PLANNING

Item 33.

SOUTHEAST LOOP HIGHWAY

(Santan Freeway)

Technical Memorandum No. 1

Drainage Concepts

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Southeast Loop Highway SR220
(Santan Freeway)
Project No. RAM 600-7-301

TECHNICAL MEMORANDUM NO. 1

DRAINAGE CONCEPTS
SOUTHEAST LOOP HIGHWAY
STATE ROUTE 220

July 19, 1988

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Arizona Department of Transportation

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1.0 INTRODUCTION

This document presents a summary of the preliminary engineering analysis completed to date for the drainage requirements along the proposed Price Expressway and Southeast Loop Highway. The purpose of the analysis is to assist with the conceptual design of highway-related drainage facilities. The analysis is not detailed, but was performed at a preliminary level to be compatible with the ongoing design concept studies.

The study area for the ongoing route selection studies does not include all areas tributary to the proposed road alignments. As a result, it was necessary, for the drainage analysis, to extend the study area to the natural or man-made watershed boundaries.

The eastern part of the drainage study area is also part of an ongoing Area Drainage Master Study (ADMS) being completed under the direction of the Flood Control District of Maricopa County. This area is east of the East Maricopa Floodway which parallels the RWCD canal on the upslope (east) side. The ADMS will identify methods and facilities to control storm runoff for this area. It is assumed that the proposed alignment for the Southeast Loop through this area can be protected from cross drainage problems by the facilities proposed in the ADMS. Detailed analysis for drainage in this area has not yet been completed.

The conceptual design of the highway drainage for the area shown on Plate 1 consists of a number of elements.

- Storm runoff is collected either in stormwater storage basins (labeled A through Q on Plate 1), or by drainage channels (labeled 1 through 21 on Plate 2) which drain to the basins.
- Each drainage channel empties into the next downslope storage basin.
- Each storage basin (except those with pumps) has a low-level gravity outlet which conveys water to the next downslope basin. Wherever the slope of the land permits, overflow from each basin will enter the respective channel and flow to the next downslope basin. If the land slope does not allow this, the low level outlet can be oversized to handle possible problem flows. Some

of the low-level gravity outlets may be determined to be unnecessary during subsequent design at any basin where a pump station will be located. The pump station could discharge to an outfall pipe or to an open channel that will convey the flow to another storage basin.

The above elements are common to all the conceptual designs investigated. A number of alternatives were investigated for the drainage of the storage basin at the lowest elevation along Price (Basin H) and along the Southeast Loop (Basin B).

The preliminary analysis completed to date can be divided into three major tasks. The first task was the estimation of required sizes and costs of the stormwater storage facilities along the proposed highways. This task required the estimation of runoff volumes from watersheds tributary to the highways. The second task concerned the estimation of required sizes and costs of drainage channels parallel to the proposed highways' rights-of-way (ROW). This task required the estimation of flowrates along highway ROW, and estimation of flowrates from watersheds immediately adjacent to the ROW. The third task was the evaluation of various alternatives to dispose of the runoff collected by the highway drainage facilities. To perform this task, the inflow hydrographs for the conceptual drainage facilities were developed.

It is noted that all analyses presented here are preliminary, and that more detailed engineering analyses may significantly alter the number and sizes of proposed facilities. In addition, the runoff storage and evacuation facilities should be reevaluated based on more detailed hydrologic studies and an acceptable evacuation time. This reevaluation should optimize the system to maximize flexibility and minimize construction and operation costs. These detailed analyses must be completed before final design of any related facilities.

Included with this document as Appendix A is correspondence relevant to the drainage studies. The following items are presented in Appendix A.

1. Gila River Indian Community (GRIC) review comments dated September 16, 1986, on the drainage portion of the preliminary Price Road Expressway Reconnaissance Report.

2. Dames & Moore's written response dated October 31, 1986, to the GRIC review comments.

3. Howard Needles Tammmen & Bergendoff (HNTB) review comments dated November 4, 1986, on the preliminary HEC-1 analyses completed for this study.

4. Dames & Moore's written response dated December 1, 1986, to the HNTB comments.

2.0 STUDY METHODOLOGY

2.1 GENERAL

As stated in the introduction, the preliminary engineering analysis presented here involved three major tasks: the estimation of sizes for storage basin facilities, the estimation of channel sizes along the highway ROW, and the evaluation of runoff disposal alternatives. The methodologies used to complete these three tasks, and their supporting assumptions, are discussed in the following sections.

2.2 STORAGE BASIN FACILITIES

In general, the sizes of basins were estimated as follows:

- Based upon a physical reconnaissance of the proposed highway routes (and recent aerial photographs), detention basins were sited to store runoff from both the proposed highways' ROW and from the watersheds intersected by the highways.
- The watersheds tributary to these basins were delineated based upon topographic maps.
- The volume of runoff from each of these tributary watersheds was estimated for 6-hour storms of varying frequency.
- The detention basins were sized to contain the storm runoff.
- The low-level outlets allowing the basins to gravity drain were sized to allow each basin (when initially full) to drain within approximately 60 hours after the start of the storm.

2.2.1 Estimating Runoff From Proposed Right-of-Way

The runoff volumes for the ROW tributary to each detention basin were estimated as follows:

- Standard cross-sections for both Price Expressway and for the Southeast Loop Highway were taken from page 12 and pages 2-4 of Southeast Loop Highway, SR220, and Price Road Highway, Tentative Right of Way Requirements, TAMS Consultants, Inc., June, 1986. The ROW for the cross-section given for Price Road is 400 feet wide, while the three cross-sections provided for the Southeast Loop Freeway have ROW widths varying between 250 and 365 feet.

- The lengths of highway ROW tributary to each proposed detention basin site were classified as one of the standard cross-sections, according to the plan published in the above cited document.

- It was assumed that for major storms, the entire highway ROW became essentially impervious so that all rainfall became runoff.

- The rainfall depths for 6-hour storms of 10-, 50-, and 100-year frequency were multiplied times the areas of standard ROW cross-sections tributary to each detention basin. The computed volumes were estimates of the volume of ROW runoff that required storage space in each basin.

2.2.2 Estimating Volume of Cross Drainage Runoff

The runoff volumes from watersheds intercepted by the proposed freeway ROW were estimated as follows:

- The watersheds tributary to each detention basin site were delineated based upon topographic maps and professional judgment. These watersheds are shown on Plate 1. This delineation involved a number of major assumptions, which include:

  - The Western Canal/Lateral 9.5 is assumed to prevent all flows from the north from entering the Price Road/Southeast Loop watersheds.

  - The watersheds lie within a large grid system formed by cross streets at half mile and one mile intervals. Since the grid is not oriented exactly with the topographic slope, water can flow downhill in more than one direction from any given road intersection. To simplify the problem, an assumption was made that water was collected along the streets running in the direction of the steepest topographic slope, and was not transferred to other streets. Therefore, it was assumed that the watersheds tributary to the basins sited along Price Road (Basins E through H on Plate 1) drain in an east to west direction, along major east-west streets, and that no water flows in a north-south direction except along one mile or less of north-south road between the major east-west roads. In similar fashion, it was assumed that in the major watersheds tributary to the Southeast Loop Freeway (Basins A through D on Plate 1), the preferred direction of drainage is along north-south major streets.

- Basins I through N were assumed to collect runoff from a half mile to one mile strip north of the Southeast Loop alignment.

- Basins O through Q were assumed to collect runoff between the Southeast Loop alignment and the Roosevelt Water Conservation District (RWCD) Canal.

- The effect of the major canals east of Price Road on drainage paths in an east-west direction has not been investigated in detail. For purposes of this study, these canals (the Consolidated and Eastern Canals) are assumed to be overtopped in major storms, and not to divert or block runoff from intercepted watersheds. Mr. Jim Geiser of Dames & Moore completed a half day tour of the watershed accompanied by Mr. Ray Jordan and Mr. Steve Martin of ADOT. The purpose of this site visit was to become familiar with site conditions and to inspect the canal and railroad alignments which cross the watershed. Each of these facilities was inspected at several locations during this site visit and at other locations on subsequent visits by Mr. Geiser. As a result of these site visits, it was decided jointly by ADOT and Dames & Moore that although the canals and railroad crossings could have an impact on surface water flows, the detailed surveying necessary to assess these impacts was not within the scope of the ongoing route selection study. Because of the uncertainty concerning the validity of the assumption that the major canals and railroad embankments do not divert or block runoff, basins along Price Road were sized for three different conditions. These are: the Consolidated Canal intercepts (and diverts from Price Road) all upslope flows; the Eastern Canal intercepts all upslope flows; and the East Maricopa Floodway (just upslope of the RWCD Canal) intercepts all upslope flows.

- Areas within each watershed were classified, using May 23, 1986 aerial photographs, as either agricultural (undeveloped), or urban (developed). These classified areas are shown on Plate 1, with the shaded areas representing urban areas.

- The urban areas were assumed to have their own stormwater retention facilities capable of retaining all or most of the runoff generated within their boundaries according to municipal policy.

The City of Chandler has a well-enforced policy that developed areas must include provisions to retain on-site runoff from the 100-year, 6-hour storm. Under existing conditions, only undeveloped areas of Chandler will produce runoff to the Price Road or Southeast Loop alignment. This study assumes that Chandler facilities will retain on site 100 percent of the 100-year, 6-hour storm runoff.

The Town of Gilbert is just starting an Area Drainage Master Study (ADMS) to control runoff within town boundaries. It was assumed that the

existing developed areas of Gilbert will soon add storm runoff retention facilities. This study assumes that these facilities will retain on site 75 percent of the 100-year, 6-hour storm runoff.

The approximate boundary between Chandler and Gilbert is shown on Plate 1.

- A U.S. Soil Conservation Service curve number (CN) of 80 was chosen as representative of the hydrologic characteristics of the typical agricultural watershed of the area.

- Two CNs were estimated for the typical Gilbert urban watershed. A CN of 75 was chosen to represent the pervious portion of the urban watershed (a combination of lawns and desert landscape). A CN of 95 was chosen to represent the impervious portion of the urban watershed (roofs, driveways, roads, etc.). Thirty-five percent of the watershed was assumed to fit the impervious category, and sixty-five percent fit the pervious category.

