PLANNING

PRICE
SANTAN
SOUTH MOUNTAIN

Arizona
Loop 101 South

Hydrology Study

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Volume I : Main Report

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HYDROLOGY STUDY REPORT

TABLE OF CONTENTS

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LIST OF TABLES

Page No.

LIST OF FIGURES

LIST OF SHEETS (In Map Pouch)

HYDROLOGY STUDY REPORT

1.0 INTRODUCTION

1.1 *Background*

In January, 1989, HDR Engineering, Inc. (HDR) entered into a General Engineering Consultant (GEC) agreement with the Arizona Department of Transportation (ADOT) for Price Expressway and the portion of the Santan Freeway from Dobson Road to Interstate 10 (I-10). As part of that agreement, HDR was to perform a hydrologic analysis on the drainage area contributing to the expressway and freeway. Prior to this GEC agreement, ADOT conducted a preliminary drainage study in which two alternative stormwater outlets were evaluated (Dames & Moore, July, 1988). One of the plans was to collect the offsite drainage in a series of detention basins along the Price and Santan alignments and to pump the impounded stormwater north to the Salt River via a gravity drain and tunnel system that is presently under construction. The second plan was to collect the drainage in basins, as per the first plan, but to pump or gravity drain the basins, south or southwesterly to the Gila River. For the present study, HDR was directed by ADOT to consider only the Salt River outlet option.

1.2 *Scope*

The Price Expressway and Santan Freeway roadways are primarily depressed below existing grades in the project area. Stormwater sheet flows originating offsite and entering the ROW cannot be conveniently passed through or under the roadway. Thus, the preferred method of handling the offsite stormwater is to contain the design storm volume (the 100-year 24-hour event) in a series of detention basins on the "upstream" side of the roadway.

Furthermore, the basins must be evacuated within several days to accommodate subsequent storm events. In the project area there is no natural stormwater outlet. The only available outlet is the Carriage Lane Outfall and Price Tunnel system which is under construction and will soon provide an outlet north to the Salt River. A constraint to pumping into this system is that the flow will be restricted to 200 cfs during design peak flow conditions, per an intergovernmental agreement between ADOT, Chandler, Mesa, and Maricopa County. (Intergovernmental Agreement, Price Drain, January 25, 1988).

The drainage area contributing offsite flows to the project is approximately 58 square miles and complicated by mixed land use, variable development criteria, and the presence of various significant barriers to flow. These are described in detail in Section 2 of this report. This required the development of a detailed hydrologic model which was the primary focus of this task and is described in Section 3.

Following the model development, the peak flows and volumes for the design storm event were calculated at "concentration points" along each ROW as shown in Section 4. A conceptual plan was then developed to contain the stormwater in regional basins which are integrated and evacuated by pumping to the Carriage Lane Outfall.

This conceptual plan is presented in Section 5. A cost estimate considering the major features of the conceptual plan is presented in Section 6.

The scope of this task did not include consideration of the onsite drainage facilities. The onsite facilities will be considered in the next task, which is to develop a preliminary design for both the offsite and onsite facilities. Because the roadway sections are depressed below grade, the onsite drainage will be pumped to the offsite basins. However, the volumes are very small relative to offsite volumes and do not materially influence the dimensions of the offsite system. Therefore, the onsite system is not further considered in this report.

During the preliminary design phase to follow, the offsite system will likely be modified to accommodate multiple use of facilities, primarily with the City of Chandler. Therefore, the conceptual plan presented herein is mainly to provide a basis for discussion and refinement during the design phase, and is not to be considered a "recommended" final plan at this time.

1.3 *Project Location*

An overall view of the project setting is shown on Figure 1 (and Sheet 1).

The drainage area boundaries for the offsite drainage analysis are the Western Canal/Lateral 9.5 on the north, the Santan Freeway alignment on the south, Interstate 10 on the west, and the Roosevelt Water Conservation District (RWCD) Canal on the east.

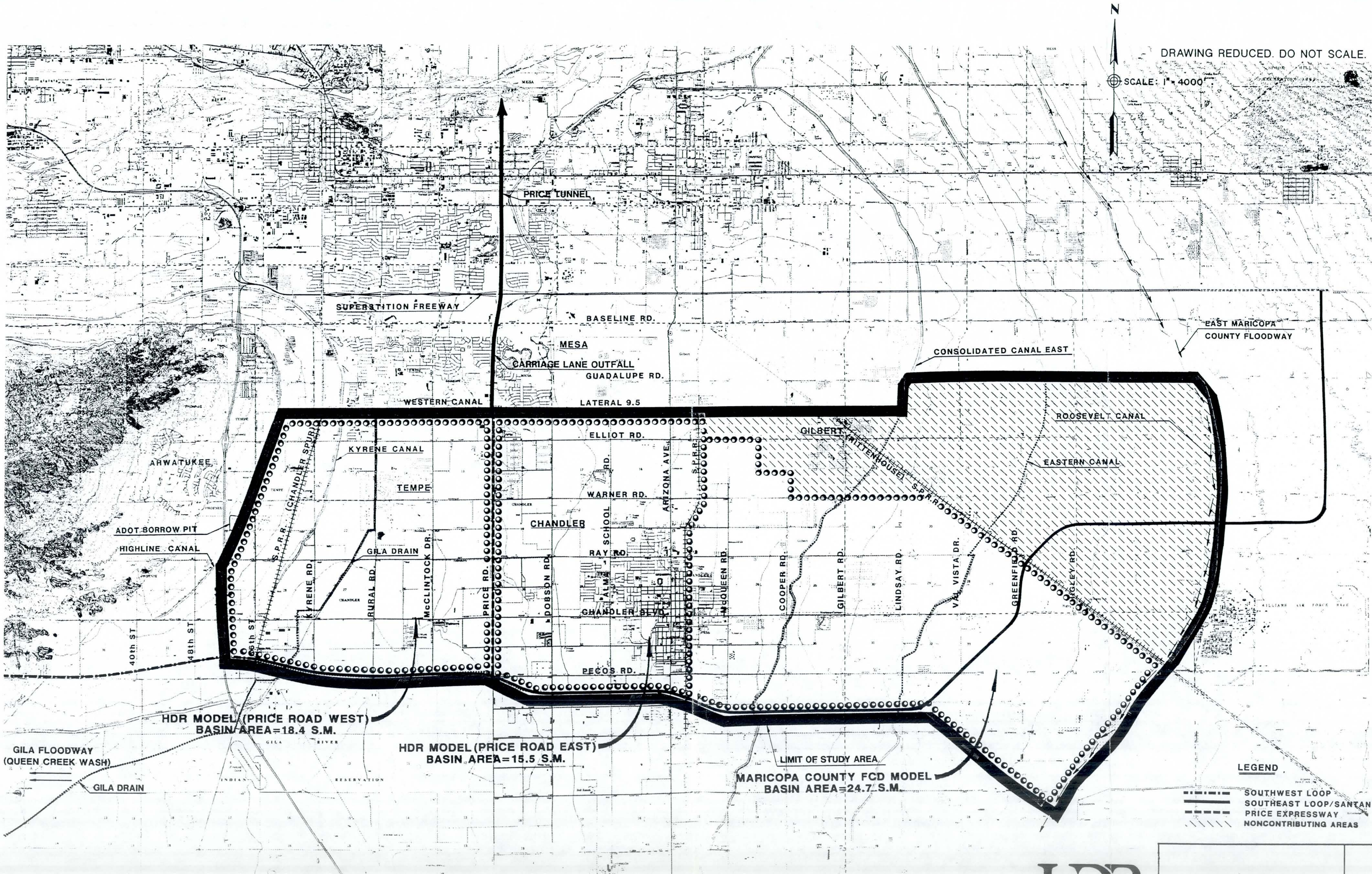
The drainage area is part of the Queen Creek watershed, which according to a 1977 Corps of Engineers (COE) report is 1000 sq.mi. in size (COE, 1977). Approximately 75% of the Queen Creek watershed east of the RWCD Canal is controlled by Soil Conservation Service flood control structures. In addition, the general drainage pattern from east to west across the study area is modified by three significant barriers, including the East Maricopa County Flood Channel constructed along the east side of the RWCD Canal, the Western Canal and Lateral 9.5, and the Rittenhouse Road branch of the Southern Pacific Railroad (SPRR). The flood channel along the RWCD Canal is a major flood control facility which is designed to divert flow to the south. It is assumed that the channel will divert southward all flood flows which would normally flow westward to Price Expressway for the storm frequencies considered in this report.

The Western Canal and Lateral 9.5 are considered to be barriers to crossflow of stormwater in the north/south direction due to the height of the canal banks and the detention facilities along both sides. Offsite drainage entering the Price ROW north of Western Canal is not included in this report, although HDR is responsible for Price roadway design from Baseline Road to Western Canal. The offsite drainage for this section of Price Expressway has been the responsibility of the DeLeuw Cather Company (DCCO) and Howard, Needles, Tamman and Bergendorf (HNTB), who have jointly planned and designed the Price Tunnel and Carriage Lane Outfall (DCCO, 1989).

During the study it was found that the Rittenhouse SPRR line, which runs diagonally across the study area, is also a significant barrier to east/west crossflow. Flood flows that reach the east side of the railroad embankment are impounded or flow northwesterly toward Gilbert (FCDMC, 1989).

Less significant barriers to east/west crossflow are the Consolidated and Eastern Canals and the Chandler SPRR embankment. The effect of these barriers on the offsite stormwater is described in detail, later in the report.

Considering the barriers, the effective combined drainage area contributing offsite runoff to the Price and Santan alignments is approximately 58 square miles, of which 40 square miles flows almost straight westerly to the Price alignment, and the remainder west of Price Road generally flows southwesterly toward the Santan alignment.



HDR

HYDROLOGY STUDY
PROJECT LOCATION MAP

FIGURE 1

2.0 WATERSHED DESCRIPTION

2.1 *General*

The watershed is located in east Maricopa County between the Salt and Gila Rivers. The natural and historic drainage pattern for the runoff generated in the project drainage area is east to west except in the vicinity of the Gila Drain in Tempe and Chandler, where flood flows turn southwesterly and converge at the I-10/Maricopa Road interchange. An irrigation canal structure known as the Gila Drain presently conveys some stormwater runoff across the Gila River Indian Reservation to the Gila River. Stormwater flows are limited to 75 cfs in the Gila Drain by a 1920 intergovernmental agreement. The Gila Drain is therefore not considered to be a stormwater outlet for the natural flows that reach the I-10/Maricopa interchange. The natural flows will continue to flow westward to a more-or-less defined wash known either as the Queen Creek Wash or Gila Floodway, as shown on Sheet 1. The Gila Drain and Gila Floodway are described in greater detail in the Dames and Moore (1988), COE (1977) and Coe and Van Loo (1979) reports. As mentioned, HDR did not consider the option to use the Gila Drain/Floodway as an outlet in this study.

The study area is partially urbanized and rapid development is taking place in the western portions. However, about two-thirds of the 58 square mile drainage area for this study is still agricultural. The extremely flat natural land slope lends itself well to the flood irrigation practices used on the agricultural areas. However, the flat slopes are not conducive to effective stormwater conveyance. The drainage area has lost most of its natural drainageways because of the sectional grid pattern of the major street network.

2.2 *Stormwater Management Policies*

The municipalities have instituted stringent stormwater retention policies on new developments in order to control the flooding problems generated by the increasing impermeability (less infiltration) due to urbanization and the lack of defined drainage outlets. There are currently four different stormwater retention policies in force in the study area. The City of Tempe has had a 100-year one-hour (2.4") volume retention policy since 1978. The City of Chandler had a 100-year six-hour (3.00") volume retention policy until 1987 when it was changed to the 100-year two-hour (2.5") volume. The City of Gilbert requires a 50-year 24-hour (3.00") retention volume. The Flood Control District of Maricopa County requires the post-development discharge not to exceed the pre-development discharge.

2.3 *Physiographic Factors Affecting Runoff*

The Soil Conservation Service (SCS) is completing a 100-year flood control channel known as the East Maricopa County Flood Channel along the upstream embankment of the RWCD irrigation canal (see sheet 1). The channel diverts the intercepted stormwater of the Queen Creek watershed to the Gila River. The flood control channel is assumed to eliminate flood flows from entering the study drainage area from the east.

Stormwater is also diverted from historic flow paths by the series of irrigation canals and railroad embankments between the RWCD canal and Price Expressway alignment. When ponding upstream of these diversions creates the required head, runoff is diverted along the upstream face of the embankments. When sufficient runoff volume is generated, the stormwater overtops the embankments and continues to flow to the west along the existing street systems. The railroad embankments and to a lesser extent, the canals, reduce the runoff volume that reaches the Price Expressway alignment.

West of the Chandler line of the Southern Pacific Railroad (SPRR) and east of Price Road, the current drainageways are along the east-west major arterials. These section line streets are typically the only flow paths for the stormwater runoff to get to the Price Expressway alignment from the east. Stormwater runoff usually ponds in the major intersections before proceeding westward to the Price Expressway alignment.

One peculiar development type in the study area is the ranchette, or 'horse property' development. These are one-half to one and one-half acre lots which are graded to accept and retain six to eight inches of irrigation water and will therefore retain the 100-year rainfall event with little or no runoff. Some of these lots are graded to accept the street runoff as well.

Area East of Chandler Branch (SPRR)

As shown in Figure 1, the area east of the Chandler branch of the Southern Pacific Railroad (SPRR), one-quarter to one-half mile east of Arizona Avenue, was modeled by Franzoy Corey Engineering Company for the Flood Control District of Maricopa County (FCDMC) in a study to delineate floodprone areas upstream of railroad and irrigation canal embankments (See Appendix C). The easternmost basin boundary of the FCDMC study was the Roosevelt Water Conservation District (RWCD) irrigation canal and East Maricopa County Flood Channel. The FCDMC model results were used as input into the HDR hydrologic model. The modeling approach for the FCDMC study area was modified to align it with the HDR modeling approach for the area downstream of the SPRR Chandler branch. HDR then reran the portion of the model that affects runoff contributing to the Price and Santan alignments.

The FCDMC study confirmed that the railroad and canal embankments significantly reduce the quantity of stormwater that reaches Price Expressway ROW between Western Canal and the Price-Santan interchange.

