arabian Way and Berneil Drive.

Plate 1 illustrates the delineation of subareas for Drainage Area No. 2, which encompasses an area of approximately 185 acres. Runoff from Sub-Basin Nos. 400 and 410 presently drain into two retention basins located west of Tatum Boulevard between Doubletree Ranch Road and Fanfol Drive (West). These basins were designed to retain nuisance water from the Tatum Foothills subdivision and overflow across Tatum Boulevard for large storm events. One flows cross Tatum Boulevard, they are diverted north to a channel running easterly through the Foothills Estates II subdivision located in Paradise Valley. This channel has an approximate downstream capacity of 110 CFS.

A decrease in retention volume for the above referenced basins may occur with the widening of Tatum Boulevard. However, the roadway profile will be raised slightly to accommodate two proposed 38"x24" concrete oval pipes at Station 32+20. A special drop inlet structure at the upstream side of the pipes will allow the basins to retain runoff up to an elevation of 1374.00. Above elevation 1374.00, runoff begins to discharge under Tatum Boulevard through the oval pipes and continue to the above mentioned drainage channel. For large storms, ponded water will cross the pavement crown and continue to flow easterly.

A unique outflow rating curve was computed for the two proposed 38" x 24" oval pipe crossings at Station 32+20 using a combination of orifice and weir flow equations. The pipe invert was set at 1372.52, the retention basin invert was found to be 1372.20, and the elevation at which water began to flow into the drop inlet was equal to 1374.00. The control section for inflow into the oval pipes was the perimeter walls for the drop inlet. Weir flow into the drop inlet structure occurred between elevations 1374.00 to 1375.20. Above elevation 1375.20, orifice flow controlled and HEC-5 methods were used to compute flow through the oval pipes. Included in the outflow rating curve were flows that overtopped Tatum Boulevard, which occurred at an elevation of 1375.40. The weir flow equation was used to compute this overflow discharge across Tatum Boulevard.

According to the Final Drainage Report for Tatum Boulevard, the 6-hour storm duration peak discharge results for Sub-Basin Nos. 400 and 410 were larger than the 24-hour duration results. Therefore, the 6-hour duration peak discharge results were reservoir routed through the retention basin using the parameters established for the new roadway profile and culvert pipes. The sub-basin peak discharge (HEC I.D. 400), reservoir routed peak discharge (HEC I.D. 400R) and peak stage results are presented in Table 2. The volume - discharge rating curve for the retention basin is presented in Appendix E. The HEC-1 hydrologic models for Sub-Basin Nos. 400 and 410 using the 6-hour duration storm are presented in Appendix E.

TABLE 2

PEAK DISCHARGE SUMMARY FOR SUB-BASINS NOS. 400/410

STORM FREQUENCY	PEAK DISCHARGE (CFS)		PEAK STAGE
(YR)	HEC I.D. 400	HEC I.D. 400R	(FT)
2	51	24	1374.64
10	179	145	1375.87
25	323	314	1376.16
50	389	381	1376.25
100	455	451	1376.31

Scuppers are proposed for Inlet Nos. 400-1 and 410-2 to 410-6 because vertical restrictions prevent catch basins from discharging flows directly to adjacent retention basins. Catch basin inverts using minimum 'v' requirements were found to be lower than the adjacent retention basin invert on the west and channel invert on the east. Scuppers located on the west half of Tatum Boulevard will discharge directly into retention basins draining Sub-Basin Nos. 400 and 410.

Pavement drainage along the east half of Tatum Boulevard will continue to drain as it has in the past, which is to flow eastward along residential streets. Scuppers (Inlet Nos. 410-4, 410-5 and 410-6) proposed on the east side will outlet into a grader ditch that will convey runoff to a drainage channel flowing through the Foothills Estates II subdivision.

Runoff from Sub-Basin Nos. 423 and 432 presently drain into onsite retention basins within the Rancho Alta Vida subdivision. These areas are considered noncontributing for the 2-year storm event. Runoff from Sub-Basin No. 422 presently crosses Tatum Boulevard at Berneil Drive (East) via a dip in the roadway. Runoff from Sub-Basin Nos. 421-1, 421-2, 431-1 and 431-2 also flow easterly across Tatum Boulevard via dips in the roadway.

A connector pipe system (Storm Drain No. 2) is proposed to collect runoff from the 2-year storm event for Sub-Basin Nos. 410-1, 421-1, 421-2, 422, 431-1 and 431-2. This pipe system will begin at Mountain View Road and proceed south along Tatum Boulevard to Station 32+20. The storm drain will curve easterly and outlet adjacent to the oval concrete pipes.

The hydrology and catch basin inlet design calculations for Drainage Area No. 2 are presented in Appendix A and Appendix B, respectively. The pipe design and hydraulic grade line calculations for Storm Drain No. 2 are presented in Appendix C. The pipe culvert computations for the two 38" x 24" oval pipes are contained in Appendix E.

Drainage Area No.3

This region contains approximately 50 percent mountainous areas and 50 percent residential areas. The mountainous areas are within the Phoenix Mountain Preserve and characterized by rugged, steep gradients sloping from south to north. Before urbanization, this region contained numerous small washes flowing northeasterly to a large unnamed wash which flowed easterly through what is now the Rancho Alta Vida subdivision. This large wash intersects Tatum Boulevard between Mountain View Road and Onyx Avenue, and thus creates the main concentration point for Drainage Area No. 3.

Plate 1 illustrates the delineation of subareas for Drainage Area No. 3, which encompasses an area of approximately 152 acres. Runoff from Sub-Basin No. 443 is collected at 44th Street and conveyed by a man-made channel through Sub-Basin Nos. 442 and 441 to six 36" x 22" CMP arch culverts. These culverts convey storm runoff under Tatum Boulevard and discharge directly into the Mountain View Estates subdivision located in the Town of Paradise Valley.

The existing six 36" x 22" CMP arch culverts at Station 52+58 have the capacity to convey 150 CFS without overtopping the west curb of Tatum Boulevard. These culverts were found to function under outlet control conditions having pipe losses equal to 2.5 feet. However, the inlet control calculations using an inlet headwater of 4.33 feet would result in a peak discharge of 234 CFS through the culvert.

Four 38" x 24" oval pipes are proposed as replacements for the six 36" x 22" CMP arch to convey storm runoff under Tatum Boulevard. These oval pipes have approximately the same conveyance capacity as the existing culverts. A larger conveyance structure at this location was not considered because of downstream restrictions on channel capacities. The downstream channel between Tatum Boulevard and 48th Place was designed to convey 210 CFS with 1.0 foot of freeboard. This channel then discharges onto Turquoise Avenue which has a street capacity of approximately 150 CFS. The City of Phoenix directed Kaminski-Hubbard Engineering, Inc. not to make conditions downstream in the Town of Paradise Valley any worse than they are today. The pipe culvert design sheets and downstream channel capacity calculations are presented in Appendix D.

Drainage Area No. 4

This region is predominantly residential and encompasses an area of approximately 64 acres. Flow occurs along residential streets and is typically northerly or easterly. Flows ultimately drain to the intersection of Gold Dust Avenue and Tatum Boulevard. This intersection has a history of flooding during heavy rains and remains a nuisance area for smaller storms.

Plate 1 illustrates the delineation of subareas for Drainage Area No. 4. Runoff from Sub-Basin Nos. 452 and 453 presently drain into onsite retention basin within the Rancho Alta Vida subdivision. These retention basins have sufficient capacity to retain runoff for a 2-year storm event. Runoff from Sub-Basin Nos. 451-1 to 451-4 currently drain along residential streets to the intersection of Tatum Boulevard and Gold Dust Avenue.

Currently, there is an existing 36-inch storm drain pipe stub along Tatum Boulevard that extends approximately 764 feet south of the Shea Boulevard 78-inch storm drain trunk line. A lateral pipe system (Storm Drain No. 3) is proposed for Tatum Boulevard which will extend the existing 36-inch storm drain pipe south to Station 59+13.50. This system will collect and convey the 2-year storm runoff from Sub-Basin Nos. 451-1, 451-2, 451-3, 451-4 to the Shea Boulevard trunk line. The hydrology and inlet design calculations are presented in Appendix A and Appendix B, respectively. The storm drain pipe design and hydraulic grade line calculations are presented in Appendix C.

APPENDIX A

HYDROLOGY CALCULATIONS

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

STORM SEWER SYSTEM DESIGN: RUNOFF CALCULATION SHEET

LOCATION DATA
Highway Tatum Blvd.
Location Phoenix, AZ
Project No.

DESIGN DATA
Frequency 2 years
P_6= — in. P_{24}= — in. P_1= — in.

RUNOFF CALCULATIONS

Drainage Area			Pav't.			Comm.			Resid.			Lot Area							Tc	I	Q
No.	Station - Station	Acre	C	A	CA	C	A	CA	C	A	CA	C	A	CA	C	A	CA	ΣCA	min.	in./hr.	cfs
	510-1								0.60	6.15	3.69							3.69	26.2	1.50	5.54
	510-2								0.60	5.46	3.27							3.27	20.0	1.75	5.73
	510-3		0.95	0.58	0.55				0.75	6.09	4.57							5.12	23.0	1.60	8.20
	510-4		0.95	0.40	0.38													0.38	10.0	2.40	0.91
	500-1		0.90	0.91	0.82				0.60	12.77	7.66	0.61	9.61	5.86				14.34	18.2	1.85	26.53
	500-2		0.95	0.36	0.34													0.34	10.0	2.40	0.83
	500-3		0.95	0.13	0.12													0.12	10.0	2.40	0.29
	500-4		0.95	0.28	0.27													0.27	10.0	2.40	0.64
	400-1		0.95	0.69	0.66													0.66	10.0	2.40	1.58
	410-1		0.95	0.44	0.41													0.41	10.0	2.40	0.99
	410-2		0.95	0.71	0.67													0.67	10.0	2.40	1.62
	410-3		0.95	0.59	0.56													0.56	10.0	2.40	1.35

Computed by: D.L.B. 8-16-90 Checked by: _____ Date: _____

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

STORM SEWER SYSTEM DESIGN: RUNOFF CALCULATION SHEET

LOCATION DATA
Highway Tatum Blvd.
Location Phoenix, AZ
Project No.

DESIGN DATA
Frequency 2 years
P_6= in. P_{24}= in. P_1= in.

RUNOFF CALCULATIONS

Drainage Area			Pav't.			Comm.			Resid.			Lot							Tc	I	Q
No.	Station - Station	Acre	C	A	CA	C	A	CA	C	A	CA	C	A	CA	C	A	CA	ΣCA	min.	in./hr.	cfs
	410-4		0.95	0.52	0.50													0.50	10.0	2.40	1.19
	410-5		0.95	0.50	0.47													0.47	10.0	2.40	1.13
	410-6		0.95	0.49	0.46													0.46	10.0	2.40	1.11
	422		0.95	1.87	1.78				0.50	8.07	4.04							5.11	22.10	1.66	9.65
	421-1		0.95	0.83	0.79				0.60	2.40	1.44							2.23	10.0	2.40	5.35
	421-2		0.95	0.84	0.79				0.60	3.36	2.02							2.81	10.0	2.40	6.75
	431-1		0.95	0.74	0.71				0.55	3.08	1.69							2.40	10.0	2.40	4.55
	431-2A		0.95	0.48	0.46				0.55	0.83	0.45							0.91	15.0	2.0	1.83
	431-2B		0.95	0.50	0.48				0.55	0.83	0.45							0.93	10.8	2.3	2.15
	451-1		0.95	0.89	0.84				0.60	2.94	1.76							2.60	14.3	2.10	5.47
	451-2								0.60	11.98	7.19							7.19	24.2	1.55	11.14
	451-3								0.60	17.45	10.47							10.47	21.3	1.70	17.80
	451-4		0.95	1.45	1.37													1.37	13.5	2.10	2.89
	452-1		0.95	0.92	0.87													0.87	10.0	2.40	2.11

Computed by: D.L.B. 8-16-90 Checked by: Date:

KAMINSKI HUBBARD engineering inc. • Surveying • Civil • Hydrology	Made by D.L.B.	Date 8-9-90	Job No. 0118
	Checked by	Date	Sheet No.
	For		

SUBBASIN NO. 510-1

(1) offsite Runoff
Drainage Area = $(13.40/2)(200)^2$ = 268,000 SF = 6.152 Ac.
(C = 0.60)

(2) Tatum Blvd. R.O.W.
Drainage Area = (505 - 192)(55) = 17,215 SF = 0.395 Ac.
(C = 0.95)

(3) Time of Conc., t_c (Use Overland Flow Time, C.O.P. Manual Pg. 21)
$L_1 = 250'$, $S_1 = 0.03'/ft$ → $\Delta t_1 = 7.5$ min.
$L_2 = 900'$, $S_2 = 0.03'/ft$ → $\Delta t_2 = 12.2$ min.
$L_3 = 170'$, $S_3 = 0.03'/ft$ → $\Delta t_3 = 6.5$ min

$\therefore t_c = 7.5 + 12.2 + 6.5 = 26.2$ min

$Q = \Sigma CAI = 4.066\ I$

Description	2Yr.	10Yr.	25Yr.	50Yr.	100Yr
I, in/hr.	1.5	2.4	2.9	3.3	3.7
① Q, CFS (INCLUDES TATUM)	6.10	9.76	11.79	13.42	15.05
② Q, CFS (OFFSITE)	5.54	8.86	10.70	12.18	13.66

① Peak Discharge includes Offsite Runoff and Tatum Blvd. R.O.W. Runoff

② Peak Discharge for Offsite Runoff Only.

KAMINSKI HUBBARD engineering inc. • Surveying • Civil • Hydrology	Made by D.L.B.	Date 8-9-90	Job No. 0118
	Checked by	Date	Sheet No.
	For		

SUBBASIN NO. 510-2

(1) OFFSITE RUNOFF

Drainage Area = $(11.88/2)(200)^2$ = 237,600 SF = 5.455 Ac.
($C = 0.60$)

(2) Tatum Blvd. R.O.W.

Drainage Area = $(192 + 305)(55)$ = 27,335 SF = 0.628 Ac.
($C = 0.95$)

(3) Time of Conc., t_c

$L = 1050'$ $S = 0.030'/ft$
$t_c = 2(10\ min.) = 20\ min.$

$Q = \Sigma CAI = 3.8696\ I$

Description	2Yr.	10Yr.	25Yr.	50Yr.	100Yr.
I, in/hr	1.75	2.8	3.3	3.7	4.2
① Q, CFS (INCLUDES TATUM)	6.77	10.83	12.77	14.32	16.25
② Q, CFS (OFFSITE)	5.73	9.16	10.80	12.11	13.75

① Peak Discharge includes Offsite Runoff and Tatum Blvd. ROW Runoff.

② Peak Discharge for Offsite Runoff Only.

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SUBBASIN NO. 510-3

(1) OFFSITE RUNOFF

Drainage Area = $(13.27/2)(200)^2$ = 265,400 SF = 6.093 Ac.
($C = 0.75$)

(2) Tatum Blvd. R.O.W.

Drainage Area = (457)(55) = 25,135 S.F. = 0.5770 Ac.
($C = 0.95$)

(3) Time of Conc., t_c (Use Overland Flow Time, C.O.P. Manual Pg. 21)

$L_1 = 125'$ $S_1 = 0.020$ '/ft $\Delta t_{c1} = 6$ min.
$L_2 = 370'$ $S_2 = 0.030$ '/ft (Asphalt) $\Delta t_{c2} = 6.2$ min.
$L_3 = 650'$ $S_3 = 0.030$ '/ft $\Delta t_{c3} = 10.8$ min.

$\therefore t_c = 6 + 6.2 + 10.8 = 23.0$ min.

$$Q = \Sigma CA\, I = [(6.093)(0.75) + (0.5770)(0.95)]\, I = 5.1179\, I$$

Description	2 Yr.	10 Yr.	25 Yr.	50 Yr.	100 Yr.
I, in/hr	1.6	2.6	3.1	3.5	3.9
① Q, CFS (INCLUDES TATUM)	8.2	13.3	15.9	17.9	20.0
② Q, CFS (OFFSITE)	7.3	11.9	14.2	16.0	17.8

① Peak Discharge includes Offsite Runoff and Tatum Blvd. R.O.W. Runoff

② Peak Discharge for Offsite Runoff Only.

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SUBBASIN NO. 510-4

Drainage Area = (315)(55) = 17325 ft^2 = 0.40 Ac.

C = 0.95 (Tatum Blvd. R.O.W.)

Assume t_c = 10 min. → $i_{2yr.}$ = 2.4 in/hr

∴ $Q_{2yr.}$ = (0.95)(2.4)(0.40) = 0.91 CFS

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SUBBASIN NO. 500-1

(1) OFFSITE RUNOFF (Thru Tatum Mt. Estates)

$C = 0.60$

Drainage Area $= (30.90/2)(200)^2 = 618{,}000$ SF $= 14.187$ Ac.

Assume 90% Contributes

$\therefore$ Contributing Drainage Area $= 556{,}200$ SF $= 12.769$ Ac.

TIME OF CONC., t_{c_1}

$t_{c_1} = (0.167 + 0.023 + 0.074) \times 60 = 15.8$ min.

(2) ONSITE RUNOFF (Thru Doubletree Canyon) *

* Drainage Area & Time of Conc. calculations per "Doubletree Canyon" Drainage Report by Collar, Williams & White, April 1979.

Total Lot Area $= 3.72 + 0.93 + 4.96 = 9.61$ Ac.
($C = 0.61$)

Total Street Area $= 0.58 + 0.09 + 0.24 = 0.91$ Ac.
($C = 0.90$)

TIME OF CONC., t_{c_2}

$L_1 = 250'$, $S_1 = 0.032$ 1/1 (Gunite Channel) $\rightarrow V = 5.8$ fps

$\therefore \Delta t_1 = 250/60 \times 5.8 = 0.7$ min

$L_2 = 600'$, $S_2 = 0.0225$ 1/1 $\rightarrow V = 6.0$ fps

$\therefore \Delta t_2 = 600/60 \times 6 = 1.7$ min.

$\therefore t_{c2} = 0.7 + 1.7 = 2.4$ min.

(3) TOTAL TIME OF CONC., t_c

$t_c = t_{c_1} + t_{c2} = 15.8 + 2.4 = 18.2$ min

$Q = \Sigma CAI = [(0.60)(12.769) + (0.61)(9.61) + (0.90)(0.91)]\, I = 14.3425\, I$

Description	2Yr.	10Yr.	25Yr.	50Yr.	100Yr.
I, in/hr	1.85	3.00	3.50	3.90	4.5
Q, CFS	26.53	43.03	50.20	55.94	64.54

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SUBAREA NO. 500-2 HYDROLOGY

Tatum Blvd. R.O.W.

Drainage Area = (287)(55) = 15,785 SF = 0.362 Ac.
(C=0.95)

Time of Conc., t_c = 10 min.

Q = CIA = 0.3439 I

Description	2Yr.	10Yr.	25Yr.	50Yr.	100Yr.
I, in/hr	2.4	3.8	4.5	5.0	5.5
Q, CFS	0.83	1.31	1.55	1.72	1.89

SUBAREA NO. 500-3

Tatum Blvd. R.O.W.

Drainage Area = (100)(55) = 5500 SF = 0.126 Ac.
(C=0.95)

Time of Conc., t_c = 10 min.

Description	2Yr	10Yr	25Yr	50Yr	100Yr
I, in/hr	2.4	3.8	4.5	5.0	5.5
Q, CFS	0.30	0.48	0.56	0.63	0.69

SUBAREA NO. 500-4

Q = CIA = 0.2675 I

Drainage Area = (223)(55) = 12,265 SF
(C=0.95) = 0.2816 Ac.
Time of Conc., t_c = 10 min

Q_{2Yr} = (0.95)(0.2816)(2.4) = 0.64 CFS

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PAVEMENT DRAINAGE FLOWING DOWN DOUBLETREE RANCH RD. - EAST
PAVEMENT DRAINAGE BETWEEN STA. 15+32 AND 22+02

Sta. 18+00 to 20+70, Lt.
D.A. = (270)(55) = 14,850 SF

Sta. 20+70 to 22+02, Lt.
D.A. = (132)(11) = 1452 SF

Sta. 15+32 to 22+02, Rt.
D.A. = (670)(49) + ½(670)(5) = 34,505 SF

Total Drainage Area = 50,807 SF = 1.166 Ac.

Assume t_c = 10 min. → I_{2yr} = 2.4 in/hr
I_{10yr} = 3.8 in/hr.

For R.O.W., C = 0.95

$Q_2 = (0.95)(2.4)(1.166) = 2.66$ CFS

$Q_{10} = (0.95)(3.8)(1.166) = 4.21$ CFS

Check Spread, T:

(1) $d_2 = \left[(2.66)/(0.56)(3333.33)(0.0071)^{1/2}\right]^{3/8} = 0.217'$

$T_2 = 0.217/0.02 = 10.85'$

(2) $d_{10} = \left[(4.21)/(0.56)(3333.33)(0.0071)^{1/2}\right]^{3/8} = 0.257'$

$T_{10} = 0.257/0.02 = 12.85'$

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SUBAREA NO. 400-1 HYDROLOGY

Drainage Area = (550)(55) = 30,250 S.F. = 0.694 Ac.

C = 0.95

Time of Conc., t_c For Q = 3 CFS & S = 0.0075'/ft,

V = 2.6 fps (p. 26, C.O.P. Drainage Manual)

$\therefore t_c = \frac{550}{2.6 \times 60} = 3.5$ min. < 10 min.

Use t_c = 10 min.

Description	2Yr	10Yr	25Yr	50Yr	100Yr
I, in/hr	2.4	3.8	4.5	5.0	5.5
Q, CFS	1.58	2.51	2.97	3.30	3.63

Check Spread upstream of Arabian Way:

S = 0.0075'/ft $\quad Q_{10yr}$ = 2.51 CFS

Sx = 0.020'/ft

$d = \left\{ 2.51 / \left[(0.56)(50/0.015)(0.0075)^{1/2} \right] \right\}^{3/8} = 0.210'$

$T = 0.210/0.02 = 10.50'$

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SUBAREA NO. 410-1

R.O.W. Area = 345 x 55 = 18,975 ft^2 = 0.4356 Ac.
(C = 0.95)
t_c = 10 min. Q = CAI = 0.4138 I

Description	2Yr.	10Yr.	25Yr.	50Yr.	100Yr.
I, in/hr	2.4	3.8	4.5	5.0	5.5
Q, CFS	0.99	1.57	1.86	2.07	2.28

SUBAREA NO. 410-2

R.O.W. Area = $(4.69/2)(100)^2 + (135)(55)$ = 30,875 SF = 0.7088 Ac.
(C = 0.95)
t_c = 10 min Q = CAI = 0.6734 I

Description	2Yr.	10Yr.	25Yr.	50Yr.	100Yr.
I, in/hr	2.4	3.8	4.5	5.0	5.5
Q, CFS	1.62	2.56	3.03	3.37	3.70

SUBAREA NO. 410-3

R.O.W. Area = (470)(55) = 25,850 SF = 0.5934 Ac.
(C = 0.95)
t_c = 10 min Q = CAI = 0.5638 I

Description	2Yr.	10Yr.	25Yr.	50Yr.	100Yr.
I, in/hr	2.4	3.8	4.5	5.0	5.5
Q, CFS	1.35	2.14	2.54	2.82	3.10

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SUBAREA NO. 410-4

R.O.W. Area = 430 x 53 = 22,790 SF = 0.5232 Ac.
(C = 0.95)
t_c = 10 min. Q = CAI = 0.4970 I

Description	2Yr	10Yr	25Yr	50Yr	100Yr
I, in/hr	2.4	3.8	4.5	5.0	5.5
Q, CFS	1.19	1.89	2.24	2.49	2.73

SUBAREA NO. 410-5

R.O.W. Area = 407 x 53 = 21,571 SF = 0.4952 Ac.
(C = 0.95)
t_c = 10 min. Q = CAI = 0.4704 I

Description	2Yr.	10Yr	25Yr	50Yr	100Yr
I, in/hr	2.4	3.8	4.5	5.0	5.5
Q, CFS	1.13	1.79	2.12	2.35	2.59

SUBAREA NO. 410-6

R.O.W. Area = 400 x 53 = 21,200 SF = 0.4867 Ac.
(C = 0.95)
t_c = 10 min. Q = CAI = 0.4624 I

Description	2Yr	10Yr	25Yr	50Yr	100Yr
I, in/hr	2.4	3.8	4.5	5.0	5.5
Q, CFS	1.11	1.76	2.08	2.31	2.54

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SUB-BASIN NO. 421

(1) SUBAREA 421-1

Assumes 90% Contributes

Residential Area = [2.90 x 200² ÷ 43560] x 0.90 = 2.40 Ac.
(C = 0.60)

R.O.W. Area = [0.90 x 200² ÷ 43560] = 0.83 Ac.
(C = 0.95)

t_c = 10 min. Q = ΣCAI = 2.2285 I

Description	2 Yr	10 Yr	25 Yr	50 Yr	100 Yr
I, in/hr	2.4	3.8	4.5	5.0	5.5
Q, CFS	5.35	8.47	10.03	11.14	12.26

(2) SUBAREA 421-2

Residential Area = [3.66 x 200² ÷ 43560] x 0.90 = 3.36 Ac.
(C = 0.60)

R.O.W. Area = [0.91 x 200² ÷ 43560] = 0.84 Ac.
(C = 0.95)

t_c = 10 min. Q = ΣCAI = 2.8140 I

Description	2 Yr	10 Yr	25 Yr	50 Yr	100 Yr
I, in/hr	2.4	3.8	4.5	5.0	5.5
Q, CFS	6.75	10.69	12.66	14.07	15.48

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SUBAREA NO. 421-3

PAVEMENT DRAINAGE BETWEEN STA. 41+52 AND 45+88 RT.

Drainage Area = (436)(41) + (165)(7) = 19,031 SF = 0.437 Ac.

Assume t_c = 10 min. → $I_{2yr.}$ = 2.4 in/hr

$I_{10yr.}$ = 3.8 in/hr

For R.O.W., C = 0.95

$Q_2 = (0.95)(2.4)(0.437) = 1.00$ CFS

$Q_{10} = (0.95)(3.8)(0.437) = 1.58$ CFS

Check Spread, T:

(1) $d_2 = \left[(1.00)/(0.56)(2666.67)(0.0052)^{1/2}\right]^{3/8} = 0.173'$

$T_2 = 0.173/0.025 = 6.92'$

(2) $d_{10} = \left[(1.58)/(0.56)(2666.67)(0.0052)^{1/2}\right]^{3/8} = 0.205'$

$T_{10} = 0.205/0.025 = 8.20'$

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SUB-BASIN NO. 422

Residential Area = $[9.76 \times 200^2 \div 43560] \times 0.90 = 8.07$ Ac.
($C = 0.50$)

R.O.W. Area = $[2.04 \times 200^2 \div 43560] = 1.87$ Ac.
($C = 0.95$)

Time of Conc., t_c.

Overland Time : $L = 350'$, $S = 0.020'/ft$, bare soil
$\Delta t_c = 12.3$ min (Page 21, C.O.P. Manual)

Gutter Flow Time: (From Graph on Page 26, C.O.P. Manual)
ALONG 47th Street - Fanfol Drive

$L_1 = 110'$, $S_1 = 0.0105'/ft$, $Q = 2.5$ CFS $\rightarrow V = 3.0$ fps $\rightarrow \Delta t_c = \frac{110}{3 \times 60} = 0.6$ min.

$L_2 = 624'$, $S_2 = 0.0020'/ft$, $Q = 3.0$ CFS $\rightarrow V = 1.6$ fps $\rightarrow \Delta t_c = \frac{624}{1.6 \times 60} = 6.5$ min.

$L_3 = 239'$, $S_3 = 0.0129'/ft$, $Q = 3.0$ CFS $\rightarrow V = 3.2$ fps $\rightarrow \Delta t_c = \frac{239}{3.2 \times 60} = 1.2$ min.

$L_4 = 272'$, $S_4 = 0.011'/ft$, $Q = 3.0$ CFS $\rightarrow V = 3.1$ fps $\rightarrow \Delta t_c = \frac{272}{3.1 \times 60} = 1.5$ min

Sub-Total = 9.8 min.

$t_c = 12.3 + 9.8 = 22.10$ min.

Description	2 Yr.	10 Yr.	25 Yr.	50 Yr.	100 Yr.
I, in/hr	1.66	2.70	3.20	3.55	4.0
Q, CFS	9.65	15.69	18.60	20.63	23.25

$$Q = \Sigma CA\,I = [(8.07)(0.50) + (1.87)(0.95)]\,I$$
$$= 5.8115\,I$$

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SUBAREA NO. 431-1 HYDROLOGY

Mountain View Rd. (South Half)

Assume 90% Contributes

Residential Area (C = 0.55) $= [3.73 \times 200^2 \div 43560] \times 0.90$
$= 3.08$ Ac.

R.O.W. Area (C = 0.95) $= [0.81 \times 200^2 \div 43560]$
$= 0.74$ Ac.

Time of Conc., t_c

(1) Overland Flow Time: $L_1 = 200'$, $S_1 = 0.02$ '/ft

$\Delta t_{c1} = 15$ min (Page 21, C.O.P. Manual)

(2) Street Flow Time: $L_2 = 184'$, $S_2 = 0.0080$ '/ft

$Q = 3$ CFS (Assumed) $\rightarrow V = 3.2$ ft/sec (Page 26, C.O.P. Manual)

$\Delta t_{c2} = \frac{184}{60 \times 2.7} = 1.1$ min.

$L_3 = 226'$, $S_3 = 0.0223$ '/ft., $Q = 3$ CFS (Assumed) $\hookrightarrow V = 3.8$ ft/sec

$\Delta t_{c3} = \frac{226}{60 \times 3.8} = 1.0$ min.

$t_c = \Delta t_{c1} + \Delta t_{c2} + \Delta t_{c3} = 15 + 1.1 + 1.0 = 17.1$ min.

$Q = \Sigma CA\,I = [(0.55)(3.08) + (0.95)(0.74)]\,I$

$= 2.397\,I$

Description	2 Yr.	10 Yr.	25 Yr.	50 Yr.	100 Yr.
I, in/hr.	1.9	3.1	3.6	4.0	4.5
Q, CFS	4.55	7.43	8.6	9.6	10.8

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SUBAREA NO. 431-2

Subdivided into 2 areas because of Grade & Alignment changes.

(1) Subarea No. 431-2A

Assume 90% Contributes

Residential Area $= [1.00 \times 200^2 \div 43560] \times 0.90$
(C = 0.55) $= 0.83$ Ac.

R.O.W. Area $= [0.52 \times 200^2 \div 43560]$
(C = 0.95) $= 0.48$ Ac.

Time of Conc., t_c

Overland Flow Time, $L_1 = 150'$, $S_1 = 0.02'/ft$, $\Delta t_{c1} = 13$ min

Street Time, $L_2 = 400'$, $S_2 = 0.017'/ft$, $Q = 2$ CFS (Assumed)
$\therefore \Delta t_{c2} = 400/60 \times 3.2 = 2.1$ min

$t_c = \Delta t_{c1} + \Delta t_{c2} = 13 + 2.1 = 15$ min.

$Q = \Sigma CAI = 0.9125\ I$

For 2 Year, $I = 2.0$ in/hr. $\therefore Q_2 = 1.83$ CFS

For 10 Year, $I = 3.2$ in/hr. $\therefore Q_{10} = 2.92$ CFS

(2) Subarea No. 431-2B

Assume 90% Contributes

Residential Area $= (1.01 \times 200^2 \div 43560) \times 0.90$
(C = 0.55) $= 0.835$ Ac.

R.O.W. Area $= [0.54 \times 200^2 \div 43560]$
(C = 0.95) $= 0.50$ Ac.

Time of Conc., t_c

Overland Flow Time, $L = 450'$, $S = 0.020'/ft$, $\Delta t_{c1} = 10$ min. (p. 21)

Street Time, $L = 75'$, $S = 0.0022'/ft$, $Q = 2$ CFS (Assumed)
$\therefore \Delta t_{c2} = 75/60 \times 1.5$ $V = 1.5$ fps (p. 26)
$= 0.8$ min.

$t_c = \Delta t_{c1} + \Delta t_{c2} = 10 + 0.8 = 10.8$ min.

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SUBAREA NO. 431-2 (Cont'd)

(2) Subarea No. 431-2B (Cont'd)

$Q = \Sigma CA\, I = 0.9343\, I$

For 2 Year, $I = 2.3$ in/hr $\therefore Q_2 = 2.15$ CFS

For 10 Year, $I = 3.7$ in/hr $\therefore Q_{10} = 3.46$ CFS

For 25 Year, $I = 4.3$ in/hr $Q_{25} = 4.02$ CFS

For 50 Year, $I = 4.8$ in/hr $Q_{50} = 4.48$ CFS

For 100 Year, $I = 5.4$ in/hr. $Q_{100} = 5.05$ CFS.