- The U.S. Army Corps of Engineers flood hydrograph program (HEC-1) was used to estimate the depth of runoff in inches from the typical urban and the typical agricultural watersheds. These depths were estimated for the 10-year 6-hour, 50-year 6-hour, and the 100-year 6-hour storms.

- The volume of runoff from each watershed was computed by completing the following three steps:

  1. Multiplying the area of agricultural land within each watershed by the depth of agricultural runoff computed by HEC-1.

  2. Multiplying the area of urban land within Gilbert within each watershed times the depth of urban runoff computed by HEC-1. This number was reduced by 75 percent to account for town stormwater retention facilities.

  3. Summing the agricultural and urban runoff.

2.2.3 Estimates for Detention Basin Sizes

The required sizes of the detention basins were estimated as follows:

- The ROW runoff and cross drainage runoff for each basin were summed. This constituted the design storage volume for each basin. Inherent in this summation is an assumption that ROW

runoff from depressed sections of the highway alignment would be pumped to the detention basins.

- Each basin was initially assumed to be square in plan, with a storage depth of 15 feet and a total depth of 19 feet. The interior side slopes of each basin were 4 horizontal to 1 vertical. Two of the basins (B and H) are proposed with depths 10 feet deeper than the others because it is proposed that the shallower basins would drain to these deeper basins.

The required storage volume, the length of a side at the ground surface, and the surface acreage for each basin are presented in Table 1. Estimates are presented for each of two cases: the 50-year, 6-hour storm, and the 100-year, 6-hour storm.

2.2.4 Estimates for Gravity Drain Sizes

All of the basins (except E, F and G) were assumed to drain to Basin B or H via gravity pipe drains between the basins. Basins E, F and G would be pumped to the north or south, depending on direction of discharge. Basins B and H were sited near the lowest elevations, and were assumed to be drained only by pumping.

The hydraulic conditions for the operation of these drains is complex. The water elevations in upstream and downstream basins are time-varying. As this is a preliminary study only, pipes were designed using the simplest possible assumptions. These include:

- Use of the Manning equation to size the pipe. This assumes that the pipe is not under pressure and therefore the design sizes given are conservative.
- Use the surface ground slope from topographic maps as the slope in the Manning equation.
- Assume circular reinforced concrete pipe, flowing full.

Pipes were sized to drain the design volume of a basin within approximately 60 hours after the start of the storm. The minimum pipe size selected was 3 feet; the maximum pipe size selected was 4 feet. Sizes required and approximate times to drain each basin (for the given pipe size) are given in Tables 1 and 2.

Table 1

BASIN DESIGN DATA
(50-YEAR, 6-HOUR STORM)

		Runoff Volume (ac-ft)			Basin Dimensions		Gravity Drain		Peak Flowrate into Basin (cfs)
		Right-of-Way	Cross Drainage	Design (Total)	Length of Side (feet)	Surface Area (acres)	Required Diameter (feet)	Time to Drain Basin (hours)	
	A	10	110	120	680	10.7	4.0	89	790
	B	7	289	296	1020	23.8	--	--	1940
	C	6	151	157	770	13.5	4.0	64	630
	D	9	123	132	710	11.6	3.5	51	350
Watershed Consolidated Canal to Price Road	E	16	207	223	900	18.5	--	--	750
	F	11	129	140	730	12.2	--	--	700
	G	11	110	121	680	10.7	--	--	560
	H	20	52	72	550	6.9	--	--	600
Watershed Eastern Canal to Price Road	E	16	404	420	1200	32.9	--	--	860
	F	11	269	280	990	22.6	--	--	1140
	G	11	224	235	920	19.3	--	--	920
	H	20	82	102	640	9.3	--	--	870
Watershed RWCD Canal to Price Road	E	16	700	716	1530	54.0	--	--	1390
	F	11	390	401	1170	31.4	--	--	1410
	G	11	224	235	920	19.3	--	--	950
	H	20	82	102	640	9.3	--	--	870
	I	9	28	37	420	4.0	3.0	24	--
	J	9	14	23	350	2.8	3.0	12	--
	K	10	34	44	450	4.6	3.0	25	--
	L	9	14	23	350	2.8	3.0	26	--
	M	9	55	64	520	6.2	3.0	23	--
	N	10	14	24	350	2.9	3.0	23	--
	O	13	137	150	750	13.0	3.5	45	--
	P	13	172	185	830	15.8	4.0	68	--
	Q	29	247	276	990	22.3	4.0	100	--

Table 2

BASIN DESIGN DATA
(100-YEAR, 6-HOUR STORM)

		Runoff Volume (ac-ft)			Basin Dimensions			Gravity Drain		Peak Flowrate into Basin (cfs)
		Right-of-Way	Cross Drainage	Design (Total)	Length of Side (feet)	Surface Area (acres)	Required Diameter (feet)	Time to Drain Basin (hours)		
	A	12	141	153	760	13.1	4.0	114		960
	B	8	370	378	1140	29.8	--	--		2270
	C	7	194	201	850	16.8	--	--		810
	D	10	158	168	790	14.3	--	--		470
Watershed Consolidated Canal to Price Road	E	19	289	308	1040	24.7	--	--		960
	F	12	177	189	830	15.9	--	--		840
	G	12	141	153	760	13.2	--	--		720
	H	23	67	90	600	8.3	--	--		780
Watershed Eastern Canal to Price Road	E	19	568	587	1400	44.8	--	--		1110
	F	12	363	375	1140	29.6	--	--		1420
	G	12	303	315	1050	25.2	--	--		1180
	H	23	105	128	700	11.3	--	--		1170
Watershed RWCD Canal to Price Road	E	19	955	974	1770	72.2	--	--		1850
	F	12	518	530	1330	40.8	--	--		1570
	G	12	303	315	1050	25.2	--	--		1200
	H	12	105	128	700	11.3	--	--		1170
	I	10	36	46	450	4.7	3.0	29		280
	J	10	18	28	370	3.2	3.0	15		170
	K	12	44	56	490	5.6	3.0	30		340
	L	10	18	28	370	3.2	3.0	32		350
	M	10	71	81	580	7.6	3.0	28		420
	N	12	17	29	380	3.3	3.0	27		330
	O	15	174	191	840	16.0	3.5	57		--
	P	15	235	250	940	20.4	4.0	68		--
	Q	33	317	350	1100	27.7	4.0	99		--

2.3 CHANNEL SIZES ALONG HIGHWAY ALIGNMENTS

Both the Southeast Loop and Price Road alignments intercept cross drainage from upslope watersheds. Therefore, channels or large culverts are required along the upstream edges of the alignments to carry flows into the stormwater storage basins sited along the alignments. Some portion of this runoff will be intercepted as sheet flow. The remainder will be channeled down small depressions or roadways. Inlet facilities will have to be located and sized during subsequent design studies to direct flows into the collector channels or closed conveyances, whichever are utilized.

The channels which were sized are shown on Plate 2. The sizes of these channels were estimated as follows:

- The watersheds tributary to each of the designated channels were delineated as shown on Plate 2. It was assumed that only the area immediately adjacent to the ROW alignment would provide flow to these channels, as the major portion of the flow would travel along the major roads and enter the detention basins directly.

- Peak flowrates from the 100-year, 6-hour storm for the watersheds were computed using HEC-1. The basic methodology and assumptions used in this computation are explained in Section 2.4.2.

- The channels were assumed to also transport runoff from half of the alignment ROW. These flows were estimated using HEC-1 and the kinematic wave method. Example runoff volumes and flowrates for the standard highway cross-sections for varying frequency and duration storms are given in Table 3.

- The Manning equation was used to estimate the channel width required to carry the design flowrate. Channels were assumed to have a 4-foot maximum depth of flow, to be lined with shotcrete, and to have 2 horizontal to 1 vertical side slopes. Bed slopes were taken from map topography.

The design flowrates, the estimated required top widths of the channels, and the lengths of the channels are given in Table 4.

Table 3

RIGHT-OF-WAY DESIGN DATA

RUNOFF VOLUMES

		10-year storm			50-year storm			100-year storm		
		6 hr.	12 hr.	24 hr.	6 hr.	12 hr.	24 hr.	6 hr.	12 hr.	24 hr.
	Precipitation (inches)	1.90	2.12	2.30	2.70	3.06	3.40	3.10	3.40	3.80
Runoff Volume (ac-ft)	Price Road	7.7	8.6	9.3	10.9	12.4	13.7	12.5	13.7	15.4
	SE Loop I	5.2	5.8	6.3	7.4	8.4	9.3	8.5	9.3	10.4
	SE Loop II	4.8	5.4	5.8	6.8	7.8	8.6	7.8	8.6	9.6
	SE Loop III	6.1	6.8	7.4	9.0	9.8	10.9	10.3	11.3	12.7

PEAK FLOWRATES (cfs) FROM
ONE MILE LENGTH OF ONE HALF OF ROW

	10-year storm	50-year storm	100-year storm
Price Road	18	28	34
SE Loop I	18	28	33
SE Loop II	23	35	40
SE Loop III	10	14	17

Notes: SE Loop I = 8 lane, at grade freeway
SE Loop II = 8 lane, depressed freeway with retaining walls and frontage roads
SE Loop III = 8 lane, depressed freeway without retaining walls or frontage roads

Table 4

CHANNEL DESIGN DATA

Channel	100-Year, 6-Hour Storm Design Flowrate (cfs)	Base Width (feet)	Top Width (feet)	Length (miles)
1	350	6	22	0.5
2	250	4	20	1.0
3	240	4	20	1.0
4	280	4	20	1.0
5	250	4	20	1.0
6	190	5	21	1.5
7	280	10	26	1.0
8	420	17	33	1.0
9	370	14	30	1.0
10	280	4	20	1.0
11	280	4	20	1.0
12	170	4	20	1.0
13	340	14	30	1.12
14	350	4	20	1.0
15	420	4	20	1.0
16	330	4	20	1.12
17	350	7	23	1.41
18	310	4	20	1.41
19	640	9	25	1.0
20	430	4	20	1.0
21	200	4	20	1.0

Channels have 4-foot depth and 2:1 side slopes.

