PLANNING

Arizona
Loop 101 South

PRICE
SANTAN
SOUTH MOUNTAIN

Concept Drainage Report

Final Report
Volume I: Main Report

Superceded
See Gila Drain Floodway Master Plan, Feb 1993

Contract 88-24
Price Expressway
General Consultant
TRACS No. H-2222-01D

Prepared for :

Arizona Department of Transportation

Prepared by:

HDR Engineering, Inc.
Phoenix, Arizona

HDR

July, 1990

CONCEPT DRAINAGE REPORT

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SECTION I

INTRODUCTION

A. Project Description

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). Figure I.1 is a project location map and Figure I.2 is an aerial photo overview of the project area.

As part of that agreement, HDR was to perform a hydrologic analysis to determine off-site stormwater intercepted by the two sections of freeway, and then to develop a concept drainage plan based upon the results of the hydrology study. The second phase of the drainage work under this contract, the development of a Concept Drainage Plan, is the subject of this report. The Concept Drainage Plan considers both on-site and off-site sources of stormwater.

Initially, the hydrology study was performed and the final report was submitted to ADOT in December, 1989 (see **Hydrology Study**, HDR Engineering, Inc.). The results of the **Hydrology Study** are summarized in Section II of this report.

The Price Expressway and Santan Freeway roadways are primarily depressed below existing grades in the project area. Stormwater sheet flows originating off-site and entering the ROW cannot be conveniently passed through or under the main roadway. Thus, the preferred method of handling the off-site 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, in the project area there is no natural stormwater outlet that can be utilized to evacuate basins at the present time. Stormwater surface or sheet flows generated in the project area presently flow generally east to west along arterials and across the proposed Price Expressway location. The flows then turn southwesterly and more-or-less concentrate in the vicinity of the I-10/Maricopa Road intersection. From there, the somewhat concentrated flows travel generally westward across the Gila River Indian Community (GRIC) tribal land through a drainage area known alternatively as the Queen Creek Wash or the Gila Floodway. Eventually, these flows reach the Gila River.

At the present time there is no existing agreement with GRIC to utilize the Queen Creek Wash, or a small irrigation tailwater drain known as the Gila Drain, to discharge project stormwater to the Gila River. Therefore, ADOT has directed HDR to develop a drainage plan utilizing an outlet to the Salt River known as the Carriage Lane Outfall (CLO)/East Valley (Price) Tunnel project. The tunnel portion of this system is currently under construction. This system is described in Section I.C.