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SUBAREA NO. 451-1 HYDROLOGY

(1) Offsite Runoff
Drainage Area = $(6.40/2)(200^2)$ = 128,000 ft^2 = 2.94 Ac.
C = 0.60

(2) Tatum Blvd. R.O.W.
Drainage Area = (702)(55) = 38,610 ft^2 = 0.886 Ac.
C = 0.95

(3) Time of Conc., t_c
L_1 = 300' (Between Paved & Bare Soil), S_1 = 0.017 ft/ft ∴ Δt_1 = 8.7 min. (p. 21)
L_2 = 702', Assume Q = 3CFS, S_2 = 0.0040 ft/ft → V = 2.1 fps (p. 26)
∴ Δt_1 = 702/60×2.1 = 5.6 min.

Total t_c = 8.7 + 5.6 = 14.3 min.

Description	2Yr	10Yr	25Yr	50Yr	100Yr
I, in/hr	2.1	3.3	3.9	4.4	4.9
Q, CFS	5.47	8.60	10.16	11.46	12.77

ΣCA = 2.61

SUBAREA NO. 451-2 HYDROLOGY

Total Drainage Area = $(43.48/2)(200)^2$ = 869,600 ft^2 = 19.96 Ac.
C = 0.60 * Assume 60% contributes; D.A. = 0.60 × 19.96 = 11.98 Ac.

Time of Conc., t_c
L_1 = 170', S_1 = 0.020 ft/ft ∴ Δt_1 = 9.2 min (p. 21)
L_2 = 2380', Assume Q = 3CFS, S_2 = 0.011 ft/ft → V = 3.0 fps (p. 26)
∴ Δt_2 = 13.2 min.
L_3 = 320', V = 3.0 fps → Δt_3 = 320/60×3.0 = 1.8 min.

Total t_c = 9.2 + 13.2 + 1.8 = 24.2 min.

Description	2Yr	10Yr	25Yr	50Yr	100Yr
I, in/hr	1.55	2.55	3.0	3.4	3.8
Q, CFS	11.14	18.33	21.56	24.44	27.31

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SUBAREA NO. 451-3 HYDROLOGY

Flow Split at 44th St. & Gold Dust Ave.
Based on field observation, 85% flows east along Gold Dust Ave and 15% north along 44th St. Therefore, assume 85% of drainage area west of 44th St. contributes to Tatum Blvd.

$$\text{Drainage Area} = 0.85\left(\frac{25.10}{2}\right)(200^2) + \left(\frac{16.68}{2}\right)(200^2)$$
$$= 760{,}300 \text{ SF} = 17.45 \text{ Ac.}$$

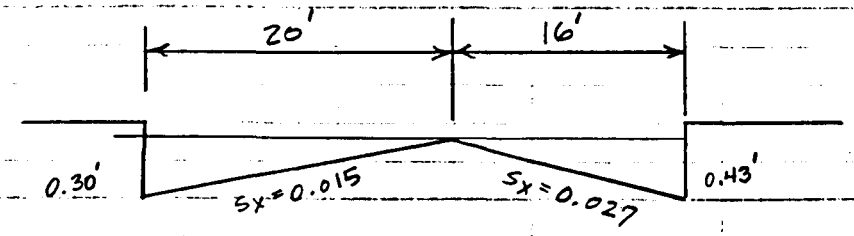
$C = 0.60$

Time of Conc., t_c Assume D.A. west of 44th St. has a $t_c = 12$ min.

For Gold Dust Ave., $Q = 10$ CFS, $S = 0.012$ ft/ft
From p. 26, $V = 4.5$ fps $\rightarrow \therefore t_c = 2520/60 \times 4.5 = 9.3$ min.

Total $t_c = 12.0 + 9.3 = 21.3$ min.

Description	2 Yr	10 Yr	25 Yr	50 Yr	100 Yr
I, in/hr	1.70	2.75	3.25	3.60	4.10
Q, CFS	17.80	28.80	34.04	37.70	42.94



½ street Capacity - Left $A = 3.0 \text{ ft}^2$ $P = 20.3$ ft $S = 0.0124$ ft/ft
$Q = 0.56(4444.4)(0.0124)^{1/2}(0.30)^{8/3} = 11.18$ CFS

½ street Capacity - Right $A = 3.456 \text{ ft}^2$, $P = 16.43$ ft $S = 0.0124$ ft/ft
$Q = 0.56(2469.14)(0.0124)^{1/2}(0.43)^{8/3} = 16.22$ CFS

For 2-Year storm, water will overflow from Left to Right.

$\therefore$ Overflow $Q = 17.80 - 11.18 = 6.62$ CFS < 16.22 CFS

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SUBAREA NO. 451-4

Drainage Area along Gold Dust Ave. = (2520)(25) = 63,000 s.f. = 1.446 Ac.
C = 0.95
Time of Conc., t_c For Q = 3 cfs, s = 0.012 ft/ft
V = 3.1 fps
t_c = 2520/60×3.1 = 13.5 min

Description	2Yr	10Yr	25Yr	50Yr	100Yr
I, in/hr	2.1	3.4	3.9	4.4	5.0
Q, cfs	2.89	4.67	5.36	6.05	6.87

Half Street Capacity

* For 2Yr Storm, Q = 6.62 + 2.89 = 9.51 cfs < 16.22 cfs OK
where Q overflow from left side of Gold Dust Ave = 6.62 cfs

SUBAREA NO. 451-5 Deleted

Drainage Area (Tatum R.O.W.) = (121)(55) + (12)(25) = 6955 ft² = 0.160 Ac.
C = 0.95

Drainage Area (Lot) = (100)(85) = 8500 ft² = 0.195 Ac.
C = 0.80

Time of Conc., t_c = 10 min. Q = ΣCAI = 0.3078 I

Description	2Yr	10Yr	25Yr	50Yr	100Yr
I, in/hr	2.4	3.8	4.5	5.0	5.5
Q, cfs	0.74	1.17	1.39	1.54	1.69

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SUBAREA NO. 452-1

56+48 → 58+00	58+00 → 60+00	60+00 → 61+86	61+86 → 62+00
57'	57'	48'	56'	110'	110'	53'	53'

Drainage Area = (152)(57) + (200)(48) + ½(200)(8) + (186)(110) + (14)(53)
(0.95) = 40,266 ft²
= 0.9244 Ac.

t_c = 10 min → I_{2yr} = 2.4 in/hr

I_{10yr} = 3.8 in/hr.

For 2 Year storm, Q = (0.95)(0.9244)(2.4) = 2.11 CFS

For 10 Year Storm, Q = (0.95)(0.9244)(3.8) = 3.34 CFS

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	For		

HYDROLOGY – TATUM ACRES II & PORTION OF SMITTY'S

Total Area = $\left(\frac{121.16}{2}\right)(100)^2(\div 43560) = 13.9073$ Ac.

Assume 80% contributes to Tatum Blvd. and use a $C = 0.60$.

$\therefore CA = (0.60)(0.80 \times 13.9073) = \underline{\underline{6.675 \text{ Ac.}}}$

Time of Conc. t_c :

$L_1 = 750'$ $S_1 = 0.015$ '/ft $\Delta t_{c_1} = 18$ min. (Overland Flow Time p. 21)

$L_2 = 960'$ $S_2 = 0.010$ '/ft Assume $V = 3.0$ fps (p. 26)

$\Delta t_{c_2} = \frac{960}{60 \times 3.0} = 5.3$ min

$\therefore$ Total t_c to Tatum Blvd. $= 18 + 5.3 = 23.3$ min.

Tatum Blvd. R/W contributing to Exist. C.B. at sta. 67+23

D.A. $= (461)(55)(\div 43560) = 0.5821$ Ac.

$C = 0.95$

$CA = 0.95 \times 0.5821 = 0.553$ Ac.

Time of Conc. t_c :

For $Q = 6$ CFS, $S = 0.0042$ '/ft, $V = 2.6$ fps

$\therefore t_c = 280/60 \times 2.6 = 1.8$ min. (p. 26)

$\Sigma CA = 6.675 + 0.553 = 7.228$ Ac.

$T_c = 23.3 + 1.8 = 25.1$ min.

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HYDROLOGY FOR EXIST. CATCH BASIN @ STA. 71+06 RT.

D.A. = 900 × 59 ÷ 43560 = 1.219 Ac.

C = 0.95

CA = 0.95 × 1.219 = 1.1581

T_c = 10 min. → $I_{2yr.}$ = 2.4 in/hr

Q_2 = CAI = 1.1581 × 2.4

= 2.78 CFS

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	For		

HYDROLOGY FOR EXIST. C.B. AT STA. 73+38, LT.

$$D.A. = \left[(223 + 298)(55) + (212)(67) + \tfrac{1}{2}(121)(12)\right] \div 43560$$
$$= 1.00 \text{ Ac.}$$

$C = 0.95$ $\qquad$ $CA = 0.95$ Ac.

$T_c = 10$ min $\longrightarrow$ $I_{2yr.} = 2.4$ in/hr

$$Q = (0.95)(1.00)(2.4) = 2.28 \text{ CFS}$$

$d = 0.233'$ $\qquad$ $s = 0.0035'/ft$

$T = 0.233/0.02 = 11.7' < 22'$ ok

HYDROLOGY FOR EXIST. C.B. AT STA. 73+76, RT.

$D.A. = 270 \times 55 \div 43560 = 0.341$ Ac.

$C = 0.95$

$CA = 0.324$ Ac

$T_c = 10$ min. $\longrightarrow$ $I_{2yr} = 2.4$ in/hr

$$Q = (0.95)(0.341)(2.4) = 0.78 \text{ CFS}$$

APPENDIX B

INLET DESIGN CALCULATIONS

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

STORM SEWER SYSTEM DESIGN: INLET CALCULATION SHEET

LOCATION DATA

Highway: Tatum Blvd.
Location: Phoenix, AZ
Project No.: 886515

DESIGN DATA

Frequency: 2 years
P_6 = – in. P_{24} = – in. P_1 = – in.
T_{all} = 22 ft.
n = 0.015

INLET CALCULATIONS

Inlet No	Inlet Station	DA No	Inlet Type	Q cfs	Q_c cfs	Q_T cfs	So %	Gutter S_x	Gutter z/n	Pavement S_x	Pavement z/n	D ft.	T ft.	a in.	Inlet Act.	Inlet Corr.	Q_i/Q	Q_i cfs	Q_c cfs
	3+19	510-1	Drop Inlet	5.54	-	5.54						0.75					1.0	5.54	0
	6+67	510-2	Drop Inlet	5.73	-	5.73						0.75					1.0	5.73	0
	9+09	510-3	Comb. Inlet P-1572, Type Q	8.30	-	8.30	3.09	0.10	645	0.02	3333	0.39	11.4	2			0.55	4.56	3.74
	11+91	510-4	Comb. Inlet P-1572, Type Q	0.91	3.74	4.65	2.53	0.10	645	0.02	3333	0.35	9.0	2			0.67	3.09	1.56
	14+72	500-2	Comb. Inlet P-1572, Type Q	0.83	1.56	2.39	2.20	0.10	645	0.02	333	0.30	6.4	2			0.83	1.97	0.42
	14+98	500-1	Comb. Inlet P-1572, Type Q	26.53	-	26.53	2.00			.006	11,111	0.30	23.0	2			Add	3.23	
			w/7 - Type R Slotted Vane Drain															10.50	12.80
	16+35	500-3	Comb. Inlet P-1567, Type K14	0.30	0.42 12.80	13.52	1.50	0.138	485	0.02	3333	0.44	13.7	2	8.53	10.67	0.38	5.14	8.38
			w/ Grate	–	8.38	8.38	1.50	0.138	485	0.02	3333	0.44	13.7	2	Grate	Grate	0.35	2.96	5.42
	17+00	500-4	Comb. Inlet P-1572, Type Q Triple	0.64	5.42	6.06	0.64	0.10	645	0.02	3333	0.30	15.0	2			1.0	6.06	–
	26+14.6	400-1	Scupper L=6'	1.58	-	1.58	0.75	0.138	485	0.02	3333	0.32	7.5	2	4.8'	6'	0.77	1.22	0.36
	27+55	410-2	Scupper L=8'	1.62	0.36	1.98	0.87	0.138	485	0.02	3333	0.33	8.8	2	6.4'	8'	0.83	1.64	0.34

Computed by: D.L.B. 4-1-91 Checked by: _____ Date: _____

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

STORM SEWER SYSTEM DESIGN: INLET CALCULATION SHEET

LOCATION DATA
Highway: Tatum Blvd.
Location: Phoenix, AZ
Project No.:

DESIGN DATA
Frequency: 2 years
P_6= in. P_{24}= in. P_1= in.
T_{all}: 22 ft
n: 0.015

INLET CALCULATIONS

Inlet No	Inlet Station	DA No	Inlet Type	Q cfs	Q_c cfs	Q_T cfs	So %	Gutter S_x	Gutter z/n	Pavement S_x	Pavement z/n	D ft.	T ft.	a in.	Inlet Act.	Inlet Corr.	Q_i/Q	Q_i cfs	Q_c cfs
	35+65	421-2	Comb. Inlet P-1567, Type K14	6.75	–	6.75	1.20	0.138	483	0.02	3333	0.43	13.2	2	L=8.53'	L=10.67'	0.57	3.81	2.94
			w/ Grate	–	2.94	2.94	1.20	0.138	483	0.02	3333	0.35	9.2	2	Grate	Grate	0.53	1.55	1.39
	35+40.9	422	Type R Mod. L=10'	9.65	–	9.65	1.10	0.132	505	0.014	4762	0.44	19.5	2	L=8.53'	L=10.67'	0.43	4.12	5.53
			w/ Grate	–	5.53	5.53	1.10	0.132	505	0.014	4762	0.38	15.5	2	Grate	Grate	0.34	1.92	3.61
	33+00	410-1	Comb. Inlet P-1567, Type K14	0.99	1.39 3.61	5.99	0.44	0.138	485	0.020	3333	0.47	15.4	2	L=8.53'	L=10.67'	0.70	4.21	1.78
			w/ Grate	–	1.78	1.78	0.44	0.138	485	0.020	3333	0.35	9.1	2	Grate	Grate	0.54	0.96	0.82
	28+75	410-3	Scupper L=6'	1.35	0.34 0.82	2.51	0.40	0.138	485	0.020	3333	0.24	11.8	2	L=4.8'	L=6'	1.0	2.51	0
	26+86	410-4	Scupper L=4'	1.19	–	1.19	0.75	0.138	485	0.020	3333	0.29	6.4	2	L=3.2'	L=4'	0.65	0.78	0.41
	30+86	410-5	Scupper L=4'	1.13	–	1.13	0.44	0.138	485	0.020	3333	0.31	7.2	2	L=3.2'	L=4'	0.72	0.82	0.31
	28+75	410-6	Scupper L=4'	1.11	0.41 0.31	1.83	0.37	0.138	485	0.020	3333	0.21	10.5	2	L=3.2'	L=4'	1.0	1.83	0
	43+00	421-1	Comb. Inlet P-1572, Type Q Triple	5.35	–	5.35	0.52			0.020	3333	0.30	15.0	2			1.0	5.35	0

Computed by: DLB 4-1-91 Checked by: Date:

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

STORM SEWER SYSTEM DESIGN: INLET CALCULATION SHEET

LOCATION DATA
Highway Tatum Blvd.
Location Phoenix, AZ
Project No.

DESIGN DATA (unless otherwise noted.)
Frequency 2 years
P_6= in. P_{24}= in. P_1= in.
T_{all} 22 ft
n 0.015

INLET CALCULATIONS

Inlet No	Inlet Station	DA No	Inlet Type	Q cfs	Q_c cfs	Q_T cfs	So %	Gutter S_x	Gutter z/n	Pavement S_x	Pavement z/n	D ft.	T ft.	a in.	Inlet Act.	Inlet Corr.	Q_i/Q	Q_i cfs	Q_c cfs
				10 Year Discharge															
	48+03.6	431-1	Comb. Inlet P-1567, Type K14	7.43	-	7.43	2.20	0.133	501	0.015	4445	0.39	14.6	2	L=8.53'	L=10.67'	0.45	3.31	4.12
			w/ Grate	-	4.12	4.12	2.20	0.133	501	0.015	4445	0.34	11.0	2	Grate	Grate	0.50	2.08	2.04
	51+50	431-2	Comb. Inlet P-1573, Type R	2.15	-	2.15	2.20	0.138	483	0.020	3333	0.40	11.7	2			0.42	0.90	1.25
			Comb. Inlet																
	59+00	451-1	P-1572 Type Q, Triple	5.47	-	5.47	0.51			0.020	3333	0.30	15.0	2			1.0	5.47	0
	61+32	451-2	MAG Det. 535 Type F	11.14	-	11.14						0.77			Grate	Only	1.0	11.14	0
	61+22.6	451-3	P-1572 Type Q Triple	11.18	-	11.18	0.88			0.018	3704	0.35	19.4	2			0.78	8.70	2.48
	61+59	451-4	P-1572 Type Q Triple	9.51	-	9.51	0.85			0.027	2469	0.35	13.0	2			0.91	8.70	0.81
	61+85	452-1	P-1567 Type K14	2.11	2.48 0.81	5.40	0.55	0.133	503	0.015	4444	0.42	16.9	2	L=8.53'	L=10.67'	0.66	3.56	1.84
			w/ Grate	-	1.84	1.84	0.55	0.133	503	0.015	4444	0.33	10.6	2	Grate	Grate	0.51	0.93	0.91
	Exist C.B. 71+06 RT		Curb Opening L=8'	2.78	0.91	3.69	0.32	0.118	567	0.023	2898	0.42	12.4	2	L=6.4'	L=8'	0.75	2.78	0.91

Computed by: D.L.B. Checked by: 4-1-91 Date:

F-44

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INLET NO. 510-1

$Q = 5.54$ CFS (2 Year Storm)

Design Catch Basin to intercept offsite 2-Year Design Flow Prior to reaching Tatum Blvd. Catch Basin will be placed behind back of sidewalk and will be considered a sump condition.

Try a $L = 6'$ Curb Opening Type Catch Basin.
Clogging Factor $= 1.25$
$h = 6''$
$d = 6 + 3 = 9''$ $\qquad d_o = 3 + 6/2 = 6''$
Since $d > 1.4h$,

$$Q_i = 0.67h(L/1.25)(2gd_o)^{1/2} = 0.67(0.5)(6/1.25)(2 \times 32.2 \times 0.5)^{1/2}$$
$$= 9.12 \text{ CFS} > 5.54 \text{ CFS}$$

Use Drop Inlet Headwall, M.A.G. Det. 501-5 with 18" Diameter Conc. Pipe

INLET NO. 510-2

$Q = 5.73$ CFS (2 Year Storm)

Design Catch Basin to intercept offsite 2-Year Storm Flow Prior to reaching Tatum Blvd. Catch Basin will be placed behind back of sidewalk and will be considered a sump condition

Try a $L = 6'$ Curb Opening Type Catch Basin.

Clogging Factor, $CF = 1.25$
$h = 6''$
$d = 6 + 3 = 9''$ $\qquad d_o = 3 + 6/2 = 6''$
Since $d > 1.4h$,

$$Q_i = 0.67(0.5)(6/1.25)(2 \times 32.2 \times 0.5)^{1/2}$$
$$= 9.12 \text{ CFS} > 5.73 \text{ CFS}$$

Use Drop Inlet Headwall, M.A.G. Det. 501-5 with 24" Diameter Conc. Pipe.

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INLET NO. 510-3

$Q = 8.30$ CFS $\qquad S_x = 0.020$ ft/ft

$S = 0.0309$ ft/ft $\qquad Z_x/n = 3333.33$

(a) Curb Opening Inlet

$a = 2$ in, $W = 17$ in. $\}$ $S_w' = 2/17 = 0.1176$ $\qquad Z_w/n = 484.7$, C.F. $= 1.25$

$S_w = 0.1176 + 0.020 = 0.1376$

Assume $d = 0.401'$ & $\quad d' = 0.401 - (1.417)(0.1376) = 0.206'$

$Q_A = 4.17$ CFS
$Q_B = 0.71$ CFS
$Q_C = 4.86$ CFS $\}$ $Q_T = 4.17 - 0.71 + 4.86 = 8.32$ CFS

Gutter Flow, $Q_w = 4.17 - 0.71 = 3.46$ CFS

$E_o = Q_w/Q = 3.46/8.32 = 0.416$

$S_e = 0.020 + (0.1176)(0.416) = 0.0689$

$$L_T = (0.6)(8.30)^{0.42}(0.0309)^{0.3}\left[1/(0.015)(0.0689)\right]^{0.6} \times 1.25$$
$$= 79.32'$$

~~(i) Try $L = 10'$, Det. P-1569, Type 'M-1'~~

~~$L/L_T = 13/34.70 = 0.375$~~

~~$E = 1 - (1 - 0.375)^{1.8} = 0.570$~~

~~$Q_i = 6.67 \times 0.570 = 3.80$ CFS~~

~~$Q_b = 6.67 - 3.80 = 2.87$ CFS~~

(ii) Try $L = 17'$, Det. P-1569, Type 'M-1'

$L/L_T = 20/79.32 = 0.252$

$E = 1 - (1 - 0.252)^{1.8} = 0.407$

$Q_i = 0.407 \times 8.30 = 3.38$ CFS

$Q_b = 8.30 - 3.38 = 4.92$ CFS

(b) Combination Inlet

Try Det. P-1572, Type Q

$S_w' = 2/24 = 0.0833$ $\qquad Z_w/n = 645$

$S_w = 0.0833 + 0.020 = 0.1033$

Assume $d = 0.394'$ & $\quad d' = 0.394 - (0.1033)(2) = 0.187'$ (.207)

$Q_A = 5.30$ CFS
$Q_B = 0.73$ CFS
$Q_C = 3.75$ CFS $\}$ $Q_T = 5.30 - 0.73 + 3.75 = 8.32$ CFS, $A = 1.455$ ft²

$V = Q/A = 8.32/1.455 = 5.70$ fps

$T = 0.187/0.02 + 2.0 = 11.4'$ ok

$E_o = Q_w/Q = (5.30 - 0.73)/8.32 = 0.549$

$Q_i = 0.549 \times 8.30 = 4.56$ CFS

$Q_b = 8.30 - 4.56 = 3.74$ CFS

Use Det. P-1572, Type Q
$Q_i = 4.56$ CFS
$Q_b = 3.74$ CFS

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INLET NO. 510-4 Located @ sta. 11+91

Flowby from Inlet No. 510-3, $Q_b = 3.74$ CFS

$Q = 0.91 + 3.74 = 4.65$ CFS $\quad S_x = 0.020$ '/ft

$S = 0.0253$ '/ft $\quad Z_x/n = 3333$

(a) Curb Opening Inlet

$a = 2$ in. $\quad W = 17$ in. $\quad S_w' = 2/17 = 0.1176 \quad Z_w/n = 484.7$

C.F. = 1.25

$S_w = 0.1176 + 0.020 = 0.1376$

Assume $d = 0.355'$ & $d' = 0.355' - (1.417)(0.1376) = 0.160'$

$Q_A = 2.73$ CFS, $Q_B = 0.33$ CFS, $Q_C = 2.24$ CFS $\quad Q_T = 2.73 - 0.33 + 2.24 = 4.64$ CFS

Gutter Flow, $Q_w = 2.73 - 0.33 = 2.40$ CFS

$E_o = Q_w/Q = 2.40/4.64 = 0.517$

$S_e = 0.020 + (0.1176)(0.517) = 0.0808$

$$L_T = 0.6(4.65)^{0.42}(0.0253)^{0.3}\left[1/(0.015)(0.0808)\right]^{0.6} \times 1.25$$
$$= 26.68'$$

(i) Try Det. P-1569, Type 'M-1', $L = 10'$

$L/L_T = 13/26.68 = 0.4873$

$E = 1 - (1 - 0.4873)^{1.8} = 0.70$

$Q_i = 0.70 \times 4.65 = 3.25$ CFS

$Q_b = 4.65 - 3.25 = 1.40$ CFS

(b) Combination Inlet, Det. P-1572, Type Q

$S_w' = 2/24 = 0.0833$

$S_w = 0.0833 + 0.020 = 0.1033 \quad Z_w/n = 645$

Assume $d = 0.347'$ & $d' = 0.347 - 0.207 = 0.140'$

$Q_A = 3.42$ CFS, $Q_B = 0.30$ CFS, $Q_C = 1.57$ CFS $\quad Q_T = 3.42 - 0.30 + 1.57 = 4.69$ CFS, $A = 0.977$ ft^2

$V = 4.69/0.977 = 4.8$ fps $\quad T = 0.14/0.02 + 2 = 9.0$ ft

$E_o = Q_w/Q = (3.42 - 0.30)/4.69 = 0.665$

$Q_i = 0.665 \times 4.65 = 3.09$ CFS

$Q_b = 4.65 - 3.09 = 1.56$ CFS

Use Det. P-1572, Type Q
$Q_i = 3.09$ CFS
$Q_b = 1.56$ CFS

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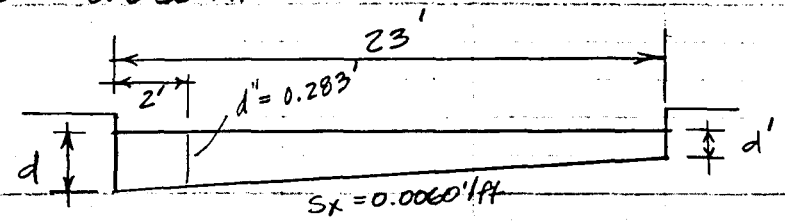
INLET NO. 500-1

$Q_2 = 26.53$ CFS $\qquad S_x = 0.006$ '/ft

$S = 0.020$ '/ft.



Try $d = 0.295'$ & $d' = 0.295 - 0.138 = 0.157'$

$A = 5.198\ ft^2$, $P = 23.45\ ft$ $\Big\}\ R = \frac{A}{P} = 0.2216$ ft.

$$Q = \frac{1.486}{0.015}(5.198)(0.2216)^{2/3}(0.020)^{1/2} = 26.67\ CFS \approx 26.53\ CFS$$

Compute Q in 2' gutter:

$A = \left(\frac{0.295 + 0.283}{2}\right)(2) = 0.578\ ft^2$, $P = 0.295 + 2 = 2.295\ ft$ $\Big\}\ R = \frac{A}{P} = 0.2519$ ft

$$Q = \frac{1.486}{0.015}(0.578)(0.2519)^{2/3}(0.02)^{1/2} = 3.23\ CFS$$

Try Neenah R-3578 Vane Grate

$K = 49.5$

$Q = Kd^{5/3} = 49.5(0.295)^{5/3}$

$= 6.47$ CFS > 3.23 CFS ok All flow in gutter is intercepted

Use 7 - 3 ft. section of slotted vane drains perpendicular to flow. According to Neenah, slotted vane drains will intercept 0.5 CFS per lineal feet.

$\therefore\ 21' \times 0.5\ CFS/ft = 10.50$ CFS

$\therefore$ Total $Q_i = 3.23 + 10.50 = 13.73$ CFS

Flowby $Q_b = 26.53 - 13.73 = 12.80$ CFS

Use Det. P-1572, Type Q w/ Type R-3578 Neenah vane grate and 7 - Type R-3599A slotted Vane Drains.

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	For		

INLET NO. 500 - 2

Flowby from Inlet #510-3

$Q = 0.83 + 1.56 = 2.39$ CFS $\qquad S_x = 0.020$ '/ft

$S = 0.0220$ '/ft $\qquad Z_x/n = 3333.33$

(a) Curb Opening Inlet

$a = 2$ in. $\}$ $S_w' = 2/17 = 0.1176$ $\qquad Z_w/n = 484.7$

$W = 17$ in. $\qquad$ C.F. = 1.25

$S_w = 0.1176 + 0.020 = 0.1376$

Assume $d = 0.307'$ & $d' = 0.307' - (1.417)^{.195}(0.1376) = 0.112'$

$Q_A = 1.73$ CFS
$Q_B = 0.12$ CFS
$Q_C = 0.81$ CFS $\Big\}$ $Q_T = 1.73 - 0.12 + 0.81 = 2.42$ CFS

Gutter Flow, $Q_w = 1.73 - 0.12 = 1.61$ CFS

$E_o = Q_w/Q = 1.61/2.42 = 0.6653$

$S_e = 0.020 + (0.1176)(0.6653) = 0.0982$

$L_T = 0.6(2.39)^{0.42}(0.022)^{0.3}\left[1/(0.015)(0.0982)\right]^{0.6} \times 1.25$
$= 17.21'$

(i) Try $L = 6'$ Det. P-1569, Type 'M-1'

$L/L_T = 9/17.21 = 0.5230$

$E = 1-(1-0.5230)^{1.8} = 0.736$

$Q_i = 0.736 \times 2.39 = 1.76$ CFS

$Q_b = 2.39 - 1.76 = 0.63$ CFS

(b) Combination Inlet

Try Det. P-1572, Type Q

$S_w' = 2/24 = 0.0833$ $\qquad Z_w/n = 645$

$S_w = 0.0833 + 0.020 = 0.1033$

Assume $d = 0.295'$ & $d' = 0.295 - (0.1033)^{.207}(2) = 0.088'$

$Q_A = 2.07$ CFS
$Q_B = 0.08$ CFS
$Q_C = 0.42$ CFS $\Big\}$ $Q_T = 2.07 - 0.08 + 0.42 = 2.41$ CFS

$A = 0.577$ ft²

$V = Q/A = 2.41/0.577 = 4.18$ fps

$T = 0.088/0.02 + 2.0 = 6.4'$ OK

$E_o = Q_w/Q = (2.07 - 0.08)/2.41 = 0.826$

$Q_i = 0.826 \times 2.39 = 1.97$ CFS

$Q_b = 2.39 - 1.97 = 0.42$ CFS

Use Det. P-1572, Type Q
$Q_i = 1.97$ CFS
$Q_b = 0.42$ CFS

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INLET NO. 500-3

$Q = 0.30 + 0.42 + 12.80 = 13.52$ CFS (0.42: Inlet #500-2 Flowby; 12.80: Inlet #500-1 Flowby)

$S = 0.0150$ '/ft

$S_x = 0.020$ '/ft

$Z_x/n = 3333.33$

(a) Curb Opening Inlet

$a = 2$ in., $W = 17$ in. $\}$ $S_w' = 2/17 = 0.1176$

$Z_w/n = 484.7$

C.F. = 1.25

$S_w = 0.1176 + 0.020 = 0.1376$

Assume $d = 0.50'$ & $d' = 0.50' - (1.417)(0.1376) = 0.305'$

$Q_A = 5.24$ CFS
$Q_B = 1.40$ CFS
$Q_C = 9.64$ CFS
$\}$ $Q_T = 5.24 - 1.40 + 9.64 = 13.48$ CFS

Gutter Flow, $Q_w = 5.24 - 1.40 = 3.84$ CFS

$E_0 = Q_w/Q = 3.84/13.48 = 0.285$

$S_e = 0.020 + (0.1176)(0.285) = 0.0535$

$L_T = 0.6\,(13.52)^{0.42}(0.015)^{0.3}\left[1/(0.015)(0.0535)\right]^{0.6} \times 1.25$
$= 45.73'$

(i) Try $L = 17'$, Det. P-1569, Type 'M-1'

$L/L_T = 20/45.73 = 0.437$

$E = 1-(1-0.437)^{1.8} = 0.644$

$Q_i = 0.644 \times 13.52 = 8.71$ CFS

$Q_b = 13.52 - 8.71 = 4.81$ CFS

$T = \frac{0.305}{0.02} + 1.42 = 16.7' < 22'$ OK

(b) Combination Inlet

(i) Try Det. P-1572, Type Q

$S_w' = 2/24 = 0.0833$

$S_w = 0.0833 + 0.020 = 0.1033$

Assume $d = 0.495'$ & $d' = 0.495 - (0.1033)(2) = 0.288'$ (.207)

$Q_A = 6.78$ CFS
$Q_B = 1.60$ CFS
$Q_C = 8.27$ CFS
$\}$ $Q_T = 6.78 - 1.60 + 8.27 = 13.45$ CFS
$A = 2.857$ ft²

$V = Q/A = 13.45/2.857 = 4.71$ fps

$T = 0.288/0.02 + 2.0 = 16.4'$

$E_0 = Q_w/Q = (6.78 - 1.60)/13.45 = 0.385$

$Q_i = 0.385 \times 13.52 = 5.21$ CFS

$Q_b = 13.52 - 5.21 = 8.31$ CFS

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INLET NO. 500-3 (Cont'd)

(b) (ii) Try Det. P-1567, Type K14

$L/L_T = (14-40/12)/45.73 = 0.233$

$E = 1-(1-0.233)^{1.8} = 0.380$

$Q_i = 0.380 \times 13.52 = 5.14$ CFS

$Q_b = 13.52 - 5.14 = 8.38$ CFS

Assume $d = 0.441'$ $\quad d' = 0.441 - (1.417)^{.195}(0.1376) = 0.246'$

$Q_A = 3.75$ CFS
$Q_B = 0.79$ CFS
$Q_C = 5.43$ CFS

$Q_T = 3.75 - 0.79 + 5.43 = 8.39$ CFS
$A = 2.001$ ft^2

$V = 8.39/2.001 = 4.19$

$T = 0.246/0.02 + 1.42 = 13.72'$

$E = Q_W/Q = (3.75-0.79)/8.39 = 0.353$

$Q_i = 0.353 \times 8.39 = 2.96$ CFS

Total $Q_i = 5.14 + 2.96 = 8.10$ CFS
$Q_b = 13.52 - 8.10 = 5.42$ CFS

Use Det. P-1567, Type K14
$Q_i = 8.10$ CFS
$Q_b = 5.42$ CFS

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INLET NO. 500-4 — Convert to ΣCA — Sump Condition - 10 Year

$\Sigma CA = 5.43/2.4 = 2.26$ Ac.