The following general conclusions were drawn from the FCDMC study:

1. The total area modelled by Franzoy Corey for Price Road was about 46.6 sq. mi., however, the Rittenhouse SPRR embankment is a significant barrier to westward flow. With the exception of two small culvert openings, and the Eastern and Consolidated Canal openings, the 100-year flood flows do not pass over or through the embankment in the study area. In addition, a small area west of Gilbert drains northward into Lateral 9.5 at the Chandler SPRR crossing. These barriers, as shown on Figure 1, remove about 21.9 sq. mi. from contributing to the Price ROW. The remaining 24.7 sq. mi. watershed presently drains to the Chandler SPRR between Western Canal and the Price-Santan interchange and crosses the railroad at two locations given in Appendix C.

   In the future when the Santan Freeway is extended east to the Superstition Freeway, the portion of the Santan between the Rittenhouse SPRR and the

RWCD Canal will either intercept or pass off-site flows depending upon the freeway design. Presently most flows would be allowed to pass unhindered because the design concept is an at-grade roadway with bridges over most arterials. However, any intercepted off-site and on-site stormwater in this reach would be expected to be contained on the east side of Rittenhouse SPRR as is presently the case and therefore, these flows are not anticipated to reach the Price/Pecos interchange area.

Any on-site or off-site flows intercepted in the portion of the Santan between RWCD Canal and the Superstition would likely be discharged to the East Maricopa County Floodway Channel.

2. The Eastern and Consolidated Canal embankments intercept and pond stormwater until there is sufficient head to move the water laterally along the upstream embankment. In the project area this movement is generally to the south. Since arterial road crossings cross over the canals, the stormwater tends to pond upstream of the crossings until the canal is overtopped. If the canal is full at this time (a conservative assumption), the flood-flows will pass undiminished across the downstream embankment. Since the Eastern Canal has a defined channel along the upstream embankment, all of the stormwater in the project area (i.e.; from Rittenhouse SPRR to Pecos Road) is transferred to an overflow point at the intersection of the Eastern Canal and Pecos Road. From there, the revised FCDMC model further indicates that the flow will continue in a west/southwesterly direction and out of the project area.

   The apparent peak flow at this location is 664 cfs and the volume is 540 ac.ft. At the present time this flow will not affect off-site facilities along Price Road or the Santan from I-10 to Dobson. In the long range future when the Santan is completed from Dobson to the Superstition, the roadway may intercept some of the off-site runoff generated in the area from Eastern Canal to the Rittenhouse SPRR flows. The intercepted and on-site flows may be transferred to the Price/Pecos Interchange via a series of basins and outflow pipes and channels. At that time further development with retention policies is likely to have reduced the off-site volumes to be handled by the project overall. Furthermore, the flows will be delayed by restricted outlets and "bled" to the Price/Pecos basin. Therefore, the on-site and potential off-site volumes for the Santan from Dobson to the Superstition are not included in the design of off-site facilities proposed herein.

3. The Consolidated Canal does not have a defined upstream flood channel and therefore overtops more frequently. The FCDMC model defines four locations where the Consolidated Canal is overtopped in the project area.

4. The SPRR embankment through downtown Chandler concentrates the sheet flows approaching from the east and discharges the flow through trestle openings at two locations. The major concentration point is just north of Chandler Boulevard in Chandler.

5. The FCDMC model indicates that Lateral 9.5 prevents flows generated north of the canal from crossing southward into the project area. However, runoff from a small area immediately south of Lateral 9.5 concentrates at the south embankment and overflows into Lateral 9.5, and thus out of the project area.

The impact of the foregoing conclusions on model assumptions is discussed in Section 3.

Area West of Chandler Branch (SPRR)

With the exception of downtown Chandler, the developed areas provide 100-year onsite retention. The City of Chandler and the City of Tempe have had strict retention policies since 1975 and 1978, respectively. From 1975 to 1987, the City of Chandler had a 100-year six-hour (3.00") retention policy. The City of Tempe has had a 100-year one-hour (2.4") retention policy since 1978.

Development sizes range from one-eighth to one and one-half square miles. Stormwater retention is provided in parks, linear basins and lakes throughout the development. The street network acts as a stormwater conveyance system to direct runoff to the provided retention facilities. Few storm drains exist in the study area.

The volume of retention provided within each development was estimated by reviewing The City of Chandler subdivision records and the development drainage reports. Where no drainage reports were available the area was field checked to verify that retention had been provided. Due to the "bowl" slope of many developments and freeboard requirements, there appears to be significant surcharge storage available which is not accounted for in the hydrologic analysis.

Downtown Chandler Area

A significant portion of the downtown Chandler area was developed prior to the City's drainage retention policy. To control stormwater runoff from this area, large regional retention basins have been built to capture a significant portion of the stormwater flows. Basin locations, sizes, and contributing drainage areas were taken from the City of Chandler Stormwater Management Master Plan (1986).

Area West of Price Road

West of Price Road and east of I-10 the arterial streets are still the primary channels for runoff, however, excess flows more-or-less converge toward the Gila Drain at I-10 and Maricopa Road . Most of the section line arterial streets are curbed and guttered and accept overflow from the adjacent subdivisions when provided retention volumes have been satisfied.

The west project boundary is assumed to be Highline Canal and I-10 (see Sheet 1). There is a large borrow pit at the intersection of Highline Canal and I-10 that serves as a detention pond for stormwater generated in the Ahwatukee area west of I-10 and in the area between I-10 and Highline Canal. The City of Tempe has constructed channels and storm sewers in the area to divert stormwater flows from Ahwatukee into the borrow pit (Tempe, 1983). The capacity of the pit is apparently well in excess of the 100-year, 24-hour runoff volume generated in the 5.5 sq.mi. contributing drainage area. Therefore, it is assumed that the storm sewer and channel system will divert 100 percent of the 100-year 24-hour rainfall into the pit, and that the pit will not overflow. In reality some flows may "escape" the diversion and collection system, but these are assumed to be small enough to be ignored. Therefore, HDR only modeled in detail the area east of I-10 and Highline Canal.

The north half of this area is in the city limits of Tempe. Tempe does not require developers to provide detention storage for arterial runoff. Instead the city provides storage for arterial runoff in regional detention basins. The south half of this area is in the Chandler city limits. Chandler requires developers to provide detention for both on-site and arterial runoff.

3.0 HYDROLOGIC MODEL DEVELOPMENT

3.1 *Approach*

The U.S. Corps of Engineers Flood Hydrograph Package, HEC-1, (COE, 1987) was utilized to model the flood hydrology of drainage areas contributing to the Price Expressway and Santan Freeway. The model simulates the rainfall-runoff process and develops runoff hydrographs, peak discharges and stormwater runoff volumes. HEC-1 can be used to analyze a complex interconnected stream network of component urban or nonurban subbasins. For this study, the stream network consisted of street surface flow paths, vestigial natural water courses, irrigation canals, and canal and railroad embankment barriers.

The HEC-1 model has numerous options for generating, connecting and routing flood hydrographs. HDR used the SCS runoff and unit hydrograph options to generate design flood hydrographs for all land use types. Combined hydrographs were routed downstream using the kinematic wave routing option. When storage structures were encountered, the modified Puls routing option was used.

Basic input requirements to HEC-1 include precipitation data, drainage area, initial abstraction (IA), runoff curve number (CN), and basin lag (L), which is a hydrograph timing factor.

3.2 *Design Storm*

The design storm used in this hydrology study was the 100-year frequency, 24-hour duration rainfall event, consistent with previous and on-going ADOT freeway and expressway designs. This design storm yields a total point precipitation depth of 3.7 inches for the project area as determined from the ADOT Hydrologic Design for Highway Drainage In Arizona (ADOT, 1968).

HEC-1 has an automated procedure for distributing the storm rainfall throughout the 24-hour period known as the "balanced storm" procedure. It creates a triangular shaped hyetograph from 5 and 15-minute and 1, 2, 3, 6, 12, and 24-hour rainfall depths. These values were derived using procedures in the ADOT Manual (ADOT, 1968). The advantage of using this HEC-1 option is that it automatically calculates the distribution for any computation time interval. The resulting distribution is shown in the linked HEC-1 model print-outs in Appendix D (separate Volume II).

An areal rainfall reduction factor was also used in this analysis. This function reduces the point precipitation amounts to an average depth of precipitation for large watersheds. HEC-1 reduces rainfall according to recommendations in Weather Bureau TP-40 (1961).

For the area east of Price Road, the contributing watershed area was calculated to be approximately 40 square miles. For the area west of Price Road, the contributing watershed area was approximately 18 square miles. When Price Expressway is completed, the watershed east of Price Road will be permanently separated from the area west of Price Road as far as runoff is concerned, therefore the two areas were treated as separate and independent watersheds with regard to areal reductions.

The rainfall reductions are not large and amount to about 4% and 2% for the two watersheds. Larger reductions up to 11% may be warranted according to NWS Hydro-40(1984). A study of impacts of such reductions on runoff is included in Appendix A. Because the runoff reductions would be significant, ADOT recommended further study and inter-agency discussions before using these reductions.

3.3 *Drainage Subbasins*

Contributing drainage areas were delineated from 7-1/2 minute USGS quadrangle maps, then subjected to a field inspection to verify general accuracy of delineation. The overall area was divided into subbasins ranging in size from one-half square mile to one and one-half square miles and an individual HEC-1 model created for each basin. Within a basin model, the basin area was further subdivided into sub-basins which shared a known or assumed common outfall point. Many subbasins have varied land use characteristics due to the sporadic development of land and a typical mixed-use approach to land development. When practical, the subbasins were delineated with a preference for size uniformity and homogeneous land use.

3.4 *Hydrologic Soil Types*

The SCS method offers the capability to classify the runoff characteristics by relating soil type and vegetative cover to a runoff parameter known as the Curve Number (CN). To assist in selection of an approximate CN, SCS has classified soils in Maricopa County into one of four general infiltration groups, A, B, C or D, (SCS, 1974). These are known as Hydrologic Soil Types. Type A soils have high intake capability and Type D soils have very low intake capability. Type B and C soils predominate in the study area and have somewhat greater than or somewhat less than average intake capability, respectively. Details are given in SCS TR-55 (1986). Type B soils comprise at least 70% of the area.

3.5 *Runoff Curve Numbers (CN)*

In addition to soil type, the CN is dependent upon vegetation and degree of soil wetness prior to the storm event. In developed areas the average CN is also dependent upon degree of imperviousness of the development (i.e.; roof, sidewalk, driveway, and street surface areas). The CN's used in the model are given in Table 1.

Table 1. Runoff Curve Number (CN) Values

Cover Description	Hydrologic Soil Group B	C
Open space, good condition (grass cover 75%)	61	74
Impervious area (pavement, roofs)	98	98
School grounds	77	85
Commercial, business	92	94
Industrial, PAD	88	91
Single family (SF-7 zoning)	84	88
Multi-family	85	90
Fallow (crop residue, good condition)	83	88
Row crop (straight row)	81	88

The CN values in Table 1 were primarily extracted from SCS TR-55(1986). The CN's are for an average soil wetness prior to the storm, which is Antecedant Moisture Condition II (AMC II). This is roughly represented by one to two inches of rainfall occurring during the 5 days preceding the design storm.

3.6 *Initial Abstractions and Loss Rates*

Runoff is dependent upon amount of initial rainfall retained on the surface (Initial Abstraction, IA) and the amount infiltrated during the storm (Infiltration). HEC-1 automatically calculates both the initial abstraction and infiltration for a given curve number as per SCS guidelines (SCS, 1971). However, HEC-1 has an option to arbitrarily change the IA without affecting the amount of infiltration. This option was used to account for storage in residential/commercial areas where detention policies were in effect, and for agricultural areas and ranchettes which were constructed to retain water. This approach was a key to HDR's modeling strategy, which was to construct models which can be regarded as site specific yet avoiding the necessity to model details such as the characteristics of each detention pond in a developed area.

The IA's were modified for these areas as follows:

Agricultural Areas

Agricultural areas were modeled as irrigated row crops on flat slopes. Most fields have berms to retain and conserve irrigation water. These were assumed to have a storage or retention effect for the design storm. A separate study by HDR was done to determine the required increase in the IA to account for the storage effect. The study concluded that an average field can retain approximately 2.5 inches of rainfall, which can be duplicated by using an IA=1.5 inches in HEC-1. Details of this study are included in Appendix B for reference.

Ranchettes

A number of residential neighborhoods in the study area are comprised mainly of "horse acreages," in which individual lots are graded as sumps and flood irrigated. The lots are capable of containing many inches of water and some may accept street runoff. A 3.0 inch IA was used for these areas which effectively eliminated excess stormwater runoff from these areas.

Residential Areas

Retention policies for various jurisdictions are listed in Table 2. Weighted average Curve Numbers were estimated for each subbasin based upon CN's obtained from Table 1 considering factors such as lot size, percentage of impervious area, and soil type.

The initial abstraction (IA) factor was adjusted to account for the onsite retention provided within each development. The value used for the IA was determined by evaluating the amount of rainfall required to produce the volume of runoff equal to the provided retention volume. The development type and density dictated the curve number to be used to transform the rainfall to runoff. The initial abstraction (IA) value was used to simulate the development retention volumes since several small, non-central basins were typically provided throughout the developments.

Table 2. Retention Policies.

Entity	Years in Effect	Rainfall Frequency/Duration	Retained Rainfall Depth
Gilbert	'75-present	50 yr.-24 hr.	3.00"
Mesa	'72-present	50 yr.-24 hr.	3.00"
Chandler	'75-'87	100 yr.-6 hr.	3.00"
	'87-present	100 yr-2 hr.	2.50"
Phoenix	pre-85	10 yr.-2 hr.	1.60"
	'85-present	100 yr.-2 hr.	2.60"
Tempe	'78-present	100 yr.-1 hr.	2.40"

Assigning an IA to account for detention policy was somewhat complicated for the project area east of Price Road because development occurred throughout the span of time during which the detention policies were changing. Therefore, available drainage reports were used to determine the actual storage provided. Most subdivisions provided more than the minimum mandated storage. However, the actual IA used in the model was never more than the minimum requirement. Subdivisions for which no drainage report was found were assigned the minimum requirement in effect at the time. In some cases a subbasin may have a weighted average IA to account for development during different time periods within the same subbasin.

West of Price Road development has primarily occurred in the 1980's when uniform and consistent policies were in effect for both Chandler and Tempe. Therefore, individual subdivision records were not investigated and uniform IA's were applied to developed subbasins in respective jurisdictions. This also simplified the model structure for this area.

Stormwater in excess of the provided retention volumes typically must surcharge the basins to reach the subdivision overflow elevation. The potential excess storage volume between the jurisdictional retention and the overflow is not accounted for in the analysis.