2.4 ALTERNATIVES EVALUATION

2.4.1 General

Various alternatives were evaluated at a conceptual level to dispose of stormwater runoff. Based upon preliminary discussions with ADOT, several alternatives were investigated at a feasibility level and preliminary cost estimates were prepared.

I. Retention for all basins with evaporation or ground-water recharge

II. Detention

A. Pumped discharge to south across GRIC

1. Basin H to Gila River along Price

B. Pumped discharge to Price Road Drain & Salt River

1. Basins E, F, G and H to tunnel @ AZ 360

2. Basins E, F, G and H to tunnel @ Western Canal

3. Basin H to tunnel @ AZ 360 with Price Basins E, F and G cascading to Basin H

4. Basin H to tunnel @ Western Canal with Price Basins E, F and G cascading to Basin H

5. Extend tunnel south of Western Canal for system optimization

III. Pass flows over road ROW as sheet flow

Retention of runoff on site could involve large basins. Long-term storage of runoff during evaporation or ground-water recharge processes could provide potential for mosquito proliferation or human drowning. Ground-water recharge technology is not yet perfected and perched ground-water conditions may be present to preclude recharge.

Passing of flows across the road alignment could be difficult to design, especially for elevated or depressed roadway sections. Some type of collection system with a downstream equalization conveyance would be

required to mitigate possible concentrations of flow and to release flows in a manner similar to the way flows approach the roadway.

Detention of stormwater runoff will require a system to discharge the water in a manner which will not cause additional flooding problems. The runoff could be conveyed to the Salt River, Gila River or the Southwest Drainage which is the low area through which the Gila Drain is aligned. All of these are located at significant distances from where the runoff will be detained. Only the Southwest Drainage is located significantly downslope from the majority of the proposed roadway. Discharges to this conveyance may cause flooding for the GRIC.

There are a variety of methods to evacuate the three Price detention basins (E, F and G). Only a few have been selected for considerations in this study. Subsequent design based on more detailed hydrologic studies should consider a full range of methods including:

- Pumping from each basin to a standpipe connected to a pressurized outfall pipe that discharges to the north.
- Pumping from each basin to the open channel conveyance structure between the basins where gravity flow will convey the flow to the adjacent basin. This system could convey flows to the north or to the south depending on the slope of the conveyances between the basins. Flows from basin H could also use this concept.
- Gravity flow through a deep pipe to the adjacent downslope basin. This system could also convey flows to the north or to the south depending on the basin invert elevations. Flows from basin H could also use this concept.

Based upon the above comments and a meeting with ADOT staff, Dames & Moore was directed to estimate costs as follows:

- Estimate construction costs for facilities to pump from four Price basins south to Gila River.
- Estimate construction costs for facilities to pump from four Price basins north to Salt River.

- Compare costs for joint use or separate facilities to convey runoff from Carriage Lane Park Detention Basin north to AZ 360 or directly to the Salt River.

These cost estimates are described in a subsequent section.

2.4.2 Inflow Hydrographs

Inflow hydrographs for the basins were developed to evaluate peak flowrates from the tributary areas. These inflow hydrographs can also be used for storm runoff routing through the basins when the runoff disposal alternative has been selected. These hydrographs were derived as follows:

- The watersheds for the hydrographs were those shown on Plate 1.

- As shown on Plate 1, each section, half section, or quarter section was designated as either an agricultural watershed or an urban watershed. These one square mile to quarter square mile areas were used as the hydrologic elements of a HEC-1 model of each watershed.

- For reasons presented in Section 2.2.2, urban watersheds in the Chandler area were assumed to not contribute flow to the hydrographs. In the Gilbert area, flood flows were routed through the urban areas using kinematic wave routing through flat-sided (50:1) triangular channels to simulate routing along streets. These Gilbert urban areas were assumed to drain into stormwater storage basins sized to contain 75 percent of the runoff from the 100-year, 6-hour storm. The runoff was routed through these basins using Modified Plus routing.

- The flood hydrographs from agricultural areas were estimated from the SCS unit hydrograph, using lag times calculated using an empirical equation from the U.S. Bureau of Reclamation publication Design of Small Dams. The lag time calculation was based upon watershed slope and maximum flow length. For the watersheds of Basins E through H, the average watershed slopes east and west of the Consolidated Canal were used for the respective areas in model elements. In these watersheds the slope is generally constant in both magnitude and direction. For the watersheds of other basins, the slopes were individually calculated for each watershed element. A CN of 80 was used.

- The watershed elements were hydrologically linked in the setup of a HEC-1 run for each watershed. Routing of hydrographs along the streets was accomplished by lagging upstream hydrographs a time period calculated using the average channel velocity in a 50:1 side slope triangular asphalt channel. The velocity was

estimated using the Manning equation, the assumed channel shape, the channel topographic slope, and the flowrate from a preliminary HEC-1 run.

The estimated peak flowrates from the 50-year, 6-hour and 100-year, 6-hour storm hydrographs are given in Table 1. The complete hydrographs are available in HEC-1 printouts.

2.4.3 Preferred Alternative

A preferred alternative has been selected for the disposal of the detained runoff. This preferred alternative was selected based only on the analysis completed to date and may not be the preferred alternative based upon more detailed hydrology and cost estimates. The primary selection criteria were construction cost and system flexibility. The preferred alternative is to pump from all four Price basins to standpipes connected to a single conveyance (shared with the City of Chandler) which outfalls to the Price Road Drain near the Carriage Lane Park Detention Basin. A conceptual design level profile of this system is presented in Figure 3. The cost analysis is presented in the following section.

3.0 COST ESTIMATES

3.1 GENERAL

Very preliminary cost estimates were completed to compare several of the alternatives and to estimate the total cost of the proposed drainage system. Recent unit costs for similar projects were used to estimate the project costs. Quantity take-offs were based on detention facilities to store the worst case 100-year storm runoff. Pumping rates were selected based on judgment for pipe sizes and basin evacuation times. Excavation costs assumed that excess soil could be used for roadway construction in the vicinity of the runoff storage basins.

No analysis or cost estimate has yet been completed for the portion of the Southeast Loop east of the East Maricopa Floodway. It is assumed that the Areas Drainage Master Study for this area will include facilities parallel to the freeway and these can be made joint use facilities to also protect the freeway.

The cost estimates presented below are separated into a basic cost common to all disposal alternatives; and costs for various disposal alternatives.

3.2 DRAINAGE COST ESTIMATE FOR FACILITIES BASIC TO ALL DISPOSAL ALTERNATIVES

The drainage facilities common to all disposal alternatives include:

- basins sized to store design storm runoff;
- overflow/collector channels between the basins; and
- low-level outlet pipes between basins without pumps.

The cost estimate includes land acquisition costs, excavation costs for basins and channels, shotcrete lining of channels, and reinforced concrete pipe installation. These costs are described in the following sections.

3.2.1 Basin Cost Estimates

The cost of the detention basins sized in Section 2.0 is the sum of a number of individual costs. These include:

- o land acquisition cost
- o excavation cost
- o cost of gravity drains

Land acquisition costs were estimated as follows. The surface area of each basin was computed and is shown in Table 1. The total acreage computed was multiplied by a cost per acre of $50,000 (Camp, Dresser and McKee, 1986), yielding a total land acquisition cost.

Excavation costs were estimated as follows. The volume of each basin was computed. These volumes are given in Tables 5 and 6. The volumes in this table differ from the design volumes given in Table 1, because the excavated volume includes the four feet additional excavation required to allow surface channels to discharge into the basins. The total volume was multiplied by a unit cost of $3.57 per cubic yard to obtain the total excavation cost. The $3.57 cost is an estimate obtained from published references (Mahoney, 1986) and assumes that the excavated earth can be used in the highway construction in the immediate vicinity. The cost of transporting the excavated material to a distant dump site is not included.

Gravity drain costs were estimated as follows. The required diameter of the gravity drain from each basin is given in Table 1. The estimated cost per foot of these pipes includes the cost of placing the pipe at a 20-foot depth. The cost for each gravity drain is the estimated cost per foot times its length. The estimated total cost is the sum of all the individual gravity drain costs.

The estimated total detention basin cost is the sum of all three of the above costs. Totals are shown in Table 5 for the 50-year, 6-hour storm design and Table 6 for the 100-year, 6-hour storm design.

Table 5

ESTIMATION OF
STORAGE BASIN COST
(50-YEAR, 6-HOUR DESIGN)
(all costs in millions of dollars)

	Basin	Basin Costs					
		Excavation Volume (cy)	Excavation Cost @ $3.57 per cy ($)	Surface Area (acres)	Land Acquisition @ $50,000 per acre ($)	Gravity Drain Cost ($)	Total Cost ($)
	A	259,400	0.93	10.7	0.53	0.61	2.07
	B	625,600	2.23	23.8	1.19		3.42
	C	337,400	1.20	13.5	0.68	0.61	2.49
	D	285,200	1.02	11.6	0.58	0.55	2.15
	I	82,600	0.29	4.0	0.20	0.51	1.00
	J	52,900	0.19	2.8	0.14	0.51	0.84
	K	99,600	0.36	4.6	0.23	0.51	1.10
	L	52,900	0.19	2.8	0.14	0.57	0.90
	M	141,100	0.50	6.2	0.31	0.51	1.32
	N	55,300	0.20	2.9	0.14	0.51	0.85
	O	322,200	1.15	13.0	0.65	0.62	2.42
	P	400,600	1.43	15.8	0.79	0.87	3.09
	Q	584,900	2.09	22.3	1.12	0.87	4.08
TOTALS			11.78		6.70	7.25	25.73
Watershed Consolidated Canal to Price Road	E	475,900	1.70	18.5	0.92		2.62
	F	301,600	1.08	12.2	0.61		1.69
	G	260,900	0.93	10.7	0.54		1.47
	H	158,300	0.57	6.9	0.35		0.31
TOTALS			4.28		2.42		6.70
Watershed Eastern Canal to Price Road	E	884,700	3.16	32.9	1.64		4.80
	F	592,600	2.12	22.6	1.13		3.25
	G	499,300	1.78	19.3	0.97		2.75
	H	222,000	0.79	9.3	0.46		1.25
TOTALS			7.85		4.20		12.05
Watershed RWCD Canal to Price Road	E	1,500,000	5.35	54.0	2.70		8.05
	F	843,500	3.01	31.4	1.57		4.58
	G	499,300	1.78	19.3	0.97		2.75
	H	222,000	0.79	9.3	0.46		1.25
TOTALS			10.93		5.7		16.63