$Q_{b,\#500-3} = 5.42$ cfs $\qquad S_x = 0.020$ '/ft

$Q_{10} = (0.27 + 2.26)3.8 = 9.61$ cfs $\qquad Z_x/n = 3333.33$

$S = 0.0064$ '/ft

(a) Curb Opening Inlet

$a = 2$ in. $\qquad h = 5$ in.

$W = 17$ in. or 1.42 ft $\qquad C.F. = 1.25$

$$Q = C_w (L + 1.8W) d^{1.5}$$

$$d = \left\{ (9.61) / [0.56(3333.33)(0.0064)^{1/2} \right\}^{3/8} = 0.357'$$

$$T = d/S_x = 0.357/0.020 = 17.9' < 20' \quad \text{OK}$$

$T_{max} = 20'$

$d_{max} = T S_x = (20)(0.020) = 0.40'$

$h = 5$ in. $\qquad a = 2$ in. $\qquad C.F. = 1.25$

Since $d_{max} < h + a$

$$Q = C_w (L/C.F. + 1.8W) d^{1.5}$$

(i) Try Det. P-1569, Type M-1, $L = 17'$

$$Q = 2.3 [20/1.25 + 1.8(17/12)](0.40)^{1.5}$$

$$= 10.79 \text{ cfs} > 9.61 \text{ cfs} \quad \underline{\text{OK}}$$

(b) Combination Inlet

$$Q = C_w P d^{1.5}$$

(i) Try Det. P-1572, Type Q Double

$P = 2(24/12) + 2(40/12) = 10.67'$

$Q_i = (3.0)(10.67)(0.40)^{1.5} = 8.10$ cfs < 9.61 cfs No Good

(ii) Try Det. P-1572, Type Q Triple

$P = 2(24/12) + 3(40/12) = 14'$

$Q_i = (3.0)(14.0)(0.40)^{1.5} = 10.63$ cfs > 9.61 cfs

Use Det. P-1572, Type Q Triple

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INLET NO. 400-1

$Q = 1.58$ CFS $\quad S_x = 0.020$ '/ft
$S = 0.0075$ '/ft $\quad Z_x/n = 3333.33$

(a) Curb Opening Inlet

$a = 2$ in. $\}$ $S_w' = 2/17 = 0.1176$ $\quad Z_w/n = 484.7$
$W = 17$ in. $\quad$ C.F. = 1.25

$S_w = 0.1176 + 0.020 = 0.1376$

Assume $d = 0.316'$ & $d' = 0.316 - (0.1376)(1.417) = 0.121'$

$Q_A = 1.09$ CFS
$Q_B = 0.08$ CFS $\}$ $Q_T = 1.09 - 0.08 + 0.58 = 1.59$ CFS
$Q_C = 0.58$ CFS

Gutter Flow, $Q_w = 1.09 - 0.08 = 1.01$ CFS

$E_o = Q_w/Q = 1.01/1.59 = 0.635$

$S_e = 0.020 + (0.1176)(0.635) = 0.0947$

$L_T = (0.6)(1.58)^{0.42}(0.0075)^{0.3}\left[1/(0.015)(0.0947)\right]^{0.6} \times 1.25$
$= 10.72'$

(i) Try Conc. Scupper, MAG Det. 206, $L = 6'$

$L/L_T = 6/10.72 = 0.560$

$E = 1 - (1 - 0.560)^{1.8} = 0.772$

$Q_i = 0.772 \times 1.58 = 1.22$ CFS $\quad T = 7.47'$ ok

$Q_b = 1.58 - 1.22 = 0.36$ CFS

(b) Combination Inlet

(i) Try Det. P-1572, Type Q

$S_w' = 2/24 = 0.0833$ $\quad Z_w/n = 645$

$S_w = 0.0833 + 0.020 = 0.1033$

Assume $d = 0.305'$ & $d' = 0.305 - (0.1033)(2.0) = 0.098'$

$Q_A = 1.32$ CFS
$Q_B = 0.06$ CFS $\}$ $Q_T = 1.32 - 0.06 + 0.33 = 1.59$ CFS
$Q_C = 0.33$ CFS $\quad Q_w = 1.32 - 0.06 = 1.26$ CFS

$E = Q_w/Q = 1.26/1.59 = 0.792$

$Q_i = 0.792 \times 1.58 = 1.25$ CFS

$Q_b = 1.58 - 1.25 = 0.33$ CFS $\quad T = 6.91'$ ok

Use Scupper with $L = 6'$ M.A.G. Det. 206	$Q_i = 1.22$ CFS $Q_b = 0.36$ CFS

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INLET NO. 410-1

$Q = 0.99 + 3.61 + 1.39 = 5.99$ CFS $\qquad S_x = 0.020$ '/ft

$S = 0.0044$ '/ft $\qquad Z_x/n = 3333.33$

(a) Curb Opening Inlet

$a = 2$ in. $\}$ $S_w' = 2/17 = 0.1176$ $\qquad Z_w/n = 484.7$

$W = 17$ in. $\qquad$ C.F. $= 1.25$

$S_w = 0.1176 + 0.020 = 0.1376$

Assume $d = 0.474'$ & $d' = 0.474 - (1.417)(0.1376) = 0.279'$

$Q_A = 2.46$ CFS
$Q_B = 0.60$ CFS
$Q_C = 4.12$ CFS $\Big\}$ $Q_T = 2.46 - 0.60 + 4.12 = 5.98$ CFS

Gutter Flow, $Q_w = 2.46 - 0.60 = 1.86$ CFS

$E_o = Q_w/Q = 1.86/5.98 = 0.311$

$S_e = 0.020 + (0.1176)(0.311) = 0.0566$

$L_T = (0.6)(5.99)^{0.42}(0.0044)^{0.3}\left[1/(0.015)(0.0566)\right]^{0.6} \times 1.25$
$= 21.75'$

(i) Try Det. P-1569, Type 'M-1', $L = 10'$

$L/L_T = 13/21.75 = 0.60$

$E = 1-(1-0.60)^{1.8} = 0.808$

$Q_i = 0.808 \times 5.99 = 4.84$ CFS $\qquad T = 15.4'$ ok

$Q_b = 5.99 - 4.84 = 1.15$ CFS

(b) Combination Inlet

(i) Try Det. P-1567, Type K14

$L/L_T = (14 - 40/12)/21.75 = 0.49$

$E = 1-(1-0.49)^{1.8} = 0.702$

$Q_i = 0.702 \times 5.99 = 4.21$ CFS

$Q_b = 5.99 - 4.21 = 1.78$ CFS

Assume $d = 0.348'$ & $d' = 0.348 - (1.417)(0.1376) = 0.153'$

$Q_A = 1.08$ CFS
$Q_B = 0.12$ CFS
$Q_C = 0.83$ CFS $\Big\}$ $Q_T = 1.08 - 0.12 + 0.83 = 1.79$ CFS
$Q_w = 1.08 - 0.12 = 0.96$ CFS

$E = Q_w/Q = 0.96/1.78 = 0.539$

$Q_i = 0.539 \times 1.78 = 0.96$ CFS

Total $Q_i = 4.21 + 0.96 = 5.17$ CFS
$Q_b = 5.99 - 5.17 = 0.82$ CFS

Use Det. P-1567, Type K14	$Q_i = 5.17$ CFS $Q_b = 0.82$ CFS

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INLET NO. 410-2

$Q = 1.62 + 0.36 = 1.98$ CFS $\qquad S_x = 0.020$ 1/ft

$S = 0.0087$ 1/ft $\qquad Z_x/n = 3333.33$

Curb Opening Inlet

$a = 2$ in. $\quad W = 17$ in. $\quad \Big\} \; S_w' = 2/17 = 0.1176 \qquad Z_w/n = 484.7$

$C.F. = 1.25$

$S_w = 0.1176 + 0.020 = 0.1376$

Assume $d = 0.331'$ & $\quad d' = 0.331 - (1.417)(0.1376) = 0.136'$

$Q_A = 1.33$ CFS

$Q_B = 0.12$ CFS

$Q_C = 0.85$ CFS

$\Big\} \; Q_T = 1.33 - 0.12 + 0.85 = 2.06$ CFS $\simeq 1.98$ CFS

Gutter Flow, $Q_w = 1.33 - 0.12 = 1.21$ CFS

$E_0 = Q_w/Q = 1.21/2.06 = 0.587$

$S_e = 0.020 + (0.1176)(0.587) = 0.089$

$$L_T = (0.6)(1.98)^{0.42}(0.0087)^{0.3}\left[1/(0.015)(0.089)\right]^{0.6} \times 1.25$$
$$= 12.77 \text{ ft.}$$

i) Try $L = 6'$ Curb Opening

$L/L_T = 6/12.77 = 0.470$

$E = 1 - (1 - 0.470)^{1.8} = 0.681$

$Q_i = 0.681 \times 1.98 = 1.35$ CFS $\qquad T = 8.8'$ ok

$Q_b = 1.98 - 1.35 = 0.63$ CFS

(ii) Try $L = 8'$ Curb Opening Scupper

$L/L_T = 8/12.77 = 0.627$

$E = 1 - (1 - 0.627)^{1.8} = 0.83$

$Q_i = 0.83 \times 1.98 = 1.64$ CFS

$Q_b = 1.98 - 1.64 = 0.34$ CFS

Use Scupper with L = 8 ft.

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INLET NO. 410-3

Sump condition – Use 10-Year Q

Convert Flowby from Inlets #410-1 & #410-2 to ΣCA :

$Q = \Sigma CA\, I$

$\Sigma CA = Q/I = (0.82 + 0.34)/2.4 = 0.483$

Total $\Sigma CA = 0.56 + 0.48 = 1.04$

$t_c = 10$ min. $\rightarrow$ $I_{10} = 3.8$ in/hr

$\therefore Q_{10} = 1.04 \times 3.8 = 3.95$ CFS

$$d = \left[\frac{3.95}{(0.56)(50/.015)(0.0040)^{1/2}}\right]^{3/8} = 0.280'$$

$T = 0.28/0.020 = 14.0'$ ok

$T_{max} = 20'$

$d_{max} = T S_x = (20)(0.020) = 0.40'$

$h = 5$ in. $a = 2$ in. C.F. = 1.25

since $d_{max} < h + a$

$$Q = C_w (L/C.F. + 1.8\,W)\, d^{1.5}$$

(i) Try $L = 8'$:

$$Q = 2.3\left[8/1.25 + 1.8(17/12)\right](0.40)^{1.5}$$
$$= 5.2 \text{ CFS} > Q_{10} = 3.95 \text{ CFS} \quad \text{ok}$$

(ii) Try $L = 6'$:

$$Q = 2.3\left[6/1.25 + 1.8(17/12)\right](0.40)^{1.5}$$
$$= 4.28 \text{ CFS} > Q_{10} = 3.95 \text{ CFS} \quad \text{ok}$$

Use Conc. Scupper, MAG Det. 206, L = 6 ft.

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INLET NO. 410-4

$Q = 1.19$ CFS
$S = 0.0075$ '/ft

$S_x = 0.020$ '/ft
$Z_x/n = 3333.33$

Curb Opening Inlet

$a = 2$ in.
$W = 17$ in. $\Big\}$ $S_w' = 2/17 = 0.1176$

$Z_w/n = 484.7$
C.F. $= 1.25$

$S_w = 0.1176 + 0.020 = 0.1376$

Assume $d = 0.294'$ & $d' = 0.294 - (1.417)(0.1376) = 0.099'$

$Q_A = 0.90$ CFS
$Q_B = 0.05$ CFS
$Q_C = 0.34$ CFS $\Big\}$ $Q_T = 0.90 - 0.05 + 0.34 = 1.19$ CFS

Gutter Flow, $Q_w = 0.90 - 0.05 = 0.85$ CFS

$E_0 = Q_w/Q = 0.85/1.19 = 0.714$

$S_e = 0.020 + (0.1176)(0.714) = 0.104$

$$L_T = (0.6)(1.19)^{0.42}(0.0075)^{0.3}\left[1/(0.015)(0.104)\right]^{0.6} \times 1.25$$
$$= 9.00 \text{ ft.}$$

i) Try $L = 4'$ Curb Opening

$L/L_T = 4/9.00 = 0.444$

$E = 1 - (1 - 0.444)^{1.8} = 0.653$

$Q_i = 0.653 \times 1.19 = 0.78$ CFS

$Q_b = 1.19 - 0.78 = 0.41$ CFS

$T = 6.38'$ ok

Use Conc. Scupper, M.A.G. Det. 206, $L = 4$ ft.

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INLET NO. 410-5

$Q = 1.13$ CFS
$S = 0.0044$ '/ft

$S_x = 0.020$ '/ft
$Z_x/n = 3333.33$

Curb Opening Inlet

$a = 2$ in.
$W = 17$ in. $\Big\}$ $S_w' = 2/17 = 0.1176$

$Z_w/n = 484.7$
C.F. $= 1.25$

$S_w = 0.1176 + 0.020 = 0.1376$

Assume $d = 0.311'$ $\quad d' = 0.311 - (1.417)(0.1376) = 0.116'$

$Q_A = 0.80$ CFS
$Q_B = 0.06$ CFS
$Q_C = 0.39$ CFS $\Big\}$ $Q_T = 0.80 - 0.06 + 0.39 = 1.13$ CFS

Gutter Flow, $Q_W = 0.80 - 0.06 = 0.74$ CFS

$E_0 = Q_W/Q = 0.74/1.13 = 0.655$

$S_e = 0.020 + (0.1176)(0.655) = 0.097$

$$L_T = (0.6)(1.13)^{0.42}(0.0044)^{0.3}\left[1/(0.015)(0.097)\right]^{0.6} \times 1.25$$
$$= 7.83'$$

i) Try $L = 4'$ Curb Opening

$L/L_T = 4/7.83 = 0.511$

$E = 1 - (1 - 0.511)^{1.8} = 0.724$

$Q_i = 0.724 \times 1.13 = 0.82$ CFS

$Q_b = 1.13 - 0.82 = 0.31$ CFS

Use Conc. Scupper, M.A.G. Det. 206, L = 4 ft.

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INLET NO. 410-6

Sump Condition - Use 10-Year Q
Convert flowby from Inlets #410-4 & #410-5 to ΣCA :

$Q = \Sigma CA I$

$\Sigma CA = Q/I = (0.41 + 0.31)/2.4 = 0.30$

Total $\Sigma CA = 0.46 + 0.30 = 0.76$

$t_c = 10\,min \rightarrow I_{10} = 3.8\ in/hr$

$\therefore Q_{10} = 0.76 \times 3.8 = 2.90\ CFS$

$$d = \left[\frac{2.90}{(0.56)(50/.015)(0.0037)^{1/2}}\right]^{3/8} = 0.253'$$

$T = 0.253/0.020 = 12.7'$ OK

$T_{max} = 20'$

$d_{max} = TS_x = (20)(0.020) = 0.40'$

$h = 5\,in.$ $a = 2\,in.$ $C.F. = 1.25$

Since $d_{max} < h + a$

$$Q = C_w(L/C.F. + 1.8W)\,d^{1.5}$$

(i) Try $L = 4'$:

$$Q = 2.3\left[4/1.25 + 1.8(17/12)\right](0.40)^{1.5}$$

$= 3.35\ CFS > Q_{10} = 2.90\ CFS$ OK

Use Conc. Scupper, M.A.G. Det. 206, L = 4 ft.

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INLET NO. 422

$Q = 9.65$ CFS $\qquad S_x = 0.0140$ '/ft

$S = 0.0110$ '/ft $\qquad Z_x/n = 4762$

(a) Curb Opening Inlet

$a = 2$ in. $\}$ $S_w' = 2/17 = 0.118$ $\qquad Z_w/n = 505$

$W = 17$ in. $\qquad$ C.F. = 1.25

$S_w = 0.118 + 0.014 = 0.132$

Assume $d = 0.44'$ & $d' = 0.44 - (1.417)(0.132) = 0.253'$

$Q_A = 3.32$ CFS
$Q_B = 0.76$ CFS
$Q_C = 7.16$ CFS
$\}$ $Q_T = 3.32 - 0.76 + 7.16 = 9.72$ CFS

Gutter Flow, $Q_w = 3.32 - 0.76 = 2.56$ CFS

$E_0 = Q_w/Q = 2.56/9.72 = 0.263$ $\qquad T = \frac{0.252}{0.014} + 1.42 = 19.5'$

$S_e = 0.014 + (0.118)(0.263) = 0.045$

$L_T = (0.60)(9.65)^{0.42}(0.0110)^{0.3}\left[1/(0.015)(0.045)\right]^{0.6}(1.25)$
$= 40.11'$

(i) Try $L = 17'$ (Det. P-1569, Type 'M-1')

$L/L_T = 20/40.11 = 0.499$

$E = 1 - (1 - 0.499)^{1.8} = 0.712$

$Q_i = 0.712 \times 9.65 = 6.87$ CFS

$Q_b = 9.65 - 6.87 = 2.78$ CFS

(b) Combination Inlet

(i) Try Det. P-1572, Type Q

$S_w' = 2/24 = 0.083$ $\qquad Z_w/n = 685$

$S_w = 0.083 + 0.0140 = 0.0973$

Assume $d = 0.435'$ & $d' = 0.435 - (2)(0.0973) = 0.240'$

$Q_A = 4.37$ CFS
$Q_B = 0.90$ CFS
$Q_C = 6.22$ CFS
$\}$ $Q_T = 4.37 - 0.90 + 6.22 = 9.69$ CFS
$A = 2.7321$ ft^2

$V = Q/A = 9.69/2.7321 = 3.55$ fps

$T = 0.240/0.014 + 2 = 19.14'$

$E_0 = Q_w/Q = (4.37 - 0.90)/9.69 = 0.358$

$Q_i = 0.358 \times 9.65 = 3.45$ CFS

$Q_b = 9.65 - 3.45 = 6.20$ CFS

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INLET NO. 422 (Cont'd)

(b) (ii) Try Det. P-1567, Type K14

$L/L_T = (14 - 40/12)/40.11 = 0.2659$

$E = 1-(1-0.2659)^{1.8} = 0.427$

$Q_i = 0.427 \times 9.65 = 4.12$ CFS

$Q_b = 9.65 - 4.12 = 5.53$ CFS

Assume $d = 0.384'$ & $d' = 0.384 - (1.417)(0.132) = 0.197'$

$Q_A = 2.31$ CFS
$Q_B = 0.39$ CFS
$Q_C = 3.67$ CFS

$Q_T = 2.31 - 0.39 + 3.67 = 5.59$ CFS

$E = Q_w/Q = 1.92/5.59 = 0.344$

$Q_i = 0.344 \times 5.59 = 1.92$ CFS

Total $Q_i = 4.12 + 1.92 = 6.04$ CFS

$E = \frac{6.04}{9.65} = 0.63$

$Q_b = 9.65 - 6.04 = 3.61$ CFS

~~Use Det. P-1567, Type K14~~
~~$Q_i = 6.04$ CFS~~
~~$Q_b = 3.61$ CFS~~

Not Recommended Due to Utility Conflicts

(c) Try Type R Modified, $L = 10'$

$L/L_T = (13 - 40/12)/40.11 = 0.241$

$E = 1-(1-0.241)^{1.8} = 0.391$

$Q_i = 0.391 \times 9.65 = 3.78$ CFS

$Q_b = 9.65 - 3.78 = 5.87$ CFS

Assume $d = 0.389'$ & $d' = 0.389 - (1.417)(0.132) = 0.202'$

$Q_A = 2.39$ CFS
$Q_B = 0.42$ CFS
$Q_C = 3.93$ CFS

$Q_T = 2.39 - 0.42 + 3.93 = 5.90$ CFS

$E = Q_w/Q = 1.97/5.90 = 0.334$

$Q_i = 0.334 \times 5.87 = 1.96$ CFS

Total $Q_i = 3.78 + 1.96 = 5.74$ CFS

$E = \frac{5.74}{9.65} = 0.59$

$Q_b = 9.65 - 5.74 = 3.91$ CFS

Use Type R Modified, $L = 10'$
$Q_i = 5.74$ CFS
$Q_b = 3.91$ CFS

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INLET NO. 421-1

Sump Condition

$Q_{2yr} = 5.35$ cfs
$Q_{10yr} = 8.47$ cfs
$S = 0.0052$ '/ft

$S_x = 0.020$'/ft
$Z_x/n = 3333.33$

For 10-Year, $d_{10} = \left\{(8.47)/[0.56(3333.33)(0.0052)^{1/2}\right\}^{3/8} = 0.355'$

$T_{10} = 0.355/0.020 = 17.8$ ft < 20 ft ok

$T_{max} = 20'$
$d_{max} = TS_x = (20)(0.020) = 0.40'$

(a) Curb Opening Inlet
$h = 5$ in. $a = 2$ in. C.F. = 1.25
Since $d < h+a$, $Q = C_w(L/C.F. + 1.8W)d^{1.5}$

(i) Try Det. P-1569, Type 'M-1', $L = 10'$
$Q = 2.3[13/1.25 + (1.8)(17/12)](0.40)^{1.5}$
$= 7.54$ cfs < 8.47 cfs No Good

(ii) Try Det. P-1569, Type 'M-2', $L = 6'$
$Q = 2.3[15/1.25 + (1.8)(17/12)](0.40)^{1.5}$
$= 8.47$ cfs ok

(b) Combination Inlet
$d_{max} = 0.40'$
Since $d < h+a$, $Q = C_w P d^{1.5} = 3.0 P d^{1.5}$

(i) Try Det. P-1572, Type Q Triple
$P = 2(24/12) + 3(40/12) = 14.0'$
$Q = (3.0)(14.0)(0.40)^{1.5}$
$= 10.63$ cfs > 8.47 cfs ok

Use Det. P-1572, Type Q Triple

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INLET NO. 421-2

$Q = 6.75$ CFS
$S = 0.0120$ '/ft

$S_x = 0.020$
$Z_x/n = 3333$

(a) Curb Opening Inlet

$a = 2$ in. } $S_w' = \frac{2}{17} = 0.118$
$W = 17$ in. }

$Z_w/n = 483$
C.F. $= 1.25$

$S_w = 0.118 + 0.020 = 0.138$
Assume $d = 0.430'$ & $d' = 0.235'$

$Q_A = 3.12$ CFS
$Q_B = 0.62$ CFS
$Q_C = 4.30$ CFS
} $Q_T = 3.12 - 0.62 + 4.30 = 6.80$ CFS ≈ 6.75 CFS

Gutter Flow, $Q_w = 3.12 - 0.62 = 2.50$ CFS
$E_o = Q_w/Q = 2.50/6.80 = 0.368$
$S_e = 0.020 + (0.118)(0.368) = 0.0634$

$T = \frac{0.235}{0.02} + 1.42 = 13.2'$

$$L_T = (0.6)(6.75)^{.42}(0.0120)^{0.3}\left[\frac{1}{(0.015)(0.0634)}\right]^{0.6}(1.25)$$
$= 28.85'$

(i) Try $L = 17'$ (P-1569, Type 'M-1')
$L/L_T = 20/28.85 = 0.693$
$E = 1 - (1 - 0.693)^{1.8} = 0.880$
$Q_i = 0.880 \times 6.75 = 5.94$ CFS
$Q_b = 6.75 - 5.94 = 0.81$ CFS

(ii) Try $L = 16'$ (Double MAG 532)
$L/L_T = 16/28.85 = 0.555$
$E = 1 - (1 - 0.555)^{1.8} = 0.767$
$Q_i = 0.767 \times 6.75 = 5.18$ CFS
$Q_b = 6.75 - 5.18 = 1.57$ CFS

(b) Combination Inlet

(i) Try Det. P-1567, K14
$L/L_T = (14 - 40/12)/28.85 = 0.370$
$E = 1 - (1 - 0.370)^{1.8} = 0.565$
$Q_i = 0.565 \times 6.75 = 3.81$ CFS
$Q_b = 6.75 - 3.81 = 2.94$ CFS

Assume $d = 0.350'$ & $d' = 0.350' - (1.417)(0.138) = 0.155'$
$Q_A = 1.80$ CFS
$Q_B = 0.21$ CFS
$Q_C = 1.42$ CFS
} $Q_T = 1.80 - 0.21 + 1.42 = 3.01$ CFS
$A = 0.959$ ft^2

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INLET NO. 421-2 (Cont'd)

(b) (i) Cont'd

$V = Q/A = 3.01/0.959 = 3.14$ fps

$T = 0.155/0.020 + 1.42 = 9.2'$

$E = (1.80 - 0.21)/3.01 = 0.528$

$Q_i = 0.528 \times 2.94 = 1.55$ CFS

Total $Q_i = 3.81 + 1.55 = 5.36$ CFS

$Q_b = 6.75 - 5.36 = 1.39$ CFS

Use Det. P-1567, Type K14

$Q_i = 5.36$ cfs

$Q_b = 1.39$ CFS

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INLET NO. 421-3 RT.

Sump Condition

$Q_{10yr} = 1.58$ CFS

$S = 0.0052$ 1/ft

$S_x = 0.025$ 1/ft

$Z_x/n = 2666.67$

$$d = \left[1.58/(0.56)(2666.67)(0.0052)^{1/2}\right]^{3/8} = 0.21'$$

$$T = d/S_x = 0.21/0.025 = 8.4' \quad \text{ok}$$

$$d_{max} = 0.50'$$

$$T_{max} = d/S_x = 0.50/0.025 = 20'$$

For Curb Opening Inlet, $d < h + a$

$$Q = 2.3(L/C.F. + 1.8W)\,d^{1.5}$$

(i) Try Conc. Scupper, $L = 4'$

$$Q = 2.3\left[4/1.25 + 1.8(17/12)\right](0.50)^{1.5}$$

$$= 4.68 \text{ CFS} > 1.58 \text{ CFS}$$

(ii) Try Conc. Scupper, $L = 2'$

$$Q = 2.3\left[2/1.25 + 1.8(17/12)\right](0.50)^{1.5}$$

$$= 3.37 \text{ CFS} > 1.58 \text{ CFS} \quad \text{ok}$$

Use Conc. Scupper, M.A.G. Det. 206, L=2'

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INLET NO. 431-1

Design Inlet to Intercept a portion of the 10 Year Q.

$Q_{10} = 7.43$ CFS $\quad S_x = 0.015$ '/ft

$S = 0.0220$ '/ft $\quad Z_x/n = 4444.67$

(a) Curb Opening Inlet

$a = 2$ in. $\quad W = 17$ in. $\quad S_w' = \frac{2}{17} = 0.118 \quad Z_w/n = 501 \quad C.F. = 1.25$

$S_w = 0.118 + 0.015 = 0.133$

Assume $d = 0.385'$ $\quad d' = 0.385' - (1.417)(0.13) = 0.197'$

$Q_A = 3.26$ CFS
$Q_B = 0.55$ CFS
$Q_C = 4.85$ CFS
$\quad Q_T = 3.26 - 0.55 + 4.85 = 7.56$ CFS

Gutter Flow, $Q_w = 3.26 - 0.55 = 2.71$ CFS

$E_o = Q_w/Q = 2.71/7.56 = 0.359$

$S_e = 0.015 + (0.118)(0.359) = 0.0574$

$$L_T = (0.6)(7.43)^{.42}(0.022)^{0.3}\left[\frac{1}{(0.015)(0.0574)}\right]^{0.6}(1.25)$$
$$= 38.26'$$

(i) Try $L = 17'$ (P-1569, Type M-1)

$L/L_T = 20/38.26 = 0.523$

$E = 1-(1-0.523)^{1.8} = 0.736$

$Q_i = 0.736 \times 7.43 = 5.47$ CFS

$Q_b = 7.43 - 5.47 = 1.96$ CFS

(b) Combination Inlet

Try Det. P-1567, Type K14

$L/L_T = (14 - 40/12)/38.26 = 0.279$

$E = 1-(1-0.279)^{1.8} = 0.445$

$Q_i = 0.445 \times 7.43 = 3.31$ CFS

$Q_b = 7.43 - 3.31 = 4.12$ CFS

Assume $d = 0.338'$ & $\quad d' = 0.338 - (1.417)(0.138) = 0.143'$

$Q_A = 2.31$ CFS
$Q_B = 0.23$ CFS
$Q_C = 2.06$ CFS
$\quad Q_T = 2.31 - 0.23 + 2.06 = 4.14$ CFS

$Q_i = 2.31 - 0.23 = 2.08$ CFS

Total $Q_i = 3.31 + 2.08 = 5.39$ CFS $\quad E = \frac{5.39}{7.43} = 0.725$

$Q_b = 7.43 - 5.39 = 2.04$ CFS

Use Det. P-1567, Type K14
$Q_i = 5.39$ CFS
$Q_b = 2.04$ CFS

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INLET NO. 431-2

$Q = 2.15$ cfs $\qquad S_x = 0.020$ '/ft

$S = 0.0022$ '/ft $\qquad Z_x/n = 3333.33$

(a) Curb Opening Inlet

$a = 2$ in. $\left.\right\} S_w' = \frac{2}{17} = 0.118 \qquad Z_w/n = 483$

$W = 17$ in.

$S_w = 0.118 + 0.020 = 0.138 \qquad C.F. = 1.25$

Assume $d = 0.400'$ & $d' = 0.400 - (1.417)(0.138) = 0.205'$

$Q_A = 1.10$ CFS
$Q_B = 0.19$ CFS $\left.\right\} Q_T = 1.10 - 0.19 + 1.28 = 2.19$ CFS OK
$Q_C = 1.28$ CFS

Gutter Flow, $Q_w = 1.10 - 0.19 = 0.91$ CFS

$E_o = Q_w/Q = 0.91/2.19 = 0.416$

$S_e = 0.020 + (0.118)(0.416) = 0.0690$

$$L_T = (0.6)(2.15)^{.42}(0.0022)^{0.3}\left[\frac{1}{(0.015)(0.0690)}\right]^{0.6}(1.25)$$
$$= 10.19'$$

(i) Try Det. P-1569, Type 'M-1', $L = 3'$

$L/L_T = 6/10.19 = 0.589$

$E = 1-(1-0.589)^{1.8} = 0.798$

$Q_i = 0.798 \times 2.15 = 1.72$ CFS

$Q_b = 2.15 - 1.72 = 0.43$ CFS

(b) Combination Inlet

Try Det. P-1573, Type R

$S_w = 0.138 \qquad Z_w/n = 483$

Assume $d = 0.40'$ & $d' = 0.40 - (1.417)(0.138) = 0.205'$

$Q_A = 1.10$ CFS
$Q_B = 0.19$ CFS $\left.\right\} Q_T = 2.19$ CFS
$Q_C = 1.28$ CFS

Gutter Flow, $Q_w = 1.10 - 0.19 = 0.91$ CFS

$T = 0.205/0.02 + 1.42 = 11.7'$

$E_o = Q_w/Q = 0.91/2.19 = 0.42$

$Q_i = 0.42 \times 2.15 = 0.90$ CFS

$Q_b = 2.15 - 0.90 = 1.25$ CFS

Use Det. P-1573, Type R $Q_i = 0.90$ CFS $Q_b = 1.25$ CFS

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INLET NO. 451-1 Sump Condition

$Q_{2yr.} = 5.47$ CFS
$Q_{10yr.} = 8.60$ CFS
$S = 0.0051$ '/ft

$S_x = 0.020$ '/ft
$Z_x/n = 3333.33$

For 10-Year, $d_{10} = \left\{(8.60)/[0.56(3333.33)(0.0051)^{1/2}]\right\}^{3/8} = 0.358'$

$T_{10} = 0.358/0.020 = 17.9$ ft < 20 ft ok

$T_{max} = 20'$
$d_{max} = T S_x = (20)(0.02) = 0.40'$

(a) Curb Opening Inlet
$h = 5$ in. $a = 2$ in. C.F. $= 1.25$
Since $d < h + a$, $Q = C_w (L/C.F. + 1.8W) d^{1.5}$

(i) Try Det. P-1569, Type 'M-1', $L = 17'$

$Q = 2.3[20/1.25 + 1.8(17/12)](0.40)^{1.5}$
$= 10.79$ CFS > 8.60 CFS ok

(ii) Try Det. P-1569, Type 'M-2', $L = 6'$

$Q = 2.3[15/1.25 + 1.8(17/12)](0.40)^{1.5}$
$= 8.47$ CFS ≈ 8.60 CFS ok

(b) Combination Inlet Since $d < h + a$, $Q = C_w P d^{1.5}$

(i) Try Det. P-1572, Type Q Double $P = 2(24/12) + 2(40/12) = 10.67'$

$Q = (3.0)(10.67)(0.40)^{1.5}$
$= 8.10$ CFS < 8.60 CFS

(ii) Try Det. P-1572, Type Q Triple $P = 2(24/12) + 2(40/12) = 14.0'$

$Q = (3.0)(14.0)(0.40)^{1.5}$
$= 10.63$ CFS > 8.60 CFS ok

Use Det. P-1572, Type Q Triple

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INLET NO. 451-2

Q = 11.14 (2-Year storm)
Sump Condition
Catch Basin will intercept offsite runoff prior to reaching Tatum Blvd.