Downtown Chandler Area

Most of Downtown Chandler was developed prior to implementation of detention policies. The City has therefore constructed regional basins to control flooding.

The drainage area contributing to each of these regional basins was modeled without onsite retention. The runoff hydrograph was routed through the regional basins (without outflow) until the basin volume was reached. The inflow in excess of the basin full capacity was modeled to by-pass the basin without attenuation or routing. Because the level pool routing was used to model the response of the basins to the stormwater inflow, a few time steps passed before the basin inflow and outflow matched following the full pool elevation.

3.7 *Lag Time*

The Lag Time (L) is a hydrograph shape factor related to the concentration time of the watershed, which is in turn related to the basin length and slope. Lag times for each subarea were calculated using procedures described in TR-55 (SCS,1986). When the overland flow velocity for an urban basin was less than one foot per second (fps) the velocity was arbitrarily raised to one fps. Agricultural velocities were not arbitrarily restricted and were as low as 0.5 fps.

3.8 *Model Construction*

Schematics of the two drainage submodels are presented in Sheet 2. As previously noted the submodel east of Price Road was much more complex than the submodel west of Price Road.

Submodel East of Price Road

For the submodel east of Price Road, HDR divided the area between Price Road and Chandler SPRR into subbasins ranging from about one half to one and one-half square miles. Each subbasin was furthermore subdivided into smaller subareas representing a common land use or runoff outlet. The total area modelled by HDR was about 15 sq. mi.

The area east of the Chandler SPRR was modeled in detail by Franzoy-Corey for FCDMC. The FCDMC model covers a large area on the order of 100 sq. mi. from Superstition Freeway on the north to Hunt Highway on the south and the RWCD Canal on the east. About 25 sq. mi. of this model was relevant to this study, and the model was very helpful in determining the impact of barriers upon runoff reaching Price Road in the project area. HDR initially requested that Franzoy Corey modify their model to make it compatible with HDR's approach. These modifications are described in Appendix C. HDR then selected the portion of this model that was relevant to the project area and reran it to determine the hydrograph characteristics of the two openings in the SPRR embankment (See PT 20 and PT 1C; Sheet 2). These in turn became hydrograph inputs to HDR's model west of the SPRR.

The subbasin boundaries are shown on Sheet 2. The subbasins are further combined into four larger areas that contribute runoff to the Price Road ROW. These are east-west strips ranging in size from about one square mile to over four square miles.

Submodel West of Price Road

The subbasins of this model were generally one square mile in size as defined by the arterial grid system. The area was previously defined as having uniform detention policies, and the canal and railroad embankments do not significantly influence drainage patterns. Also, the runoff from Ahwatukee is assumed to be diverted into the ADOT borrow pit southeast of the I-10/Warner Road Intersection, and will therefore not be of concern in the project area.

The Gila Drain is located along the approximate lowest elevation contour for this area. Runoff tends to flow toward the drain and then generally south and southwesterly toward the present I-10/Maricopa Road interchange area. The subbasins were therefore combined into three larger areas that concentrate flows along the proposed Santan Freeway Alignment. (See Sheet 2).

The term "concentration" requires definition. In reality, flows that reach the Price or Santan ROW are sheet flows and are concentrated only to the extent that the arterial roads can be considered to be limited shallow channels. Because the area is flat and the topographic mapping is limited to the ten-foot contour interval USGS maps, it is not possible to precisely define where flows will concentrate along either ROW. The models represent the best estimate based upon available information and field observation.

Subbasin Aggregation and Optimization

In developing the models, the watershed was initially subdivided into subbasins and then subareas within subbasins. Each subbasin is therefore a substantial model by itself. If all the subbasins were to be linked in a single model, the resulting combined model would have been very cumbersome to use because of long run times, large print-outs, and the amount of detail in the print-out.

To overcome the inefficiencies of this approach, the subbasin output hydrographs were duplicated by using a lumped model of the subbasin and the HEC-1 calibration option. Thus, the lumped basin hydrograph was optimized by varying the parameters of the SCS method for a single basin until the outflow hydrograph closely matched the linked subarea hydrograph.

Suitable optimized hydrographs were obtained in two or three trials. The effort was directed toward closely matching volumes rather than peaks. However, differences in peaks between the lumped and detailed models were normally insignificant after routing to the next combination point downstream. The routing functions essentially smooth the sharper peaks (and sometimes multiple peaks) of the detailed subarea models.

Results of subbasin models and the optimization runs are presented for each subbasin in Appendix E. A watershed map, model input and output summaries, and a plot comparing detailed and lumped area hydrographs are given for each subbasin. Appendix E is bound separately in Volume II of this report. Optimized subbasin hydrograph parameters are listed in Tables 3 and 4. Because of the optimization procedure, the CN's, IA's and L's only approximately represent averaged values for the subbasins. In some cases these values were arbitarily adjusted to achieve a better calibration.

In summary, the modeling approach resulted in models that:

1) accurately represent detailed considerations of land uses, soil types, and mandatory detention storage throughout the area;

2) can be run efficiently on a micro-computer;

3) are very flexible and easy to modify;

4) allows a longer time step computation interval to be used in HEC-1.

With regard to Item 4, two time steps, 5 and 12 minutes are compared in Tables 3 and 4.

Table 3 . Subbasin Characteristics - Area West of Price Road

						FIVE MIN. TABULATION INTERVAL		TWELVE MIN. TABULATION INTERVAL		
BASIN NAME	SUBAREA HYDROGRAPH DESIGNATION	DRAINAGE AREA (SQ. MI.)	INIT. AB-STRACTION (INCHES)	SCS CURVE NUMBER	LAG TIME (HOURS)	RUNOFF VOLUME (AC-FT)	PEAK FLOW (CFS)	RUNOFF VOLUME (AC-FT)	PEAK FLOW (CFS)	TIME TO PEAK (HOURS)
KYELL	SECT9	0.33	1.94	83.0	0.44	14	94	13	86	12.75
ELRUR	SECT10(1)	0.50	2.41	84.0	0.36	13	65	13	62	12.75
ELRUR	SECT10(2)	0.19	2.25	88.0	0.53	7	37	7	35	12.92
ELRUR	SECT10(3)	0.03	1.50	83.0	0.92	2	9	2	9	13.17
ELRUR	SECT11	0.51	2.41	86.0	0.71	14	51	14	50	13.25
ELMCL	SECT12	0.50	3.00	86.0	0.58	4	7	4	6	15.08
MCWAR	SECT13	1.00	2.72	88.0	0.32	19	54	19	54	12.92
WRRUR	SECT14	1.00	2.83	75.7	0.75	8	15	8	15	14.50
RURWR	SECT15(1)	0.69	2.18	81.0	0.82	20	81	20	78	13.25
RURWR	SECT15(2)	0.31	1.45	83.0	0.98	18	94	18	89	13.25
KYWAR	SECT16(1)	0.14	1.54	83.0	0.41	8	69	8	61	12.67
KYWAR	SECT16(2)	0.50	1.52	83.0	0.54	28	215	28	198	12.75
KYWAR	SECT16(3)	0.23	3.00	88.0	0.39	2	4	2	4	14.83
56RAY	SECT20	0.29	1.51	83.0	0.43	16	145	16	145	12.67
KYRAY	SUB21(1)	0.42	1.41	83.0	0.61	26	186	26	170	12.83
KYRAY	SECT21(2)	0.59	1.60	83.0	0.60	31	217	32	199	12.83
KYRAY	SUB22(1)	0.82	2.26	83.0	0.38	24	143	24	133	12.75
KYRAY	SECT22(2)	0.13	1.45	83.0	0.33	8	79	8	71	12.50
RURAY	SECT23	1.00	2.97	83.0	0.78	8	14	8	14	15.17
MCRAY	SECT24	1.00	2.77	83.0	0.60	13	27	13	27	13.75
MCLCH	SECT25	1.00	1.81	84.0	0.91	47	222	47	211	13.25
RURCH	SECT26	1.00	1.82	85.0	0.96	48	221	48	221	13.33
KYRCH	SECT27	1.00	2.18	88.0	1.38	38	119	39	117	13.92
KYCHN	SECT28(1)	0.69	2.51	85.0	0.35	16	71	16	66	12.83
KYCHN	SUB28(2)	0.30	1.97	85.0	0.58	13	76	13	71	12.92
56CHN	SECT29	0.50	2.12	85.0	0.62	18	99	18	90	12.92
56SWL	SECT32	0.33	1.50	83.0	0.60	19	135	19	123	12.83
56SWL	SECT33(1)	0.53	2.22	82.0	0.18	15	126	15	116	12.50
56SWL	SECT33(2)	0.26	1.53	86.0	0.54	16	126	16	116	12.75
KYSWL	SECT34	0.67	2.04	83.0	1.05	24	93	25	90	13.50
RUSWL	SECT35	1.09	2.38	83.0	0.23	27	177	27	155	12.58
MCSWL	SECT36	0.83	1.95	84.0	0.94	34	149	41	197	13.33
TOTALS		18.38				598		607		

Table 4 . Subbasin Characteristics - Area East of Price Road

						FIVE MIN. TABULATION INTERVAL		TWELVE MIN. TABULATION INTERVAL		
BASIN NAME	SUBAREA HYDROGRAPH DESIGNATION	DRAINAGE AREA (SQ. MI.)	INIT. AB-STRACT (INCHES)	SCS CURVE NUMBER	LAG TIME (HOURS)	RUNOFF VOLUME (AC-FT)	PEAK FLOW (CFS)	RUNOFF VOLUME (AC-FT)	PEAK FLOW (CFS)	TIME TO PEAK (HOURS)
PEL	OPT7	0.56	2.20	89.7	0.32	22	137	22	130	12.67
ED	OPT8	0.49	1.90	75.0	1.66	13	37	14	37	14.33
EAS	OPT9	0.44	2.11	75.8	1.35	10	28	11	28	14.00
EA	OPT15	0.92	2.20	82.5	0.91	25	84	26	82	13.50
WAS	OPT16	0.88	2.64	85.0	0.85	14	29	15	29	14.17
DM	OPT17	1.00	2.45	82.3	1.14	19	45	20	45	14.08
PW	OPT18	0.80	2.50	75.8	0.44	11	32	11	31	13.08
WP	OPT19	1.10	2.43	89.6	0.30	32	149	32	149	12.75
WD	OPT20	0.74	2.53	90.9	0.29	20	77	20	73	12.83
AR	OPT21	1.45	1.22	77.5	1.16	79	348	80	335	13.42
GRB	OPT28	1.00	0.90	93.0	0.75	110	777	110	734	12.92
CD	OPT29(A&B)	1.12				20	76	20	75	14.25
PG	OPT30A	0.39	0.60	63.1	0.90	20	116	21	111	13.08
PC	OPT30B	0.41	0.92	69.8	0.85	22	124	22	118	13.08
PCH	OPT31A	0.55	2.64	95.0	0.34	17	51	17	50	13.08
PF	OPT31B	1.40	1.90	79.4	1.20	47	159	48	155	13.83
PP	OPT31C	0.69	0.32	75.0	1.10	58	297	58	283	13.25
DRB	OPT33/OPT33B	0.82				30	255	31	235	12.67
PP(2)	OPT456	0.77	1.50	81.0	3.15	39	75	39	75	15.80
SPRR	EAST OF SPRR					440	277	440	277	
TOTALS		15.53				1048		1057		

NOTE: VOLUMES DO NOT ACCOUNT FOR ARROWHEAD AND DENVER BASIN STORAGE

2) DRAINAGE AREA IS BETWEEN PRICE ROAD AND CHANDLER SPUR OF SPRR. AREA EAST OF CHANDLER SPRR MODELLED BY FRANZOY COREY IS APPROX. 25 SQ. MI. TOTAL BASIN AREA EAST OF PRICE ROAD IS 40 SQ. MI.

Without the intermediate step of lumping small subareas into larger sub-basins,the computation interval is limited to 4 or 5 minutes. For the 300-ordinate HEC-1 program, this allows a simulation for a maximum of 25 hours. This is not adequate to compute complete inflow hydrographs to the Price and Santan ROW since the time base of several basins is longer than 25 hours. Thus, the runoff volume computed for these basins is greater for the 12-minute runs as shown in Tables 3 and 4. Since runoff volumes determine the size of detention basins, it is important to be able to use longer time intervals for basin sizings.

With the longer time interval, runoff peak flows are generally lower. Therefore, for channel design purposes, the 5-minute peak flow values will be used.

4.0 RESULTS

4.1. *Price Expressway*

Runoff volumes and discharge peaks for the 100-year 24-hour design event are shown at key locations on Sheet 3. As noted on Sheet 3, the volumes were taken from the 12-minute interval linked HEC-1 runs which are included in Appendix D (Volume II). However, the peak discharges were taken from the 5-minute interval HEC-1 runs which are not included in Appendix D. The peak discharges for the 5-minute runs are typically somewhat larger than the peaks for the 12-minute runs. The hydrograph volumes at major concentration (or outflow) points along Price Road were used to design the offsite drainage system.

The characteristics at the Price Road concentration points are shown on Sheet 3 and listed in Table 5.

Table 5. 100-Year 24-Hour Volumes and Peak Flows At Concentration Points Along Price Road

Location	Identification	Volume acre-feet	Peak Flow cfs
Elliot To Warner	Outlet E	189	298
Warner to Ray	Outlet F	32	149
Ray to Chandler	Outlet G	216	975
Chandler to Pecos	Outlet H	732	805
	Total	1169	

4.2 *Santan Freeway*

Volumes and peak flows are also shown on Sheet 3 for the area between Price Road and I-10. Volumes and flows for the concentration points determined from the HEC-1 runs are listed in Table 6.

TABLE 6. 100-Year 24-Hour Volumes and Peak Flows at Concentration Points along the Santan Freeway

Location	Identification	Volume acre-feet	Peak Flow cfs
Price to McClintock	Outlet A	81	360
McClintock to Gila Drain	Outlet B	391	988
Gila Drain to I-10	Outlet C	134	670
	Total	606	

5.0 DETENTION POND REQUIREMENTS

A conceptual drainage plan for the offsite runoff was completed as part of this investigation. A detailed drainage plan for offsite and onsite drainage facilities will follow this report as a separate task.

As stated, the basic plan for handling offsite runoff is to contain the 100-year 24-hour runoff volume in detention basins which will be evacuated by pumping the impounded stormwater to the Salt River via the Carriage Lane Basin Outfall Pipe and the Price Tunnel. At the present time, the City of Chandler and ADOT can each nominally pump a maximum of 100 cfs into the Price Tunnel at the peak of the 100-year storm event for the basin. Considerably higher flows can be pumped during off-peak times.