Total cost, assuming Basins E,F,G,H are filled from watersheds west of Consolidated Canal, is 25.73 + 6.70 = 32.43 million dollars

Total cost, assuming Basins E,F,G,H are filled from watersheds west of Eastern Canal, is 25.73 + 12.05 = 37.78 million dollars

Total cost, assuming Basins E,F,G,H are filled from watersheds west of RWCD Canal, is 25.73 + 16.63 = 42.36 million dollars

Table 6

ESTIMATION OF
STORAGE BASIN COST
(100-YEAR, 6-HOUR DESIGN)
(all costs in millions of dollars)

	Basin	Basin Costs					
		Excavation Volume (cy)	Excavation Cost @ $3.57 per cy ($)	Surface Area (acres)	Land Acquisition @ $50,000 per acre ($)	Gravity Drain Cost ($)	Total Cost ($)
	A	327,300 104,300	1.17	13.1	0.66 .24	0.61	2.44
	B	796,400 227,700	2.84	29.8	1.49 .47		4.33
	C	427,800 151,000	1.53	16.8	0.84 .33	0.61	2.98
	D	360,600 53,000	1.29	14.3	0.72 .16	0.61	2.62
	I	101,900	0.36	4.7	0.24	0.51	1.10
	J	64,200	0.23	3.2	0.16	0.51	0.90
	K	123,400	0.44	5.6	0.28	0.51	1.23
	L	64,200	0.23	3.2	0.16	0.57	0.96
	M	176,600	0.63	7.6	0.38	0.51	1.52
	N	67,000	0.24	3.3	0.17	0.51	0.92
	O	407,000	1.45	16.0	0.80	0.62	2.87
	P	531,300	1.90	20.4	1.02	0.87	3.79
	Q	738,300 284 cts	2.64	27.7	1.39	0.87	4.90
TOTALS			14.95		8.31	7.31	30.56
Watershed Consolidated Canal to Price Road	E	651,800	2.33	24.7	1.24		3.57
	F	403,580	1.44	15.9	0.79		2.23
	G	329,000	1.17	13.2	0.66		1.83
	H	195,400	0.70	8.3	0.41		1.11
TOTALS			5.64		3.1		8.74
Watershed Eastern Canal to Price Road	E	1,230,700	4.39	44.8	2.24		6.63
	F	791,300	2.83	29.6	1.48		4.31
	G	665,700	2.38	25.2	1.26		3.64
	H	276,900	0.99	11.3	0.57		1.56
TOTALS			10.59		5.55		16.14
Watershed RWCD Canal to Price Road	E	2,030,100 719,800	7.25	72.2	3.61 1.35		10.86
	F	1,112,300 300,300	3.97	40.8	2.04 .61		6.01
	G	665,700 331,300	2.38	25.2	1.26 .66		3.64
	H	276,900 65,500	0.99	11.3	0.57 .16		1.56
TOTALS			14.59		7.48		22.07

Total cost, assuming Basins E,F,G,H are filled from watersheds west of Consolidated Canal, is 30.56 + 8.74 = 39.30 million dollars

Total cost, assuming Basins E,F,G,H are filled from watersheds west of Eastern Canal, is 30.56 + 16.14 = 46.70 million dollars

Total cost, assuming Basins E,F,G,H are filled from watersheds west of RWCD Canal, is 30.56 + 22.07 = [redacted] million dollars

3.2.2 Channel Cost Estimates

The cost of the channels in Table 4 was estimated as the sum of the cost of excavation and the cost of shotcrete. It is assumed the channels will lie within the highway ROW and will require no additional land acquisition costs.

The excavated volume was computed from the estimated required channel dimensions given in Table 4 and the channel length. Cost per cubic yard was estimated to be $3.57, as explained in Section 3.2.1. Estimated total excavation cost is, therefore, volume (276,100 cy) times cost per cubic yard ($3.57), or $1.0 million. The cost for shotcrete was estimated as the channel perimeter times its length times the cost per square yard of reinforced shotcrete. This cost per square yard was estimated at $1.35. The estimated total shotcrete cost for all the channels is $4.1 million.

The total cost for the channels is the sum of the two constituent costs, or $5.1 million.

3.3 DISPOSAL ALTERNATIVE COST ESTIMATES

At the request of ADOT, construction cost estimates were prepared for several of the stormwater disposal alternatives. This was not a detailed cost analysis but rather a feasibility-level effort to assist ADOT with decisions regarding how to dispose of the stormwater and whether or not ADOT's participation in the Price Road Drain would be beneficial.

Each of the disposal alternatives includes a basic assumption that stormwater will be discharged from the system only from Basins B, E, F, G and H. The other basins will generally gravity drain to these basins. For discharges to the north, it was assumed that Basins E, F and G would have pump stations rather than allowing the stormwater to flow south to Basin H from which it would be pumped to the north. Costs for the option of pumping from Basin B to Basin H were also estimated although discharge to the Southwest Drainage is still feasible from an engineering viewpoint.

Some of the cost estimates and unit costs were taken from other reports for similar projects in the area. It must be noted that some unit costs, particularly for the tunnel construction, may change significantly and the total costs for some alternatives may also change significantly.

The results of the cost estimates for the various stormwater disposal alternatives are presented in Table 7.

3.4 TOTAL COST

A summary of estimated total costs for drainage facilities is presented at the bottom of Table 7. The pumping of stormwater from Basin B to Basin H is an option which may be included with any of the other options. It was not included in the total costs presented at the bottom of Table 7.

Table 7

SUMMARY OF COST ESTIMATES[a]

	Pump Stormwater From Basin H					Pump Stormwater From Basin B to Basin H[b]
	Pump to South Outfall to Gila River	Pump to North Outfall to Salt River				
		Pump from Basins E, F, G & H to near Carriage Lane Park Detention Basin (CLPDB) PRCP = $4,357 4 Pump Stations = 2,100 Inlets, Outlets & Canal Crossing = 690 Total = $7,147				
		CLPDB to Price Road Drain @ AZ 360 Note: FDC Q = 285 cfs			CLPDB to Salt River	
	Q=200 cfs 102-in. PRCP= $20,500 Collector Pipe=$1,478 Inlets,Outfall &Channel=$422 4 Pump Stations=$2,100	168-in. Tunnel Q=485 cfs Tunnel=$5,754 Drop Shaft=$300 Net Cost to ADOT=$2,218	84-in. RCP Q=200 cfs Pipe=$3,117	108-in 114-in. RCP Q=485 cfs Pipe=$4,987 Freeway Crossing=$1,000 Net Cost to ADOT=$1,151	102-in. RCP Q=200 cfs Pipe=$12,300 Pump Station= $600 Outlet=$200	54-in. PRCP Q=25 cfs Pipe=$2,520
Option Cost	$24,500	$9,365	$10,264	$8,298	$20,247	$3,120[b]
Common Facilities Cost	$62,830	$62,830	$62,830	$62,830	$62,830	---
Total Cost	$87,330	$72,195	$73,004	$71,128	$83,077	---

a)All Costs x $1,000

b)This is an independent option and may or may not be added to any of the Basin H stormwater disposal alternatives.

Numbers do not check 52.63 + 5.1 = 57.7

4.0 REFERENCES

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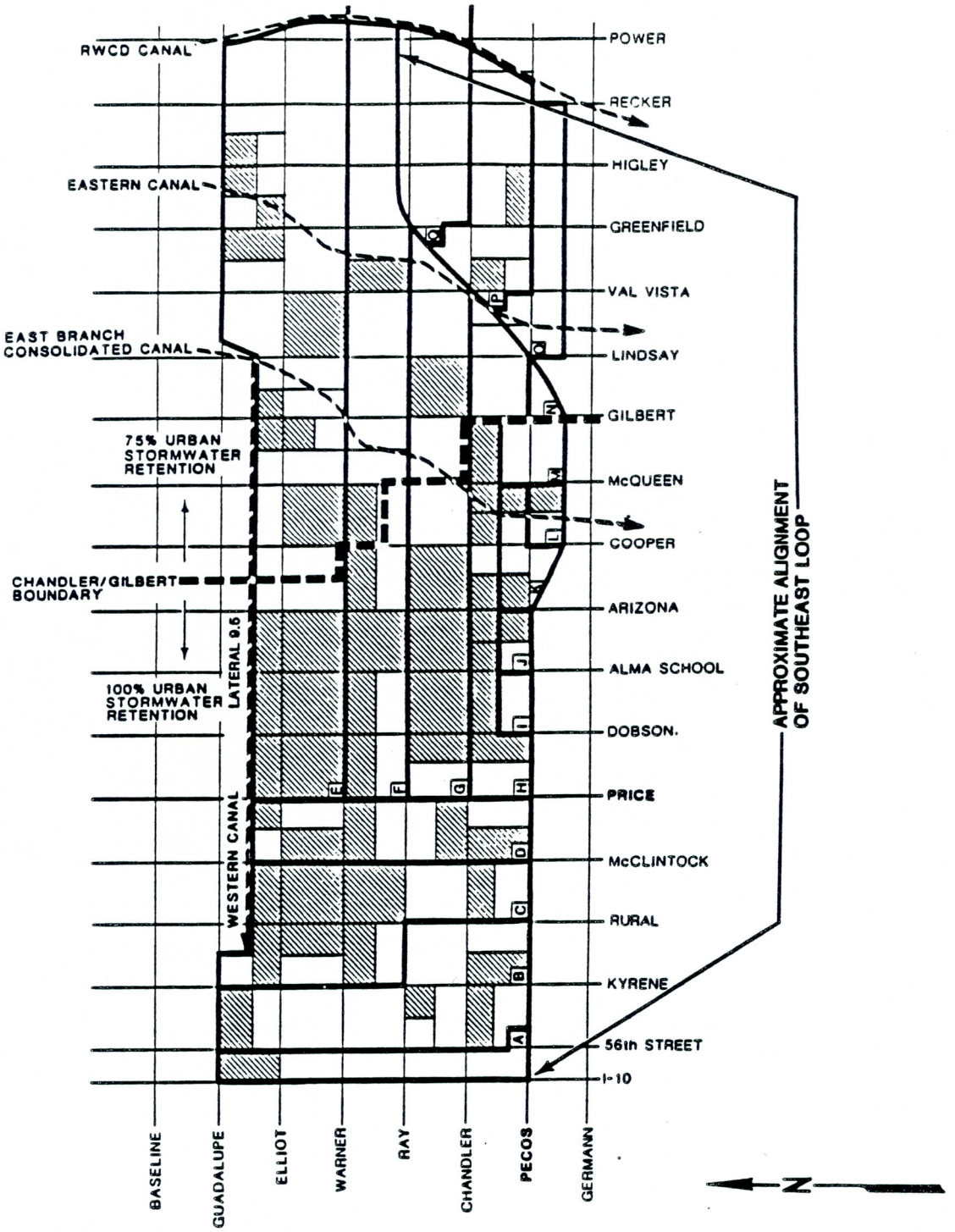
U.S. Army Corps of Engineers. 1981, Users Manual - HEC-1 Flood Hydrograph Package, Hydrologic Engineering Center, computer program 723-X6-L2010, September 1981.