Determine depth d necessary to intercept the above discharge using the Catch Basin, MAG Det. 535, Type F.

$P_{actual} = 2(2.5 + 3.0) = 11.0 \text{ ft}$

$A_{\substack{actual \\ clear\ opening}} = (2.5)(3.17) - 20(2.5)(\frac{0.5}{12}) - 4(3.17)(\frac{1}{12}) = 4.79 \text{ ft}^2$

check using Weir Flow: $Q_i = C_w P d^{1.5}$

$d = \left[11.14/(3)(11/2)\right]^{1/1.5} = \underline{\underline{0.77'}} < 0.8'$

Check using Orifice Flow: $Q_i = C_o A (2gd)^{0.5}$

$11.14 = 0.67\left(\frac{4.79}{2}\right)(2 \times 32.2\, d)^{0.5}$

$d = 0.75'$

Use M.A.G. Det. 535, Type F

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INLET NO. 451-3

SUMP CONDITION

$Q = 11.18$ CFS
$S = 0.0088$ 1/ft

$S_x = 0.018$ 1/ft
$Z_x/n = 3704$

(a) Curb Opening Inlet
$a = 2$ in.
$W = 17$ in.

$h = 5$ in.
C.F. = 1.25

Max. Depth at Gutter, $d = 0.35'$ (Before Overtopping)
Since $d < h + a$,

$$Q = 2.3(L/1.25 + 1.8W)\,d^{1.5}$$

(i) Try Det. P-1569, Type M-1, $L = 10'$

$$Q = 2.3\left[13/1.25 + 1.8(17/12)\right](0.35)^{1.5}$$
$$= 6.17 \text{ CFS}$$

$$Q_b = 11.18 - 6.17 = 5.01 \text{ CFS} \qquad E = \frac{6.17}{11.18} = 0.55$$

(ii) Try Det. P-1569, Type M-1, $L = 17'$

$$Q = 2.3\left[20/1.25 + 1.8(17/12)\right](0.35)^{1.5}$$
$$= 8.83 \text{ CFS}$$

$$Q_b = 11.18 - 8.83 = 2.35 \text{ CFS} \qquad E = \frac{8.83}{11.18} = 0.79$$

(b) Combination Inlet

Since $d < h + a$, $\quad Q = 3.0\,P\,d^{1.5}$

(i) Try Type Q Double, Det. P-1572

$$Q = 3.0\left[2(2) + 2'(40/12)\right](0.35)^{1.5} = 6.63 \text{ CFS}$$
$$Q_b = 11.18 - 6.63 = 4.55 \text{ CFS} \qquad E = 6.63/11.18 = 0.59$$

(ii) Try Type Q Triple, Det. P-1572

$$Q = 3.0\left[2(2) + 3'(40/12)\right](0.35)^{1.5} = 8.70 \text{ CFS}$$
$$Q_b = 11.18 - 8.70 = 2.48 \text{ CFS} \qquad E = 8.70/11.18 = 0.78$$

Use Det. P-1572, Type Q Triple

$Q_i = 8.70$ CFS

$Q_b = 2.48$ CFS

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INLET NO. 451-4

SUMP CONDITION

$Q = 9.51$ CFS
$S = 0.0085$ 1/ft

$S_x = 0.027$ 1/ft
$Z_x/n = 2469$

(a) Curb Opening Inlet
$a = 2$ in. $h = 5$ in.
$W = 17$ in. C.F. = 1.25

Max. Depth at Gutter, $d = 0.35'$ (Before Overtopping)
Since $d < h + a$,

$$Q = 2.3\left(L/1.25 + 1.8W\right) d^{1.5}$$

(i) Try Det. P-1569, Type M-1, $L = 10'$

$$Q = 2.3\left[13/1.25 + (1.8)(17/12)\right](0.35)^{1.5}$$
$$= 6.17 \text{ CFS}$$

$Q_b = 9.51 - 6.17 = 3.34$ CFS $\quad E = \frac{6.17}{9.51} = 0.65$

(ii) Try Det. P-1569, Type M-1, $L = 17'$

$$Q = 2.3\left[20/1.25 + (1.8)(17/12)\right](0.35)^{1.5}$$
$$= 8.83 \text{ CFS}$$

$Q_b = 9.51 - 8.83 = 0.68$ CFS $\quad E = \frac{8.83}{9.51} = 0.93$

(b) Combination Inlet

Since $d < h + a$, $\quad Q = 3.0\, P d^{1.5}$

(i) Try Type Q Double, Det. P-1572

$$Q = 3.0\left[2(2) + 2(40/12)\right](0.35)^{1.5} = 6.63 \text{ CFS}$$
$Q_b = 9.51 - 6.63 = 2.88$ CFS $\quad E = 6.63/9.51 = 0.70$

(ii) Try Type Q Triple, Det. P-1572

$$Q = 3.0\left[2(2) + 3(40/12)\right](0.35)^{1.5} = 8.70 \text{ CFS}$$
$Q_b = 9.51 - 8.70 = 0.81$ CFS $\quad E = 8.70/9.51 = 0.91$

Use Det. P-1572, Type Q Triple

$Q_i = 8.70$ CFS
$Q_b = 0.81$ CFS

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INLET NO. 452-1

$Q_{b,\,inlet\,\#451-3} = 2.48$ CFS $\qquad Q_{b,\,inlet\,\#451-4} = 0.81$ CFS

$Q = 2.11 + 2.48 + 0.81 = 5.40$ CFS
$S = 0.0055$ 1/ft
$S_x = 0.015$ 1/ft $\longrightarrow$ $Z_x/n = 4444.44$

(a) Curb Opening Inlet

$a = 2$ in., $W = 17$ in. $\Big\}$ $S_w' = \frac{2}{17} = 0.1176$ $\qquad Z_w/n = 503$, $C.F. = 1.25$

$S_w = 0.1176 + 0.015 = 0.1326$ 1/ft

Assume $d = 0.42'$ & $d' = 0.420 - (1.417)(0.1326) = 0.232'$

$Q_A = 2.07$ CFS, $Q_B = 0.42$ CFS, $Q_C = 3.75$ CFS $\Big\}$ $Q_T = 2.07 - 0.42 + 3.75 = 5.40$ CFS OK

$Q_w = 2.07 - 0.42 = 1.65$ CFS

$E_o = Q_w/Q_T = 1.65/5.40 = 0.3056$

$S_e = 0.015 + (0.1176)(0.3056) = 0.0509$

$L_T = 0.6(5.40)^{0.42}(0.0055)^{0.3}\left[1/(0.015)(0.0509)\right]^{0.6} \times 1.25$
$= 23.71'$

(i) Try Det. P-1569, Type 'M-2', $L = 6'$
$L/L_T = 15'/23.71' = 0.6326$
$E = 1-(1-0.6326)^{1.8} = 0.835$
$Q_i = 0.835 \times 5.40 = 4.51$ CFS
$Q_b = 5.40 - 4.51 = 0.89$ CFS

(ii) Try Det. P-1569, Type 'M-1', $L = 10'$
$L/L_T = 13'/23.71' = 0.5483$
$E = 1-(1-0.5483)^{1.8} = 0.761$
$Q_i = 0.761 \times 5.40 = 4.11$ CFS
$Q_b = 5.40 - 4.11 = 1.29$ CFS

(b) Combination Inlet
Try Det. P-1567, Type K14
$L/L_T = (14 - 40/12)/23.71 = 0.45$
$E = 1-(1-0.45)^{1.8} = 0.659$
$Q_i = 0.659 \times 5.40 = 3.56$ CFS
$Q_b = 5.40 - 3.56 = 1.84$ CFS

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INLET NO. 452-1 (cont'd)

(b) cont'd.

Assume $d = 0.325'$ & $d' = 0.325 - (1.417)(0.1326) = 0.137'$

$Q_A = 1.04$ CFS
$Q_B = 0.10$ CFS
$Q_C = 0.92$ CFS

$Q_T = 1.04 - 0.10 + 0.92 = 1.86$ CFS
$Q_W = 1.04 - 0.10 = 0.94$ CFS
$A = 0.954$ ft²

$V = 1.86/0.954 = 1.95$ fps
$T = 0.137/0.015 + 1.42 = 10.6'$
$E = Q_W/Q = 0.94/1.86 = 0.5054$
$Q_i = 0.5054 \times 1.84 = 0.93$ CFS

Total $Q_i = 3.56 + 0.93 = 4.49$ CFS
$Q_b = 5.40 - 4.49 = 0.91$ CFS

$E = \frac{4.49}{5.40} = 0.83$

Use Det. P-1567, Type K14
$Q_i = 4.49$ CFS
$Q_b = 0.91$ CFS

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	For		

INLET CALCULATIONS AT STA. 71+06 RT.

$Q_{b,\ inlet\ \#452\text{-}1} = 0.91$ CFS $\qquad S_x = 0.023$ '/ft

$Q_T = 2.78 + 0.91 = 3.69$ CFS $\qquad Z_x/n = 2898.55$

$S = 0.0032$ '/ft

(a) Curb Opening Inlet

$S_w = 2/17 = 0.1176$

$Z_w/n = 567$

Assume $d = 0.420'$ & $d' = 0.420 - (1.417)(0.1176) = 0.253'$

$Q_A = 1.78$ CFS
$Q_B = 0.46$ CFS
$Q_C = 2.35$ CFS

$Q_T = 1.78 - 0.46 + 2.35 = 3.67$ CFS
$Q_W = 1.78 - 0.46 = 1.32$ CFS

$E_o = Q_W/Q = 1.32/3.67 = 0.360$

$S_e = 0.023 + (0.1176)(0.360) = 0.0653$

$$L_T = (0.6)(3.69)^{.42}(0.0032)^{0.3}\left[\frac{1}{(0.015)(0.0653)}\right]^{0.6}(1.25)$$
$$= 14.79'$$

Check $L = 8'$ Curb Opening:

$L/L_T = 8/14.79 = 0.541$

$E = 1 - (1 - 0.541)^{1.8} = 0.754$

$Q_i = 0.754 \times 3.69 = 2.78$ CFS

$Q_b = 3.69 - 2.78 = 0.91$ CFS

Existing Catch Basin is sufficient to intercept 2-Year Storm Runoff.

APPENDIX C

STORM DRAIN SYSTEM DESIGN

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

STORM DRAIN No. 1

STORM SEWER SYSTEM DESIGN: SEWER CALCULATION SHEET

LOCATION DATA
Highway Tatum Blvd.
Location Phoenix, AZ
Project No. 886515

DESIGN DATA
Frequency 2 years
P_6 = – in. P_{24} = – in. P_1 = – in.
Outlet TW Elev. = 1377.70
Pipe n = 0.012

SEWER CALCULATIONS

Location		Drainage Area			Time of Flow, Min.			I	Q	Sewer Profile				Sewer Design			Hydraulic Grade Line				
Line No.	From - To	No.	Increm. CA	Σ CA	Inlet	Sewer	Design	in./hr	cfs	Crown Elevation Inlet	Crown Elevation Outlet	Length ft.	Slope ft./ft.	Diam. in.	Q_{full} cfs	V f.p.s.	S_f ft./ft.	h_L ft.	Elevation Inlet	Elevation Outlet	
1	9+09 - 11+91		4.17	4.17	23.0	–	23.0	1.60	6.67	94.76	87.71	282	.025	18	18.0	9.5	.0034	0.96	93.89	86.84	0.63'
2	11+91 - 14+73		–	4.17	10.0	0.5	23.5	1.60	6.67	87.71	80.66	282	.025	18	18.0	9.5	.0034	0.96	86.84	80.73	0.63'
3	14+73 - 18+03		15.18	19.35	18.2	0.5	24.0	1.57	30.38	80.66	78.68	330	.0060	30	34.4	7.9	.0047	1.55	80.73	79.18	1.83'
4	18+03 - 21+25		–	19.35	–	0.7	24.7	1.55	30.0	78.68	77.70	322	.0030	30	24.3	6.1	.0046	1.48	79.18	77.70	full

Computed by: D.L.B. Checked by: _____ Date: _____
4-25-91

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KAMINSKI HUBBARD engineering inc. • Surveying • Civil • Hydrology	Made by D.L.B.	Date 8-17-90	Job No. 0118
	Checked by	Date	Sheet No.
	For		

LINE NO. 182

Q = 6.67 CFS L = 282 ft.

Try S_o = 0.025 ft./ft. d/D = 0.42
Try D, Dia. = 18 in. d = 0.63'
C_a (Table 2) = 0.313
Area, A = 0.704 ft^2
Then, V = Q/A = 9.5 FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_a (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_a (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_a (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

$K_{18''}$ = 113.8

$S_{min} = \left(\frac{6.67}{113.8}\right)^2$ $Q_{full} = 113.8(0.025)^{1/2} = 18.0$ cfs

$= 0.0034$ ft/ft

K =

S_{min} = Q_{full} =

K =

S_{min} = Q_{full} =

K =

S_{min} = Q_{full} =

KAMINSKI HUBBARD engineering inc. • Surveying • Civil • Hydrology	Made by D.L.B.	Date 9-5-90	Job No. 0118
	Checked by	Date	Sheet No.
	For		

LINE NO. 3

Q = 30.38 CFS L = 330 ft.

Try S_o = 0.0060 ft./ft. d/D = -
Try D, Dia. = 24 in. d = - } Full Flow
C_a (Table 2) = -
Area, A = 3.142 ft^2
Then, V = Q/A = 9.67 FPS

Try S_o = 0.0060 ft./ft. d/D = 0.73
Try D, Dia. = 30 in. d = 1.83'
C_a (Table 2) = 0.614
Area, A = 3.838 ft^2
Then, V = Q/A = 7.9 FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_a (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_a (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

$K_{24''}$ = 245.1

$S_{min} = \left(\frac{30.38}{245.1}\right)^2$ $Q_{full} = 245.1\,(0.0060)^{1/2} = 19.0$ CFS

$= 0.0154$ 1/ft

$K_{30''}$ = 444.3

$S_{min} = \left(\frac{30.38}{444.3}\right)^2$ $Q_{full} = 444.3\,(0.0060)^{1/2} = 34.4$ CFS

$= 0.0047$ 1/ft

K =

S_{min} = Q_{full} =

K =

S_{min} = Q_{full} =

KAMINSKI HUBBARD engineering inc. • Surveying • Civil • Hydrology	Made by D.L.B.	Date 9-5-90	Job No. 0118
	Checked by	Date	Sheet No.
	For		

LINE NO. 4

Q = 30.0 CFS L = 322 ft.

Try S_o = 0.0030 ft./ft. d/D = –
Try D, Dia. = 30 in. d = –
C_a (Table 2) = – } Full Flow
Area, A = 4.909 ft^2
Then, V = Q/A = 6.11 FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_a (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_a (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_a (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

$K_{30''}$ = 444.3

$S_{min} = \left(\frac{30.0}{444.3}\right)^2$ $Q_{full} = 444.3(0.0030)^{1/2} = 24.34$ CFS

$= 0.0046$ '/ft

K =

S_{min} = Q_{full} =

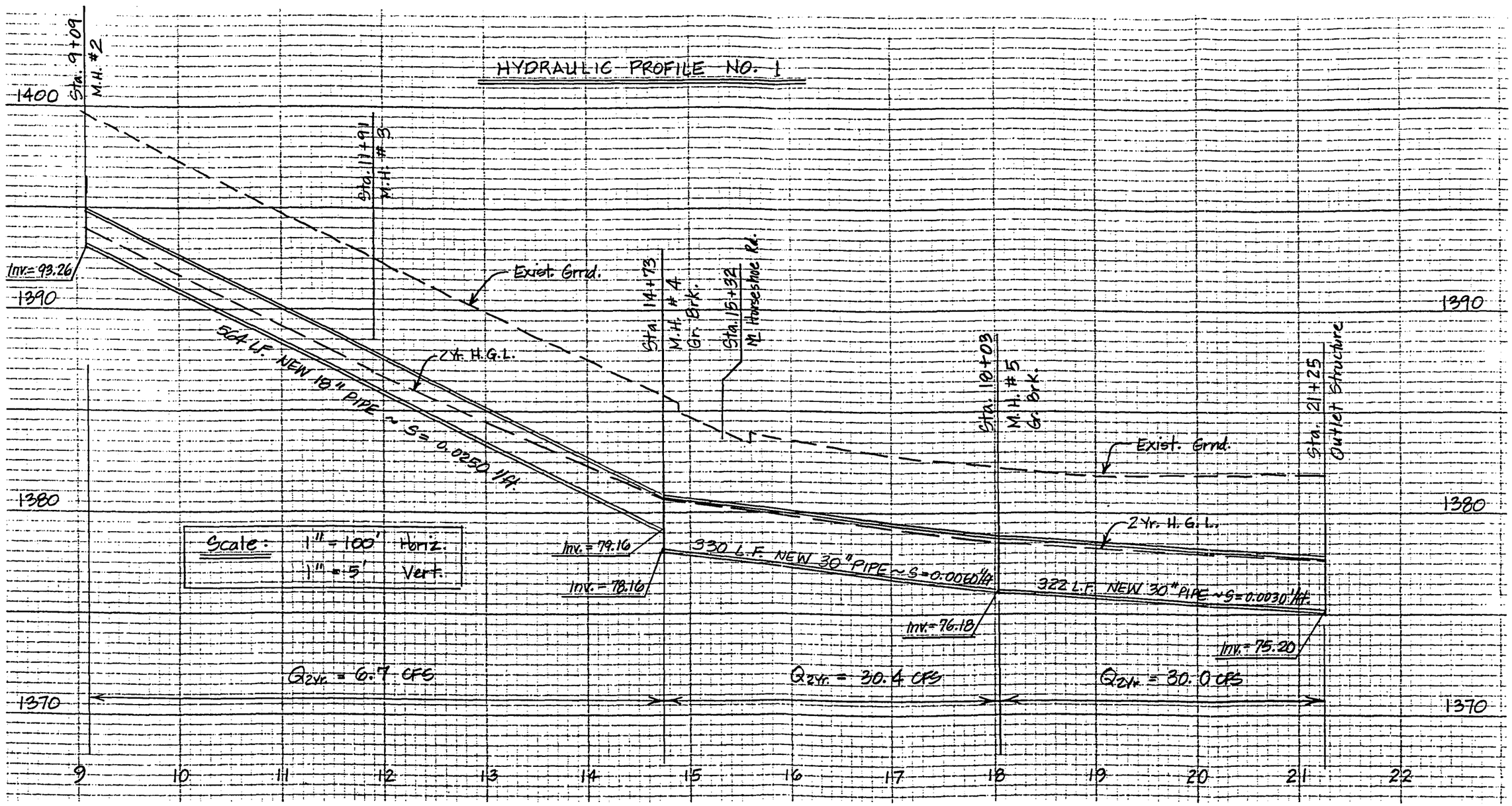
K =

S_{min} = Q_{full} =

K =

S_{min} = Q_{full} =

KAMINSKI HUBBARD engineering inc. • Surveying • Civil • Hydrology	Made by D.L.B.	Date 4-25-91	Job No. 0118
	Checked by	Date	Sheet No.
	For		



STORM DRAIN Nº 2

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

STORM SEWER SYSTEM DESIGN: SEWER CALCULATION SHEET

LOCATION DATA
Highway Tatum Blvd.
Location Phoenix, AZ
Project No.

DESIGN DATA
Frequency 2 years
P_6= — in. P_{24}= — in. P_1= — in.
Outlet TW Elev. = 1374.80
Pipe n = 0.012

SEWER CALCULATIONS

Location		Drainage Area			Time of Flow, Min.			I	Q	Sewer Profile				Sewer Design			Hydraulic Grade Line				
Line No.	From - To	No.	Increm. CA	Σ CA	Inlet	Sewer	Design	in./hr	cfs	Crown Elevation Inlet	Crown Elevation Outlet	Length ft.	Slope ft./ft.	Diam. in.	Q_{full} cfs	V f.p.s.	S_f ft./ft.	h_L ft.	Elevation Inlet	Elevation Outlet	
5	48+00 - 44+70		3.31	3.31			17.1	1.90	6.3	79.51	78.02	330	.0045	18	7.6	4.8	.0031	1.02	79.05	77.78	1.04'
6	44+70 - 41+40		2.23	5.54			18.1	1.85	10.2	78.02	77.03	330	.0030	24	13.4	4.7	.0017	0.56	77.78	77.22	1.30'
7	41+40 - 38+10		—	5.54			19.1	1.80	10.0	77.03	76.04	330	.0030	24	13.4	4.7	.0017	0.56	77.22	76.66	1.29'
8	38+10 - 35+03		8.63	14.17			20.1	1.75	24.8	76.04	75.43	307	.0020	30	19.9	5.1	.0031	0.95	76.66	75.71	Full
9	35+03 - 31+88		0.41	14.58	22.1		22.1	1.65	24.1	75.43	74.80	315	.0020	30	19.9	4.9	.0029	0.91	75.71	74.80	Full

Computed by: D.L.B. 4-15-91 Checked by: _____ Date: _____

KAMINSKI HUBBARD engineering inc. • Surveying • Civil • Hydrology	Made by D.L.B.	Date 4-12-91	Job No. 0118
	Checked by	Date	Sheet No.
	For		

LINE NO. 5

Q = 6.3 CFS L = 330 ft.

Try S_o = 0.0030 ft./ft. d/D = –
Try D, Dia. = 18 in. d = –
C_m (Table 2) = – } Full Flow
Area, A = 1.767 ft^2
Then, V = Q/A = 3.6 FPS

Try S_o = 0.0040 ft./ft. d/D = 0.727
Try D, Dia. = 18 in. d = 1.09'
C_m (Table 2) = 0.610
Area, A = 1.373 ft^2
Then, V = Q/A = 4.6 FPS

Try S_o = 0.0045 ft./ft. d/D = 0.693
Try D, Dia. = 18 in. d = 1.04'
C_m (Table 2) = 0.581
Area, A = 1.307 ft^2
Then, V = Q/A = 4.8 FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_m (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

.0040

$K_{18''} = 113.8$

$S_{min} = \left(\frac{6.30}{113.8}\right)^2 = 0.0031$ 1/ft. $Q_{full} = (113.8)(0.0030)^{1/2} = 6.2$ CFS

K =

S_{min} = $Q_{full} = (113.8)(0.0040)^{1/2} = 7.2$ CFS

K =

S_{min} = $Q_{full} = (113.8)(0.0045)^{1/2} = 7.6$ CFS

K =

S_{min} = Q_{full} =

KAMINSKI HUBBARD engineering inc. • Surveying • Civil • Hydrology	Made by D.L.B.	Date 4-15-91	Job No. 0118
	Checked by	Date	Sheet No.
	For		

LINE NO. 6

Q = 10.2 CFS L = 330 ft.

Try S_o = 0.0030 ft./ft. d/D = 0.65
Try D, Dia. = 24 in. d = 1.30'
C_m (Table 2) = 0.54
Area, A = 2.160 ft²
Then, V = Q/A = 4.7 FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_m (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_m (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_m (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

$K_{24''} = 245.1$

$S_{min} = \left(\frac{10.2}{245.1}\right)^2 = 0.0017'/ft$ $Q_{full} = (245.1)(0.0030)^{1/2} = 13.4$ cfs

K =

S_{min} = Q_{full} =

K =

S_{min} = Q_{full} =

K =

S_{min} = Q_{full} =

KAMINSKI HUBBARD engineering inc. • Surveying • Civil • Hydrology	Made by D.L.B.	Date 4-15-91	Job No. 0118
	Checked by	Date	Sheet No.
	For		

LINE NO. 7

Q = 10.0 CFS L = 330 ft.

Try S_o = 0.0032 ft./ft. d/D = -
Try D, Dia. = 18 in. d = -
C_m (Table 2) = - } Full Flow
Area, A = 1.767 ft²
Then, V = Q/A = 5.7 FPS

Try S_o = 0.0032 ft./ft. d/D = 0.63
Try D, Dia. = 24 in. d = 1.26'
C_m (Table 2) = 0.521
Area, A = 2.084 ft²
Then, V = Q/A = 4.8 FPS

Try S_o = 0.0025 ft./ft. d/D = 0.685
Try D, Dia. = 24 in. d = 1.37
C_m (Table 2) = 0.575
Area, A = 2.300 ft²
Then, V = Q/A = 4.35 FPS

Try S_o = 0.0030 ft./ft. d/D = 0.645
Try D, Dia. = 24 in. d = 1.29'
C_m (Table 2) = 0.534
Area, A = 2.137 ft²
Then, V = Q/A = 4.7 FPS

$K_{18''} = 113.8$

$S_{min} = \left(\frac{10.0}{113.8}\right)^2 = 0.0077$ 1/ft $Q_{full} = (113.8)(0.0032)^{1/2} = 6.4$ CFS

$K_{24''} = 245.1$

$S_{min} = \left(\frac{10.0}{245.1}\right)^2 = 0.0017$ 1/ft $Q_{full} = (245.1)(0.0032)^{1/2} = 13.9$ CFS

K =

S_{min} = $Q_{full} = (245.1)(0.0030)^{1/2} = 13.4$ CFS

K =

S_{min} = Q_{full} =

KAMINSKI HUBBARD engineering inc. • Surveying • Civil • Hydrology	Made by D.L.B.	Date 4-15-91	Job No. 0118
	Checked by	Date	Sheet No.
	For		

LINE NO. 8

Q = 24.8 CFS L = 307 ft.

Try S_o = 0.0020 ft./ft. d/D = –
Try D, Dia. = 30 in. d = –
C_m (Table 2) = – } Full Flow
Area, A = 4.909 ft^2
Then, V = Q/A = 5.1 FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_m (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_m (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_m (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

$K_{30''}$ = 444.3

$S_{min} = \left(\frac{24.8}{444.3}\right)^2 = 0.0031\ 1/ft$ $Q_{full} = (444.3)(0.0020)^{1/2} = 19.9$ CFS

K =

S_{min} = Q_{full} =

K =

S_{min} = Q_{full} =

K =

S_{min} = Q_{full} =

KAMINSKI HUBBARD engineering inc. • Surveying • Civil • Hydrology	Made by D.L.B.	Date 4-12-91	Job No. 0118
	Checked by	Date	Sheet No.
	For		

LINE NO. 9

Q = 24.1 CFS L = 315 ft.

Try S_o = 0.0030 ft./ft. d/D = -
Try D, Dia. = 24 in. d = - } Full Flow
C_a (Table 2) = -
Area, A = 3.142 ft^2
Then, V = Q/A = 7.67 FPS

Try S_o = 0.0030 ft./ft. d/D = 0.80
Try D, Dia. = 30 in. d = 3.30'
C_a (Table 2) = 0.647
Area, A = 4.044 ft^2
Then, V = Q/A = 6.0 FPS

Try S_o = 0.0020 ft./ft. d/D = -
Try D, Dia. = 30 in. d = - } Full Flow
C_a (Table 2) = -
Area, A = 4.909 ft^2 ✓
Then, V = Q/A = 4.9 FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_a (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

$K_{24''}$ = 245.1

$S_{min} = \left(\frac{24.1}{245.1}\right)^2$ $Q_{full} = (245.1)(0.0030)^{1/2} = 13.4$ CFS

$= 0.0097$ '/ft

$K_{30''}$ = 444.3

$S_{min} = \left(\frac{24.1}{444.3}\right)^2$ $Q_{full} = (444.3)(0.0030)^{1/2} = 24.3$ CFS

$= 0.0029$ '/ft

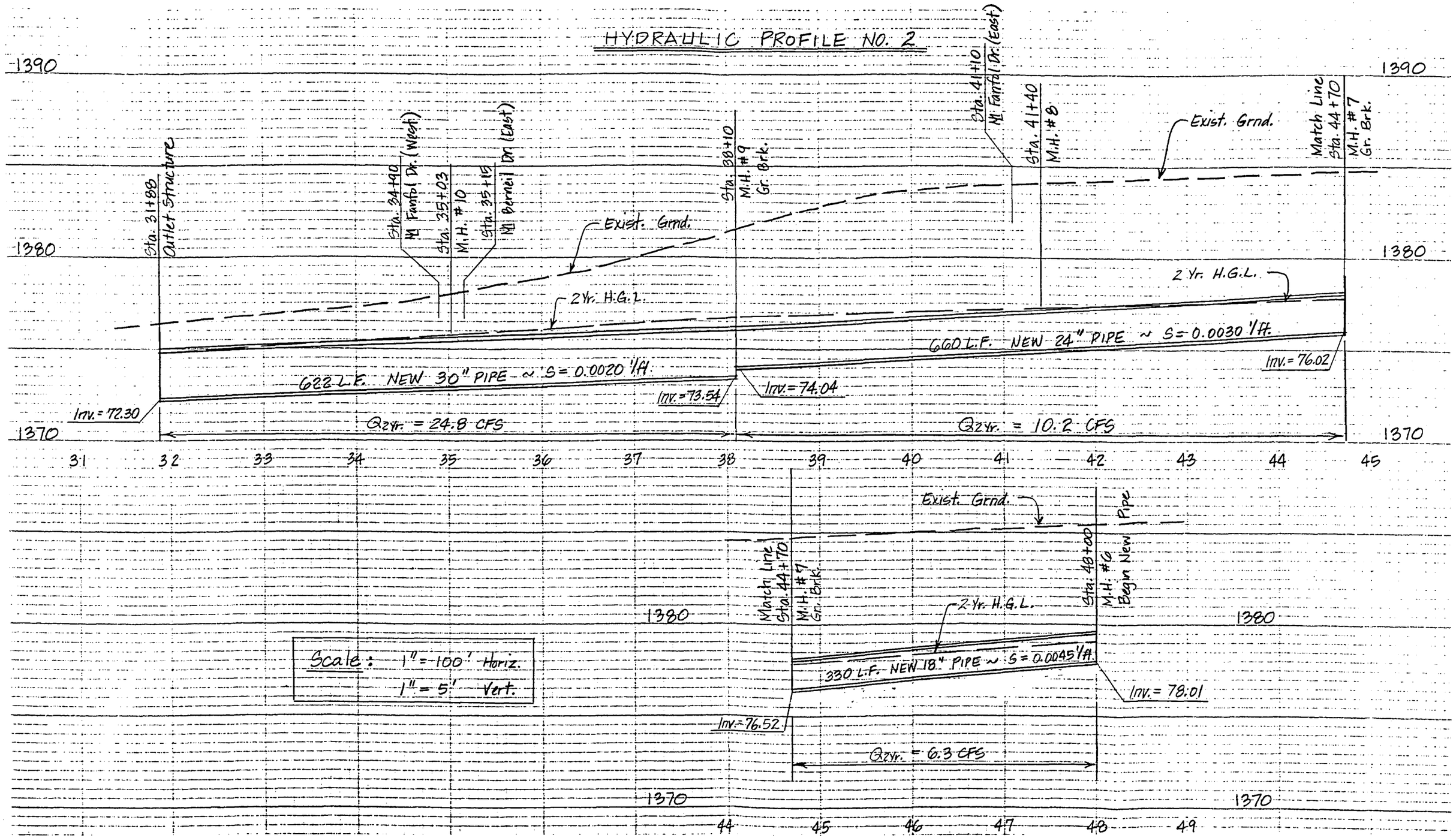
K =

S_{min} = $Q_{full} = (444.3)(0.0020)^{1/2} = 19.9$ CFS

K =

S_{min} = Q_{full} =

KAMINSKI HUBBARD engineering inc. • Surveying • Civil • Hydrology	Made by D.L.B.	Date 4-16-91	Job No. 0118
	Checked by	Date	Sheet No.
	For		



STORM DRAIN Nº 3

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

STORM SEWER SYSTEM DESIGN: SEWER CALCULATION SHEET

LOCATION DATA
Highway Tatum Blvd.
Location Phoenix, AZ
Project No.

DESIGN DATA
Frequency 2 years
P_6= — in. P_{24}= — in. P_1= — in.
Outlet TW Elev. =
Pipe n = 0.012

SEWER CALCULATIONS

Location		Drainage Area			Time of Flow, Min.			I	Q	Sewer Profile				Sewer Design			Hydraulic Grade Line				
Line No.	From - To	No.	Increm. CA	Σ CA	Inlet	Sewer	Design	in./hr	cfs	Crown Elevation Inlet	Crown Elevation Outlet	Length ft.	Slope ft./ft.	Diam. in.	Q_{full} cfs	V f.p.s.	S_f ft./ft.	h_L ft.	Elevation Inlet	Elevation Outlet	
10	59+13.5 - 63+53.5		22.52	22.52	24.2	—	24.2	1.55	34.9	77.90	75.70	440	.0050	36	51.1	7.8	.0023	1.01	76.37	75.36	1.82'
11	63+53.5 - 67+93.5		7.23	29.75	25.1	0.9	25.1	1.53	45.5	75.70	73.50	440	.0050	36	51.1	8.2	.0040	1.76	75.36	73.60	2.21'
12	67+93.5 - 71+21.5		1.16	30.91	12.3	0.9	26.0	1.50	46.4	73.50	71.86	328	.0050	36	51.1	8.2	.0041	1.34	73.60	72.26	2.24'
13	71+21.5 - 74+47.5		1.27	32.18	10.0	0.7	26.7	1.48	47.6	71.86	70.23	326	.0050	36	51.1	8.2	.0043	1.40	72.26	70.86	2.29'

Computed by: D.L.B. 4-16-91 Checked by: Date:

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KAMINSKI HUBBARD engineering inc. • Surveying • Civil • Hydrology	Made by D.L.B.	Date 4-16-91	Job No. 0118
	Checked by	Date	Sheet No.
	For		

LINE NO. 10

Q = 34.9 CFS L = 440 ft.