The required size of a detention basin is basically dependent upon the inflow hydrograph volume and outflow pumping rate. The design inflow hydrographs were discussed in Section 4.0.

5.1 *Price Expressway*

For the purpose of this conceptual level design, the following assumptions were made:

1. A reasonable average pumping rate into the Carriage Lane Outfall is 250 cfs. This will allow complete evacuation in approximately 85 hours (100 hours from the beginning of the storm event).
2. Since over 40 percent of the overall offsite system cost is land acquisition, the basins were assumed to be deep (20 feet and 30 feet respectively) and as rectangular as possible to minimize surface area requirements.
3. Basin locations were consistent with the Dames and Moore design concept plan. On Price Road, basins were located on previously identified sites.
4. The basins do not consider multiple use options.
5. The overall system is not necessarily optimized with respect to pumping rates and pump station sizes versus number, location, and size of detention basins.

It should be noted that ADOT intends to explore multiple use options with the City of Chandler during the preliminary design that will follow this study. This will likely influence the final plan for both basins and pump stations. Recreational use of some basins will likely influence location and configuration, and therefore, cost.

This plan differs from the Dames and Moore design concept plan in that Basin F is eliminated, pump stations at Basins F and G are eliminated and the remaining basins will control the 100-year 24-hour volume as compared to the 100-year 6-hour volumes used by Dames and Moore. The difference between the 24-hour and 6-hour volumes is substantial. HDR compared the volume of runoff from a 100-year 24-hour event versus a 100-year 6-hour event on three sample sub-basins in the Price contributing area. On the average, the 24-hour event produced more than twice the volume of runoff than the 6-hour event. However, because the barriers in the contributing area divert and store stormwater runoff, the actual runoff volumes considered in this conceptual plan are nearly equivalent to the volumes considered in the Dames and Moore plan which was based on 100-year 6-hour volumes. Basin F is located in a subbasin that is quite small relative to the other subbasins. The stormwater generated in this area can be more efficiently collected in a channel and discharged into Basin E.

The offsite system consists of five large rectangular RCB collector channels, three basins, two evacuation pump stations, and one gravity outlet pipe. The collector channels are underground for the purpose of preserving ROW. Basin G can likely be evacuated at lower cost by gravity draining, to Basin H with a small outlet pipe.

During the preliminary design phase to follow, variations to this basic plan will be investigated. Some of these variations may be:

1. Using wide-open grass-lined channels for collector drains to provide continuous recreational or open space corridors between detention ponds.
2. Distributing basins in various ways to enhance joint utilization. Due to the flatness of the topography between Elliot Road and Chandler Boulevard, the location of basins is not critical with respect to gravity drainage.
3. Investigating the capacity of evacuation pump stations with respect to Carriage Lane Outfall capacity and considering current and future intergovernmental agreements and other constraints.

5.2 *Santan Freeway*

The conceptual plan for offsite drainage for the Santan Freeway between Price Road and I-10 is shown on Sheet 5. The plan consists of a lined collector channel, one basin and one pump station. This differs from the design concept plan in that two basins have been eliminated (Basins A and C). There is enough fall along the freeway alignment to efficiently drain flows to one basin located at the Gila Drain crossing which is near the low point and is therefore, the concentration point for the entire drainage area.

However, the detention basin is larger than the combined total in the original plan because the 100-year 24-hour volumes is more than twice the 100-year 6-hour volume used in the design concept plan, and barriers do not reduce runoff volumes as significantly as for the area east of Price Road.

The Santan Freeway east of the Price/Santan interchange is to be constructed in the somewhat distant future. Any stormwater generated between Price Road and the Consolidated Canal that will in the future be intercepted by this roadway has been included in the volume calculations for Basin H. If current development patterns and retention regulations are continued into the future, the runoff intercepted by this roadway will be smaller in volume than has been considered in this conceptual plan. Also, the roadway may be designed to pass or divert some flows out of the project area rather than concentrating additional volumes. Therefore, Basin H is more likely to be oversized rather than undersized with respect to future development.

6.0 Preliminary Cost Estimate

A cost estimate was prepared for the major components of the conceptual plans shown on Sheets 4 and 5. The cost estimate is included as Table 7 in this section. The cost estimate contains a listing of facilities included in the plan. Separate costs were calculated for the Price and Santan systems.

Collector drains were sized to carry the peak flows shown on Sheet 3 at reasonable slopes for the area using Manning's equation. Concrete box culvert costs were based on standard quantities. Pumping rates were developed considering the basin volumes in proportion to the total volume of the system and considering an allowable average outflow rate of 250 cfs to the Carriage Lane Head Structure. Heads were developed considering basin depths, friction losses in the outlet pipe and the hydraulic grade line at the outlet. Pump station costs were developed from cost figures in a pump station alternatives report done for ADOT (Boyle Engineering Corporation, 1986). The costs for elements required for an evacuation pump station were further analyzed and a station power versus cost curve developed.

Basin costs are primarily land acquisition and earthwork costs. Land was valued at $5 per square foot and earthwork was valued at $3 per cubic yard.

Other pipe costs were developed from various sources, including ADOT bid tabulations.

6.1 *Price Expressway*

Estimated total construction costs, including land acquisition for the major offsite drainage system components along Price Expressway shown on Sheet 4 are summarized as follows (See Table 7 for a detailed listing of quantities):

	Cost ($)	Percent
Pump Stations	$ 4,050,000	14
Pressure Pipe	3,876,000	13
Detention Ponds (Land)	12,240,000	43
Detention Ponds (Excavation)	4,317,000	15
Gravity Underdrains	385,000	1
Offsite Gravity Collectors	3,880,000	14
Sub-Total A:	$28,748,000	100
Other Items (15% of Sub-Total A)	4,312,000	
Sub-Total B:	$33,060,000	
Contingencies & Engineering (12% of Sub-Total B)	3,967,000	
Estimated Total Project Cost (Offsite Drainage System)	$37,027,000	

As the summary indicates, land acquisition costs represent more than 40 percent of the project cost. The 15% and 12% factors are applied to land cost as well as the structures costs because land acquisition is considered to be at least as unpredictable as construction costs.

6.2 *Santan Freeway* (From I-10 to Dobson Road)

Estimated costs for the Santan system are summarized below for the major components shown on Sheet 5 (See Table 7 for a detailed listing of quantities):

	Cost ($)	Percent
Pump Stations	$ 1,500,000	8
Pressure Pipe	2,024,000	11
Detention Ponds (Land)	6,512,000	36
Detention Ponds (Excavation)	2,952,000	16
Collector Drains/Culverts	1,956.000	11
Collector Drains (Land)	3.346.000	18
Sub-Total A:	$18,290,000	100
Other Items (15% of Sub-Total A)	2.744.000	
Sub-Total B:	$21,034,000	
Contingencies & Engineering (12% of Sub-Total A)	2.524.000	
Estimated Total Project Cost (Offsite Drainage System)	$23,558,000	

The total estimated project cost for the combined Price and Santan offsite systems is therefore approximately $60 million.

Table 7. Preliminary Cost Estimate - Base Case

OFFSITE SYSTEM- All Price/Santan Stormwater Pumped North to Salt River

Price Expressway

- **(1)Collector Drain System (Pipes)**
- **(2)Detention Ponds (E,G,H)**
- **(3)Pump Stations (E,H)/Pressure Pipe**

Santan Frwy

- **(1)Open Channel Collector Drain System**
- **(2)Detention Pond (B)**
- **(3)Pump Station B to Basin H**

QUANTITIES/CAPACITIES (PRICE ROAD)

A. Gravity Collector Drains

	Q	Ht (ft) or Diam (in)	Width (ft)	Length (ft)	Slope (ft/ft)	Area (sq.ft)	Velocity (fps)
Elliot - Basin E	175	78"	-	2200	0.001	33	5.3
Ray - Basin E	152	78"	-	7580	0.001	33	4.6
Ray - Basin G	116	66"	-	2600	0.001	24	4.8
Chandler - Basin G	917	10	10	1200	0.001	100	9.0
Chandler - Basin H	200	84"	-	4000	0.001	39	5.2

B. Detention Ponds

Detention Basin	Area (Acres)	Storage Volume (Ac-ft)	Depth (ft) (Inc.4'FB)	Dewater Q (cfs)
Basin E	15.5	150	20	30
Basin G	17.4	182	20	30
Basin H	23.3	370	30	105
Total	56.2	702		165

C. Pump Stations

Pump Station	Flow (cfs)	Static H (ft)	Length (ft)	Pipe Diam (ft)	Velocity (ft/sec)	Sf	Headloss (ft)	Head (ft)	Pump Eff.	Power (hp)
Basin H	190	31	17600	6.0	6.72	0.00185	32.6	64	.8	1725
Basin E	250	22	5200	6.0	8.84	0.00330	17.2	39	.8	1382

COST ESTIMATE (PRICE ROAD)

D. Gravity Collector Drains

	Ht (ft) or Diam (in)	Width (ft)	Length (ft)	Unit Cost ($/lf)	Cost ($)
Elliot - Basin E	78"	-	2200	210	462,000
Ray - Basin E	78"	-	7580	210	1,591,800
Ray - Basin G	66"	-	2600	160	416,000
Chandler - Basin G	10	10	1200	375	450,000
Chandler - Basin H	84"	-	4000	240	960,000
Total					3,879,800

E. Detention Ponds

Detention Basin	Area (Acres)	Excav. Volume (Ac-ft) (4' FB)	Land		Excavation		Total Cost ($)
			Unit Cost ($/sq.ft)	Land Cost ($)	Unit Cost ($/cu.yd.)	Cost ($)	
Basin E	15.5	205	5	3,375,900	3	992,200	4,368,100
Basin G	17.4	239	5	3,789,720	3	1,156,760	4,946,480
Basin H	23.3	448	5	5,074,740	3	2,168,320	7,243,060
Total	56.2	892		12,240,360		4,317,280	16,557,640

F. Gravity Outlet Pipe

Item	Length (ft)	Q (cfs)	Diameter (ft)	V (ft/sec)	Unit Cost ($/ft)	Cost ($)
Basin G to Basin H	5660	30	3	4.2	68	384,880
Total						384,880

G. Pump Station Cost

Pump Station	Power (hp)	Length (ft)	Pipe Diam (ft)	Pipeline Unit Cost ($/ft)	Pipeline Cost ($)	Pump Sta. Cost ($)	Total Cost ($)
Basin H	1725	17600	6.0	170	2,992,000	2,200,000	5,192,000
Basin E	1382	5200	6.0	170	884,000	1,850,000	2,730,000
Total					3,876,000	4,050,000	7,920,000

H. Cost Summary (Price Road)

	Cost ($)	Percent
Pump Stations	4,050,000	14.1%
Pressure Pipe	3,876,000	13.5%
Detention Ponds (Land)	12,240,360	42.5%
Detention Ponds (Excavation)	4,317,280	15.0%
Gravity Outlet Pipe	384,880	1.3%
Gravity Collector Drains	3,879,800	13.6%
Total Construction Cost	28,748,320	100.0%

QUANTITIES/CAPACITIES (SANTAN FRWY)

I. Gravity Collector Drain Channels

	Q (cfs)	Length (ft.)	B (ft)	SS (h:v)	dw (ft)	FB (ft)	Area (sq.ft.)	Excav (cu.yd.)	Lining A. (sq.yd.)
I-10 to 56th St.	135	2,500	4.0	2:1	2.3	1.7	48	4,444	6,080
56th St. to Basin B	697	3,200	12.0	2:1	4.0	2.0	144	17,067	10,627
Price to McClintock	99	5,280	2.0	2:1	2.5	1.5	40	7,822	11,668
McClintock to Rural	466	5,280	9.5	2:1	4.0	2.0	129	25,227	16,068
Rural to Kyrene	599	5,280	10.0	2:1	4.0	2.0	132	25,813	16,361
Kyrene to Basin B	1,164	1,250	10.0	2:1	5.0	2.5	188	6,681	6,047
							Total	87,054	66,851

J. Culverts

	Q (cfs)	Length (ft)	BBL's	Size H	W	Conc. (cy/ft)	Conc.Vol. (cu.yd.)
56th St.	350	100	1	5	10	1.023	102.3
Gila Drain and RR	697	150	2	5	10	2.033	305.0
McClintock	404	100	2	4	10	1.940	194.0
Kyrene	1,051	100	3	5	10	2.932	293.2
					Total		894.5

Note: 1. Estimated from peak flow combinations

2. Assumed inlet control with hw/d = 1

K. Gravity Collector Drain Channels - Land

Item	Length (ft)	Width (ft)	Area (sf)
I-10 to 56th Street	2500	20	50,000
56th Street to Basin B	3200	36	115,200
Price to McClintock	5280	18	95,000
McClintock to Rural	5280	34	179,500
Rural to Kyrene	5280	34	179,500
Kyrene to Basin B	1250	40	50,000
Total			669,200

L. Detention Ponds

Detention Basin	Area (Acres)	Depth (ft) (Inc.4'FB)	Storage Volume (Ac-ft)	Dewater Q (cfs)
Basin B	29.9	30	507	85.0
Total	29.9		507	85.0

M. Pump Station & Pipeline

Pump Station	Flow (cfs)	Static H (ft)	Length (ft)	Pipe Diam (ft)	Velocity (ft/sec)	Sf (ft/ft)	Headloss (ft)	Head (ft)	Pump eff.	Power (hp)
Basin B	85	54	17,600	4.5	5.34	0.00165	29.0	83	0.8	1000

COST ESTIMATE (SANTAN FRWY)

N. Collector Drain Channels / Culverts / Land

Item	Excavation ($/cu.yd)	Lining ($/sq.yd)	Culverts ($/cy)	Land ($/sf)	Excavation ($)	Lining ($)	Culverts ($)	Land ($)	Total ($)
I thru K	3	22	250	5	261,200	1,470,700	223,600	3,346,000	5,301,500

O. Detention Pond

Detention Basin	Area (Acres)	Excav. Volume (Ac-ft) (4' fb)	Land		Excavation		Total Cost ($)
			Unit Cost ($/sq.ft)	Land Cost ($)	Unit Cost ($/cu.yd.)	Cost ($)	
Basin B	29.9	610	5	6,512,200	3	2,952,400	9,464,600

P. Pump Station

Pump Station	Power (hp)	Length (ft)	Pipe Diam (ft)	Pipeline Unit Cost ($/ft)	Pipeline Cost ($)	Pump Sta. Cost ($)	Total Cost ($)
Basin B	1000	17600	4.5	115	2,024,000	1,500,000	3,524,000