U.S. Bureau of Reclamation. Design of Small Dams, Washington, 1977.

U.S. Geological Survey. Current 7.5 minute quadrangle maps of the study area.



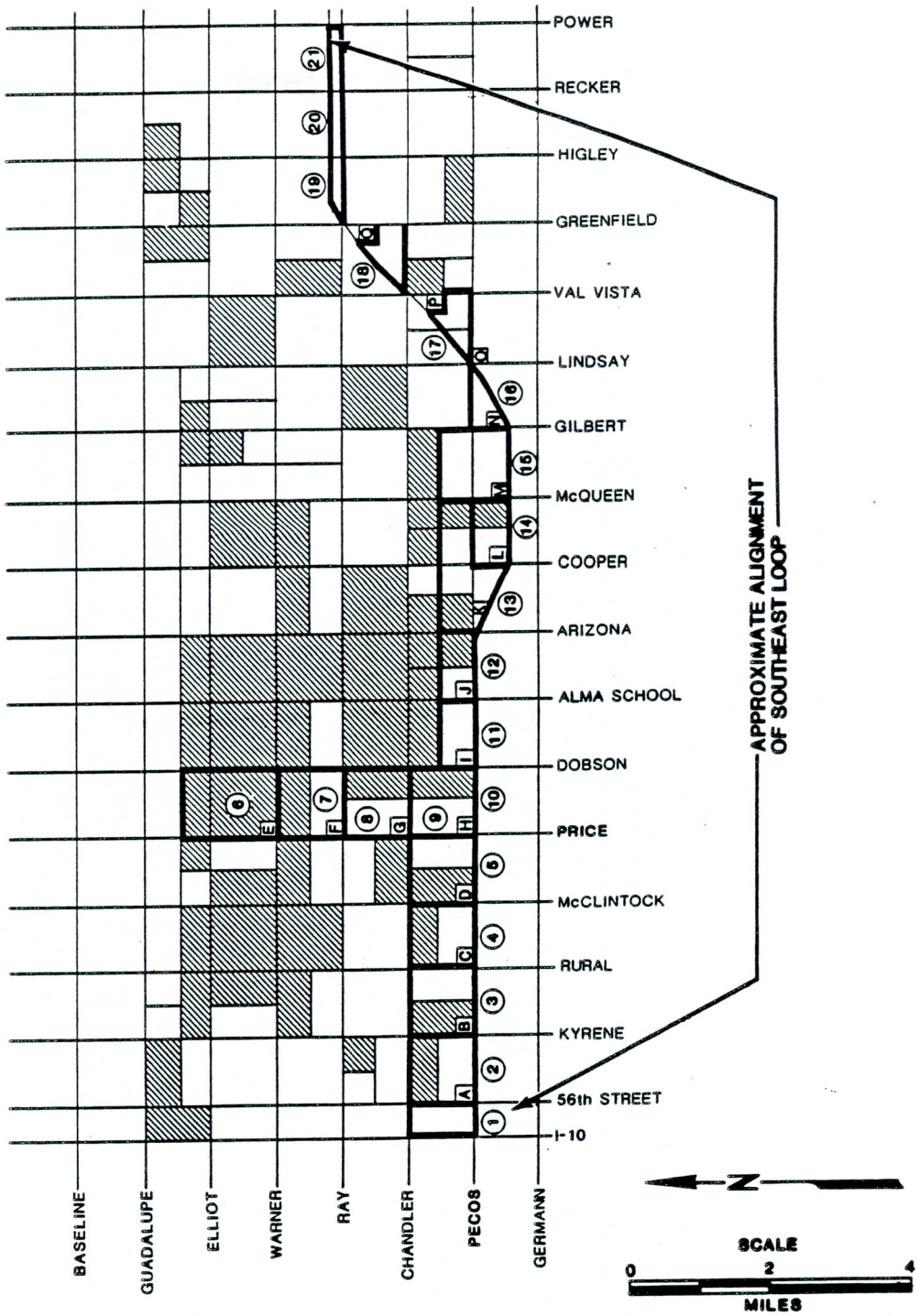
LEGEND:

- DEVELOPED AREAS
- NON-DEVELOPED OR AGRICULTURAL AREAS
- DETENTION BASIN WITH IDENTIFICATION
- DRAINAGE AREA BOUNDARY FOR BASIN DESIGN

STUDY AREA SHOWING DRAINAGE AREAS FOR BASIN DESIGN

NOTE: BASIN LOCATIONS ARE SCEMATIC ONLY

BY **Dames & Moore** Plate 1



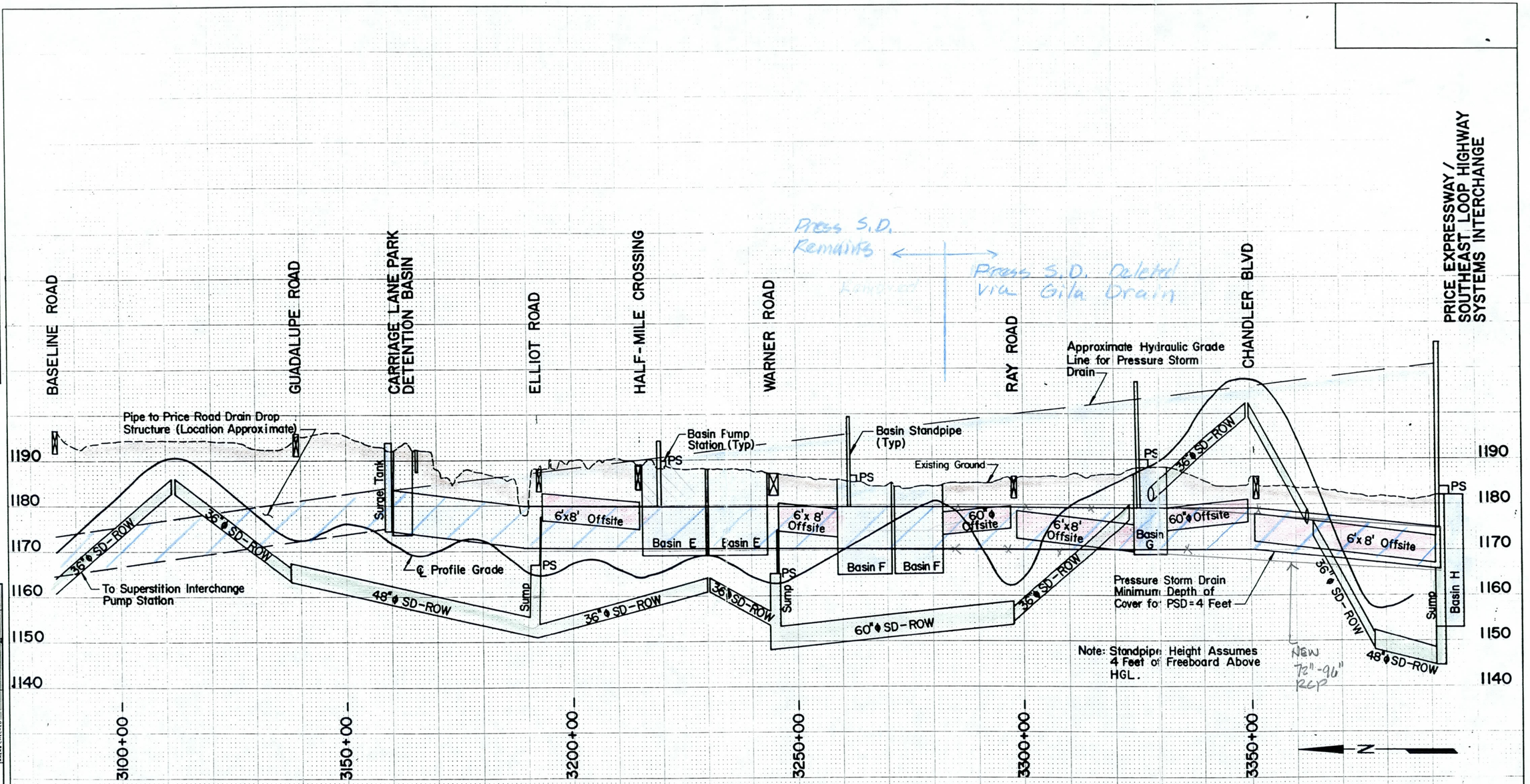
LEGEND:

- DEVELOPED AREAS
- NON-DEVELOPED OR AGRICULTURAL AREAS
- DETENTION BASIN WITH IDENTIFICATION
- CHANNEL REACH WITH IDENTIFICATION
- DRAINAGE AREA BOUNDARY FOR CHANNEL DESIGN

STUDY AREA SHOWING DRAINAGE AREAS FOR CHANNEL DESIGN

NOTE: BASIN LOCATIONS ARE SCHEMATIC ONLY.