Try S_o = 0.0050 ft./ft. d/D = –
Try D, Dia. = 30 in. d = –
C_a (Table 2) = – } Full Flow
Area, A = 4.909 ft²
Then, V = Q/A = 7.1 FPS

Try S_o = 0.0050 ft./ft. d/D = 0.606
Try D, Dia. = 36 in. d = 1.82'
C_a (Table 2) = 0.498
Area, A = 4.482 ft²
Then, V = Q/A = 7.8 FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_a (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_a (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

$K_{30''} = 444.3$

$S_{min} = \left(\frac{34.9}{444.3}\right)^2 = 0.0062$ 1/ft $Q_{full} = (444.3)(0.0050)^{1/2} = 31.4$ CFS

$K_{36''} = 722.2$

$S_{min} = \left(\frac{34.9}{722.2}\right)^2 = 0.0023$ 1/ft $Q_{full} = (722.2)(0.0050)^{1/2} = 51.1$ CFS.

K =

S_{min} = Q_{full} =

K =

S_{min} = Q_{full} =

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LINE NO. 11

Q = 45.5 CFS L = 440 ft.

Try S_o = 0.0050 ft./ft. d/D = 0.737
Try D, Dia. = 36 in. d = 2.21'
C_a (Table 2) = 0.619
Area, A = 5.571 ft^2
Then, V = Q/A = 8.2 FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_a (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_a (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_a (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

$K_{36''}$ = 722.2

$S_{min} = \left(\frac{45.5}{722.2}\right)^2$ $Q_{full} = (722.2)(0.0050)^{1/2} = 51.1$ CFS

= 0.0040 '/ft

K =

S_{min} = Q_{full} =

K =

S_{min} = Q_{full} =

K =

S_{min} = Q_{full} =

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LINE NO. 12

Q = 46.4 CFS L = 328 ft.

Try S_o = 0.0050 ft./ft. d/D = 0.747
Try D, Dia. = 36 in. d = 2.24'
C_m (Table 2) = 0.630
Area, A = 5.670 ft²
Then, V = Q/A = 8.2 FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_m (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_m (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_m (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

$K_{36''} = 722.2$

$S_{min} = \left(\frac{46.4}{722.2}\right)^2 = 0.0041'/ft$ $Q_{full} = (722.2)(0.0050)^{1/2} = 51.1$ CFS

K =

S_{min} = Q_{full} =

K =

S_{min} = Q_{full} =

K =

S_{min} = Q_{full} =

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	Checked by	Date	Sheet No.
	For		

LINE NO. 13

Q = 47.6 CFS L = 326 ft.

Try S_o = 0.0050 ft./ft. d/D = 0.762
Try D, Dia. = 36 in. d = 2.29'
C_m (Table 2) = 0.642
Area, A = 5.778 ft²
Then, V = Q/A = 8.2 FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_m (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_m (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

Try S_o = _____ ft./ft. d/D = _____
Try D, Dia. = _____ in. d = _____
C_m (Table 2) = _____
Area, A = _____
Then, V = Q/A = _____ FPS

$K_{36''} = 722.2$

$S_{min} = \left(\frac{47.6}{722.2}\right)^2$ $Q_{full} = (722.2)(0.0050)^{1/2} = 51.1$ CFS

$= 0.0043'/ft.$

K =

S_{min} = Q_{full} =

K =

S_{min} = Q_{full} =

K =

S_{min} = Q_{full} =

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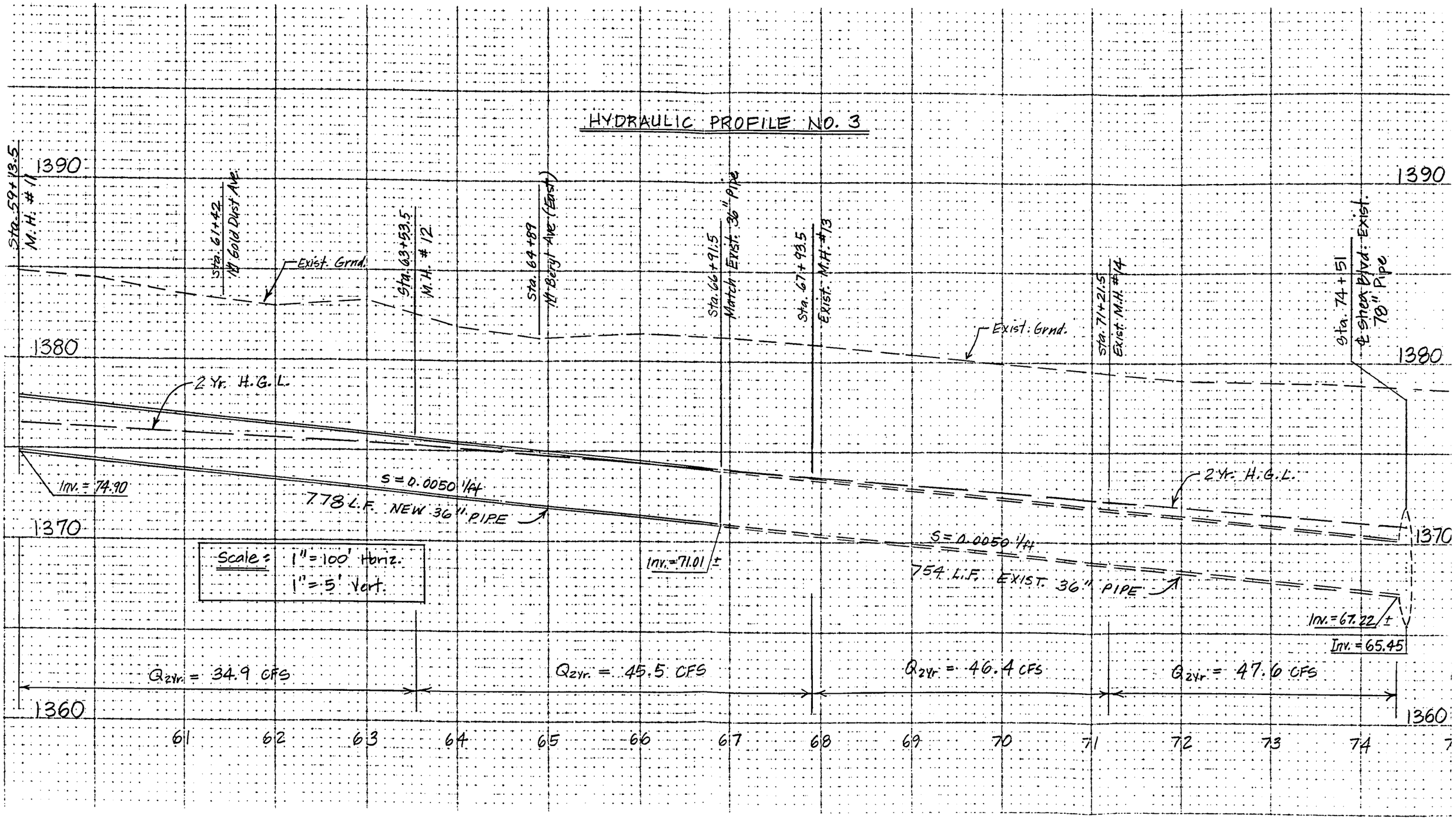
	For		



APPENDIX D

PIPE CULVERT DESIGN SHEETS

PROJECT: Tatum Blvd.
Proj. No. 886515

STATION: 20+98
SHEET ____ OF ____

CULVERT DESIGN FORM
DESIGNER / DATE: D.L.B. / 4-25-91
REVIEWER / DATE: ____ / ____

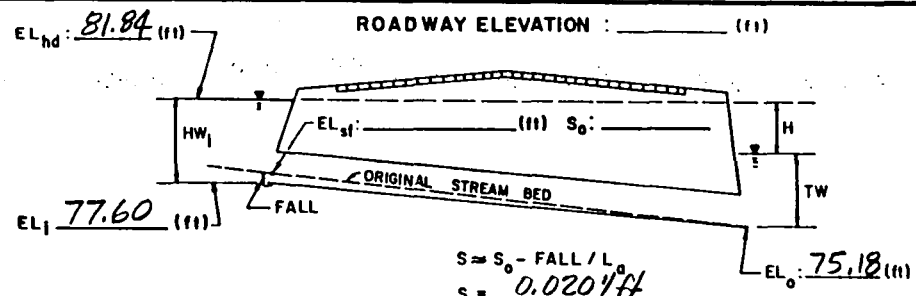
HYDROLOGICAL DATA

SEE ADD'L. SHTS.

- ☐ METHOD: ____
- ☐ DRAINAGE AREA: ____ ☐ STREAM SLOPE: ____
- ☐ CHANNEL SHAPE: ____
- ☐ ROUTING: ____ ☐ OTHER: ____

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)



$S \approx S_o$ - FALL / L_o
S = 0.020 ft/ft
L_o = 121

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
Conc.-Rect.- 2-8'x2'	272	17	2.10	4.20	–	81.80	4.21	2.0	2.0	4.21	0.5	1.38	80.77	81.84	8.5	
" " " "	240	15	1.75	3.50	–	81.10	4.21	1.92	1.96	4.21	0.5	1.58	80.97	81.10	8.5	

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) FALL = $HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) h_o = TW or $(d_c + D/2)$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29 n^2 L)/R^{1.33}\right] V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:
SIZE: 2-8'x2'
SHAPE: Rectangular
MATERIAL: Conc. n 0.012
ENTRANCE: Bevel 45°

PROJECT: Tatum Blvd.

STATION: 32+20

SHEET 1 OF 2

CULVERT DESIGN FORM

DESIGNER / DATE: D.L.B. / 4-25-91

REVIEWER / DATE: ____ / ____

HYDROLOGICAL DATA

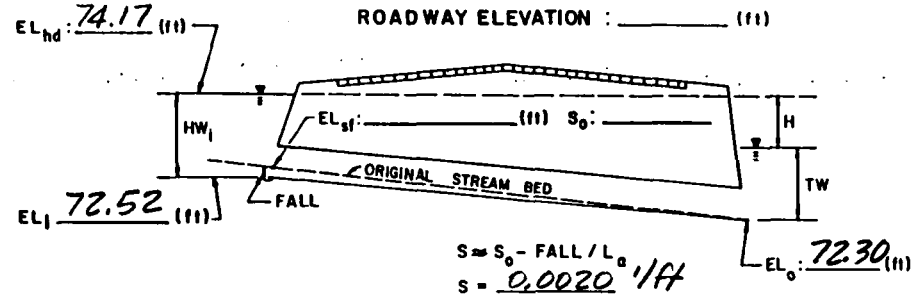
SEE ADD'L. SHTS.

- ☐ METHOD: ____
- ☐ DRAINAGE AREA: ____ ☐ STREAM SLOPE: ____
- ☐ CHANNEL SHAPE: ____
- ☐ ROUTING: ____ ☐ OTHER: ____

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)

ROADWAY ELEVATION: ____ (ft)



$S \approx S_o - FALL / L_a$

$S = 0.0020$ '/ft

$L_a = 110$

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_hi (4)	OUTLET CONTROL TW (5)	d_c	(d_c+D)/2	h_o (6)	k_e	H (7)	EL_ho (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
Conc.-Oval-38"x24"	16	16	0.83	1.65	-	74.17		1.15	1.58	1.58	0.5	1.19	75.07	75.07		
" " " "		12	0.70	1.40	-	73.92		0.95	1.48	1.48	0.5	1.45	75.23	75.23		
Conc.-Oval-38"x24"		20	0.96	1.92	-	74.44		1.27	1.64	1.64	0.5	1.03	74.97	74.97		
" " " "		24	1.10	2.20	-	74.72		1.40	1.70	1.70	0.5	0.88	74.88	74.88		
Conc.-Oval-38"x24"		30	1.35	2.70	-	75.22		1.56	1.78	1.78	0.5	0.75	74.83	75.22		

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) h_o = TW or $(d_c + D/2)$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29 n^2 L) / R^{1.33}\right] V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: ____

SHAPE: ____

MATERIAL: ____

ENTRANCE: ____

PROJECT: Tatum Blvd.	STATION: 32+20	CULVERT DESIGN FORM
	SHEET 2 OF 2	DESIGNER / DATE: D.L.B. / 5-6-91
		REVIEWER / DATE: /

HYDROLOGICAL DATA

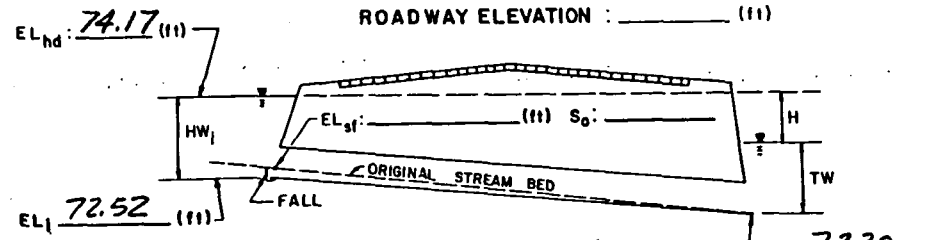
SEE ADD'L. SHTS.

- ☐ METHOD:
- ☐ DRAINAGE AREA: ☐ STREAM SLOPE:
- ☐ CHANNEL SHAPE:
- ☐ ROUTING: ☐ OTHER:

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)

ROADWAY ELEVATION: (ft)



$S = S_o - FALL / L_a$

$S = 0.0020'/ft$

$L_a = 110'$

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_hi (4)	OUTLET CONTROL TW (5)	d_c	(d_c+D)/2	h_o (6)	k_e	H (7)	EL_ho (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
Conc. - Oval - 30"x19"		10	0.88	1.39	-	73.91	-	0.95	1.27	1.27	0.5	0.76	74.33	74.33		
		15	1.18	1.87	-	74.39	-	1.20	1.39	1.39	0.5	0.57	74.26	74.39		

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) h_o = TW or $(d_c + D/2)$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29n^2 L)/R^{1.33}\right] V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: 30" X 19"

SHAPE: Oval

MATERIAL: Conc. n=.012

ENTRANCE:

PROJECT: Tatum Blvd.	STATION: 52+57	CULVERT DESIGN FORM
	SHEET ____ OF ____	DESIGNER / DATE: DLB / 4-17-91
		REVIEWER / DATE: ____ / ____

HYDROLOGICAL DATA

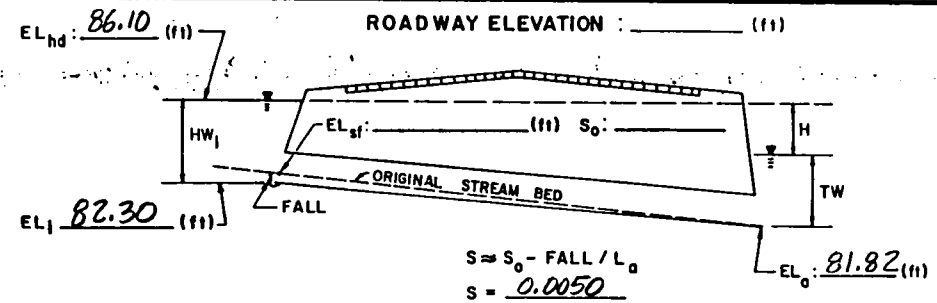
SEE ADD'L. SHTS.

- ☐ METHOD: ____
- ☐ DRAINAGE AREA: ____ ☐ STREAM SLOPE: ____
- ☐ CHANNEL SHAPE: ____
- ☐ ROUTING: ____ ☐ OTHER: ____

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW (cfs)	TW (ft)

ROADWAY ELEVATION: ____ (ft)



$S \approx S_o - FALL / L_a$

$S = 0.0050$

$L_a = 97'$

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
Conc.-Oval-4-38"x24"	160	40	1.90	3.80	–	86.10	–	1.78	1.89	1.89	0.5	2.23	85.94	86.10	8.0	
Conc.-Oval-4-38"x24"	150	37.5	1.73	3.46	–	85.76	–	1.73	1.87	1.87	0.5	1.97	85.66	85.76	7.5	
Conc.-Circ.-7-24"	168	24	1.90	3.80	–	86.10	–	1.75	1.88	1.88	0.5	2.50	86.20	86.2	7.6	
Conc.-Circ.-7-24"	147	21	1.57	3.14	–	85.44	–	1.65	1.83	1.83	0.5	1.90	85.55	85.55	6.7	

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_f/D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) h_o = TW or $(d_c + D/2)$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29n^2 L)/R^{1.33}\right] V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

a. APPROXIMATE
f. CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho. HEADWATER IN OUTLET CONTROL
i. INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw. TAILWATER

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: 4-38"x24"

SHAPE: Oval

MATERIAL: Conc. n 0.012

ENTRANCE: Bevel Edge 45°

PROJECT: Tatum Blvd Check Exist. 6-36"x22" CMP A Capacity	STATION: 52+52 SHEET ____ OF ____	CULVERT DESIGN FORM DESIGNER / DATE: DLB / 9-4-90 REVIEWER / DATE: ____ / ____

HYDROLOGICAL DATA

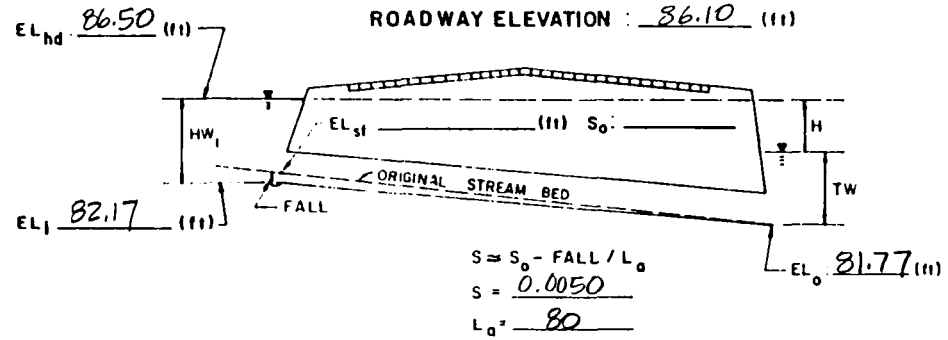
SEE ADD'L SHTS.

- ☐ METHOD: ____
- ☐ DRAINAGE AREA: ____ ☐ STREAM SLOPE: ____
- ☐ CHANNEL SHAPE: ____
- ☐ ROUTING ____ ☐ OTHER ____

DESIGN FLOWS/TAILWATER

R I (YEARS)	FLOW (cfs)	TW (ft)

ROADWAY ELEVATION: 86.10 (ft)



CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q (cfs)	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_hi (4)	OUTLET CONTROL TW (5)	d_c	(d_c+D)/2	h_o (6)	k_e	H (7)	EL_ho (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS
6-36"x22"x80' CMP A	234	39	2.36	4.33	—	86.50	—	1.64	1.74	1.74	0.5	6.0	89.51	89.51		89.51 > 86.5
" " "	150	25	1.35	2.48	-	84.65	-	1.32	1.58	1.58	0.5	2.5	85.85	85.85		85.85 < 86.5
6-36" x 22" x 80' CMPA	168	28	1.52	2.79	-	84.96	-	1.40	1.62	1.62	0.5	3.1	86.49	86.49		86.49 ≈ 86.50

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) h_o = TW or $(d_c + D/2)$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (29n^2 L)/R^{1.33}\right] V^2/2g$

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

a. APPROXIMATE
f CULVERT FACE
hd. DESIGN HEADWATER
hi. HEADWATER IN INLET CONTROL
ho HEADWATER IN OUTLET CONTROL
i INLET CONTROL SECTION
o. OUTLET
sf. STREAMBED AT CULVERT FACE
tw TAILWATER

COMMENTS / DISCUSSION:

Exist. 6-36"x22"x80' CMP A will convey 168 CFS.

CULVERT BARREL SELECTED:

SIZE: ____
SHAPE: ____
MATERIAL: ____
ENTRANCE: ____

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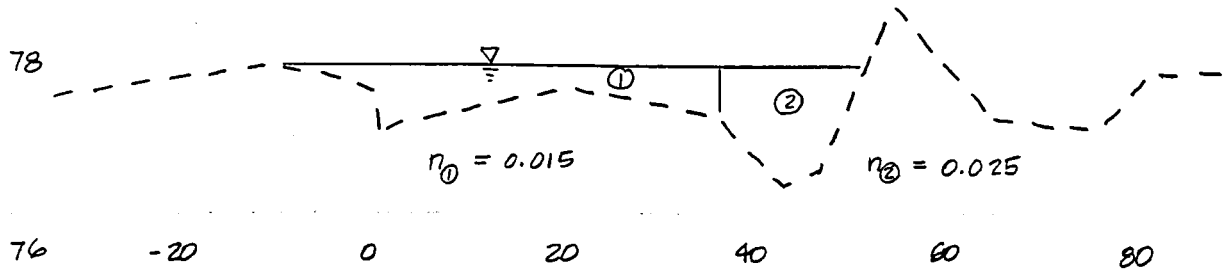
Existing Capacity - Doubletree Ranch Road

$S = 0.010$ ft/ft



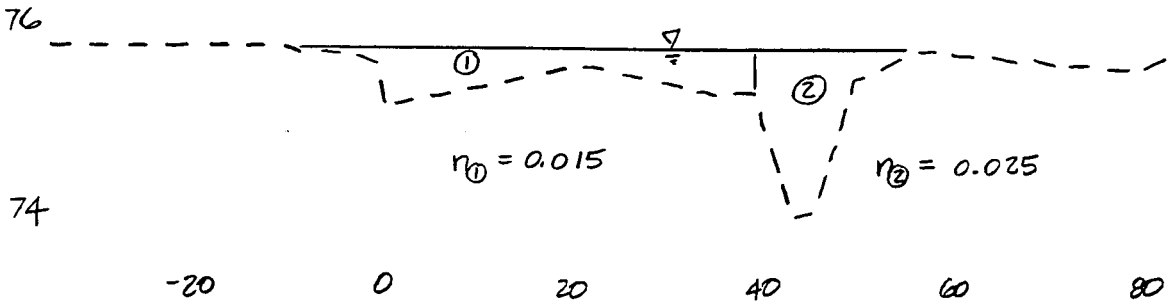
$A_① = 16.8\ ft^2$
$P_① = 47.4\ ft$

$\therefore Q_① = \frac{1.486}{0.015}(16.8)\left(\frac{16.8}{47.4}\right)^{2/3}(0.010)^{1/2}$
$= 83.3$ CFS

$A_② = 12.4\ ft^2$
$P_② = 15.3\ ft$

$Q_2 = \frac{1.486}{0.025}(12.4)\left(\frac{12.4}{15.3}\right)^{2/3}(0.010)^{1/2}$
$= 64.1$ CFS

$\therefore Q_T = 83.3 + 64.1 = \underline{\underline{147.4\ CFS}}$



$A_① = 12.4\ ft^2$
$P_① = 44.1\ ft$

$Q_① = \frac{1.486}{0.015}(12.4)\left(\frac{12.4}{44.1}\right)^{2/3}(0.008)^{1/2}$
$= 47.2$ CFS

$A_② = 13.2\ ft^2$
$P_② = 17.5\ ft$

$Q_② = \frac{1.486}{0.025}(13.2)\left(\frac{13.2}{17.5}\right)^{2/3}(0.008)^{1/2}$
$= 58.2$ CFS

$\therefore Q_T = 47.2 + 58.2 = 105.4$ CFS

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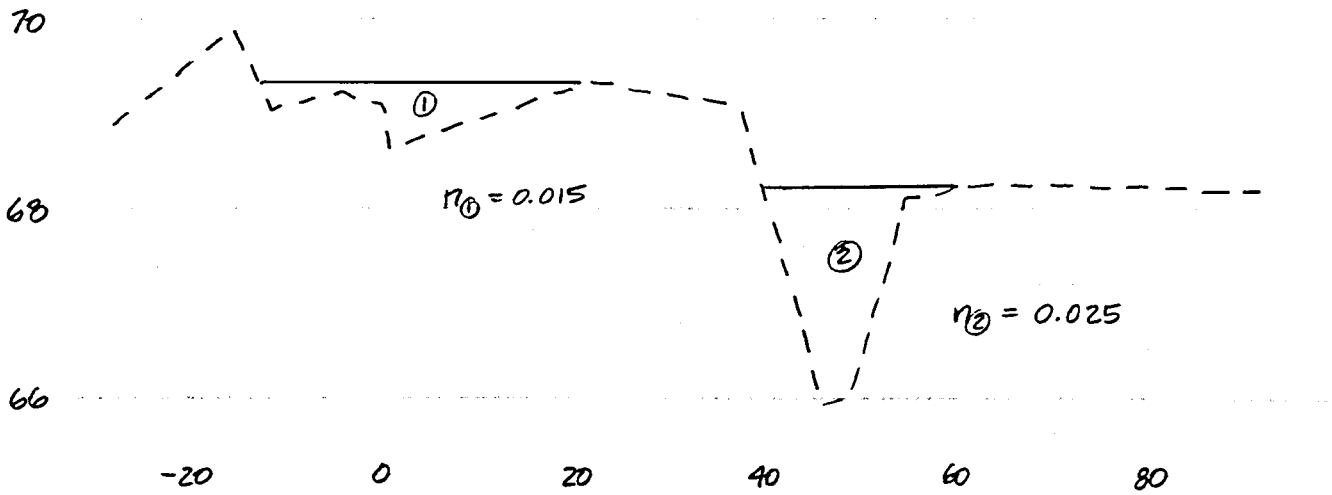
	For		

Existing Capacity - Double tree Ranch Road (cont'd)



$A_① = 10.8\ ft^2$
$P_① = 34.9\ ft$

$\therefore Q_① = \frac{1.486}{0.015}(10.8)\left(\frac{10.8}{34.9}\right)^{2/3}(0.008)^{1/2}$
$= 43.8\ CFS$

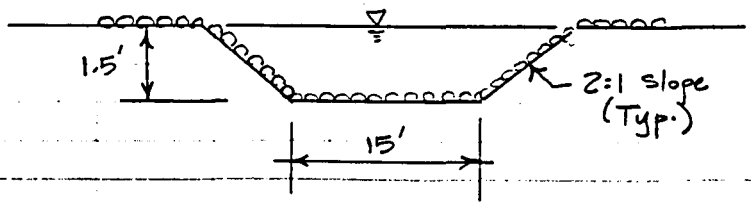
$A_② = 20.8\ ft^2$
$P_② = 24.2\ ft$

$Q_② = \frac{1.486}{0.025}(20.8)\left(\frac{20.8}{24.2}\right)^{2/3}(0.008)^{1/2}$
$= 100.0\ CFS$

KAMINSKI HUBBARD engineering inc. • Surveying • Civil • Hydrology	Made by D.L.B	Date 9-5-90	Job No. 0118
	Checked by	Date	Sheet No.
	For		

Capacity of Downstream Channel @ Sta. 32+20 Rt.

Location: Foothills Estates II subdivision



$n = 0.030$

$S = 0.0050'/ft$

$A = (15)(1.5) + (3)(1.5) = 27.0\ ft^2$

$P = 15 + 2\sqrt{3^2 + 1.5^2} = 21.7\ ft$

$$Q = \frac{1.486}{0.030}(27.0)\left(\frac{27.0}{21.7}\right)^{2/3}(0.0050)^{1/2}$$

$$= \underline{\underline{109.4\ CFS}}$$

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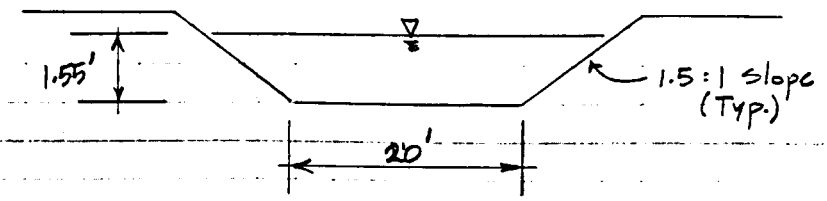
	Checked by	Date	Sheet No.
	For		

Capacity of Downstream Channel in Mountain View Estates

(a) Exist. 6- 36"×22" CMPA discharge into :



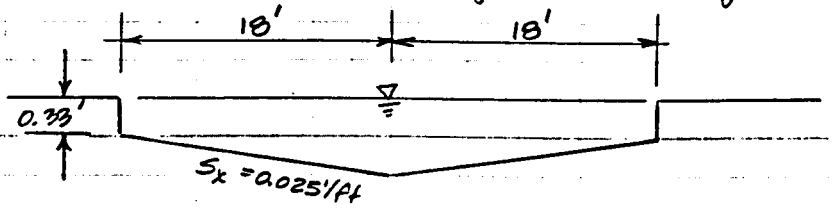
$n = 0.025$
$S = 0.0050$ '/ft

$A = (1.55)(2.33) + (1.55)(20) = 34.6 \text{ ft}^2$
$P = 2(2.79) + 20 = 25.6 \text{ ft}$

$$Q = \frac{1.486}{0.025}(34.6)\left(\frac{34.6}{25.6}\right)^{2/3}(0.0050)^{1/2}$$

$$= \underline{\underline{178 \text{ CFS}}}$$

(b) The above channel discharges onto Turquoise Ave.



$n = 0.015$
$S = 0.012$ '/ft

$A = (0.33)(36) + (0.025)(18)(18) = 20.0 \text{ ft}^2$
$P = 2(18) + 2(0.33) = 36.7 \text{ ft}$

$$Q = \frac{1.486}{0.015}(20.0)\left(\frac{20.0}{36.7}\right)^{2/3}(0.012)^{1/2}$$

$$= \underline{\underline{145 \text{ CFS}}}$$

APPENDIX E

HYDROLOGIC ANALYSIS FOR SUB-BASIN NO. 400/410

KAMINSKI HUBBARD engineering inc. • Surveying • Civil • Hydrology	Made by D.L.B.	Date 5-8-91	Job No. 0118
	Checked by	Date	Sheet No. 1
	For		

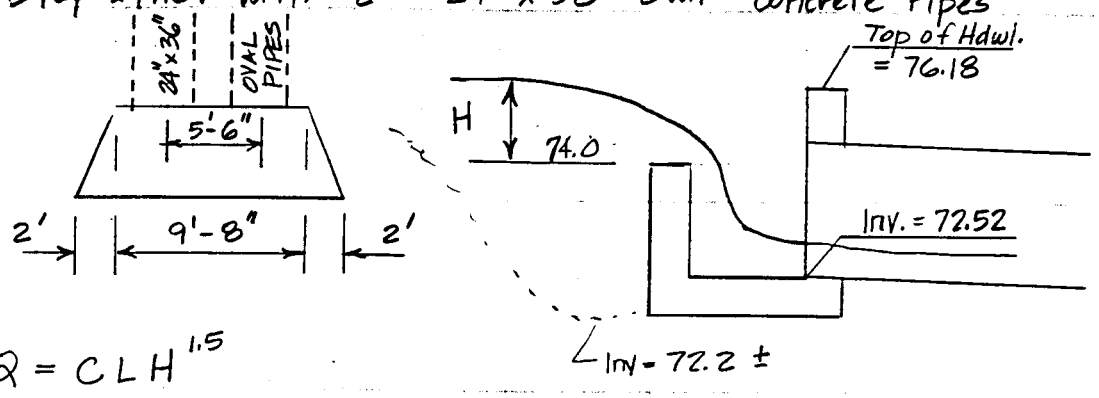
STORAGE - OUTFLOW RELATIONSHIP FOR CULVERT AT STA. 32+20

(1) STORAGE

ELEV. (FT.)	72.2	72.5	73.0	74.0	75.0	75.4	76.0	76.5		
STORAGE (AC-FT)	0	0.03	0.14	0.54	1.21	1.71	2.66	3.59		

(2) OUTFLOW

Drop Inlet with 2 - 24" x 38" Oval Concrete Pipes



$Q = CLH^{1.5}$

use $C = 3.3$

For Weir Flow:

ELEV. (FT)	1374.0	1374.5	1375.0	1375.4	1375.75	1376.0
Q (CFS)	0	15.9	45.1	74.7	104.4	127.6

There is a division between weir flow and orifice flow through the culverts. Use Chart 29 of the HEC-5 Manual to determine the allowable head to convey a specific discharge through 2 - 24" x 38" Oval pipes.