Q. Cost Summary (Santan Frwy)

	Cost ($)	Percent
Pump Stations	1,500,000	8.2%
Pressure Pipe	2,024,000	11.1%
Detention Ponds (Land)	6,512,200	35.6%
Detention Ponds (Excavation)	2,952,400	16.1%
Collector Drains/Culverts/Land	5,301,500	29.0%
Total Construction Cost	18,290,100	100.0%

COST ESTIMATE COMBINED

R. Total Costs

	Cost ($)
Price Expressway	28,748,000
Santan Freeway	18,290,000
Sub-Total A:	47,038,000
Other Items (15% of Sub-Total A)	7,056,000
Sub-Total B:	54,094,000
Contingencies of Engr. (12% of Sub-total B)	6,491,000
Total Project Cost	60,585,000

7.0 REFERENCES

1. Dames & Moore, Drainage Concepts, Price Expressway, Technical Memorandum No. 4 and Drainage Concepts, Southeast Loop Highway (Santan Freeway), Technical Memorandum No. 1, Arizona Department of Transportation, July 19, 1988.
2. U.S. Army Corps of Engineers, Los Angeles District, Gila Floodway Summary Report for Flood Control, September, 1977.
3. U.S. Army Corps of Engineers, HEC-1 Flood Hydrograph Package, 2 August 1988, Version.
4. Arizona Department of Transportation, Hydrologic Design for Highway Drainage in Arizona, March 1969.
5. DeLeuw, Cather & Company, Stanley D. Polasik, Personal Communication, July 1989.
6. City of Tempe, Surface Water Hydrology Report, Evans, Kuhn & Associates, Inc., 1983.
7. Flood Control District of Maricopa County, Flood Insurance Study, Appendix A Hydrologic Analyses, (Preliminary), Franzoy-Corey, March, 1989.
8. Soil Conservation Service, Soil Survey Eastern Maricopa and Northern Pinal Counties Area, Arizona, November, 1977.
9. National Oceanic and Atmospheric Administration, Depth Area Ratios In The Semi-Arid Southwest United States, Technical Memorandum NWS Hydro-40, August, 1984.
10. City of Phoenix, Storm Drain Design Manual, Storm Drains with Paving of Major Streets, July 1987 and Subdivision Drainage Design, September, 1985.
11. City of Chandler, Arizona, Stormwater Management Master Plan, Camp Dresser & McKee, Inc., October, 1986.
12. Soil Conservation Service, Urban Hydrology for Small Watersheds, Technical Release 55, June, 1986.
13. Boyle Engineering Corporation, Storm Drainage Pump Station Study, Outer Loop Freeway, August, 1986.
14. Coe & Van Loo, Gila Drain Preliminary Design Report, Flood Control District of Maricopa County, April, 1979.

Volume I

Appendix A

Hydrology Study

Note on Areal Reductions

HDR Internal Memorandum

To: Arizona Dept. of Transportation

Date: June 29, 1989

From: Jerry Zovne, Steve Miller

Subject: Price Expressway Drainage

Attention: Ray Jordan

This is in response to your request on June 27, 1989 that we summarize our findings with regard to areal precipitation reduction factors. To date we have been using NOAA Atlas 2, (i.e. TP40) reduction factors which results in a 4% reduction of the 100-year/24-hour rainfall for our 40-sq.mi. watershed.

However NWS Hydro-40 indicates a reduction of 11% for a 40-sq. mi. area in Central Arizona (Areas A and C) which includes Phoenix. (See map attached).

Our first cut at determining effects was to attempt to achieve the 11% reduction by using a large area (400 sq. mi.) on the PH card (TP40) which results in a 9% reduction in rainfall, and runoff volume reductions of 24 to 32% for the sample watersheds. (See "Sample Hydrographs" attached).

To obtain the 11% (NWS Hydro-40) reductions we plotted the longer duration rainfalls on log-log paper and extended the curve as a straight line. (See attached).

The new "NWS HYDRO-40" PH card resulted in even greater reductions in runoff volumes. The three rainfall reduction scenarios are compared for two sample subareas in the last figure (4%,9%, and 11%).

/jm/aak

HDR

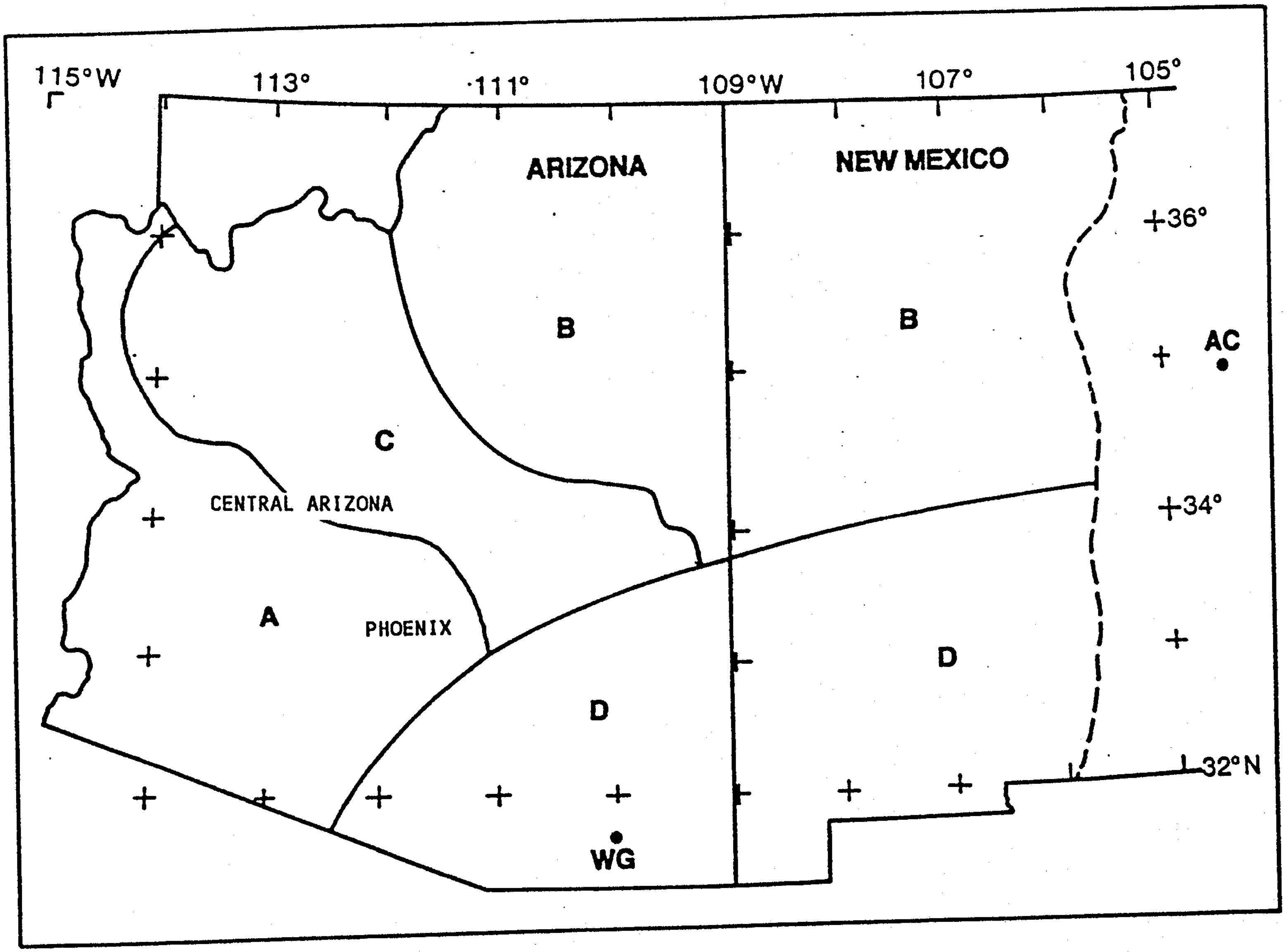
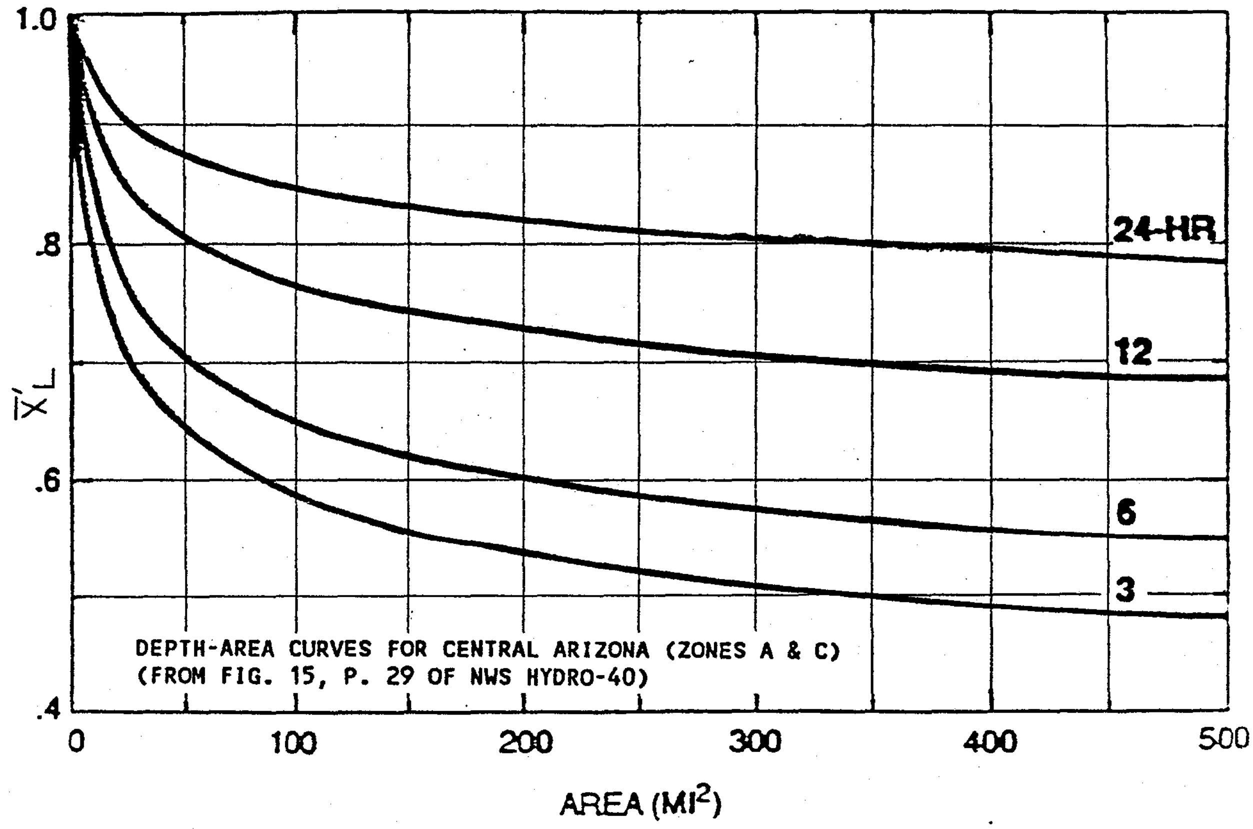
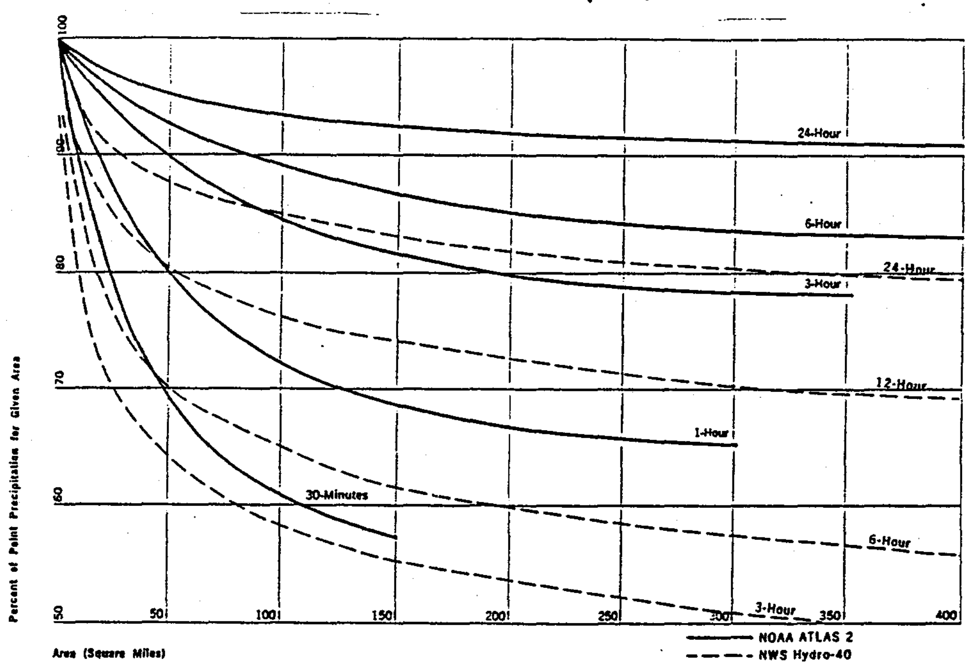


Figure 13.—Map of study area and depth-area zones.



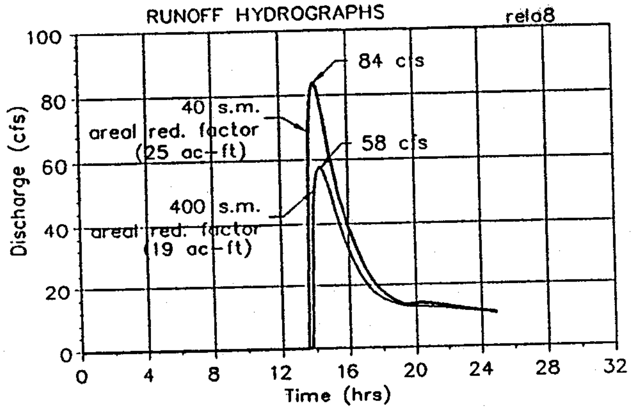


Depth-Area curves.