BY **Dames & Moore** Plate 2

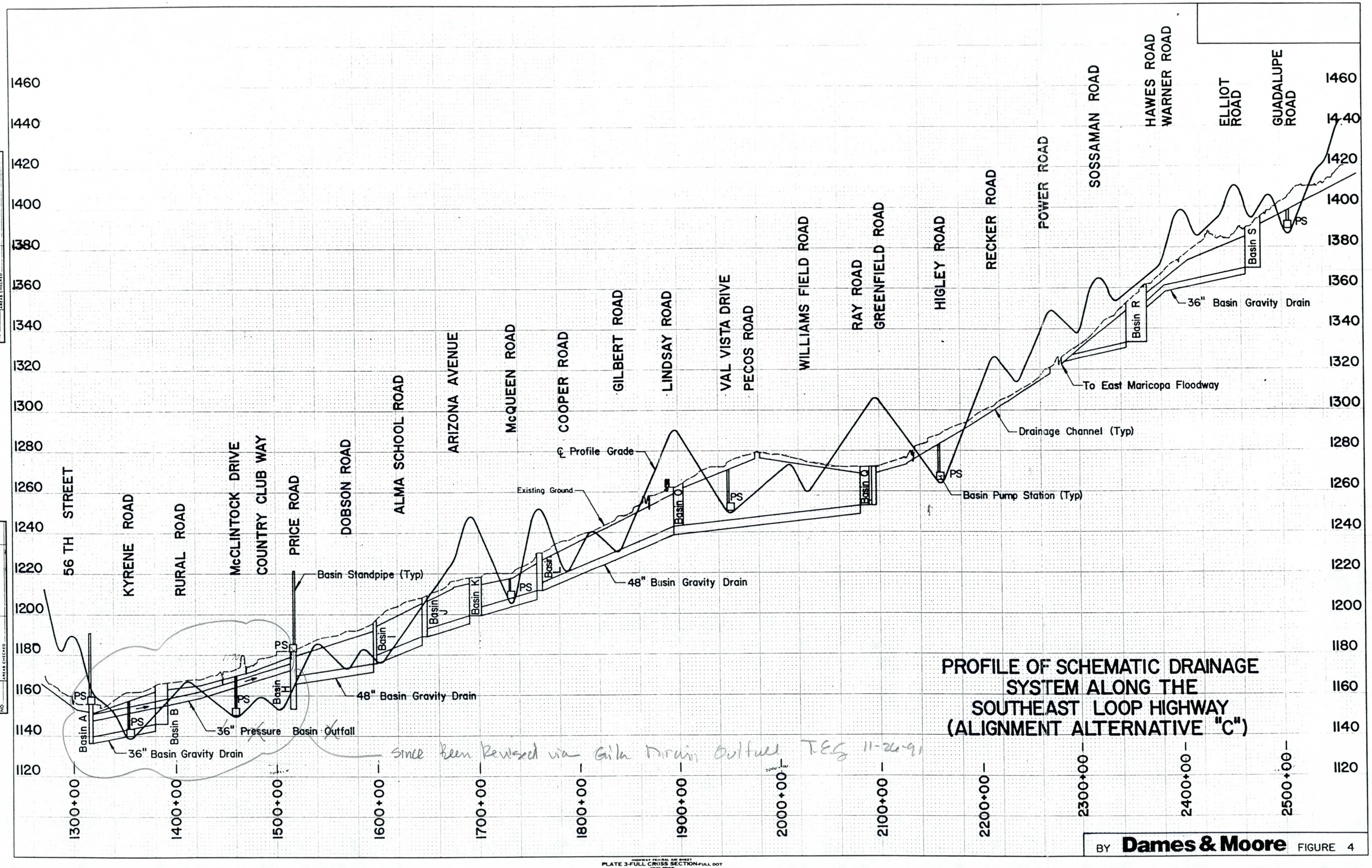


PROFILE OF SCHEMATIC DRAINAGE SYSTEM ALONG THE PRICE EXPRESSWAY

BY **Dames & Moore** FIGURE 3

FINAL SURVEY — SURVEYED, PLOTTED, TEMPLATE, AREA, AREAS CHECKED, NOTE BOOK, NO., BY, DATE

ORIGINAL SURVEY — SURVEYED, PLOTTED, TEMPLATE, AREA, AREAS CHECKED, NOTE BOOK, NO., BY, DATE



PROFILE OF SCHEMATIC DRAINAGE SYSTEM ALONG THE SOUTHEAST LOOP HIGHWAY (ALIGNMENT ALTERNATIVE "C")

BY **Dames & Moore** FIGURE 4

FINAL SURVEY — SURVEYED, PLOTTED, TEMPLATE, AREA, AREAS CHECKED, NOTE BOOK, NO., BY, DATE

ORIGINAL SURVEY — SURVEYED, PLOTTED, TEMPLATE, AREA, AREAS CHECKED, NOTE BOOK, NO., BY, DATE

HIGHWAY FEDERAL AID SHEET
PLATE 3-FULL CROSS SECTION-FULL DOT
SPTEL8096
PRINTED IN U.S.A.

APPENDIX A

SUPPORTING CORRESPONDENCE

GILA RIVER INDIAN COMMUNITY

SACATON, AZ. 85247

OFFICE OF PLANNING & EVALUATION
P.O. BOX 338 – (602) 562-3311

September 16, 1986

DAMES & MOORE
☐ S.E. LOOP
☐ PRICE ROAD
SEP 18 1986

Mr. A. Terry Clapham
Project Manager
Dames and Moore
Pointe Corporate Center
7500 Dreamy Draw Drive
Suite 145
Phoenix, AZ 85020

Dear Mr. Clapham,

We have reviewed the "Preliminary Draft Price Expressway Location/Design Study Reconaissance Report" dated September 11, 1986. There are a number of statements in the Report relating to drainage which are erroneous, or based on misassumptions.

A concept presented in its formative stages in the Report is the use of the Gila Drain to convey stormwater to the Gila River. There are three errors in this concept. First, the Gila Drain is not a "natural" (p. 2-22) watercourse, either in the sense of being a channel formed by natural hydrologic forces, or in the sense of being the course by which water would flow within the area it drains. In fact, it cuts across the natural contour of the land.

Second, the Gila Drain is not the natural interceptor of water to the Gila River (pp.2-27, 2-28); rather, it is the interceptor for drainage from the Queen Creek basin, which is separated from the Gila River basin by a complex drainage ridgeline. It is, therefore, misleading to discuss conveyance of stormwaters from Price Road to the Gila River without acknowledging that this transport violates the natural divide between the Queen Creek and Gila River drainage basins. To convey stormwater from Price Road to the Gila River is to transport water out of one basin into another.

Third, there may be a legal impediment to using the Gila Drain to convey stormwater from GRIC lands (p. 2-23). The Drain specifically is limited to use for conveyance of tailwater from Salt River Project irrigated lands. Use for conveyance of stormwater discharged from GRIC lands (p. 2-25) is questionable, and use for this purpose has been prevented physically by a series of berms constructed by Salt River Project. The volume of tailwater which can be conveyed is limited to 75 cfs where the drain is on Gila River Indian Community lands; this water is part of the GRIC's surface water rights. The tailwater is a useable asset for Community farms irrigation. The mixture of stormwater with irrigation tailwater may, therefore, be unacceptable.

The Gila Drain, then, cannot be assumed to be part of the solution to stormwater management on the Price Road Expressway because of hydrologic and probable legal constraints.

There is reference on p. 2-25 to stormwater volumes of "existing conditions." This term needs to be precisely defined, in light of the disputes over allowable flows onto the GRIC presently arising over development along South Mountain.

Also on p. 2-25 there is the statement that the Tribe will "install a drainage system to convey runoff from the northern part of their property to the Gila Drain." The GRIC has an adopted Master Drainage Plan prepared as part of the the North Central Area Plan. However, at this time, the Community does not have the fiscal resources to implement the plan. As you are aware, the watershed is part of a very complex drainage system which is under the management authority of the Maricopa County Flood Control District. Before acceptable solutions to the drainage needs of the Price Road Expressway can be put forward, a general plan, including feasible implementation, must be prepared for the drainage basins which the Price Road Expressway would affect. To address the drainage of the Price Expressway in isolation from the larger context of drainage issues is not a sound planning approach.

We would be more than willing to discuss with your staff again the Community's North Central Area Drainage Plan as a starting point to finding a mutually acceptable solution to the drainage from the planned Price Expressway.

Sincerely,

DOROTHY HALLOCK
Comprehensive Planner

WILLIAM TALBOW
Director, Physical Resources Department

DH/WT/dh

cc: Walter Mills, BIA PAO
Vernon Palmer, BIA Pima Agency
Stan Smith, Manager, Maricopa County Flood Control District
Rod Lewis, GRIC Law Office
Cecil Antone, GRIC Land Use Planning
Carleton Giff, Chairman, GRIC Natural Resources Committee

Dames & Moore

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7500 N. Dreamy Draw Drive
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Phoenix, Arizona 85020
(602) 371-1110

October 31, 1986

Dorothy Hallock, Comprehensive Planner
William Talbow, Physical Resources Department
Gila River Indian Community
Office of Planning & Evaluation
P.O. Box 338
Sacaton, AZ 85247

Dear Ms. Hallock and Mr. Talbow:

Thank you for taking the time to review and send us your comments on the Preliminary Draft Price Expressway Reconnaissance Report.

Below are responses to your comments. The numbered responses below refer to the numbers annotated on the attached copy of your memo:

1. There has been confusion regarding the term "Gila Drain". We will revise the text to refer to the drainage area through which the "Gila Drain" passes as the "Southwest Drainage" and only use the term "Gila Drain" to refer to the interceptor.

2. Water draining to or on the proposed Price Road Expressway would naturally drain to the Gila River via the Queen Creek drainage. Utilizing the "Gila Drain" would transfer water from the Queen Creek drainage to the Gila River via a non-"natural" route.

3. We recognize the argument you are making relative to the difference between tailwater from irrigated lands and stormwater. We will reflect your argument in the Final Reconnaissance Report.

4. We have not recommended or chosen any solution at this time. The hydrologic and potential legal constraints you bring up will be considered in the evaluation of alternative.

5. GRIC representatives used the term "existing condition" in a meeting with ADOT, HDR, and Dames & Moore regarding drainage from both the Southwest and Southeast Loops. The basis for the statement "It is anticipated that the GRIC will not permit any stormwater to enter their property in any manner except similar to existing conditions" was comments made by GRIC representatives at the meeting. It would be beneficial

to the study to have a precise definition of what GRIC feels "existing conditions" means.