Compare the Above Weir Flowrates vs. HEC-5 Flowrates:

Elev.	Q_T	$Q/N=2$	CHART 29 H_w/D	H_w	W.S.	Comments
74.5	15.9	8	0.545	1.09	73.61	Weir Flow OK
75.0	45.1	22.5	1.05	2.10	74.62	Weir Flow OK
75.2	59.3	29.7	1.35	2.70	75.22	Use HEC-5
75.4	74.7	37.4	1.72	3.44	75.96	Use HEC-5

KAMINSKI HUBBARD engineering inc. • Surveying • Civil • Hydrology	Made by D.L.B.	Date 5-8-91	Job No. 0118
	Checked by	Date	Sheet No. 2
	For		

Compute Flowrates For the Following W.S. Elevations Using HEC-5:

W.S. Elev. (Ft.)	HW (Ft.)	HW/D	Q/N (CFS)	Q_T (CFS)
75.40	2.88	1.44	32	64
75.75	3.23	1.62	35	70
76.00	3.48	1.74	38	76
76.25	3.73	1.87	40	80
76.50	3.98	1.99	42	84

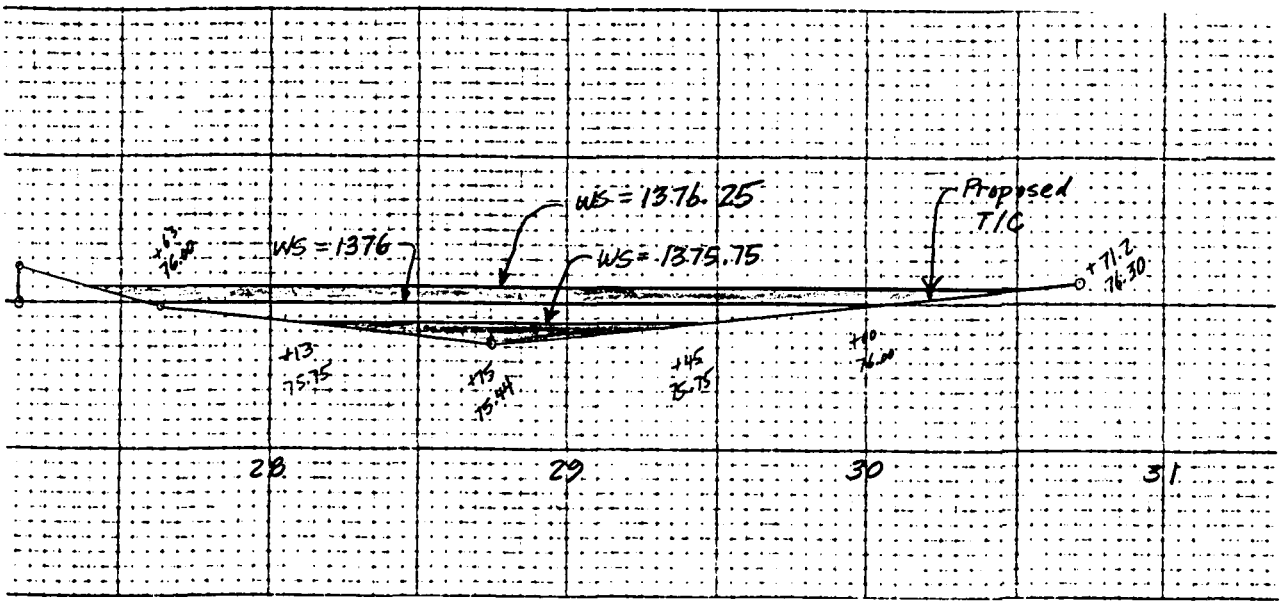
DISCHARGE - ELEVATION RELATIONSHIP FOR TATUM BLVD.

BETWEEN STA. 27+63 TO 30+00

W.S. ELEV. [FT.]	OUTFLOW FOR 2-24"x38" OVAL CONC. PIPES. [CFS]	OVERFLOW ACROSS TATUM BLVD. [CFS]	TOTAL DISCHARGE [CFS]
1372.2	0	0	0
1372.5	0	0	0
1373.0	0	0	0
1374.0	0	0	0
1374.5	16	0	16
1375.0	45	0	45
1375.2	59	0	59
1375.4	64	0	64
1375.75	70	26	96
1376.0	76	118	194
1376.25	80	300	380
1376.50	84	596	680

KAMINSKI HUBBARD engineering inc. • Surveying • Civil • Hydrology	Made by D.L.B.	Date 5-8-91	Job No. 0118
	Checked by	Date	Sheet No.
	For		

TATUM BLVD. OVERFLOW DISCHARGE AT STA. 28+75



Use Weir Eq. : $Q = CLH^{1.5}$ where $C = 3.0$

(1) Compute Q for WS Elev. = 1375.75

$$Q = \Sigma CLH^{1.5} = (3.0)\left[(37)\left(\frac{0.19}{2}\right)^{1.5} + (25)\left(\frac{0.19+0.31}{2}\right)^{1.5} + (25)\left(\frac{0.31+0.20}{2}\right)^{1.5} + (45)\left(\frac{0.20}{2}\right)^{1.5}\right]$$

$$= 26.6 \text{ CFS}$$

(2) Compute Q for WS Elev. = 1376

$$Q = \Sigma CLH^{1.5} = (3.0)\left[(50)\left(\frac{0.25}{2}\right)^{1.5} + (37)\left(\frac{0.25+0.44}{2}\right)^{1.5} + (25)\left(\frac{0.44+0.56}{2}\right)^{1.5} + (25)\left(\frac{0.56+0.45}{2}\right)^{1.5} + (45)\left(\frac{0.45+0.25}{2}\right)^{1.5} + (55)\left(\frac{0.25}{2}\right)^{1.5}\right]$$

$$= 117.8 \text{ CFS}$$

(3) Compute Q for WS Elev. = 1376.25

$$Q = \Sigma CLH^{1.5} = (3.0)\left[(36.5)\left(\frac{0.25}{2}\right)^{1.5} + (25)\left(\frac{0.25}{2}\right)^{1.5} + (50)\left(\frac{0.25+0.50}{2}\right)^{1.5} + (37)\left(\frac{0.50+0.69}{2}\right)^{1.5} + (25)\left(\frac{0.69+0.81}{2}\right)^{1.5} + (25)\left(\frac{0.81+0.70}{2}\right)^{1.5} + (45)\left(\frac{0.70+0.50}{2}\right)^{1.5} + (55)\left(\frac{0.50+0.25}{2}\right)^{1.5} + (60)\left(\frac{0.25}{2}\right)^{1.5}\right]$$

$$= 300.0 \text{ CFS}$$

KAMINSKI HUBBARD engineering inc. • Surveying • Civil • Hydrology	Made by DLB	Date 5-8-91	Job No. 0118
	Checked by	Date	Sheet No.
	For		

(4) Compute Q for WS Elev. = 1376.50

$$Q = \Sigma CLH^{1.5} = (3.0)\left[(34)\left(\frac{0.25}{2}\right)^{1.5} + (36.5)\left(\frac{0.25+0.50}{2}\right)^{1.5} + (23.5)\left(\frac{0.25}{2}\right)^{1.5}\right.$$

$$+ (25)\left(\frac{0.25+0.50}{2}\right)^{1.5} + (50)\left(\frac{0.50+0.75}{2}\right)^{1.5} + (62)\left(\frac{0.75+1.06}{2}\right)^{1.5}$$

$$+ (70)\left(\frac{1.06+0.75}{2}\right)^{1.5} + (55)\left(\frac{0.75+0.50}{2}\right)^{1.5} + (60)\left(\frac{0.50+0.25}{2}\right)^{1.5} + (57)\left(\frac{0.25}{2}\right)^{1.5}$$

$$= 596 \text{ cfs}$$

MADE BY DLB DATE 5-7-91 JOB NO. 0118

CHECKED BY _____ DATE _____

CALCULATIONS FOR

SHEET NO. 1

W.S. ELEV. = 1376.0

STATION	DIST. FT.	EXCAVATION END AREA SQ. FT.	EXCAVATION ½(SUM OF END AREAS)	EXCAVATION VOLUME CU. ~~YDS.~~ FT.
2300		0		
	100		15.4	1540
2400		30.8		
	100		41.7	4170
2500		52.6		
	100		58.0	5800
2600		63.4		
	100		44.3	4430
2700		25.2		
	50		92.6	4630
2750		160.0		
	50		177.2	8860
2800		194.4		
	50		209.1	10,455
2850		223.8		
	50		234.3	11,715
2900		244.8		
	50		238.3	11,915
2950		231.8		
	50		220.5	11,025
3000		209.2		
	50		197.6	9880
3050		186.0		
	50		171.0	8550
3100		156.0		
	50		143.5	7175
3150		131.0		
	50		116.5	5825
3200		102.0		
	50		85.4	4270
3250		68.8		
	50		57.8	2890
3300		46.8		
	50		35.2	1760
3350		23.6		
	50		15.3	765
3400		7.0		
	50		4.8	240
3450		2.6		
	25		1.3	32.5
3475		0		
			Total =	115,928 ft³

W.S. ELEV. = 1376.5

STATION	DIST. FT.	EMBANKMENT END AREA SQ. FT.	EMBANKMENT ½(SUM OF END AREAS)	EMBANKMENT VOLUME CU. ~~YDS.~~ FT.
2300		0		
	100		29.9	2990
2400		59.8		
	100		66.2	6620
2500		72.6		
	100		78.0	7800
2600		83.4		
	100		76.9	7690
2700		70.4		
	50		139.7	6985
2750		209.0		
	50		227.7	11,385
2800		246.4		
	50		264.0	13,200
2850		281.6		
	50		294.7	14,735
2900		307.8		
	50		298.3	14,915
2950		288.8		
	50		274.5	13,725
3000		260.2		
	50		247.1	12,355
3050		234.0		
	50		218.3	10,912
3100		202.5		
	50		190.0	9500
3150		177.5		
	50		163.9	8197
3200		150.4		
	50		122.5	6125
3250		94.6		
	50		82.7	4135
3300		70.8		
	50		57.8	2890
3350		44.8		
	50		30.6	1530
3400		16.4		
	50		12.4	620
3450		8.4		
	25		4.2	105
3475		0		
			Total =	156,414 ft³

MADE BY DLB DATE 5-7-91 JOB NO. 0118

CHECKED BY _____ DATE _____

CALCULATIONS FOR

SHEET NO. 2

W.S. ELEV. = 1375.4

STATION	DIST. FT.	EXCAVATION END AREA SQ. FT.	EXCAVATION ½(SUM OF END AREAS)	EXCAVATION VOLUME CU. ~~YDS.~~ FT.	STATION	DIST. FT.	EMBANKMENT END AREA SQ. FT.	EMBANKMENT SUM OF END AREAS	EMBANKMENT VOLUME CU. YDS.
23+67	33	0	4.0	132					
24+00	100	8.0	19.8	1980					
25+00	100	31.6	37.0	3700					
26+00	100	42.4	21.2	2120					
27+00	50	0	52.0	2600					
27+50	50	104.0	119.5	5975					
28+00	50	135.0	146.4	7320					
28+50	50	157.8	164.4	8220					
29+00	50	171.0	167.2	8360					
29+50	50	163.4	155.7	7785					
30+00	50	148.0	133.4	6670					
30+50	50	118.8	110.4	5520					
31+00	50	102.0	94.2	4710					
31+50	50	86.4	79.0	3950					
32+00	50	71.6	57.2	2860					
32+50	50	42.8	33.6	1680					
33+00	50	24.4	15.8	790					
33+50	50	7.2	3.6	180					
34+00		0							
				74,552 ft³					

MADE BY DLB DATE 5-7-91 JOB NO. 0118

CHECKED BY _____ DATE _____

CALCULATIONS FOR

SHEET NO 3

WATER SURFACE ELEVATION = 1375.0

LT. STATION	DIST. FT.	RETENTION VOLUME END AREA SQ. FT.	½(SUM OF END AREAS)	VOLUME CU. FT.
24+00		0		
	100		9.8	980
25+00		19.6		
	100		24.8	2480
26+00		30.0		
	100		15.0	1500
27+00		0		
	50		37.9	1895
27+50		75.8		
	"		82.9	4145
28+00		90.0		
	"		96.0	4800
28+50		102.0		
	"		108.0	5400
29+00		114.0		
	"		115.2	5760
29+50		116.4		
	"		114.2	5710
30+00		112.0		
	50		105.2	5260
30+50		98.4		
	"		90.9	4545
31+00		83.4		
	"		75.7	3785
31+50		68.0		
	"		61.0	3050
32+00		54.0		
	"		41.6	2080
32+50		29.2		
	"		21.0	1050
33+00		12.8		
	50		6.4	320
33+50		0		
			TOTAL =	52,760 ft³

W.S. ELEV. = 1374.0

STATION	DIST. FT.	RETENTION VOLUME END AREA SQ. FT.	½(SUM OF END AREAS)	VOLUME CU. FT.
25+00		0		
	100		4.0	400
26+00		8.0		
	100		4.0	400
27+00		0		
	50		17.6	880
27+50		35.2		
	"		39.4	1970
28+00		43.6		
	"		46.6	2330
28+50		49.6		
	"		51.5	2575
29+00		53.4		
	"		55.6	2780
29+50		57.8		
	50		59.1	2955
30+00		60.4		
	"		57.2	2860
30+50		54.0		
	"		48.6	2430
31+00		43.2		
	"		39.5	1975
31+50		35.8		
	"		27.8	1390
32+00		19.8		
	50		12.3	615
32+50		4.8		
	30		2.4	72
32+80		0		
			TOTAL =	23,632 ft³

MADE BY ULB DATE 5-7-91 JOB NO. 0118

CHECKED BY _____ DATE _____

CALCULATIONS FOR

SHEET NO 4

W.G. ELEV. = 1373.0

STATION	DIST. FT.	RETENTION VOLUME ~~EXCAVATION~~ END AREA SQ. FT.	½(SUM OF END AREAS)	VOLUME FT. CU. ~~YDS.~~
27+00		0		
	50		3.8	190
27+50		7.6		
	50		9.8	490
28+00		12.0		
	50		12.7	635
28+50		13.4		
	50		13.5	675
29+00		13.6		
	50		15.3	765
29+50		17.0		
	50		18.8	940
30+00		20.6		
	50		19.5	975
30+50		18.4		
	50		16.0	800
31+00		13.6		
	50		9.8	490
31+50		6.0		
	50		3.0	150
32+00		0		
				6,110 ft³

W.S. ELEV. = 1372.5

STATION	DIST. FT.	RETENTION VOLUME ~~EMBANKMENT~~ END AREA SQ. FT.	½(SUM OF END AREAS)	VOLUME FT. CU. ~~YDS.~~
27+50		0		
	50		1.4	70
28+00		2.8		
	50		3.0	150
28+50		3.2		
	50		2.6	130
29+00		2.0		
	50		3.0	150
29+50		4.0		
	50		5.7	285
30+00		7.4		
	50		6.8	340
30+50		6.2		
	50		5.1	255
31+00		4.0		
	50		2.0	100
31+50		0		
				1,480 ft³

```
*****************************************
*                                       *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *
*              MAY   1991               *
*           VERSION 4.0.1E              *
*                                       *
*  RUN DATE  01/14/93  TIME  09:55:20   *
*                                       *
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*****************************************
*                                       *
*    U.S. ARMY CORPS OF ENGINEERS       *
*    HYDROLOGIC ENGINEERING CENTER      *
*          609 SECOND STREET            *
*       DAVIS, CALIFORNIA 95616         *
*           (916) 551-1748              *
*                                       *
*****************************************
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X     X  X        X              X
XXXXXXX  XXXX     X       XXXXX  X
X     X  X        X              X
X     X  X        X     X        X
X     X  XXXXXXX   XXXXX         XXX
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THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

```
****************************************                    ****************************************
*                                      *                    *                                      *
*  FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                    *   U.S. ARMY CORPS OF ENGINEERS       *
*             MAY   1991               *                    *   HYDROLOGIC ENGINEERING CENTER      *
*           VERSION 4.0.1E             *                    *         609 SECOND STREET            *
*                                      *                    *      DAVIS, CALIFORNIA 95616         *
* RUN DATE  01/14/93  TIME  09:55:20   *                    *         (916) 551-1748               *
*                                      *                    *                                      *
****************************************                    ****************************************
```

CITY OF PHOENIX PROJECT NO. P-886515
TATUM BLVD. - MOCKINGBIRD LANE TO SHEA BLVD.
KAMINSKI - HUBBARD ENGINEERING, INC. - JOB NO. 0118
HEC-1 FILENAME : 118-29.DAT
2 YEAR - 6 HOUR STORM

```
 7 IO        OUTPUT CONTROL VARIABLES
                   IPRNT          5  PRINT CONTROL
                   IPLOT          0  PLOT CONTROL
                   QSCAL         0.  HYDROGRAPH PLOT SCALE

   IT        HYDROGRAPH TIME DATA
                    NMIN          2  MINUTES IN COMPUTATION INTERVAL
                   IDATE     7FEB90  STARTING DATE
                   ITIME       1000  STARTING TIME
                      NQ        721  NUMBER OF HYDROGRAPH ORDINATES
                  NDDATE     8FEB90  ENDING DATE
                  NDTIME       1000  ENDING TIME
                   ICENT         19  CENTURY MARK

             COMPUTATION INTERVAL    0.03 HOURS
                  TOTAL TIME BASE   24.00 HOURS

       ENGLISH UNITS
            DRAINAGE AREA          SQUARE MILES
            PRECIPITATION DEPTH    INCHES
            LENGTH, ELEVATION      FEET
            FLOW                   CUBIC FEET PER SECOND
            STORAGE VOLUME         ACRE-FEET
            SURFACE AREA           ACRES
            TEMPERATURE            DEGREES FAHRENHEIT


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


          **************
          *            *
 18 KK    *     400R   *              Subarea No. 400/410
          *            *
          **************

 20 KO        OUTPUT CONTROL VARIABLES
                   IPRNT          3  PRINT CONTROL
                   IPLOT          0  PLOT CONTROL
                   QSCAL         0.  HYDROGRAPH PLOT SCALE

            HYDROGRAPH ROUTING DATA

 21 RS        STORAGE ROUTING
                   NSTPS          1  NUMBER OF SUBREACHES
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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 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
HYDROGRAPH AT	400	51.	4.07	4.	1.	1.	0.08		
ROUTED TO	400R	24.	4.23	3.	1.	1.	0.08	1374.64	4.23

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

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****************************************
*                                      *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *
*              MAY   1991              *
*           VERSION 4.0.1E             *
*                                      *
*  RUN DATE  01/14/93  TIME  09:54:41  *
*                                      *
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****************************************
*                                      *
*    U.S. ARMY CORPS OF ENGINEERS      *
*    HYDROLOGIC ENGINEERING CENTER     *
*          609 SECOND STREET           *
*       DAVIS, CALIFORNIA 95616        *
*           (916) 551-1748             *
*                                      *
****************************************
```

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X     X  XXXXXXX   XXXXX          X
X     X  X        X     X        XX
X     X  X        X              X
XXXXXXX  XXXX     X       XXXXX  X
X     X  X        X              X
X     X  X        X     X        X
X     X  XXXXXXX   XXXXX         XXX
```

THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

```
****************************************                    ****************************************
*                                      *                    *                                      *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *                    *    U.S. ARMY CORPS OF ENGINEERS      *
*              MAY   1991              *                    *    HYDROLOGIC ENGINEERING CENTER     *
*           VERSION 4.0.1E             *                    *          609 SECOND STREET           *
*                                      *                    *       DAVIS, CALIFORNIA 95616        *
*  RUN DATE  01/14/93  TIME  09:54:41  *                    *          (916) 551-1748              *
*                                      *                    *                                      *
****************************************                    ****************************************
```

CITY OF PHOENIX PROJECT NO. P-886515
TATUM BLVD. - MOCKINGBIRD LANE TO SHEA BLVD.
KAMINSKI - HUBBARD ENGINEERING, INC. - JOB NO. 0118
HEC-1 FILENAME : 118-28.DAT
10 YEAR - 6 HOUR STORM

```
 7 IO        OUTPUT CONTROL VARIABLES
                   IPRNT          5  PRINT CONTROL
                   IPLOT          0  PLOT CONTROL
                   QSCAL         0.  HYDROGRAPH PLOT SCALE

   IT        HYDROGRAPH TIME DATA
                    NMIN          2  MINUTES IN COMPUTATION INTERVAL
                   IDATE     7FEB90  STARTING DATE
                   ITIME       1000  STARTING TIME
                      NQ        721  NUMBER OF HYDROGRAPH ORDINATES
                  NDDATE     8FEB90  ENDING DATE
                  NDTIME       1000  ENDING TIME
                   ICENT         19  CENTURY MARK

              COMPUTATION INTERVAL    0.03 HOURS
                   TOTAL TIME BASE   24.00 HOURS

        ENGLISH UNITS
             DRAINAGE AREA          SQUARE MILES
             PRECIPITATION DEPTH    INCHES
             LENGTH, ELEVATION      FEET
             FLOW                   CUBIC FEET PER SECOND
             STORAGE VOLUME         ACRE-FEET
             SURFACE AREA           ACRES
             TEMPERATURE            DEGREES FAHRENHEIT


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


          **************
          *            *
 18 KK    *      400R  *                Subarea No. 400/410
          *            *
          **************

 20 KO       OUTPUT CONTROL VARIABLES
                   IPRNT          3  PRINT CONTROL
                   IPLOT          0  PLOT CONTROL
                   QSCAL         0.  HYDROGRAPH PLOT SCALE

             HYDROGRAPH ROUTING DATA

 21 RS       STORAGE ROUTING
                   NSTPS          1  NUMBER OF SUBREACHES
```

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 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
HYDROGRAPH AT	400	179.	4.03	14.	4.	4.	0.11		
ROUTED TO	400R	145.	4.13	13.	3.	3.	0.11	1375.87	4.13

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

```
****************************************
*                                      *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *
*              MAY   1991              *
*           VERSION 4.0.1E             *
*                                      *
*  RUN DATE  01/14/93  TIME  09:53:56  *
*                                      *
****************************************
```

```
****************************************
*                                      *
*    U.S. ARMY CORPS OF ENGINEERS      *
*    HYDROLOGIC ENGINEERING CENTER     *
*          609 SECOND STREET           *
*       DAVIS, CALIFORNIA 95616        *
*           (916) 551-1748             *
*                                      *
****************************************
```

```
X     X  XXXXXXX   XXXXX          X
X     X  X        X     X        XX
X     X  X        X               X
XXXXXXX  XXXX     X        XXXXX  X
X     X  X        X               X
X     X  X        X     X         X
X     X  XXXXXXX   XXXXX         XXX
```

THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

```
****************************************                    ****************************************
*                                      *                    *                                      *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *                    *   U.S. ARMY CORPS OF ENGINEERS       *
*              MAY   1991              *                    *   HYDROLOGIC ENGINEERING CENTER      *
*           VERSION 4.0.1E             *                    *         609 SECOND STREET            *
*                                      *                    *      DAVIS, CALIFORNIA 95616         *
*  RUN DATE  01/14/93  TIME  09:53:56  *                    *          (916) 551-1748              *
*                                      *                    *                                      *
****************************************                    ****************************************
```

CITY OF PHOENIX PROJECT NO. P-886515
TATUM BLVD. - MOCKINGBIRD LANE TO SHEA BLVD.
KAMINSKI - HUBBARD ENGINEERING, INC. - JOB NO. 0118
HEC-1 FILENAME : 118-27.DAT
25 YEAR - 6 HOUR STORM

```
 7 IO        OUTPUT CONTROL VARIABLES
                   IPRNT          5  PRINT CONTROL
                   IPLOT          0  PLOT CONTROL
                   QSCAL         0.  HYDROGRAPH PLOT SCALE

   IT        HYDROGRAPH TIME DATA
                    NMIN          2  MINUTES IN COMPUTATION INTERVAL
                   IDATE     7FEB90  STARTING DATE
                   ITIME       1000  STARTING TIME
                      NQ        721  NUMBER OF HYDROGRAPH ORDINATES
                  NDDATE     8FEB90  ENDING DATE
                  NDTIME       1000  ENDING TIME
                   ICENT         19  CENTURY MARK

              COMPUTATION INTERVAL    0.03 HOURS
                   TOTAL TIME BASE   24.00 HOURS

        ENGLISH UNITS
            DRAINAGE AREA          SQUARE MILES
            PRECIPITATION DEPTH    INCHES
            LENGTH, ELEVATION      FEET
            FLOW                   CUBIC FEET PER SECOND
            STORAGE VOLUME         ACRE-FEET
            SURFACE AREA           ACRES
            TEMPERATURE            DEGREES FAHRENHEIT


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


          **************
          *            *
 18 KK    *     400R   *                Subarea No. 400/410
          *            *
          **************

 20 KO        OUTPUT CONTROL VARIABLES
                   IPRNT          3  PRINT CONTROL
                   IPLOT          0  PLOT CONTROL
                   QSCAL         0.  HYDROGRAPH PLOT SCALE

            HYDROGRAPH ROUTING DATA

 21 RS        STORAGE ROUTING
                   NSTPS          1  NUMBER OF SUBREACHES
```

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 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
HYDROGRAPH AT	400	323.	4.03	26.	6.	6.	0.15		
ROUTED TO	400R	314.	4.07	25.	6.	6.	0.15	1376.16	4.07

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

```
****************************************
*                                      *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *
*              MAY   1991              *
*           VERSION 4.0.1E             *
*                                      *
*  RUN DATE  01/14/93  TIME  09:53:17  *
*                                      *
****************************************
```

```
****************************************
*                                      *
*    U.S. ARMY CORPS OF ENGINEERS      *
*    HYDROLOGIC ENGINEERING CENTER     *
*          609 SECOND STREET           *
*       DAVIS, CALIFORNIA 95616        *
*           (916) 551-1748             *
*                                      *
****************************************
```

```
X     X  XXXXXXX   XXXXX          X
X     X  X        X     X        XX
X     X  X        X               X
XXXXXXX  XXXX     X       XXXXX   X
X     X  X        X               X
X     X  X        X     X         X
X     X  XXXXXXX   XXXXX         XXX
```

THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

```
****************************************
*                                      *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *
*              MAY   1991              *
*            VERSION 4.0.1E            *
*                                      *
*  RUN DATE  01/14/93  TIME  09:53:17  *
*                                      *
****************************************
```

```
****************************************
*                                      *
*    U.S. ARMY CORPS OF ENGINEERS      *
*    HYDROLOGIC ENGINEERING CENTER     *
*          609 SECOND STREET           *
*       DAVIS, CALIFORNIA 95616        *
*           (916) 551-1748             *
*                                      *
****************************************
```

CITY OF PHOENIX PROJECT NO. P-886515
TATUM BLVD. - MOCKINGBIRD LANE TO SHEA BLVD.
KAMINSKI - HUBBARD ENGINEERING, INC. - JOB NO. 0118
HEC-1 FILENAME : 118-26.DAT
50 YEAR - 6 HOUR STORM

```
 7 IO        OUTPUT CONTROL VARIABLES
                  IPRNT          5  PRINT CONTROL
                  IPLOT          0  PLOT CONTROL
                  QSCAL         0.  HYDROGRAPH PLOT SCALE

   IT        HYDROGRAPH TIME DATA
                   NMIN          2  MINUTES IN COMPUTATION INTERVAL
                  IDATE     7FEB90  STARTING DATE
                  ITIME       1000  STARTING TIME
                     NQ        721  NUMBER OF HYDROGRAPH ORDINATES
                 NDDATE     8FEB90  ENDING DATE
                 NDTIME       1000  ENDING TIME
                  ICENT         19  CENTURY MARK

             COMPUTATION INTERVAL    0.03 HOURS
                  TOTAL TIME BASE   24.00 HOURS

        ENGLISH UNITS
             DRAINAGE AREA         SQUARE MILES
             PRECIPITATION DEPTH   INCHES
             LENGTH, ELEVATION     FEET
             FLOW                  CUBIC FEET PER SECOND
             STORAGE VOLUME        ACRE-FEET
             SURFACE AREA          ACRES
             TEMPERATURE           DEGREES FAHRENHEIT


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


          **************
          *            *
 18 KK    *     400R   *                Subarea No. 400/410
          *            *
          **************

 20 KO        OUTPUT CONTROL VARIABLES
                  IPRNT          3  PRINT CONTROL
                  IPLOT          0  PLOT CONTROL
                  QSCAL         0.  HYDROGRAPH PLOT SCALE

             HYDROGRAPH ROUTING DATA

 21 RS        STORAGE ROUTING
                  NSTPS          1  NUMBER OF SUBREACHES
```

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 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
HYDROGRAPH AT	400	389.	4.03	32.	8.	8.	0.15		
ROUTED TO	400R	381.	4.07	30.	8.	8.	0.15	1376.25	4.07

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

```
****************************************
*                                      *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *
*              MAY   1991              *
*           VERSION 4.0.1E             *
*                                      *
*  RUN DATE  01/14/93  TIME  09:50:55  *
*                                      *
****************************************
```

```
****************************************
*                                      *
*    U.S. ARMY CORPS OF ENGINEERS      *
*    HYDROLOGIC ENGINEERING CENTER     *
*          609 SECOND STREET           *
*       DAVIS, CALIFORNIA 95616        *
*           (916) 551-1748             *
*                                      *
****************************************
```

```
X     X  XXXXXXX   XXXXX          X
X     X  X        X     X        XX
X     X  X        X              X
XXXXXXX  XXXX     X       XXXXX  X
X     X  X        X              X
X     X  X        X     X        X
X     X  XXXXXXX   XXXXX         XXX
```

THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

```
****************************************
*                                      *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *
*              MAY   1991              *
*           VERSION 4.0.1E             *
*                                      *
*  RUN DATE  01/14/93  TIME  09:50:55  *
*                                      *
****************************************
```

```
****************************************
*                                      *
*    U.S. ARMY CORPS OF ENGINEERS      *
*    HYDROLOGIC ENGINEERING CENTER     *
*          609 SECOND STREET           *
*       DAVIS, CALIFORNIA 95616        *
*           (916) 551-1748             *
*                                      *
****************************************
```

CITY OF PHOENIX PROJECT NO. P-886515
TATUM BLVD. - MOCKINGBIRD LANE TO SHEA BLVD.
KAMINSKI - HUBBARD ENGINEERING, INC. - JOB NO. 0118
HEC-1 FILENAME : 118-25.DAT
100 YEAR - 6 HOUR STORM

```
 7 IO        OUTPUT CONTROL VARIABLES
                  IPRNT          5  PRINT CONTROL
                  IPLOT          0  PLOT CONTROL
                  QSCAL         0.  HYDROGRAPH PLOT SCALE

   IT        HYDROGRAPH TIME DATA
                   NMIN          2  MINUTES IN COMPUTATION INTERVAL
                  IDATE     7FEB90  STARTING DATE
                  ITIME       1000  STARTING TIME
                     NQ        721  NUMBER OF HYDROGRAPH ORDINATES
                 NDDATE     8FEB90  ENDING DATE
                 NDTIME       1000  ENDING TIME
                  ICENT         19  CENTURY MARK

             COMPUTATION INTERVAL    0.03 HOURS
                  TOTAL TIME BASE   24.00 HOURS

       ENGLISH UNITS
            DRAINAGE AREA          SQUARE MILES
            PRECIPITATION DEPTH    INCHES
            LENGTH, ELEVATION      FEET
            FLOW                   CUBIC FEET PER SECOND
            STORAGE VOLUME         ACRE-FEET
            SURFACE AREA           ACRES
            TEMPERATURE            DEGREES FAHRENHEIT


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


          **************
          *            *
 18 KK    *     400R   *                Subarea No. 400/410
          *            *
          **************

 20 KO        OUTPUT CONTROL VARIABLES
                  IPRNT          3  PRINT CONTROL
                  IPLOT          0  PLOT CONTROL
                  QSCAL         0.  HYDROGRAPH PLOT SCALE

            HYDROGRAPH ROUTING DATA

 21 RS        STORAGE ROUTING
                  NSTPS          1  NUMBER OF SUBREACHES
```