COMPARISON OF NOAA ATLAS 2 (TP40) AND NWS HYDRO-40 (FIG. 15)

		DURATION							
		HOURS						MINS.	
		24	12	6	3	2	1	15	5
(PERCENT OF POINT PRECIPITATION FOR	TP40	96	-	94	91.5	-	83	-	- (%)
40 S.M. WATERSHED)	NWS HYDRO-40	89	82	72.5	67	-	-	-	- (%)

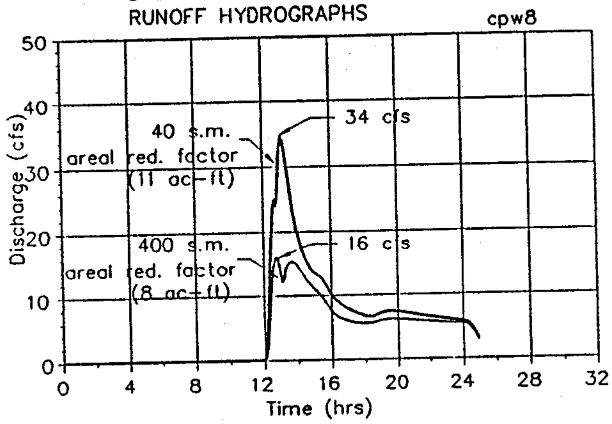
BASIN "ELA" PTS. SECS. 10 AND 15
RUNOFF HYDROGRAPHS



TP40
AREAL REDUCTION FACTOR

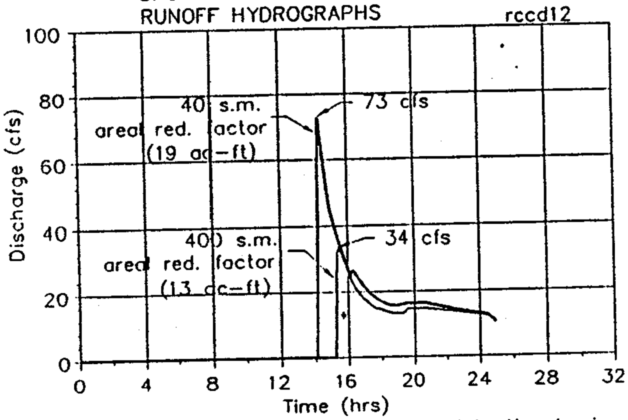
	0	40	400
Rainfall (in.) (100 yr–24 hr)	3.70	3.55	3.37
Runoff volume (ac–ft)	–	25	19 (–24%)

BASIN "PW" SEC. 18
RUNOFF HYDROGRAPHS



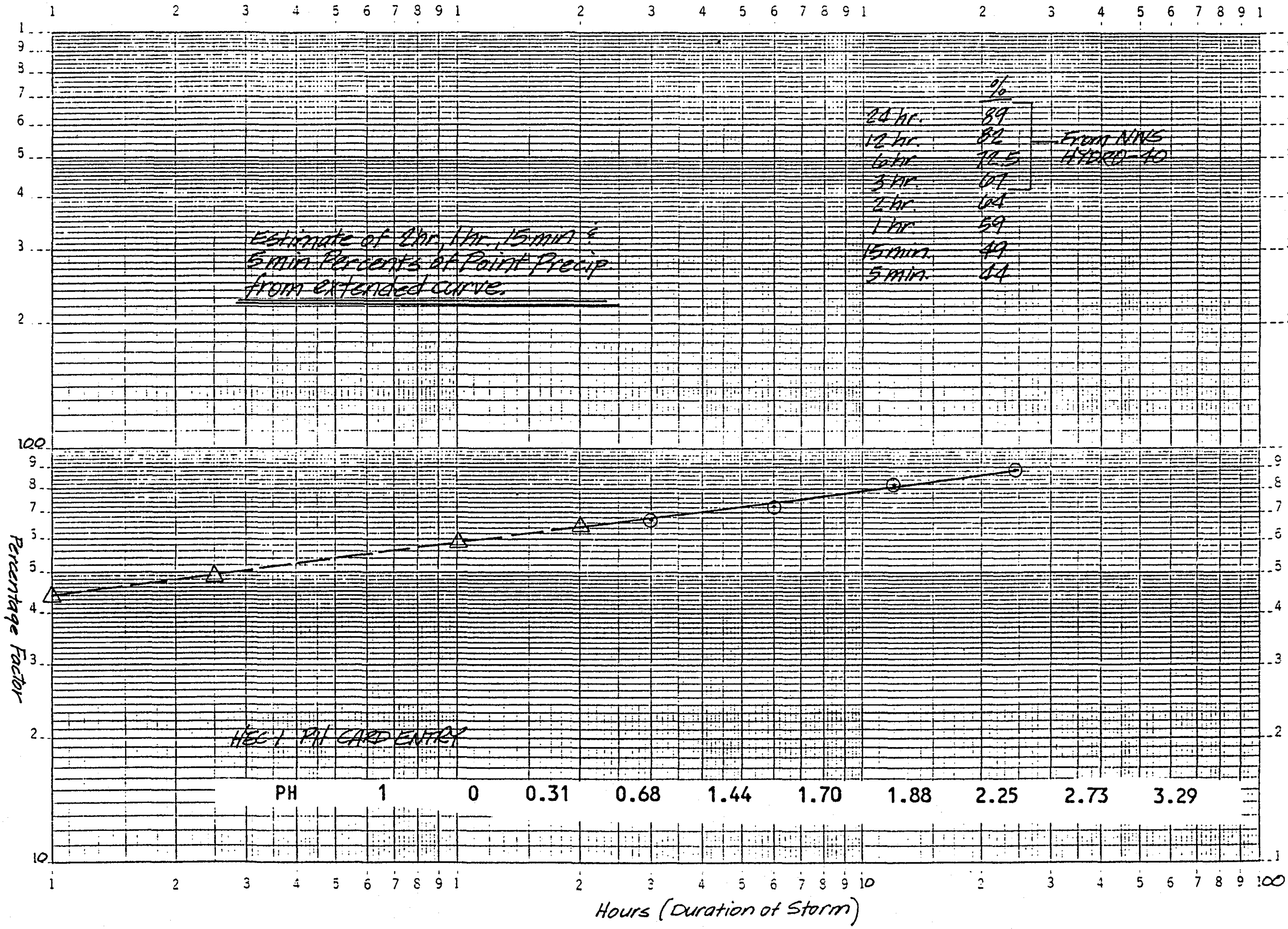
	0	40	400
Rainfall (in.) (100 yr–24 hr)	3.70	3.55	3.37
Runoff volume (ac–ft)	–	11	8 (–27%)

BASIN "CD" SEC. 29
RUNOFF HYDROGRAPHS

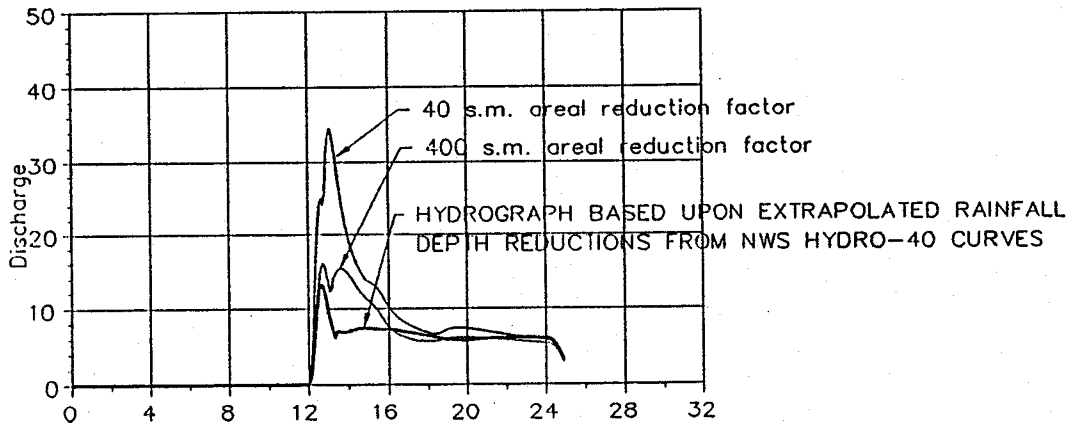


*influenced by existing detention basins

	0	40	400
Rainfall (in.) (100 yr–24 hr)	3.70	3.55	3.37
Runoff volume (ac–ft)	–	19	13 (–32%)

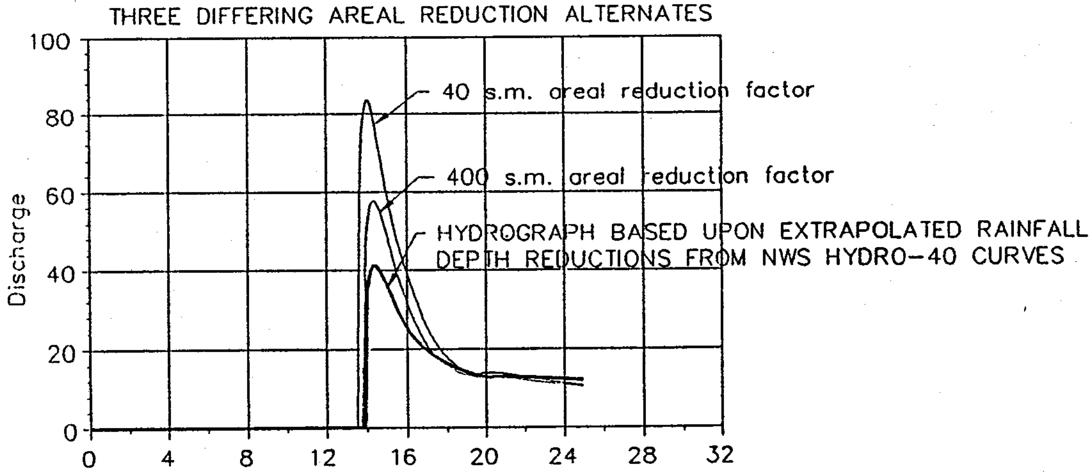

Estimate of 2hr, 1hr, 15min & 5min Percents of Point Precip. from extended curve.
%
24 hr. 89
12 hr. 82
6 hr 72.5
3 hr. 67
2 hr. 64
1 hr. 59
15 min. 49
5 min. 44
From NWS HYDRO-40
Percentage Factor
HEC 1 PH CARD ENTRY
PH 1 0 0.31 0.68 1.44 1.70 1.88 2.25 2.73 3.29
Hours (Duration of Storm)

BASIN "PW" HYDROGRAPH COMPARISON
FOR THREE DIFFERING AREAL REDUCTION ALTERNATES



	40	400	Extrap.	
Rainfall (in.)	3.55	3.37	3.29	(100 yr-24 hr)
Runoff volume (ac-ft)	11	8	7	
Peak Q (cfs)	34	16	13	

BASIN "ELA" HYDROGRAPH COMPARISON FOR
THREE DIFFERING AREAL REDUCTION ALTERNATES



	40	400	Extrap.	
Rainfall (in.)	3.55	3.37	3.29	(100 yr-24 hr)
Runoff volume (ac-ft)	25	19	17	
Peak Q (cfs)	84	58	41	

Volume I

Appendix B

Hydrology Study

Note on Agricultural Area Modeling

HDR Internal Memorandum

To: Arizona Department of Transportation

Date: June 29, 1989

From: Jerry Zovne, Steve Miller

Subject: Price Expressway Drainage

Attention: Ray Jordan and Rich DeBoer

Attached is HDR's response to your request of June 21, 1989, to summarize our approach to agricultural area modeling. We have included some test HEC-1 runs for your review. This section will be included in our Initial Hydrology Report. Please advise us on any comments or concerns.

/jm/aaj

HDR

AGRICULTURAL AREA MODELING APPROACH

HDR Engineering, Inc. (HDR) is developing the offsite hydrology for the future Price Expressway and a portion of the Santan Freeway under a GEC contract with the Arizona Department of Transportation. The drainage area consists of urban residential and commercial development with significant onsite retention, irrigated farmlands and very little undeveloped land. The drainage area has a significant percentage of irrigated land which potentially produces more runoff than urban land developed under the current retention policies. HDR has developed the following approach to modeling irrigated land. While there appears to be significant guidance in the literature for urban developed land, there is less guidance for irrigated land. HDR's approach was therefore developed using commonly accepted procedures, supplemented by field observations and informal discussion with others who are dealing with similar problems.

Previous reports dealing with irrigated land vary in opinions about the magnitude of important parameters in the runoff modeling process. The three parameters are the Antecedent Moisture Condition (AMC), the Initial Abstraction (IA), and the Infiltration Rate (F). HDR is using the U.S. Soil Conservation Service (SCS) methodology as represented in the U.S. Army Corps of Engineers HEC-1 Flood Hydrograph Package. The HEC-1 package computes the IA and F automatically for a specified Curve Number (CN), unless the user specifies an IA. HDR uses this option to account for additional abstractions for an irrigated field. Curve numbers correspond to an AMC-II antecedent moisture condition.

The agricultural irrigation practices in Eastern Maricopa County are typified by flat sloped, furrowed parcels of one quarter to one full section. Irrigation water is delivered to the upslope end of the field and is entrapped at the downslope end of the field by earthen berms. Irrigation water is a valuable resource in the arid southwest, and runoff of irrigation water prevented. Current farming practice includes zero-sloped fields to insure maximum utilization of irrigation waters.

Application rates for irrigation waters are typically 3-4 inches applied uniformly over a 12 hour period. This volume of water is equal to the 100-year 24-hour rainfall volume, though the application rate is considerably less intense. The soils in the study area have permeability rates between a range of 0.2-0.6 inches per hour for the clay loams to a range of 2.0-6.0 inches per hour for the gravelly loams. Peak intensity rainfall for the 100-year 24-hour storm can exceed 8.5 inches per hour (5 minute duration) causing runoff from even the most permeable soils.

The irrigated farmlands have parcel slopes of less than or equal to 0.002 feet-per-foot. Furrows are six to eight inches deep with tailwater berms on the downslope end of the fields varying from six to twelve inches in height.

Irrigated parcels range in size from one quarter-section to one full section, but are typically one quarter-section in size. A quarter-section field theoretically has a 2640-foot long furrow length and a width of 2640 feet.

The soils in this area are consistently clay loams, sandy loams, or gravelly loams. The predominant soil type is hydrologic soil group B with an infiltration rate range of 0.6 to 2.0 inches per hour. A conservative runoff coefficient for straight row crops on a B soil is CN=81 which results in a fifty percent loss or 1.85 inches on 100-year 24-hour, 3.7 inch storm (ignoring storage and tailwater infiltration).