6. This statement will be changed to reflect that at this time GRIC does not have the fiscal resources to implement the plan.

7. The Maricopa County Flood Control District is part of the Technical Advisory Committee for the Price Expressway Study. The Price Expressway drainage study is being done in the larger context including other basins and the plans of others.

Sincerely,

Terry Clapham

Terry Clapham
Project Manager

HNTB Review Comments

11057-11-09	COPY TO	INFO	ACTION
file		✓	
RDM	✓		
TCW	✓		
RWL	✓		
RPB	✓		

Bob Luscombe - Phoenix

November 4, 1986

Pete Jarchow, Dick Schwab - Kansas City

DAMES & MOORE HEC-1 HYDROLOGY MODELS FOR AREAS SOUTH OF WESTERN CANAL
HNTB PROJECT NO. 11057-11-09

The hydrology models received from Dames and Moore have a combined drainage area of approximately ~~48~~ 60 square miles, lying west of the Roosevelt RWCD Canal, south of the Western Canal, east of the Price Road corridor and north of the proposed South-East Loop corridor. The drainage area was divided into 5 east-west strips, ranging in width from 0.5 to 1.0 miles and in length from 8 to 12 miles; runoff from each strip is determined by a HEC-1 model. The results of the HEC-1 modeling are 5 hydrogrpahs of the flow impacting the Price Road Corridor, each hydrograph representing flow between the major east-west streets (generally spaced 1 mile apart) at the north-south portion of corridor. The peaks range from 1,100 to 1,900 cfs; nothing was received pertaining to proposed storage or conveyance of the model flows to Carriage Lane basin, thence to the Outer Loop project outlet.

We have several comments about the hydrologic modeling. The HEC-1 models are obviously not intended as detailed, final models. The models are presumably intended to roughly quantify expected flows only to the degree of detail required for a corridor analysis, where only relative magnitudes of peaks and volumes are required. All agricultural subareas are modeled identically except for subarea sizes; the same is true of urban subareas. Virtually no area - specific data was used.

There are several factors adversely affecting the compatability of the Dames and Moore HEC-1 models and our HEC-1 models.

1) Dames and Moore are using a 100-year, 6-hour storm. We, Boyle, Yost & Gardner and most others in the area have used a 24-hour storm. Usage of the 100-year 24-hour storm would increase peaks 10 to 20 percent and runoff volumes 30 to 40 percent.

2) Dames and Moore use the kinematic wave method to determine runoff from urbanized subareas, and the SCS method to determine runoff from agricultural areas. Boyle and HNTB used the SCS method to determine runoff from all areas, regardless of degree of urbanization.

3) Dames and Moore route hydrographs simply by "lagging" them, delaying them by an estimated travel time. Boyle and HNTB used the kinematic wave routing method.

4) Dames and Moore applied areal reduction factors to the input rainfall depths; while it is certainly justifiable, it results in usage of different rainfall values from those used by Boyle and HNTB.

5) The HEC-1 runs are based upon a 4-minute ordinate interval. HNTB used a 5-minute interval, and reran the Boyle model (originally based upon a 10-minute interval) for 5 minutes. Due to a program limitation, 5 minutes is the shortest interval that can be used with a 24-hour storm. A shorter interval can increase peaks.

6) The Dames and Moore model includes large and frequent variances in drainage area sizes; they have a 4:1 range of size for their standard basins and have a 6:1 ratio of sizes from their smallest to their largest. Large variations in subarea sizes can affect peak runoff rates unless lag times are carefully determined.

7) Major existing topographic features that were used as subarea and overall area boundaries in the HNTB and Boyle models were completely ignored in the Dames and Moore model. No allowance was made for the storage or flow redirection affects of the raised embankments of the two alignments of the Southern Pacific Railroad as it passes through the area, or of the affects of the Consolidated and Eastern Canals upon the runoff.

8) There is no distinction in the model between routing runoff

hydrographs through urbanized subareas (many that were obviously assumed to have storage capacity for all local runoff) and routing flow through agricultural area.

9) The lag times used for agricultural subareas seem short for very flat areas with few if any existing drainage features.

Because of these factors we feel that the current results of the Dames and Moore modeling should not be used directly to derive additional inputs to our hydraulic model.

However, because of constraints such as the head required to convey S.E. Loop flows to Carriage Lane, it would seem that usable hydrographs could be developed based upon storage versus pumping requirements and the Dames and Moore hydrology. For consistency of hydrologic methodology we suggest that the incompatability factors listed be resolved; because of the likely requirement of large quantities of detention, the factors relating to runoff volume are the most important (items no. 1, 4 and 7 above, plus an analysis of any excess runoff that would be expected from communities using a 6-hour design storm for detention storage design, if it is decided to use the 24-hour storm).

DII100.18

Pen changes by Jim Geiser
(during meeting)

MINUTES OF MEETING
NOVEMBER 6, 1986
PRICE ROAD DRAIN MEETING

A meeting was held at DeLeuw-Cather offices on Thursday, November 6, 1986. HNTB was in attendance to discuss the HEC-1 modeling Dames & Moore had completed to estimate peak flows and runoff volumes from the areas tributary to the Price Road alignment. The meeting was scheduled during a ADOT TAC meeting and attendees included Larry Olsen and Stan Polasik of DeLeuw-Cather, Bob Luscombe and Pete Jarchow of HNTB and Jim Geiser of Dames & Moore.

HNTB presented to DeLeuw-Cather their comments (copy attached) on Dames & Moore's hydrology. All parties agreed that the preliminary study was sufficient to complete a corridor study but that final design would require a much more detailed study. In particular, areas of "flood irrigation" where all runoff might be contained needed to be identified. More detailed analysis of the three canals and the two railroad embankments is necessary and detailed coordination with engineering staff of both Gilbert and Chandler should be completed to compare assumptions and techniques.

The following comments are presented in response to HNTB comments.

1. Using the 24-hour storm may increase the peak flowrate by a small percentage. However, the preliminary peaks provided were sufficiently accurate to evaluate ADOT participation in the Price Road Drain.

2. The kinematic wave method is particularly well suited for and is probably more representative for modeling agricultural areas.

3. No HEC-1 method is very accurate at modeling flows down streets where the street can not contain the flow. The kinematic wave routing method is more suited to channel flow. Either method is appropriate to estimate lag times for this level of study.

4. Aerial reduction should be used, especially for areas of less than one square mile.

5. As explained by HNTB they were limited to 5 minute time intervals. The 4 minute time interval can be more accurate.

6. Variances in subbasin size from 1/4 to 1 square mile are not significant for this level of study involving about 60 square miles.

7. ADOT staff members accompanied Jim Geiser on a tour of the watershed prior to the HEC-1 modeling. It was agreed at that time that the level of detail necessary for the corridor study did not require detailed analysis of these types of features.

8. Flows were assumed to pass down the major streets as reported by residents of the area. Streets adjacent to agricultural areas are considered similar to residential streets for this level of study.

9. Estimation of lag times was based on empirical formulas and could be revised during more detailed analysis based on thorough site investigations.

General discussions and comments included the following:

a. All agreed that pumping would be required to discharge to north or south from the Price Road area under study.

b. All attendees agreed that joint probability of 10-year Salt River flow, 100-year peak at Superstition Freeway and 100-year peak flow to Price Road south of Western Canal is very low. Hence, there is probably excess capacity in the proposed tunnel when Price Road drainage is available for discharge. Tunnel may need not be enlarged to carry Price Road drainage.

c. If the tunnel does not need to be enlarged, then the tunnel design and construction would not be delayed as a result of adding Price Road drainage.

d. Conveyance as proposed by FCD from Carriage Lane Park to Superstition tunnel inlet may have to be redesigned because Chandler and Gilbert flows are nearly constant and of long duration. This should be based on the more detailed studies of the areas tributary to Price Road.

e. Jim explained briefly the meeting with GRIC representatives.

f. HNTB stated 14-foot tunnel is now estimated at $800/ft.

g. Jim pointed out that Dames & Moore's contour maps showed Carriage Lane basin invert about 7 feet lower than in HNTB report.

Meeting went well and was beneficial to all.

APPENDIX B

MEMORANDUM CONCERNING REDESIGN OF DETENTION BASINS

MEMORANDUM CONCERNING REDESIGN OF DETENTION BASINS

May 1988

Dames & Moore
Pointe Corporate Centre
7500 N. Dreamy Draw Drive, Suite 145
Phoenix, Arizona 85020

D&M Job Numbers 11344-022-22
11344-030-22

1.0 INTRODUCTION

As explained in the our Summary of Preliminary Drainage Studies For Southeast Loop And Price Expressway, the existing design of the detention basins along Price Road and the Southeast Loop was very sensitive to four major assumptions: 1) the Southern Pacific Railroad embankment did not intercept flows along its right-of-way and 2) 100% of agricultural land runoff would travel to the highway rights-of-way 3) the design would be based upon land use estimated from 1986 photographs 4) the major canals which cross the watershed do not have a significant effect on storm runoff. These four assumptions, as explained previously, were made because of a lack of detailed data and were conservative. These assumptions therefore led to a "worst" case design.

Since the resulting design appeared too conservative and costly, it was decided that the collection of additional data concerning the railroad embankment and the agricultural lands was warranted. This memorandum discusses this effort to formulate less conservative assumptions, and presents the new detention basin designs based upon the new assumptions.

2.0 RESULTS OF INVESTIGATION

2.1 SOUTHERN PACIFIC RAILROAD EMBANKMENT STORAGE

Two separate lines of the Southern Pacific Railroad are aligned across the watershed in the area east of Price Road and west of the RWCD Canal. These lines are supported by embankments varying in height from 1 to 6 feet above the surrounding fields or urban areas, with an average height of about 3.5 feet. Road crossings in this region are without exception at-grade crossings.