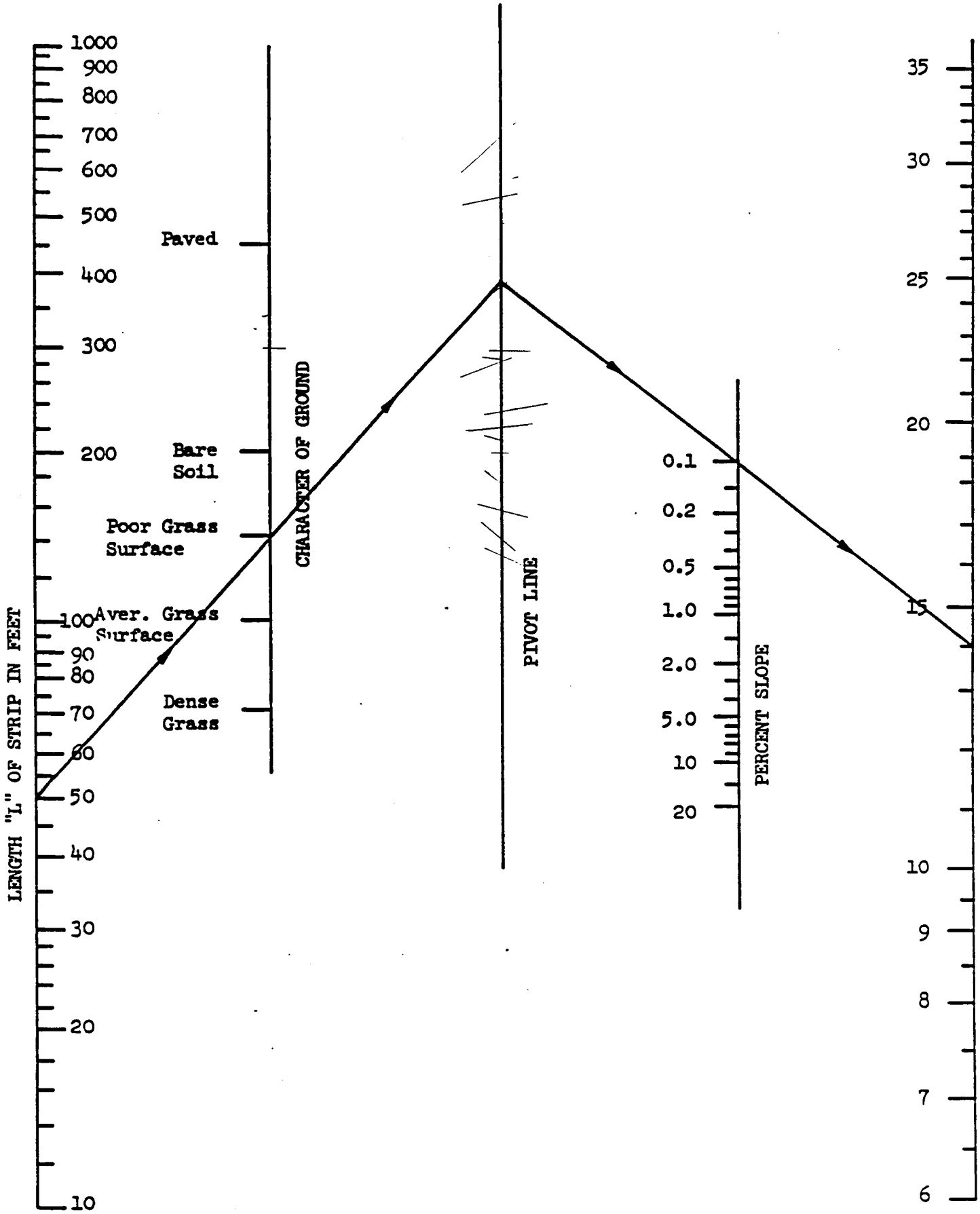
RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
HYDROGRAPH AT	400	455.	4.03	37.	9.	9.	0.15		
ROUTED TO	400R	451.	4.07	36.	9.	9.	0.15	1376.31	4.07

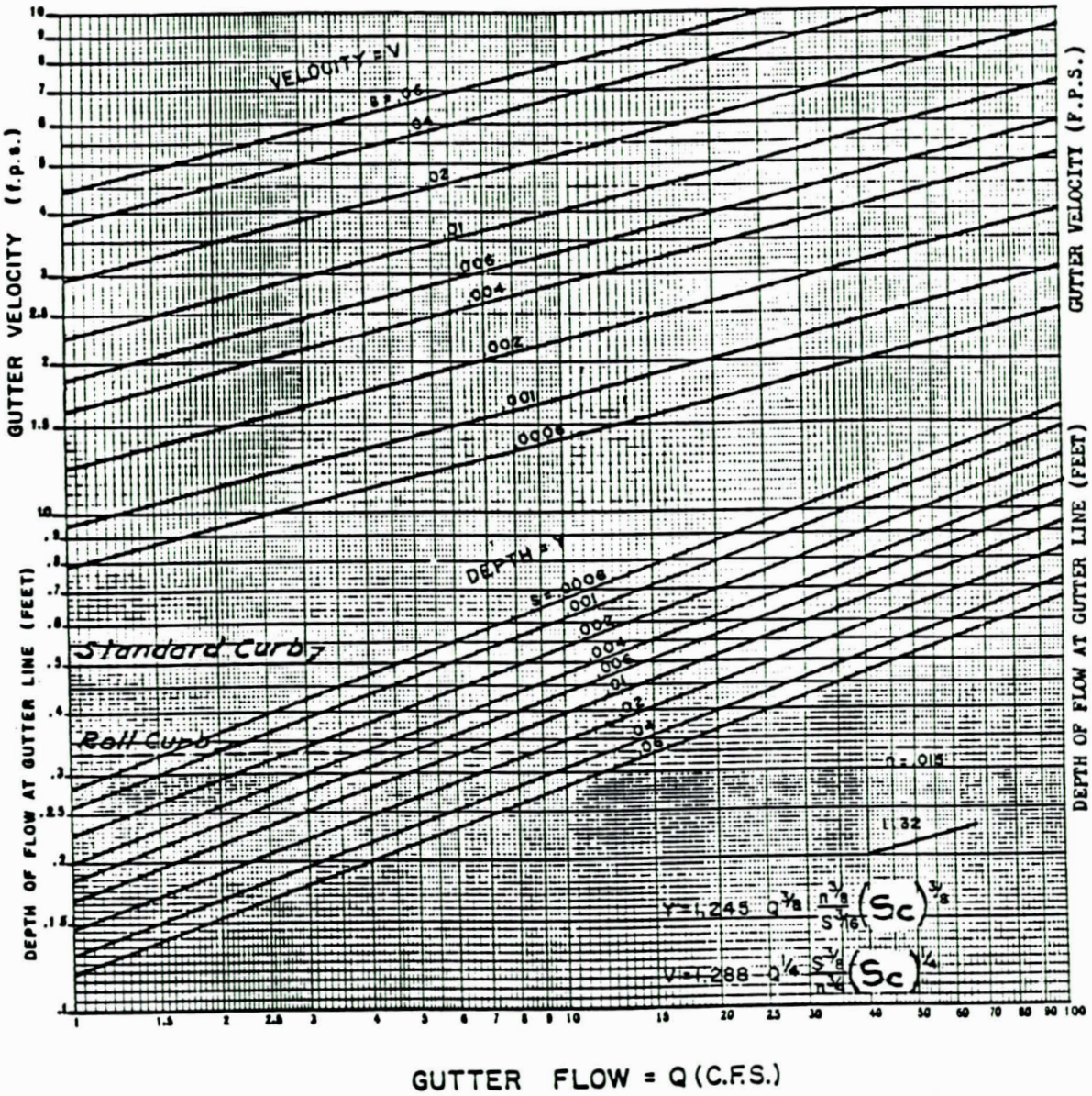
*** NORMAL END OF HEC-1 ***

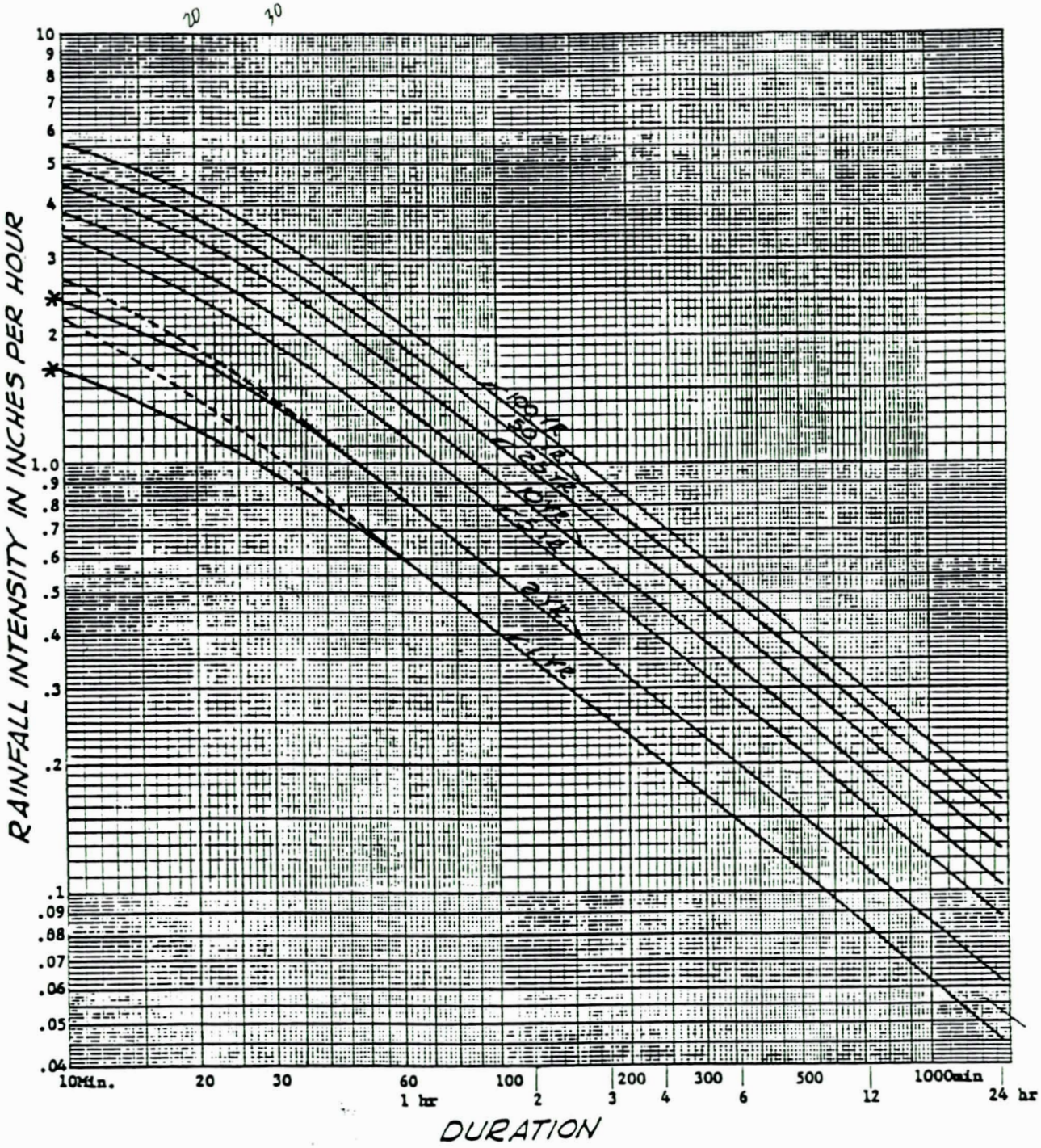
APPENDIX F

HYDROLOGIC AND HYDRAULIC CHARTS AND TABLES



OVERLAND FLOW TIME

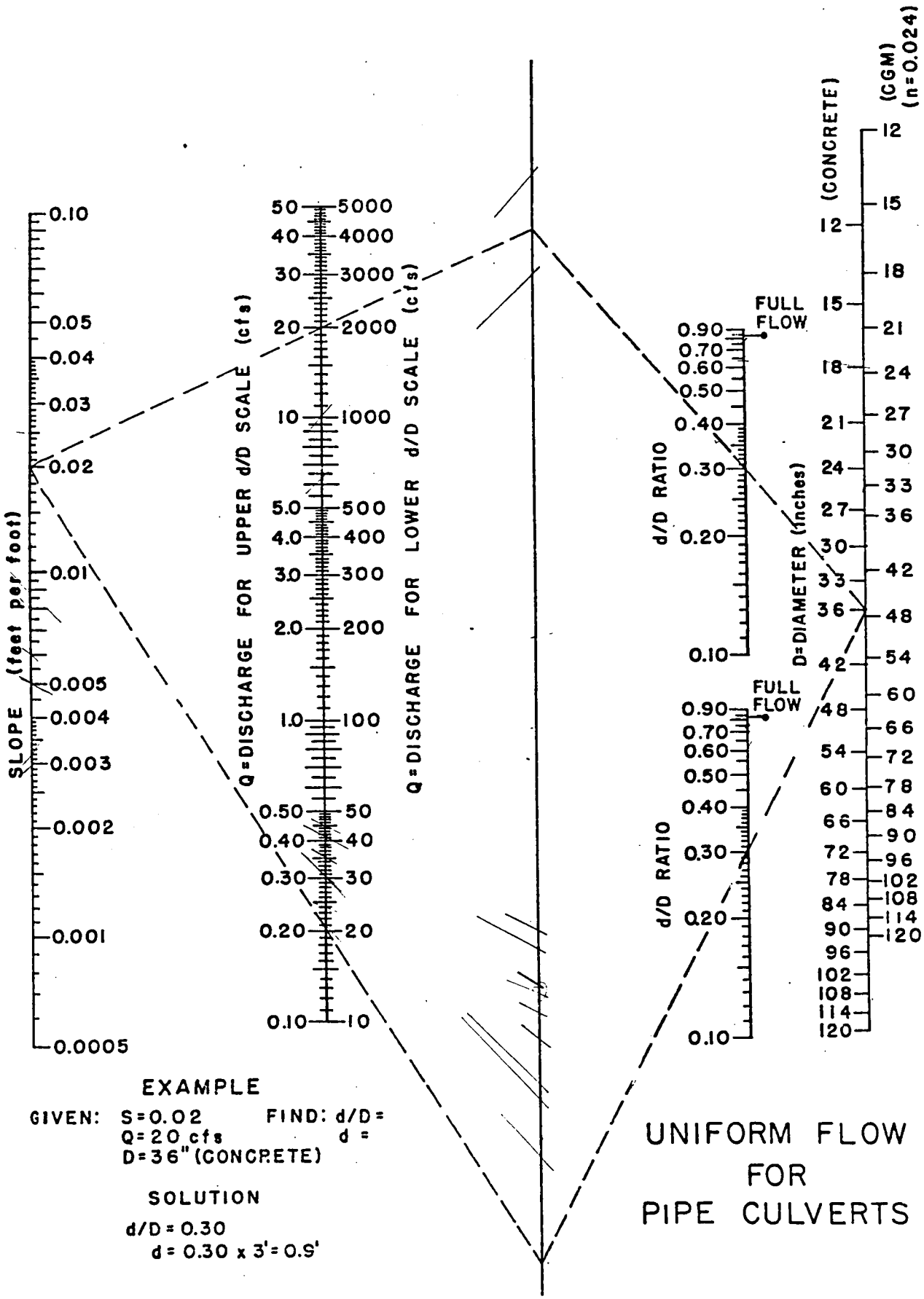

VELOCITY = V
S = .06
.04
.02
.01
.006
.004
.002
.001
.0006
DEPTH = Y
S = .0006
.001
.002
.004
.006
.01
.02
.04
.06
Standard Curb
Roll Curb
n = .015
1:32
$Y = 1.245\ Q^{3/8}\ \frac{n^{3/8}}{S^{3/16}} (Sc)^{3/8}$
$V = 1.288\ Q^{1/4}\ \frac{S^{1/8}}{n^{1/4}} (Sc)^{1/4}$
GUTTER VELOCITY (f.p.s.)
DEPTH OF FLOW AT GUTTER LINE (FEET)
GUTTER VELOCITY (F.P.S.)
DEPTH OF FLOW AT GUTTER LINE (FEET)
GUTTER FLOW = Q (C.F.S.)



RAINFALL INTENSITY-DURATION-FREQUENCY RELATION
FOR PHOENIX, ARIZONA
(Partial Duration Series)

Curves are based on methods of U.S. Weather Bureau Technical Papers Nos. 28 and 40 and rainfall data prepared by U.S. Weather Bureau Office of Hydrology for the Soil Conservation Service, March 1967

* Curves revised June 1975 to reflect new information from WR-44.



EXAMPLE

GIVEN: S=0.02
Q=20 cfs
D=36" (CONCRETE)

FIND: d/D=
d =

SOLUTION

d/D = 0.30
d = 0.30 x 3' = 0.9'

UNIFORM FLOW FOR PIPE CULVERTS

AHD
Structures Section
Hydraulics Branch
10-15-72

Chart 15

Texas
Highway
Department

Table 4-17 Solution of Manning's Formula—Pipe Running Full

Diam. in Inches	Area in Square Feet	Hydraulic Radius	$R^{1/6}$	$AR^{1/6}$	$\frac{1.486}{n} \times R^{1/6} \times A$ for various values of n				
					n=.012	n=.015	n=.019	n=.021	n=.024
6	.196	.125	.250	.049	6.068	4.854	3.832	3.467	3.034
8	.349	.167	.303	.106	13.13	10.50	8.290	7.501	6.563
10	.545	.208	.351	.191	23.65	18.92	14.94	13.52	11.83
12	.785	.250	.397	.312	38.64	30.91	24.40	22.08	19.32
15	1.227	.3125	.461	.566	70.09	56.07	44.27	40.05	35.04
18	1.767	.375	.520	.919	113.8	91.04	71.88	65.03	56.90
21	2.405	.437	.576	1.385	171.5	137.2	108.3	98.01	85.75
24	3.142	.50	.630	1.979	245.1	196.1	154.8	140.0	122.5
30	4.909	.625	.731	3.588	444.3	355.5	280.6	253.9	222.2
36	7.069	.75	.825	5.832	722.2	577.8	456.1	412.7	361.1
42	9.621	.875	.915	8.803	1090	872.1	688.5	622.9	545.0
48	12.566	1.00	1.00	12.566	1556	1245	982.8	889.2	778.0
54	15.904	1.125	1.082	17.208	2131	1705	1346	1218	1065
60	19.635	1.25	1.16	22.777	2821	2256	1781	1612	1410
66	23.758	1.375	1.236	29.365	3636	2909	2297	2078	1818
72	28.274	1.50	1.310	37.039	4587	3669	2897	2621	2293
78	33.183	1.625	1.382	45.859	5679	4543	3587	3245	2839
84	38.485	1.75	1.452	55.880	6920	5536	4370	3954	3460
90	44.179	1.875	1.521	67.196	8321	6657	5255	4755	4161
96	50.266	2.00	1.587	79.722	9878	7903	6239	5645	4939
108	63.617	2.25	1.717	109.230	13526	10820	8543	7729	6763
114	70.882	2.375	1.780	126.170	15624	12500	9868	8928	7812
120	78.54	2.5	1.842	144.671	17915	14330	11320	10240	8957
27	3.976	0.563	0.681	2.708	335.3				

$Q = AV$

$Q = A \times \frac{1.486}{n} \times R^{1/6} \times S^{1/2}$ $V = \frac{1.486}{n} \times R^{1/6} \times S^{1/2}$

V = Velocity in fps R = Hydraulic Radius. S = Hydraulic slope = fall in ft divided by length in feet.
Q = Capacity in cfs A = Area of pipe in square ft. n = Coefficient of Roughness.

Note: To obtain Q for any diameter pipe, multiply the figure shown in the column under the proper value of n by the sq root of the slope.

MIN. SLOPE REQ'D $= (Q/K)^2$

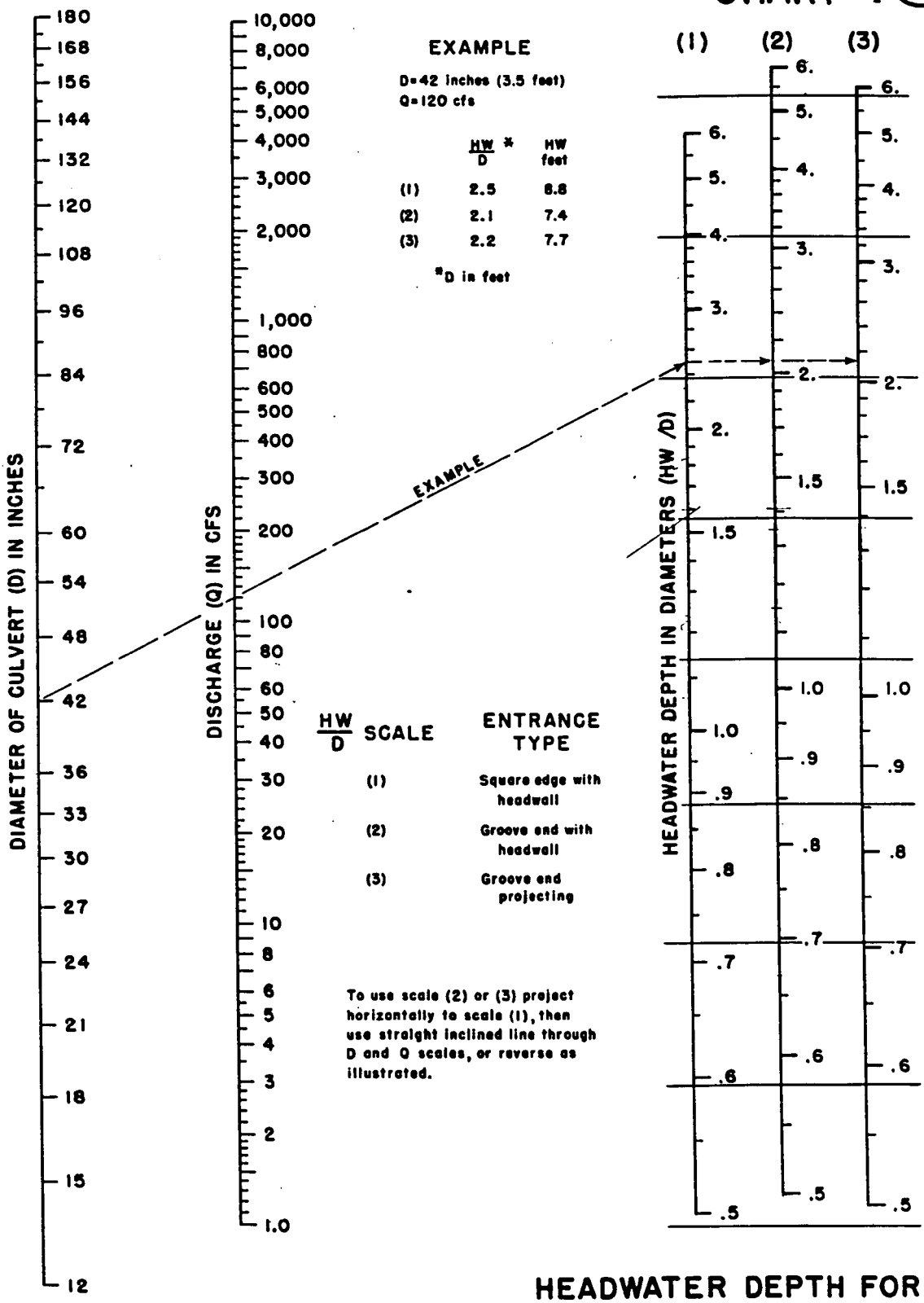
For Determining the Area "a" of the Cross Section
of a Circular Conduit Flowing Part Full

Let $\frac{\text{depth of water}}{\text{diameter of channel}} = \frac{D}{d}$ and C_a = the tabulated value. Then $a = C_a d^2$.

$\frac{D}{d}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0	.0000	.0013	.0037	.0069	.0105	.0147	.0192	.0242	.0294	.0350
.1	.0409	.0470	.0534	.0600	.0668	.0739	.0811	.0885	.0961	.1039
.2	.1118	.1199	.1281	.1365	.1449	.1535	.1623	.1711	.1800	.1890
.3	.1982	.2074	.2167	.2260	.2355	.2450	.2546	.2642	.2739	.2836
.4	.2934	.3032	.3130	.3229	.3328	.3428	.3527	.3627	.3727	.3827
.5	.393	.403	.413	.423	.433	.443	.453	.462	.472	.482
.6	.492	.502	.512	.521	.531	.540	.550	.559	.569	.578
.7	.587	.596	.605	.614	.623	.632	.640	.649	.657	.666
.8	.647	.681	.689	.697	.704	.712	.719	.725	.732	.738
.9	.745	.750	.756	.761	.766	.771	.775	.779	.782	.784