Urban Hydrology for Small Watersheds, TR-55, is the accepted guide for selecting runoff curve numbers for the SCS Method. Frequently no adjustment is made to the aforementioned curve number to reflect the additional abstractions that are likely to occur because of the flatness of these fields and the storage effects of tailwater berms. HDR accounts for these effects in the following way:

To estimate the total losses from the 100-year 24-hour storm the following assumptions were made for a typical quarter-section field with slope of 0.002 and furrow length of 2640 feet. The tailwater berm height of 8 inches is considered typical, the average ponding time in the tailwater pond is assumed to be 3 hours, and the average infiltration rate is assumed to be 0.60 inches per hour.

Tailwater pond spread = (8"/(12"ft.))/0.002'/ft.
= 333 ft.

Storage volume per foot of field = (8"/(12"/ft.))x333'x0.5
= 111 ft ^{3}/ft.

Runoff stored (depth averaged over field) = ((111 ft.^{3}/ft.)/2640ft.)x(12"/ft.)
= 0.505 inches

Infiltration (depth = 333 ft. x (3hrs x 0.60"/hr)/2640 ft.
averaged over field) = 0.22 inches

TOTAL LOSSES DURING 100-YEAR 24-HOUR STORM EVENT

Initial losses + Storage losses + Infiltration losses =
1.85 inches + 0.50 inches + 0.22 inches =

TOTAL LOSSES = 2.57 inches

HDR approached modeling of the agricultural areas by using a typical curve number for straight row crops and utilizing the initial abstraction (IA) parameter to account for the excess storage and infiltration amounts realized for the irrigation practices used in the study area. To determine the proper IA value to use to account for the 2.5 inches of losses, a sensitivity analysis was performed for a range of curve numbers for different soil types and crop conditions. The total losses for IA's of 1.00, 1.50, and 2.00 inches were calculated. An initial abstraction of 1.5 inches resulted in about 2.5 inches of losses for curve numbers between 81 and 85. Table 1 summarizes the results from the sensitivity study.

Table 1 Total Losses by CN and IA

Curve Number	Initial Abstraction Inches	Total Losses Inches
71	1.0	2.57
71	1.5	2.86
71	2.0	3.12
78	1.0	2.34
78	1.5	2.69
78	2.0	3.00
81	1.0	2.22
81	1.5	2.59
81	2.0	2.93
83	1.0	2.14
83	1.5	2.52
83	2.0	2.88
85	1.0	2.04
85	1.5	2.45
85	2.0	2.83

The Phoenix office of SCS is presently using runoff curve numbers between 50 and 65 to model runoff from agricultural areas in Maricopa County and the surrounding vicinity (1). A curve number of 50 is used when the irrigated field has been regraded to zero percent slope and a curve number of 65 represents fields with some longitudinal slope. A curve number of 50 represents a total loss of 3.45 inches of rainfall and the 65 represents a loss of 2.84 inches. With respect to HDR's modeling approach, this could be interpreted as an assumption of dryer antecedent moisture conditions. At the present, HDR is using AMC-II Condition which is considered normal for many parts of the country, but wetter than normal for Phoenix. This accounts for the good possibility that the fields were irrigated within a week prior to the design storm.

Another observation is that the irrigated fields here may be better represented by contoured and terraced row crops in good condition, which for B soils, AMC-II, yields a curve number of 71. HEC-1 yields a loss of 2.51 inches for the design storm (0.2S IA) for this case, which is equivalent to HDR's assumption of CN = 81 and IA = 1.50 inches.

Neither TR-55 nor its parent publication NEH-4 (2) address runoff from irrigated fields. HDR does not believe that this is sufficient reason to make the "worst case" assumption of straight row crops in poor condition (i.e. CN=81 for B soil, AMC-II) without accounting in some way for increased infiltration due to flat slopes and tailwater ponding. HDR's use of IA = 1.50 in. results in a 0.5 in. reduction in runoff for the design storm, which amounts to 26 acre-feet reduction per section. This is significant considering that the Price/Santan Watershed contains about 25 square miles of irrigated land.

1. Harry Milsaps, Personal Communication, Soil Conservation Service, May, 1989.

2. National Engineering Handbook, Section 4, Hydrology, USDA, Soil Conservation Service, 1964.

JJZ/jm/aai

Volume I

Appendix C

Hydrology Study

Note on Flood Control District of Maricopa County Model

HYDROLOGY STUDY
FOR FREEWAYS IN THE SOUTHEAST VALLEY

GILBERT-CHANDLER FLOOD INSURANCE STUDY MODIFICATION

AUGUST 1989

HDR ENGINEERING/ARIZONA DEPARTMENT OF TRANSPORTATION

CONTRACT NUMBER LA-89-08

Prepared by
FRANZOY COREY ENGINEERING COMPANY
7776 Pointe Parkway South, Suite 290
Phoenix, Arizona 85044
(602) 438-2200

Registered Professional Engineer (Civil)
CERTIFICATE NO.
RUSSELL L.
MIRACLE
Date Signed 8/23/89
ARIZONA, U.S.A.

GILBERT-CHANDLER FLOOD INSURANCE STUDY MODIFICATION

1. INTRODUCTION

In 1987 the Flood Control District of Maricopa County (FCDMC) contracted with Franzoy Corey Engineering Company (FC) to delineate the flood zones along hydraulic barriers in the area of the cities of Gilbert and Chandler southeast of Phoenix, Arizona. The study was conducted following Federal Emergency Management Agency (FEMA) standards. The purpose of the Gilbert-Chandler Flood Insurance Study was to map the flood-prone areas south of the Superstition Freeway (State Route 360) along the Eastern and Consolidated canals and the Southern Pacific Railroad which splits at Baseline and proceeds southeast along Rittenhouse Road and south along Arizona Avenue. The southern border of the study area is the county line (Hunt Highway). The Roosevelt Water Conservation District (RWCD) canal and East Maricopa County Floodway form the eastern boundary for the study area. A portion of the study area extends into the Dobson Ranch area of the City of Mesa bounded by the Tempe Canal on the west and the Western Canal on the south. It was assumed the East Maricopa County Floodway will intercept all flows from the east. The area was divided up into 45 separate subareas with each having a number of subbasins contributing to the total runoff. The subbasins generally do not exceed one-half of a square mile in area and are of one land-use classification. The HEC-1 computer model (Corps of Engineers, 1988) was used to determine runoff flows and volumes. The methodologies and assumptions for the hydrology for the Gilbert-Chandler study are attached to this report as appendix A.

The Arizona Department of Transportation (ADOT) is developing a major transportation network in the Phoenix metropolitan area. ADOT has contracted with HDR

Engineering, Inc. (HDR) as general engineering consultant for services on the Price Expressway and portions of the Southeast Loop Freeway. These freeways will intercept flood flows from an area which includes the Gilbert-Chandler study area. Areas west of the Gilbert-Chandler study area are being modeled by HDR. After HDR reviewed the methodologies used in the Gilbert-Chandler Flood Insurance study, Franzoy Corey was requested to modify hydrologic parameters used in the Gilbert-Chandler study to be more consistent with other studies conducted by ADOT for other areas along the freeway network. With these modifications, the results will be used as input to the HDR model. The two models combined will predict runoff from the drainage areas contributing to flow along the freeway alignment on the Price Expressway and portions of the Southeast Loop Freeway.

2. MODIFICATIONS TO THE GILBERT-CHANDLER AREA - HEC-1 MODEL

The purpose of the Flood Insurance Study (FIS) in the Gilbert-Chandler area was to determine the extent of flooding along the hydrologic barriers in the area. Flood zones along the barriers will be mapped and used for floodplain management decisions.

Hydrologic parameters selected to determine the extent of flooding were generally conservative.

HDR is responsible for sizing the drainage features of the Price Road and southeast loop freeways. In order to be consistent with the HDR hydrologic modeling of the area contributing to the flow crossing the freeway alignment, HDR requested the flowing changes be made to the FIS HEC-1 model: (1) use the storm distribution recommended by ADOT, (2) apply areal reduction to precipitation volumes and, (3) include an initial abstraction in the loss computation for the areas designated as agricultural land use. In the following sections of this chapter each of the model modifications will be discussed.

2.1 Storm Distribution

The Gilbert-Chandler flood insurance study uses the Soil Conservation Service (SCS) Type II storm distribution as provided by FCDMC. Most ADOT studies use the Phoenix distribution or a hypothetical storm distribution procedure described in the manual Hydrologic Design for Highway Drainage in Arizona, (ADOT, 1975). The method described in this publication uses National Oceanic and Atmospheric Administration (NOAA) precipitation maps and allows the development of a table showing rainfall duration depth values. In the east valley FCDMC recommended a storm producing 3.8 inches of precipitation in 24 hours (100-year return frequency). This total amount of rainfall was used to develop the input data for the PH card. Table 1 shows the duration-depth values used in the modified HEC-1 model.

TABLE 1
Depth-Duration Values
August 1989

Duration (minutes)	Depth (inches)
5	0.66
15	1.30
60	2.28
120	2.53
180	2.69
360	3.00
720	3.34
1440	3.80

Source: Franzoy Corey

2.2 Areal Reduction

Areal reduction is adjusting a point precipitation to the area of the subbasin (COE HEC-1 Manual, 1981). The areal reduction number is related to the area over which the storm is centered. The contributing area impacting the Price Expressway was determined from the land use/subbasin maps. This includes areas where overtopping of the canal barriers contributes to downstream flooding. The floodflows eventually reach the Southern Pacific Railroad which parallels Arizona Avenue. The railroad is the western boundary for the Gilbert-Chandler flood study. The railroad barrier causes ponding upstream, and outflow is through a series of culverts and by overtopping of the railroad due to the inadequate capacity of the culverts. The total size of this contributing area is combined with an area west of the railroad which was modeled by HDR. The Gilbert-Chandler flood study contributing area is 24.70 square miles. The HDR study contributing area is 14.75 square miles for a total of 39.45 square miles. An areal reduction factor of 40 was inserted on field 2 of the PH card in the HEC-1 input file.

2.3 Initial Abstraction

HDR expected that more retention capacity was available in the agricultural areas and that future development in the same areas would also reduce runoff. The Gilbert-Chandler flood study used an initial abstraction computed by the HEC-1 computer program for the SCS curve number technique. An initial abstraction of 1.5 inches, as determined by HDR, was added to the LS card for subbasins with agricultural land-use.

3. RESULTS

The modified HEC-1 model predicts floodflows crossing the Southern Pacific Railroad at a location near Chandler Boulevard and some lesser flows near Elliot Road. Also floodflows concentrate at the intersection of the Tempe Canal (at Price Road) and the Western Canal (lateral 9.5). There are several locations along the railroad alignment from the Superstition Freeway to the Maricopa County line where water ponds and is released through wooden bridges or culverts. The largest contributing area drains to an area near Chandler Boulevard. Runoff from the area bounded by Ray Road on the north to Pecos Road on the south ponds against the railroad and eventually flows toward Chandler Boulevard. There is a small retention basin about one-quarter mile north of Chandler Boulevard where the flood flows are initially concentrated. The basin and the small culvert under the railroad near the center of the basin are soon flooded and the basin then overtops the railroad in the same vicinity. Runoff between Ray Road and Warner Road is contained in a ponded area one-half mile south of Warner Road. A railroad culvert between Elliot Road and Warner Road drains the area at that location. At the confluence of the Tempe and Western canals the Carriage Lane retention basin overflows toward the west or southwest.

4. DATA SUMMARY AND DESCRIPTION

Hydrographs at locations impacting the proposed freeways were written to files and are provided separate on a 5-1/4 inch diskette. The drainage map also shows land-use and concentration points in each subarea. A printout of the complete computer model summary along with the input listed and a flow routing diagram are provided separate from this report. The methodologies used for the development of the model may be found in appendix A, with modifications described herein.

5. REFERENCES

<u>HEC-1 Flood Hydrograph Package Manual</u>. September 1981 (revised March 1987).

<u>Hydrologic Design for Highway Drainage in Arizona</u>. December 1, 1969 (revised March 1969). Jencsok, Eugene I.

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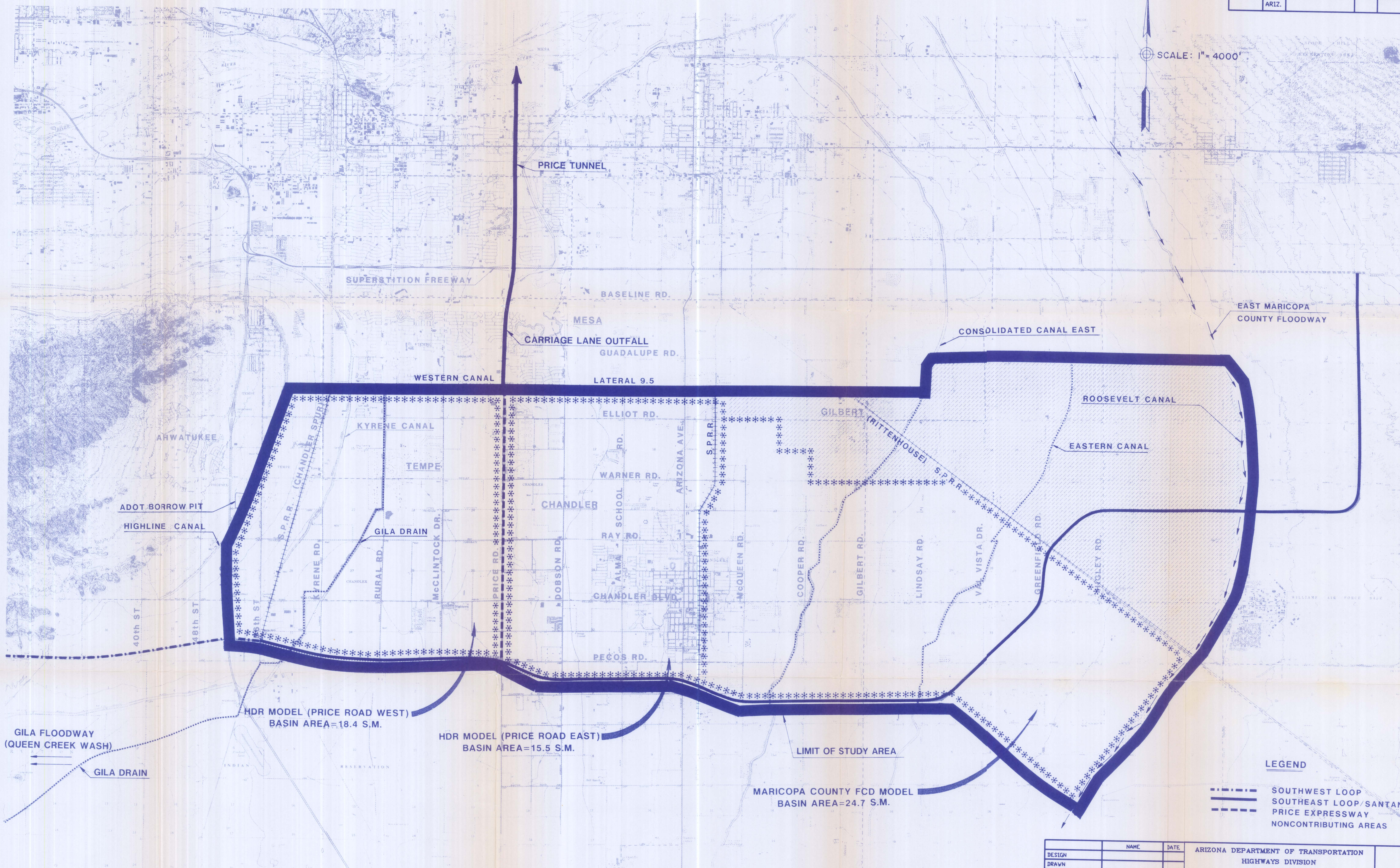
F.H.W.A. REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	AS BUILT
	ARIZ.				