Shotcrete-lined irrigation laterals 2 feet wide and from 3 to 4 feet deep extend along the edge of the major east-west roads in this region. The railroad embankments pass over these lateral channels at approximately one mile intervals as shown in Plate 1. Any irrigation water or storm runoff which enters these laterals can pass through the railroad embankments

with flowrates dependent on the physical conditions at each opening. Open channel flow through each of these openings in the embankment could reach 130 cubic feet per second (cfs), based on a bed slope similar to the general slope of the adjacent land. With water ponded against the embankment, flows could reach 200 cfs at each opening. The elevation difference between top of channel bank for these lateral channels and the surrounding fields or urban areas varies between 1 and 2 feet, and therefore there is considerable detention storage volume available at these embankment crossings. This storage was estimated as follows.

The storage below the channel top of bank on the upslope side of the bank was given a rough survey in the region of the crossings. The total storage was calculated by extending a horizontal line from the top of the channel in each direction (along the railroad embankment and along a line perpendicular to the railroad) for a distance sufficient to intercept the sloping ground. Where these slopes could not be measured, a distance of a half mile was assumed. From these two distances and the storage depth (channel bank elevation minus surrounding area elevation) at the embankment, a storage volume was calculated. These volumes are presented in Table 1.

Table 1

Storage Volumes Upslope of Embankments

Location (shown on Plate 1)	Storage (ac-ft)	Basin Affected
a	17.4	E
b	56.0	E
c	69.7	F
d	26.0	H
e	40.0	E
f	14.9	E
TOTAL	224.0	

2.2 AGRICULTURAL LAND STORAGE

The assumption made in the earlier design was that 100% of the runoff from the region's agricultural lands would join the runoff draining towards the highways. From a tour of the study area it is apparent that the amount of stormwater from fields which would drain onto local road rights-of-way is highly variable. It is apparent, however, that the earlier assumption of zero field storage (i.e. 100% runoff) is very conservative.

By observation, the majority of the regional fields, at the corner of the lowest elevation, are 1 to 2 feet below the lateral channel top of bank, or local road grade. Assuming that the fields follow the general topography upslope from these corners yields a storage volume between 25% and 100% of the total field runoff. Therefore, an assumption of an average 50% detention on agricultural land is reasonable.

2.3 LAND USE ESTIMATES

Current data was reviewed to estimate land use in 1993. The estimated areas of 1986-1993 development within the tributary watershed is shown on Plate 1.

The earlier conceptual design was based upon an assumption that the City of Chandler urban areas were all capable of retaining 100% of the 100-year, 6-hour storm runoff on site, while Gilbert areas were capable of 75% retention. These assumptions are still used in the current revised conceptual design.

2.4 CANAL STORAGE

The earlier conceptual design provided three different estimated basin sizes, each corresponding to an assumption as to which of the major canals (Consolidated, Eastern, RWCD) intercepted all the upstream flow. This was done because of the lack of detailed data concerning the over-topping of these canals in major floods.

A reconnaissance of the Consolidated Canal shows that lateral channels from the east, and several urban and road storm drainage culverts empty into the canal. Discharge west towards Chandler could occur through gated and ungated lateral channels leading from the canal, or by overtopping the canal bank. Storage volume against the upslope canal bank is frequently zero (because of stormwater discharge into the canal) but can rise to a depth of about 2 feet at some locations. The conservative assumption, because of the highly variable nature of the storage, is to assume zero storage, as was done previously. An accurate assessment of the effect of this canal upon cross-drainage would be very involved.

A reconnaissance of the Eastern Canal shows that immediately upstream of the canal there is a channel about 6 feet deep with a 3-foot-wide bottom and 2:1 (horizontal:vertical) side slopes. The bed slope is extremely flat, and the channel is interrupted at road crossings where erratically maintained culverts provide drainage under the roads. The conservative assumption is that this channel provides little conveyance and provides storage only. This storage amounts to about 10.9 acre-feet per linear mile of canal.

The East Maricopa Floodway is still assumed to intercept all upstream drainage not already intercepted by other upslope facilities.

2.5 ESTIMATION OF REQUIRED DETENTION BASIN SIZES

The above assumptions were incorporated into a re-estimation of required detention basin sizes along Price Road and the SE Loop. First, the 1993 land use distribution was used to re-estimate watershed urban and agricultural acreages. The urban runoff was estimated, using the assumed retention percentages for Gilbert and Chandler. Agricultural runoff was computed as per the method described for the previous estimate, then reduced by 50% to account for assumed field storage. Storage along the Eastern Canal and along the Southern Pacific Railroad embankments was subtracted from watershed total runoff, where appropriate. Required basin sizes were estimated to contain the remaining runoff. The resulting estimates are presented in Table 2.

Table 2

ESTIMATED DETENTION BASIN SIZES AND COSTS[a]

FOR DETENTION OF 50-YEAR, SIX HOUR STORM:

Basin	Width Required (feet)	Area Required (acres)	Volume Required (cy)	Area Cost[b]	Volume Cost[c]
A	370	3.1	61300	160	220
B	550	7.0	160900	350	570
C	460	4.8	104200	240	370
D	290	1.9	32300	90	120
E	770	13.6	340400	680	220
F	540	6.7	152200	330	540
G	630	9.2	218900	460	780
H	380	3.3	67200	170	240
TOTALS		50.0	1137300	2480	4060
			TOTAL COST		$6,540

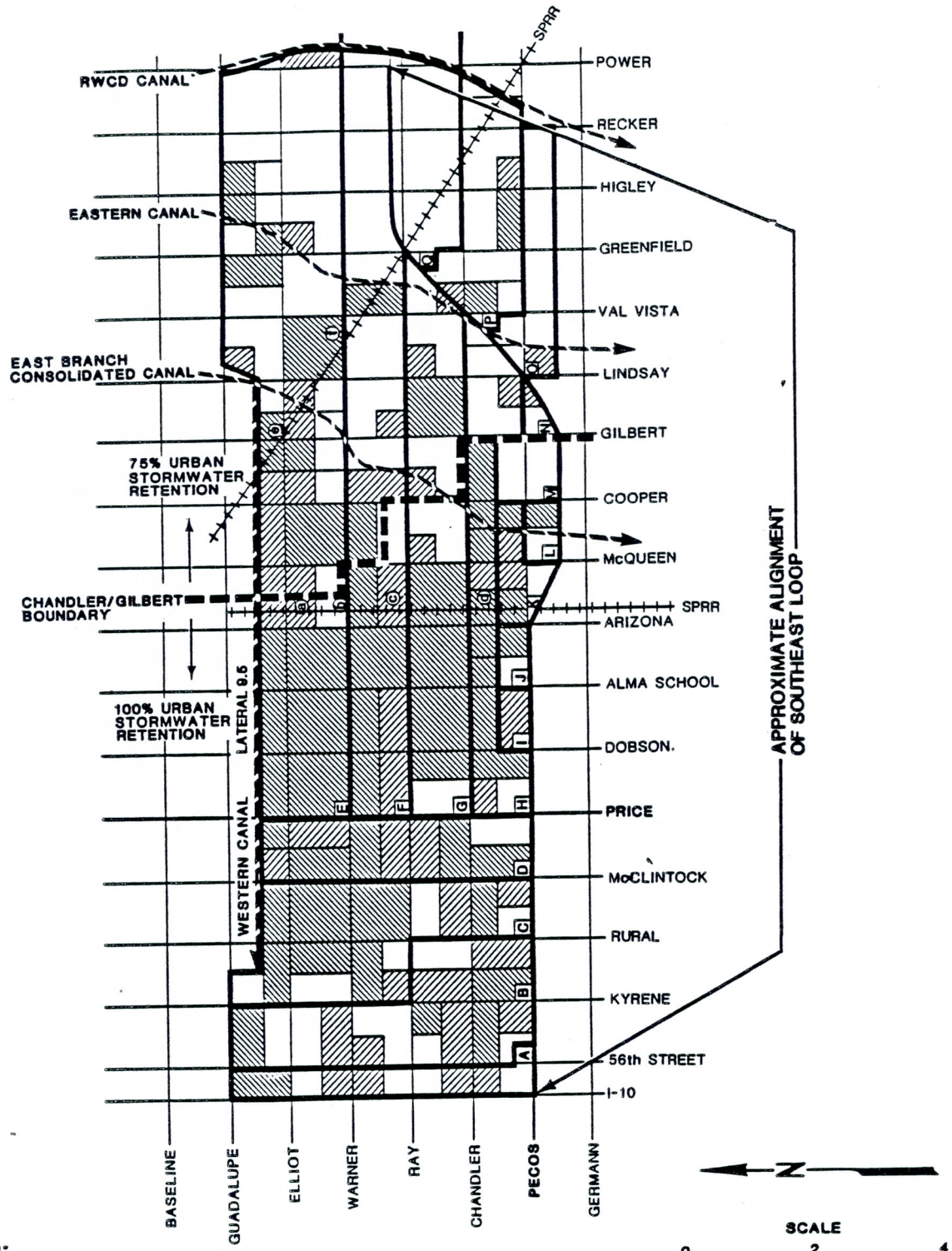
FOR DETENTION OF THE 100-YEAR, 6-HOUR STORM:

Basin	Width Required (feet)	Area Required (acres)	Volume Required (cy)	AF	Area Cost[b]	Volume Cost[c]
A	460	4.8	104800	87	240	370
B	640	9.5	227700	141	470	810
C	540	6.6	151000	94	330	540
D	370	3.2	63000	39	160	230
E	1090	27.1 24.1	719800	446	1350	2570
F	730	12.2	300300	186	610	1070
G	760	13.3	331300	205	660	1180
H	380	3.3	65500	41	160	230
TOTALS		80.0	1963400		3980	7000
			TOTAL COST			$10,980

[a] all costs x $1,000
[b] based on $50,000 per acre
[c] based on $3.57/cy

Basin D = 30' A = 6.7 acre D = 30'
L/W Top = 540
L/W Bottom = 300'

$$V = \frac{300^2 + 540^2}{2}(30) = 130 \text{ af}$$



LEGEND:

- DEVELOPED AREAS (1985)
- DEVELOPED AREAS (1993 PROJECTION)
- NON-DEVELOPED OR AGRICULTURAL AREAS
- A DETENTION BASIN WITH IDENTIFICATION
- DRAINAGE AREA BOUNDARY FOR BASIN DESIGN

STUDY AREA SHOWING DRAINAGE AREAS FOR BASIN DESIGN

NOTE: BASIN LOCATIONS ARE SCHEMATIC ONLY.

BY **Dames & Moore**

Plate B – 1