Table 2

CHART 1

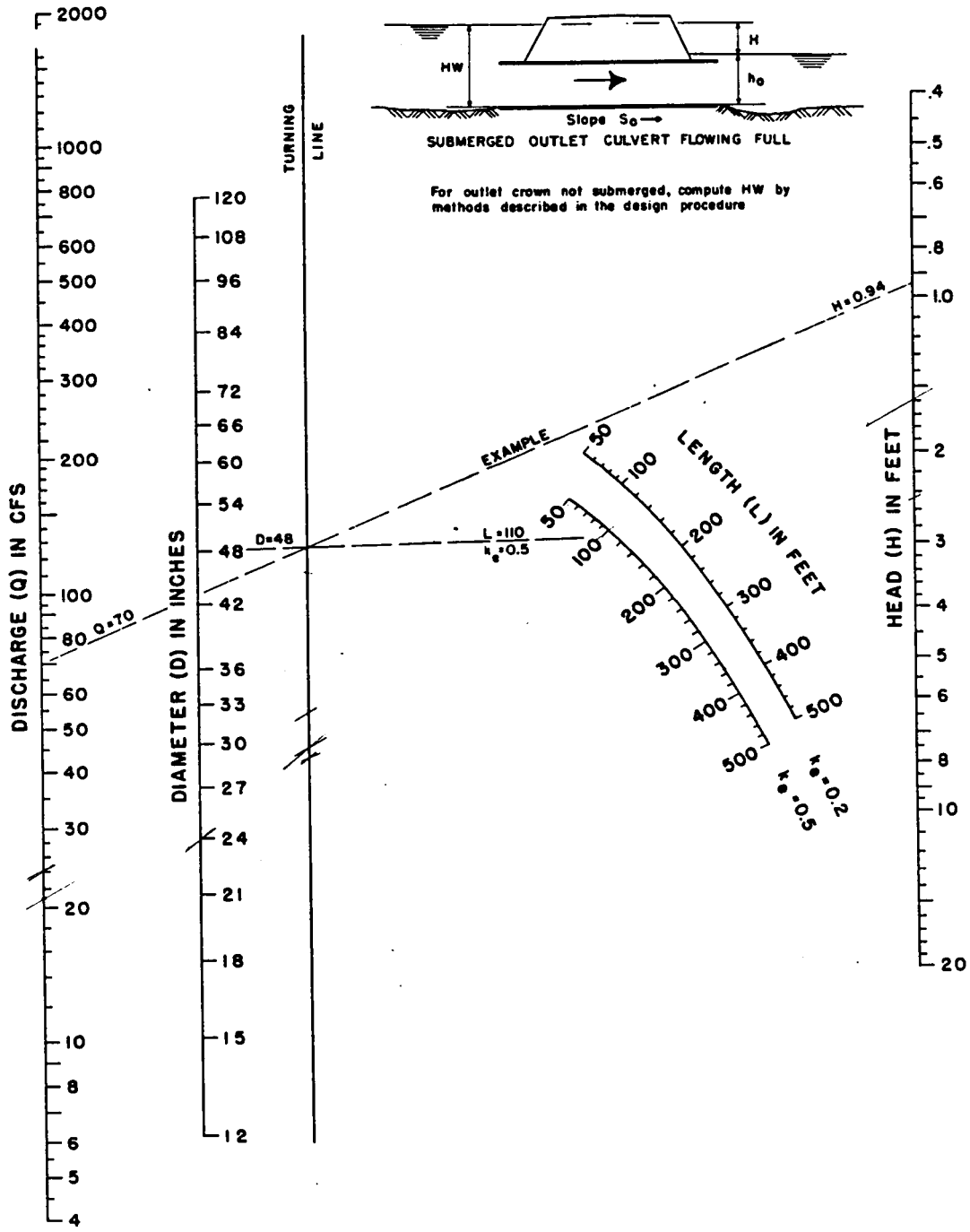


HEADWATER SCALES 2&3
REVISED MAY 1964

BUREAU OF PUBLIC ROADS JAN. 1963

HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

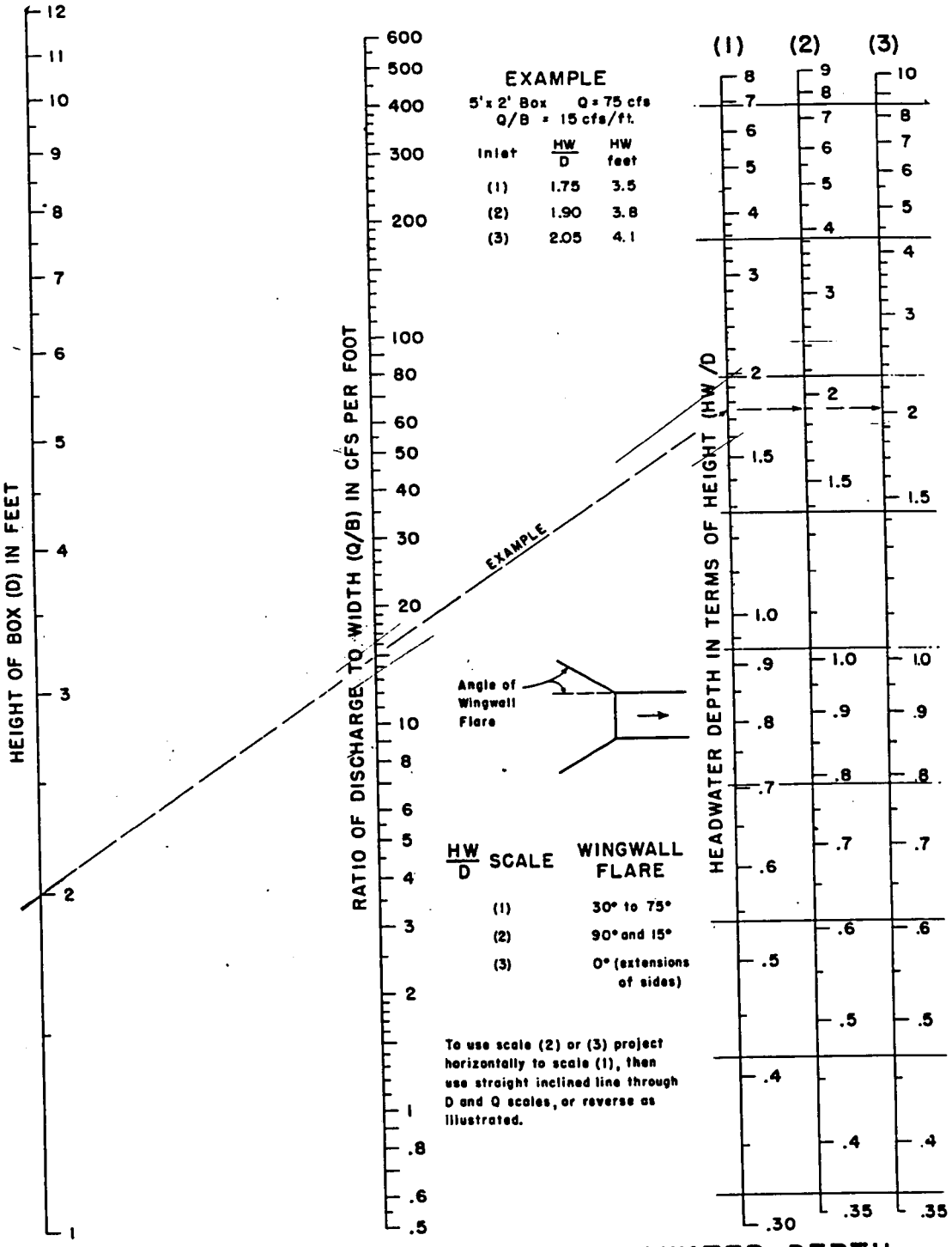
CHART 5



HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

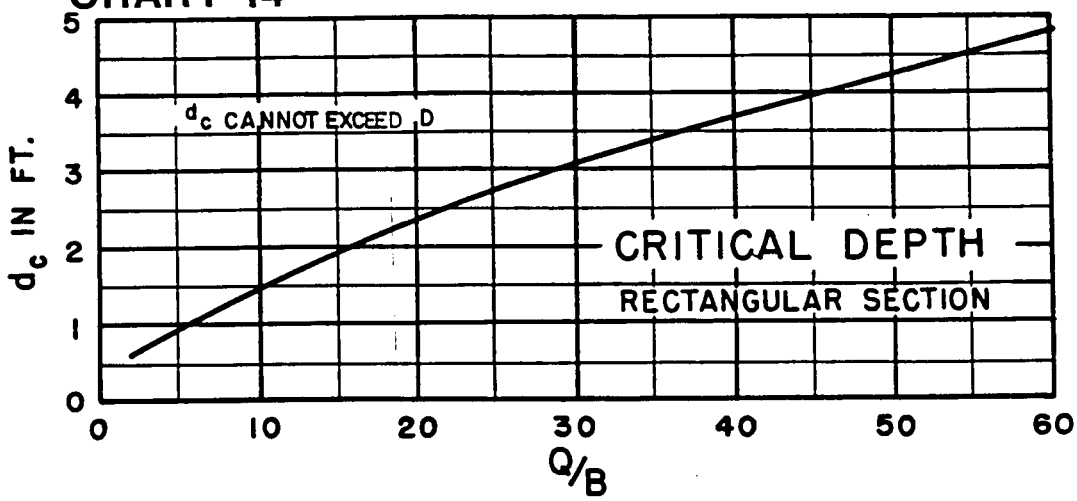
CHART 8

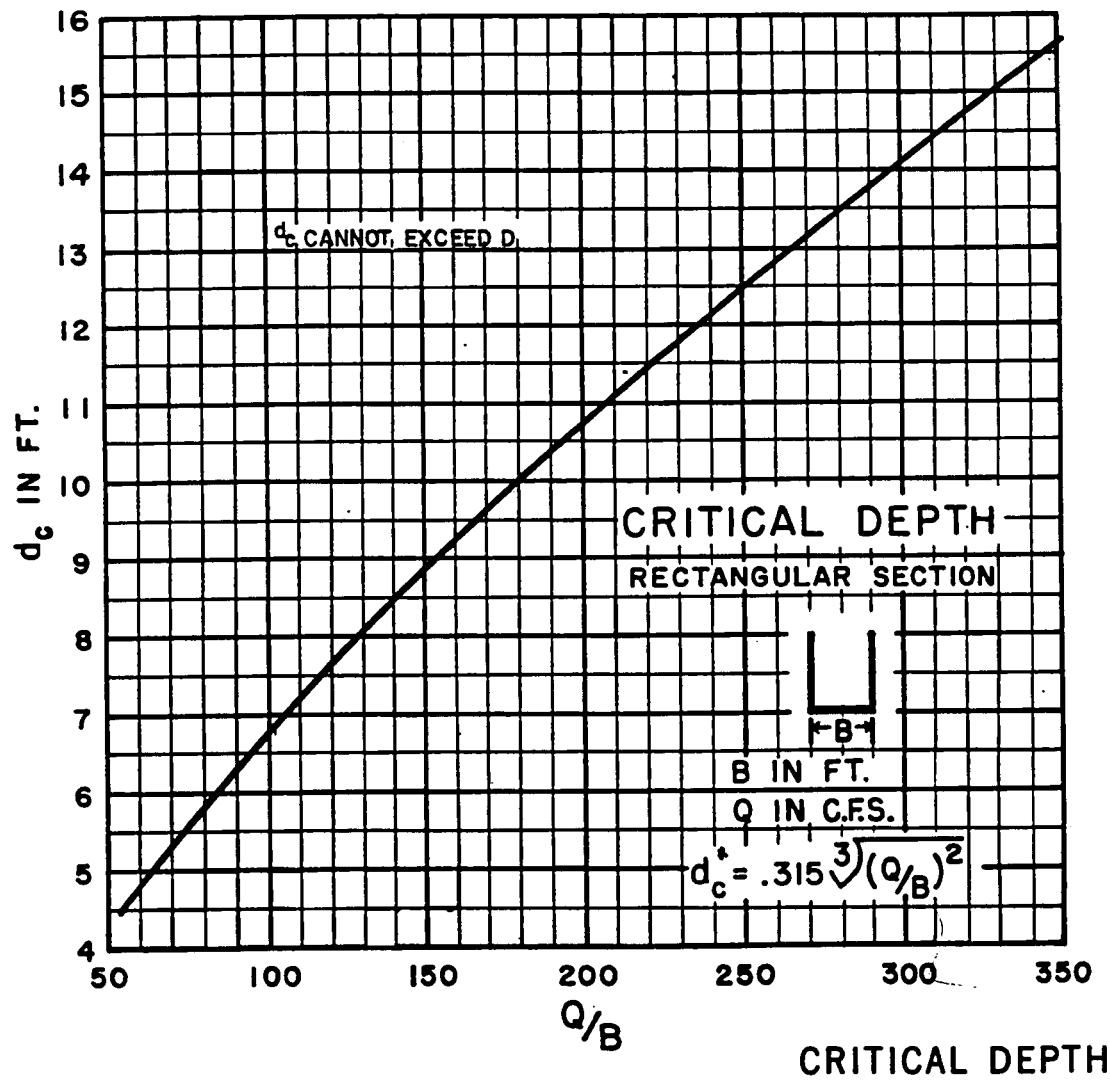


HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

CHART 14

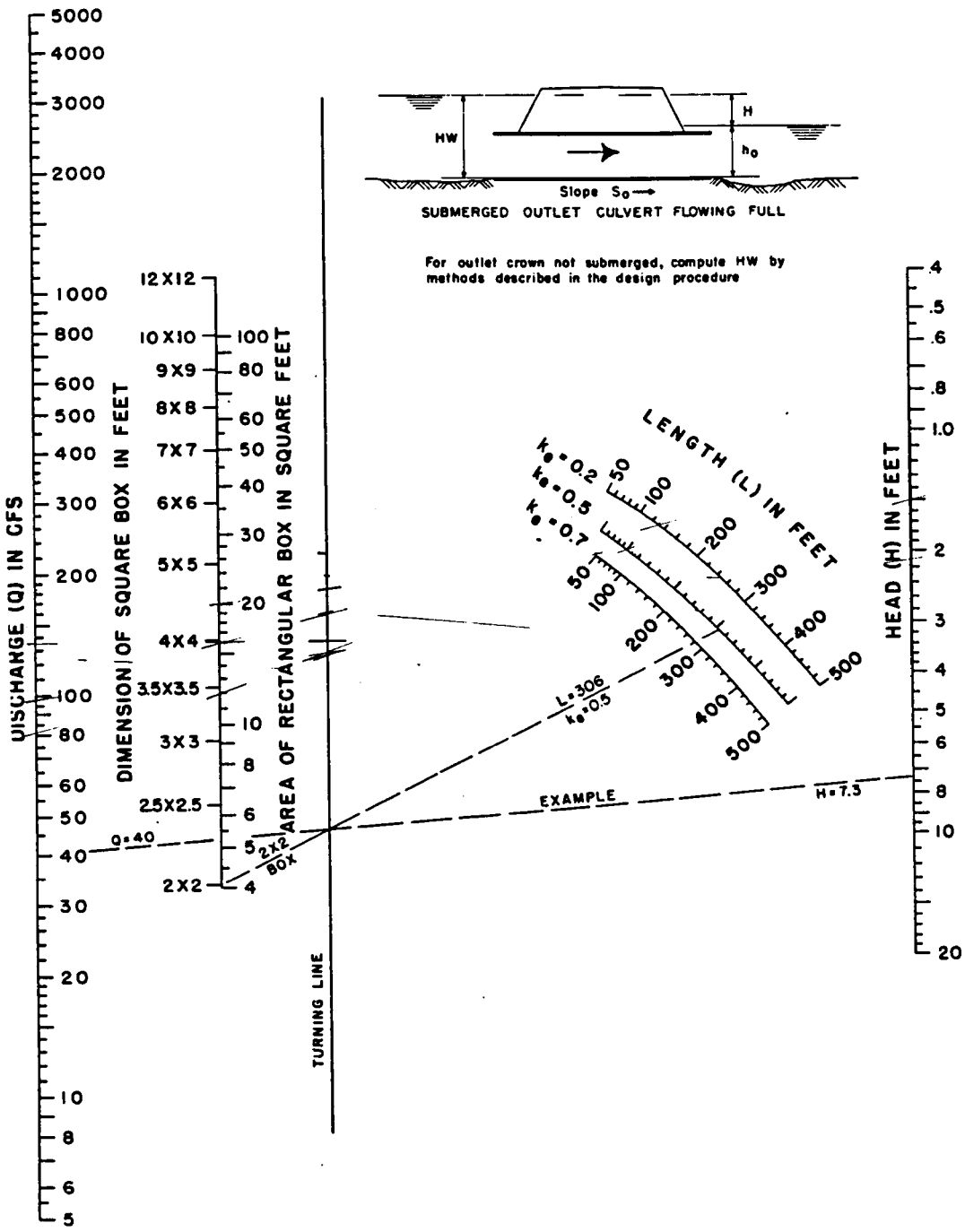




BUREAU OF PUBLIC ROADS JAN. 1963

CRITICAL DEPTH
RECTANGULAR SECTION

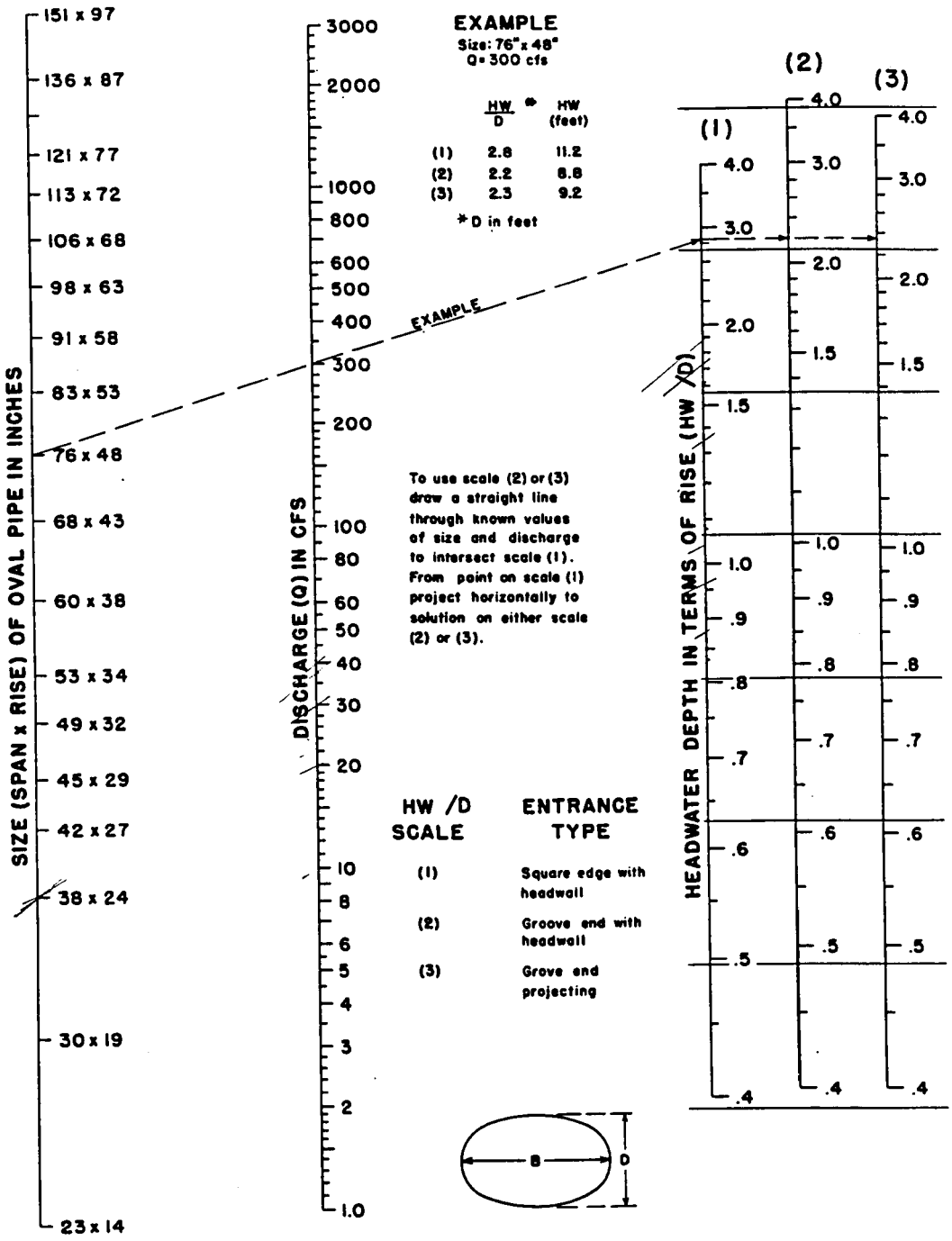
CHART 15



HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
n = 0.012

AU OF PUBLIC ROADS JAN. 1963

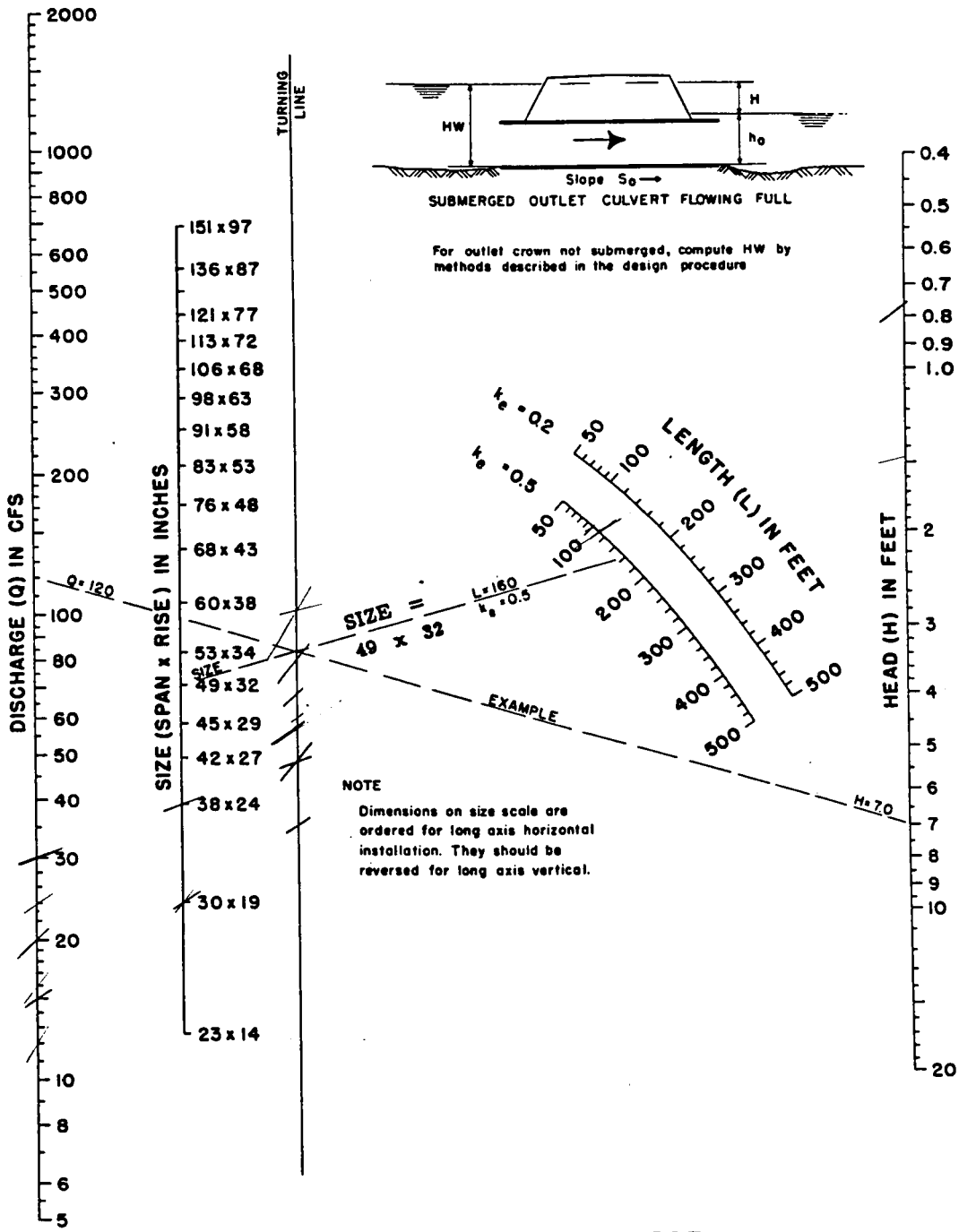
CHART 29



HEADWATER DEPTH FOR OVAL CONCRETE PIPE CULVERTS LONG AXIS HORIZONTAL WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

CHART 33

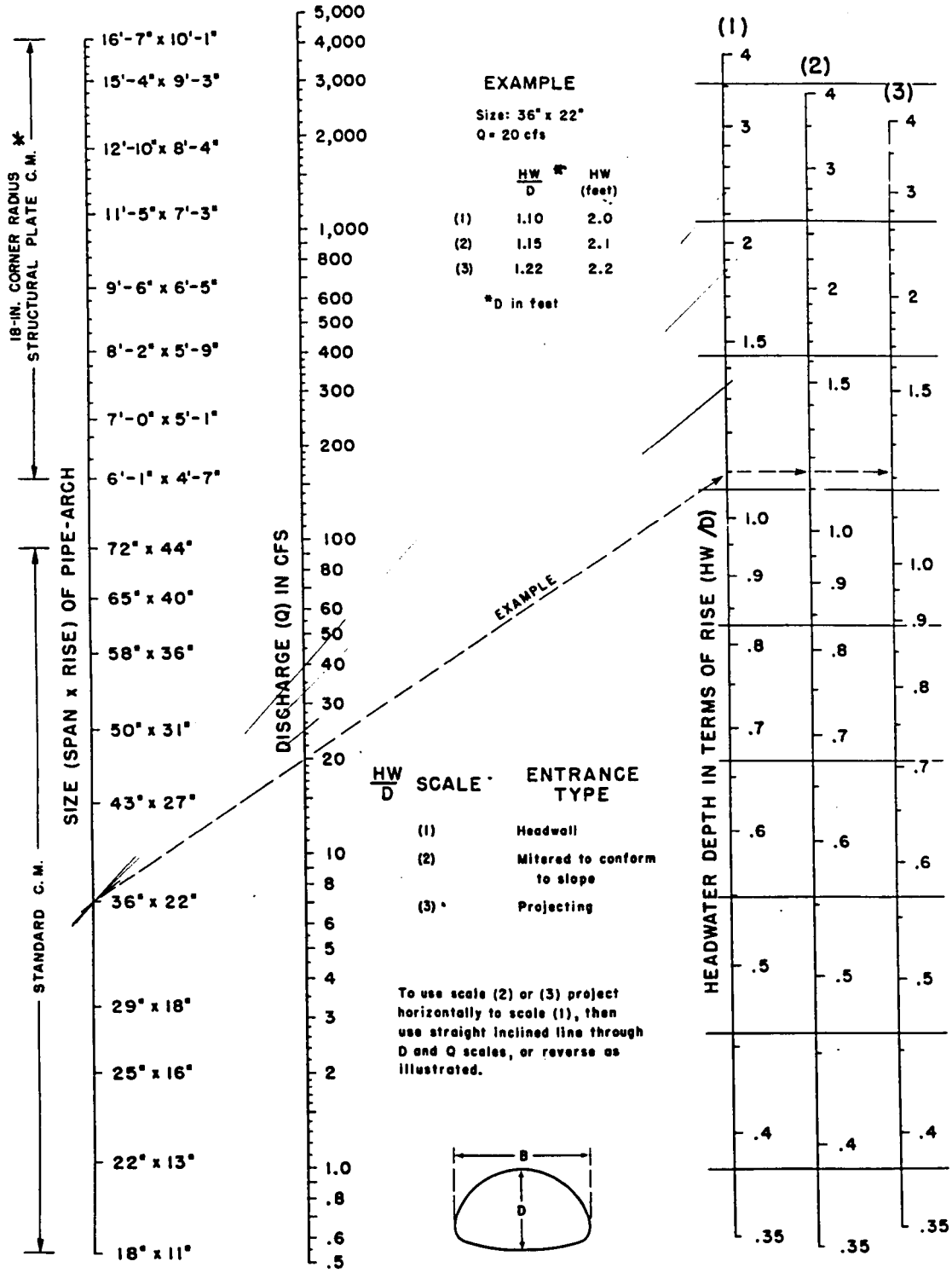


HEAD FOR OVAL CONCRETE PIPE CULVERTS LONG AXIS HORIZONTAL OR VERTICAL FLOWING FULL

n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

CHART 34

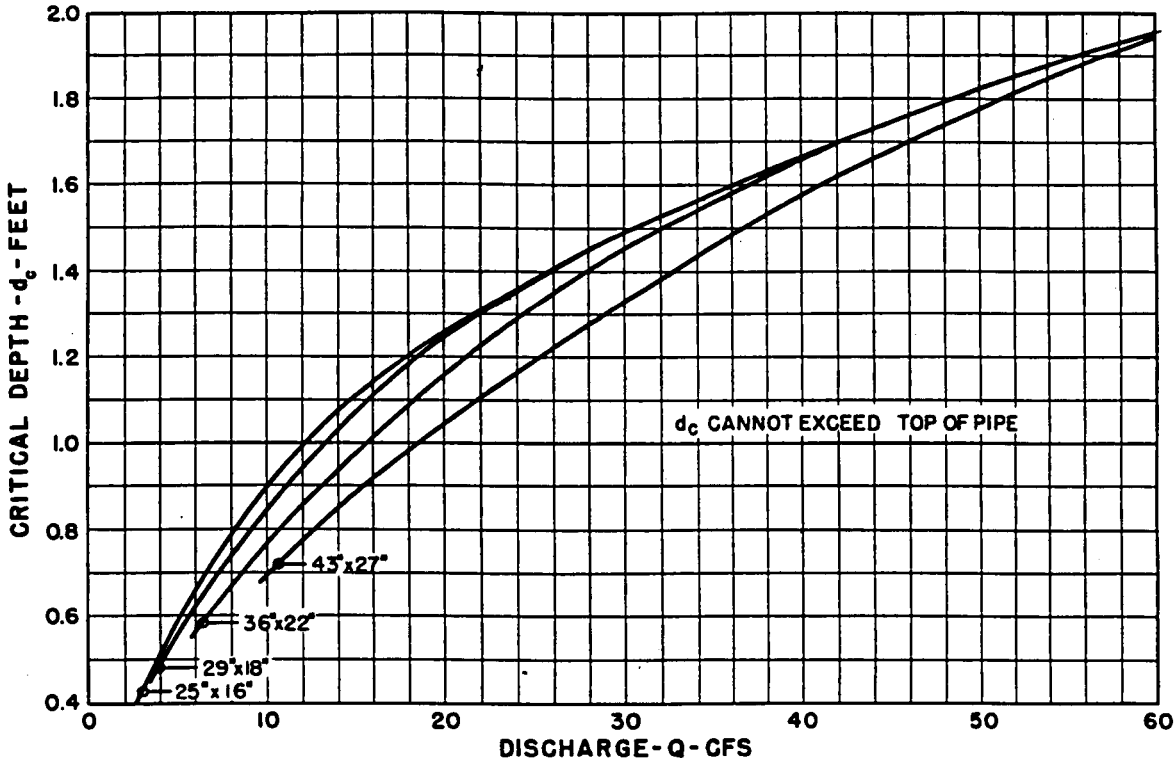


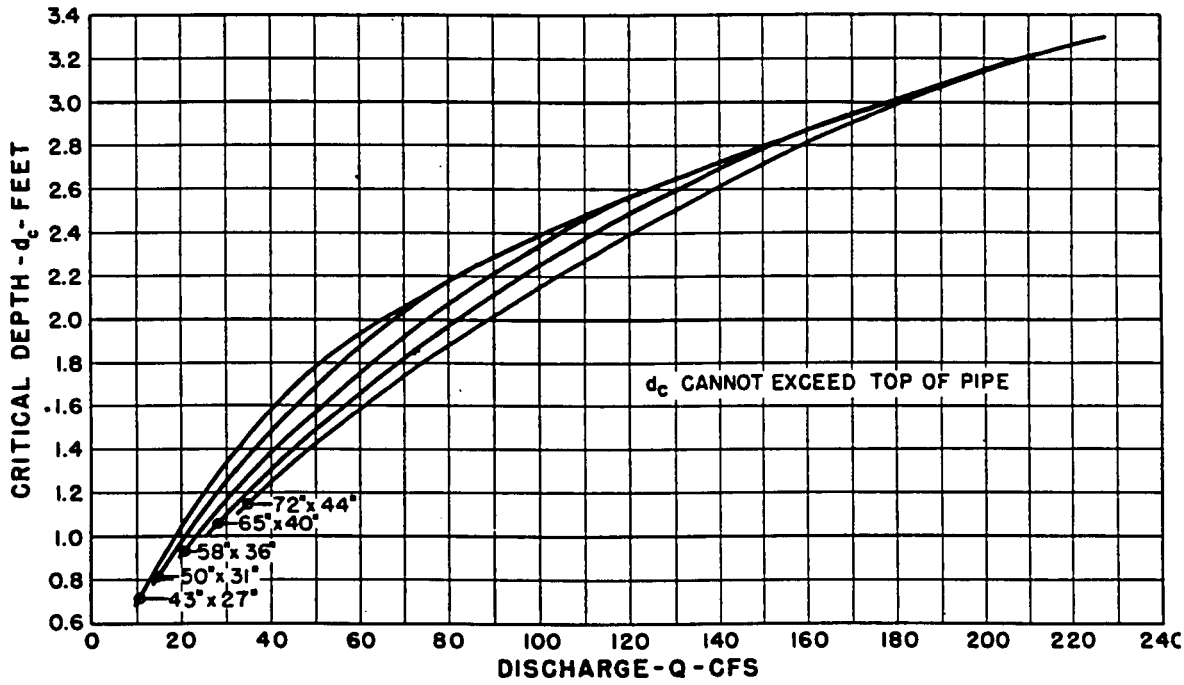
*ADDITIONAL SIZES NOT DIMENSIONED ARE LISTED IN FABRICATOR'S CATALOG

BUREAU OF PUBLIC ROADS JAN. 1963

HEADWATER DEPTH FOR C. M. PIPE-ARCH CULVERTS WITH INLET CONTROL

CHART 37

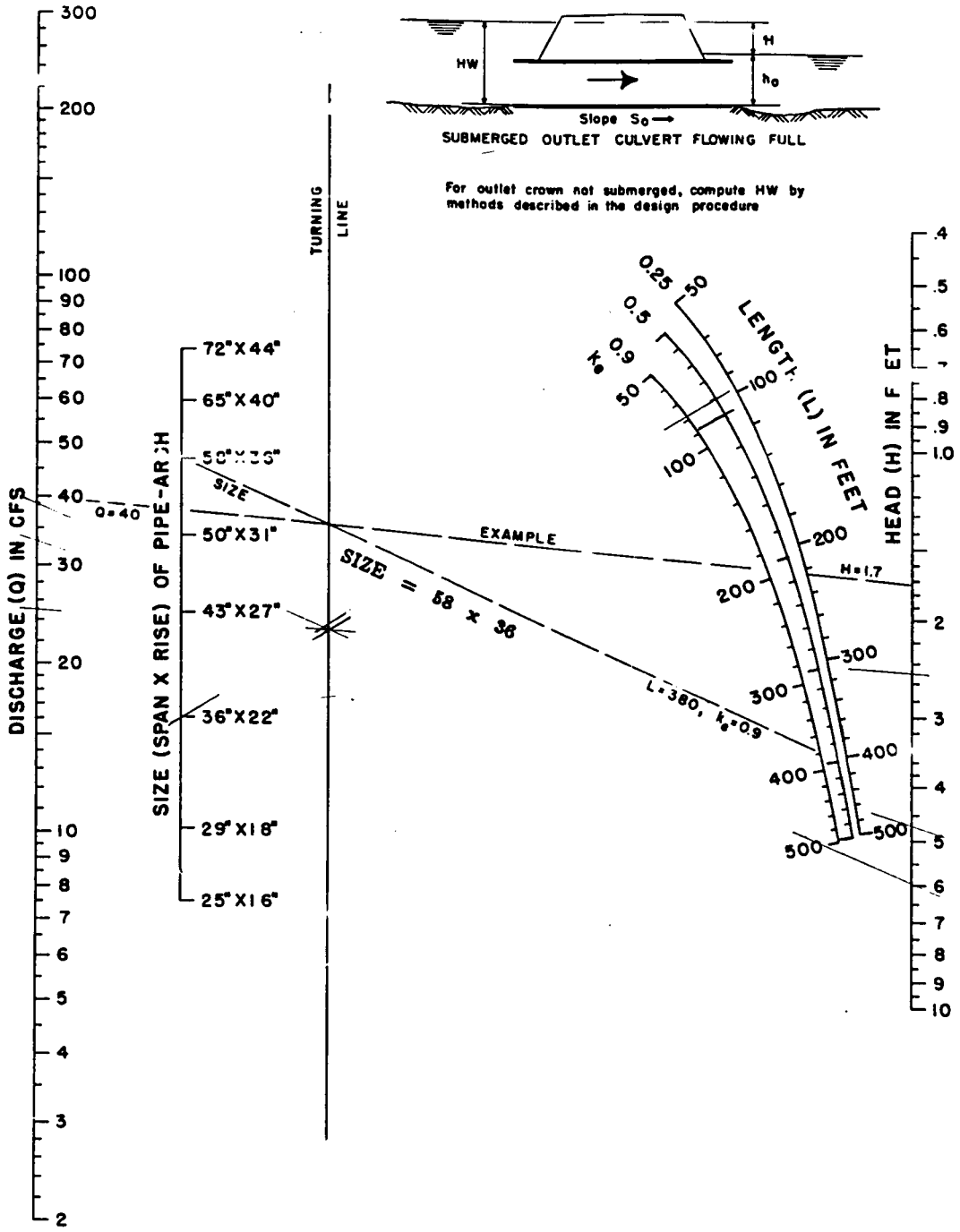




BUREAU OF PUBLIC ROADS
JAN. 1964

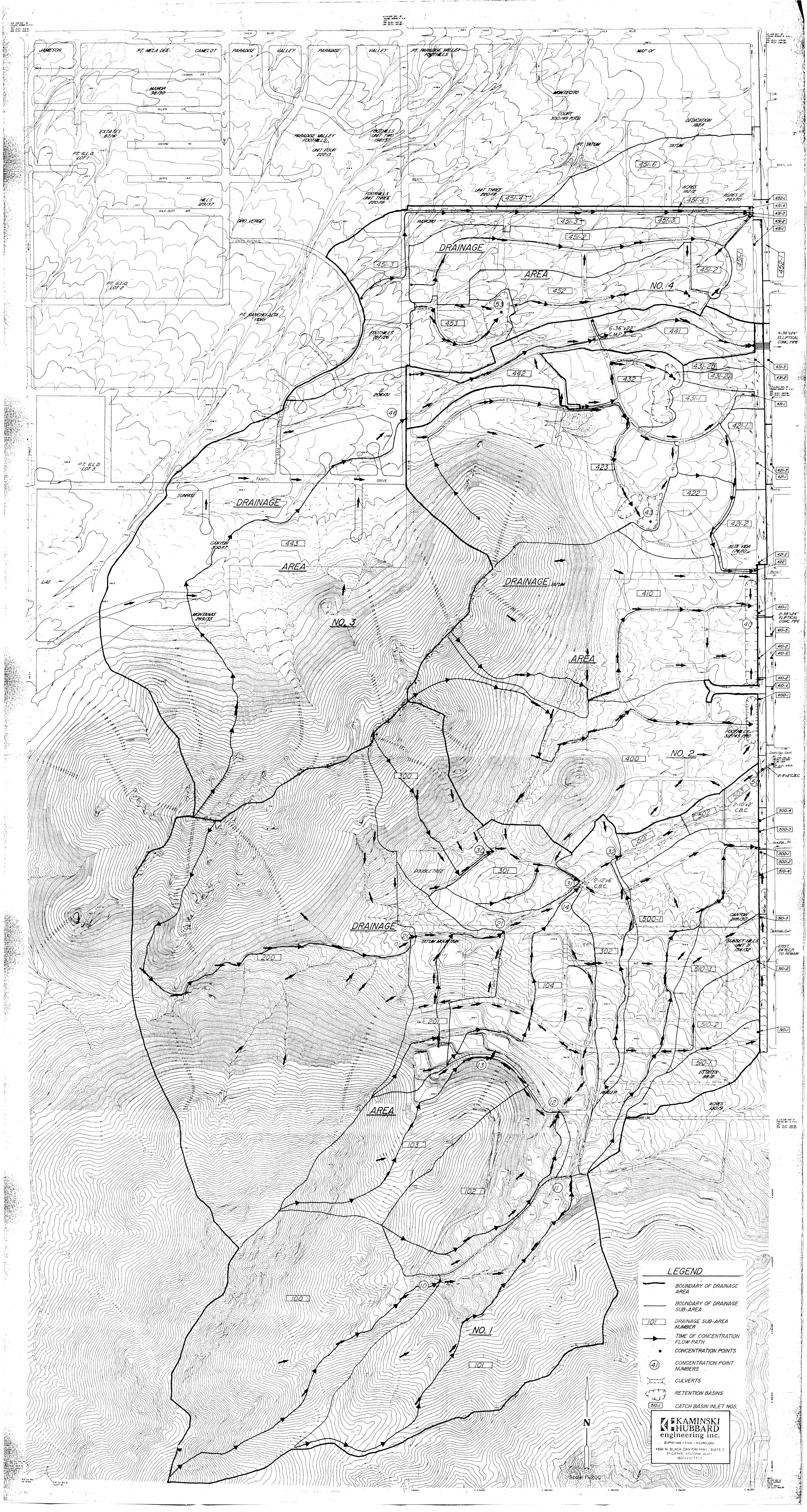
CRITICAL DEPTH
STANDARD C.M. PIPE-ARCH

CHART 39



HEAD FOR
STANDARD C. M. PIPE-ARCH CULVERTS
FLOWING FULL
n=0.024

BUREAU OF PUBLIC ROADS JAN. 1963


DRAINAGE AREA NO. 1
DRAINAGE AREA NO. 2
DRAINAGE AREA NO. 3
DRAINAGE AREA NO. 4
LEGEND
BOUNDARY OF DRAINAGE AREA
BOUNDARY OF DRAINAGE SUB-AREA
DRAINAGE SUB-AREA NUMBER
TIME OF CONCENTRATION FLOW PATH
CONCENTRATION POINTS
CONCENTRATION POINT NUMBERS
CULVERTS
RETENTION BASINS
CATCH BASIN INLET NOS.
KAMINSKI HUBBARD engineering inc.
SURVEYING • CIVIL • HYDROLOGY
4550 N. BLACK CANYON HWY., SUITE C
PHOENIX, ARIZONA 85017
(602) 242-7515
N
Scale: 1"=200'