LEGEND

- SOUTHWEST LOOP
- SOUTHEAST LOOP/SANTAN
- PRICE EXPRESSWAY
- NONCONTRIBUTING AREAS

SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

	NAME	DATE
DESIGN		
DRAWN		
CHECKED		

HDR Engineering, Inc.

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION
HIGHWAY PLANS

HYDROLOGY STUDY
PROJECT LOCATION MAP

ROUTE	MILEPOST	LOCATION

SHEET 1 OF 5

H-2222-01D

OF

F.H.W.A. REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	AS BUILT
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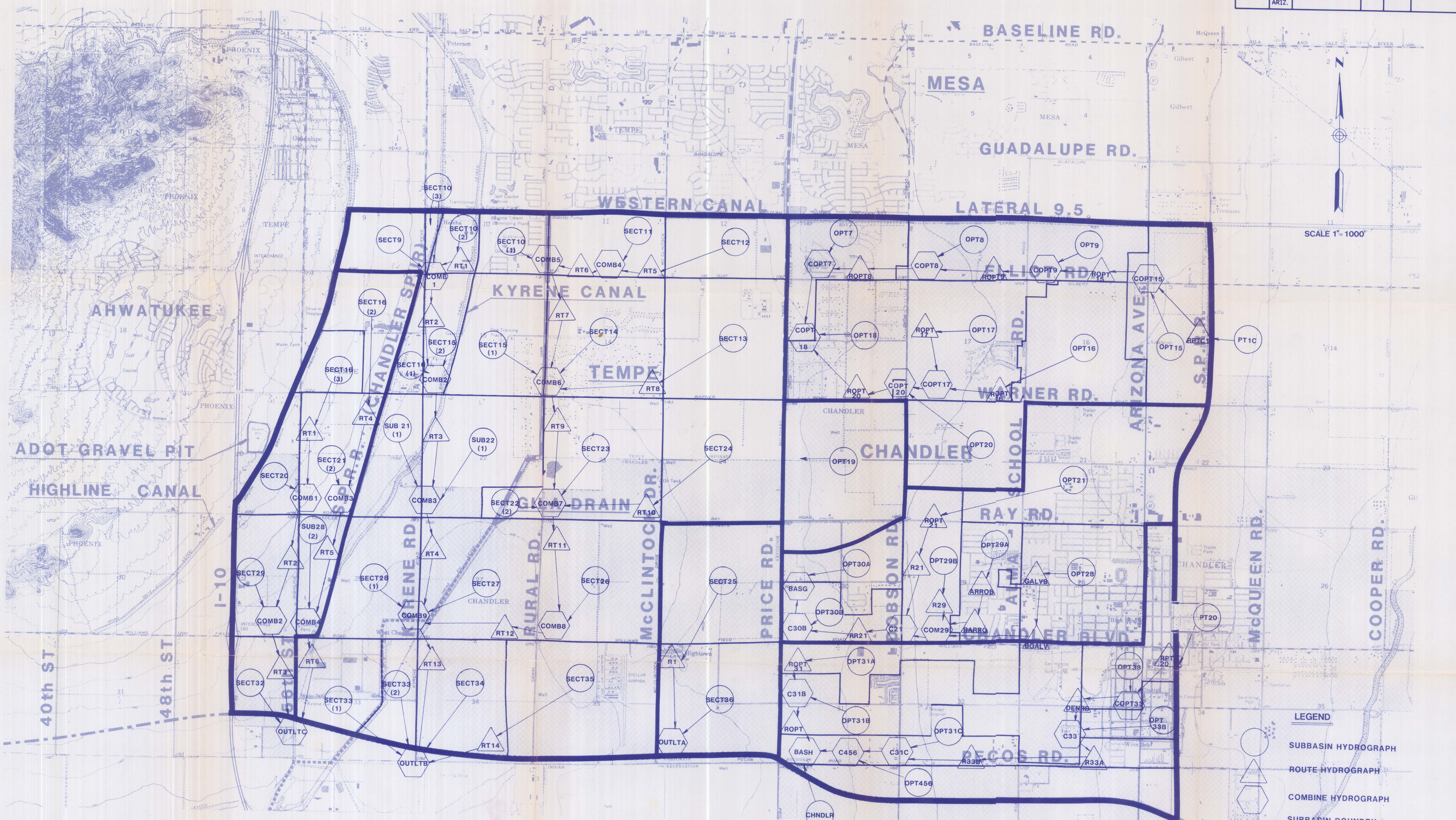

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CHECKED		

HDR Engineering, Inc.

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION
HIGHWAY PLANS

HYDROLOGY STUDY
DRAINAGE AREA MAP
AND HEC-1 MODEL SCHEMATIC

ROUTE | MILEPOST | LOCATION

SHEET 2 OF 5

H-2222-01D

___ OF ___

SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

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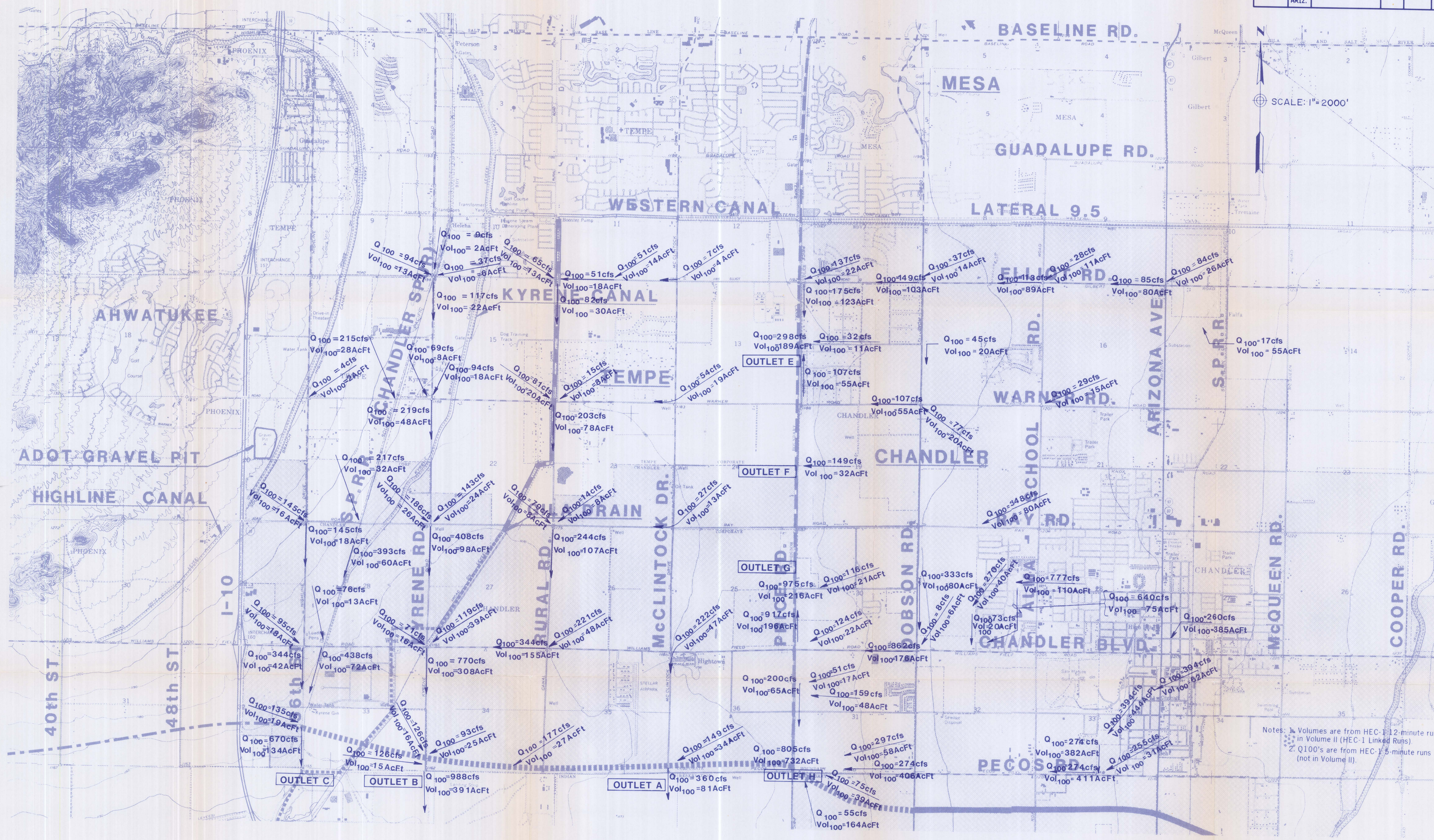

	NAME	DATE
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CHECKED		

HDR Engineering, Inc.

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION
HIGHWAY PLANS

HYDROLOGY STUDY
PEAK DISCHARGE AND VOLUMES

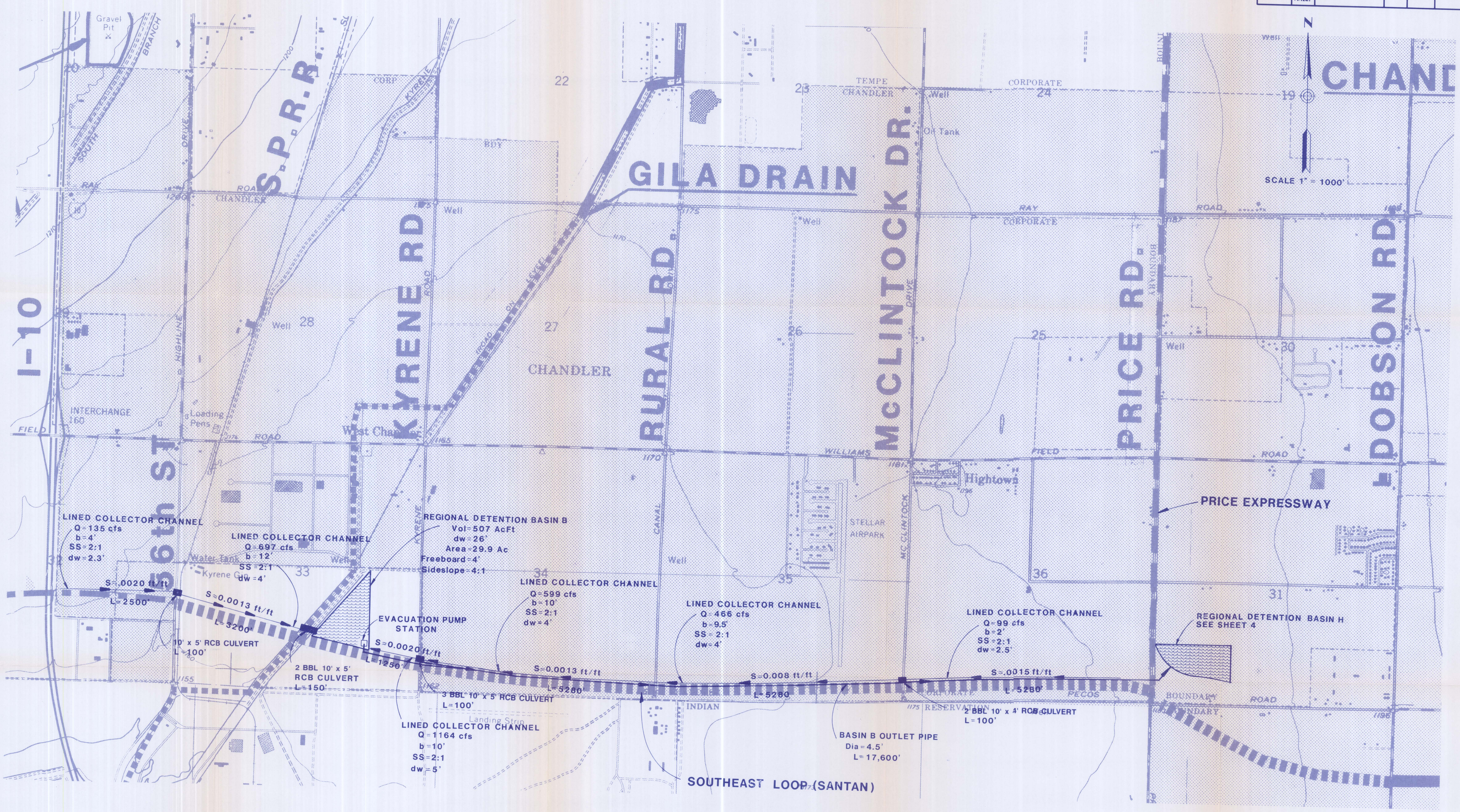
ROUTE | MILEPOST | LOCATION

SHEET 3 OF 5

H-2222-01D

SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

F.H.W.A. REGION	STATE	PROJECT NO.	SHEET NO.	TOTAL SHEETS	AS BUILT
	ARIZ.				



SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

	NAME	DATE
DESIGN		
DRAWN		
CHECKED		

HDR Engineering, Inc.

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION
HIGHWAY PLANS

HYDROLOGY STUDY
CONCEPTUAL OFFSITE DRAINAGE PLAN
SOUTHEAST LOOP

ROUTE	MILEPOST	LOCATION

SHEET 5 OF 5

H-2222-01D

___ OF ___