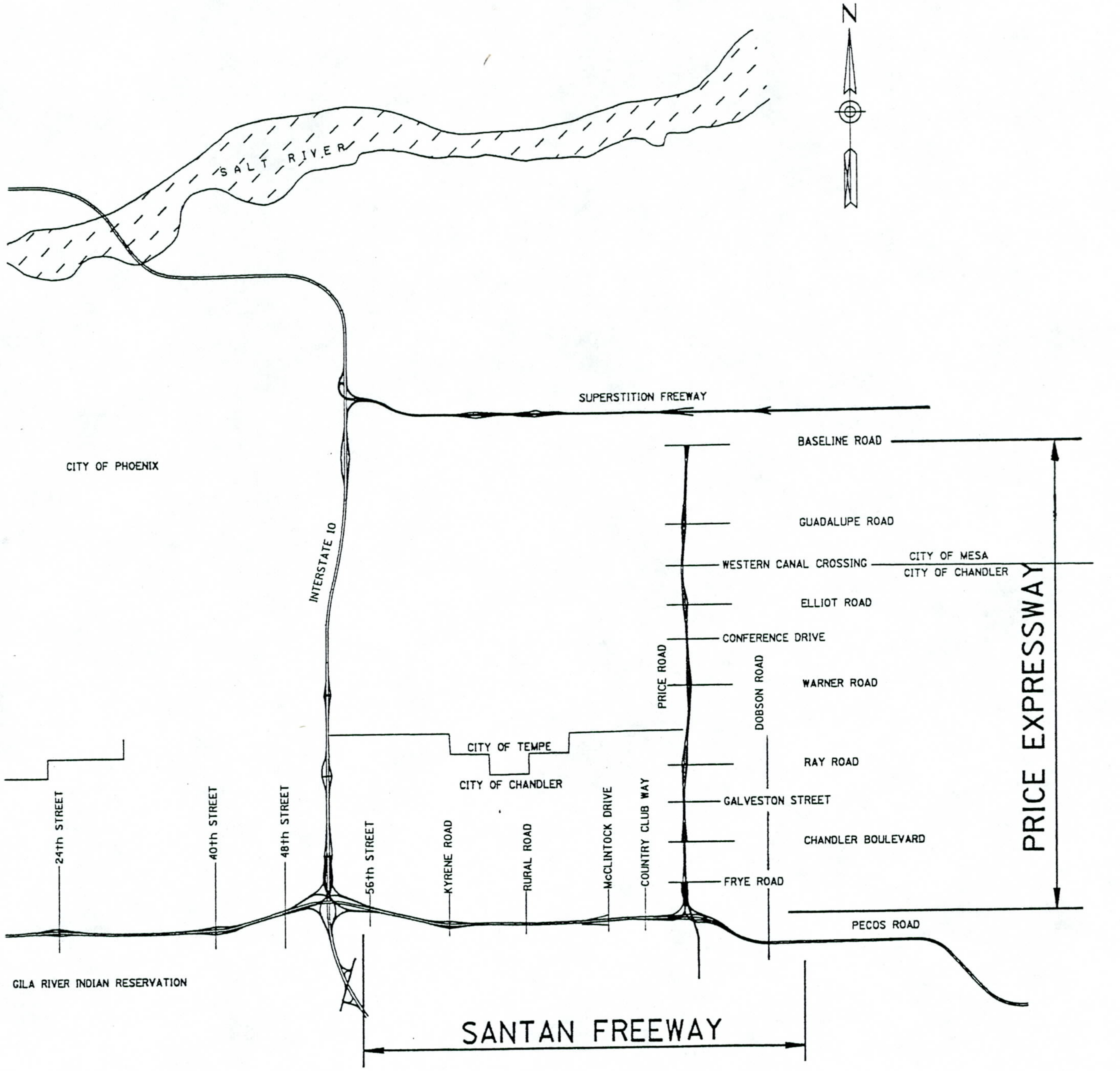

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FIGURE I.1

PROJECT LOCATION MAP

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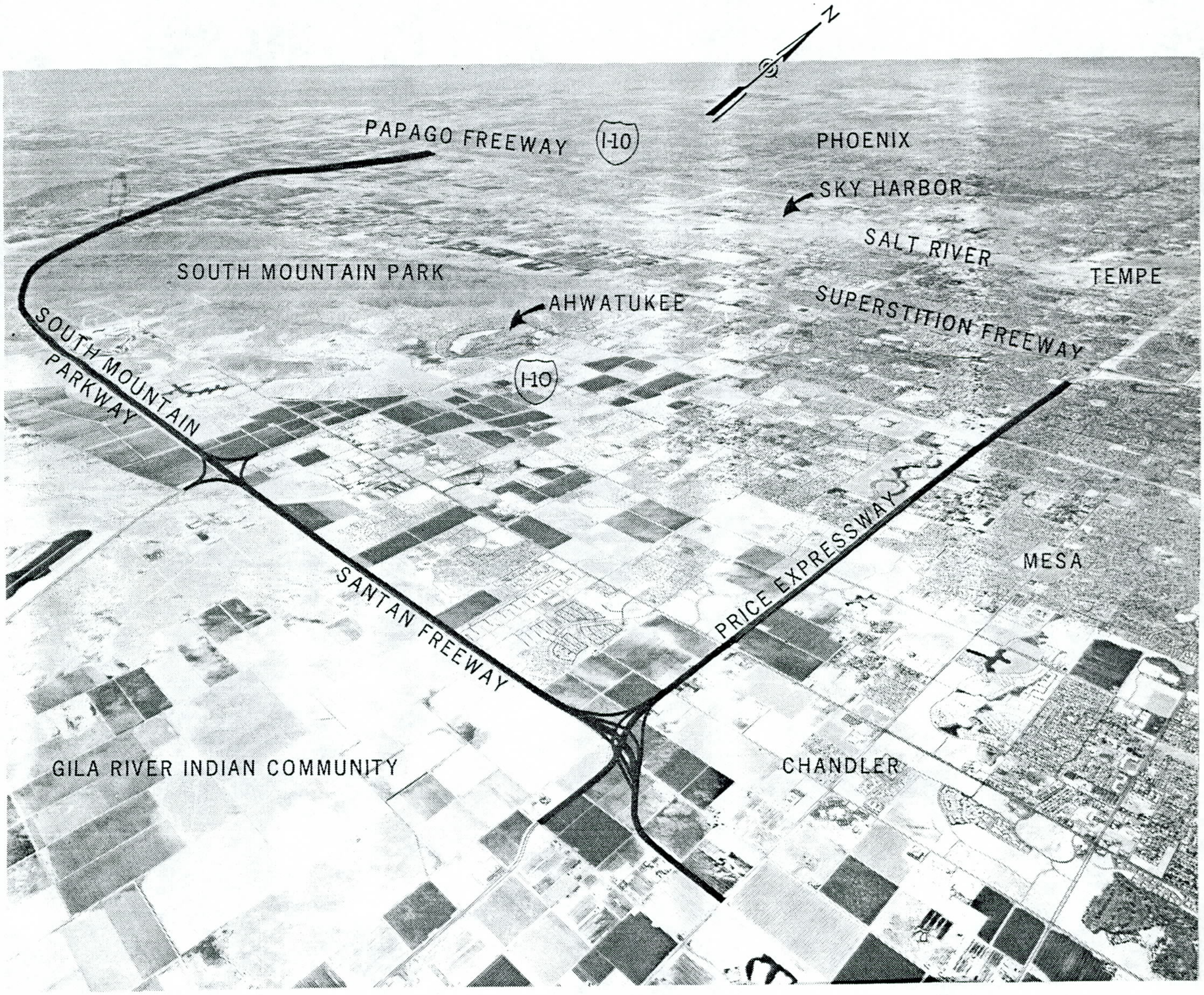

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FIGURE I.2

PROJECT OVERVIEW

HDR

B. General Drainage Plan

An overview of the concept drainage plan is presented in this section. The plan covers both off-site and on-site systems. The major facilities are shown in Figures I.3 and I.4. The general plan includes five (5) detention basins; three (3) basin evacuation pump stations and discharge pipelines; five on-site pump stations discharging to the off-site collector system; an off-site collector system of gravity sewers under the East Frontage Road (EFR) of Price Expressway, and a concrete-lined open-channel along the north side of Santan Freeway; and an on-site collector sewer system along the east edge of the depressed portions of the Price mainline roadway and the north edge of the depressed portions of the Santan mainline roadway.

The system is intended to function as follows:

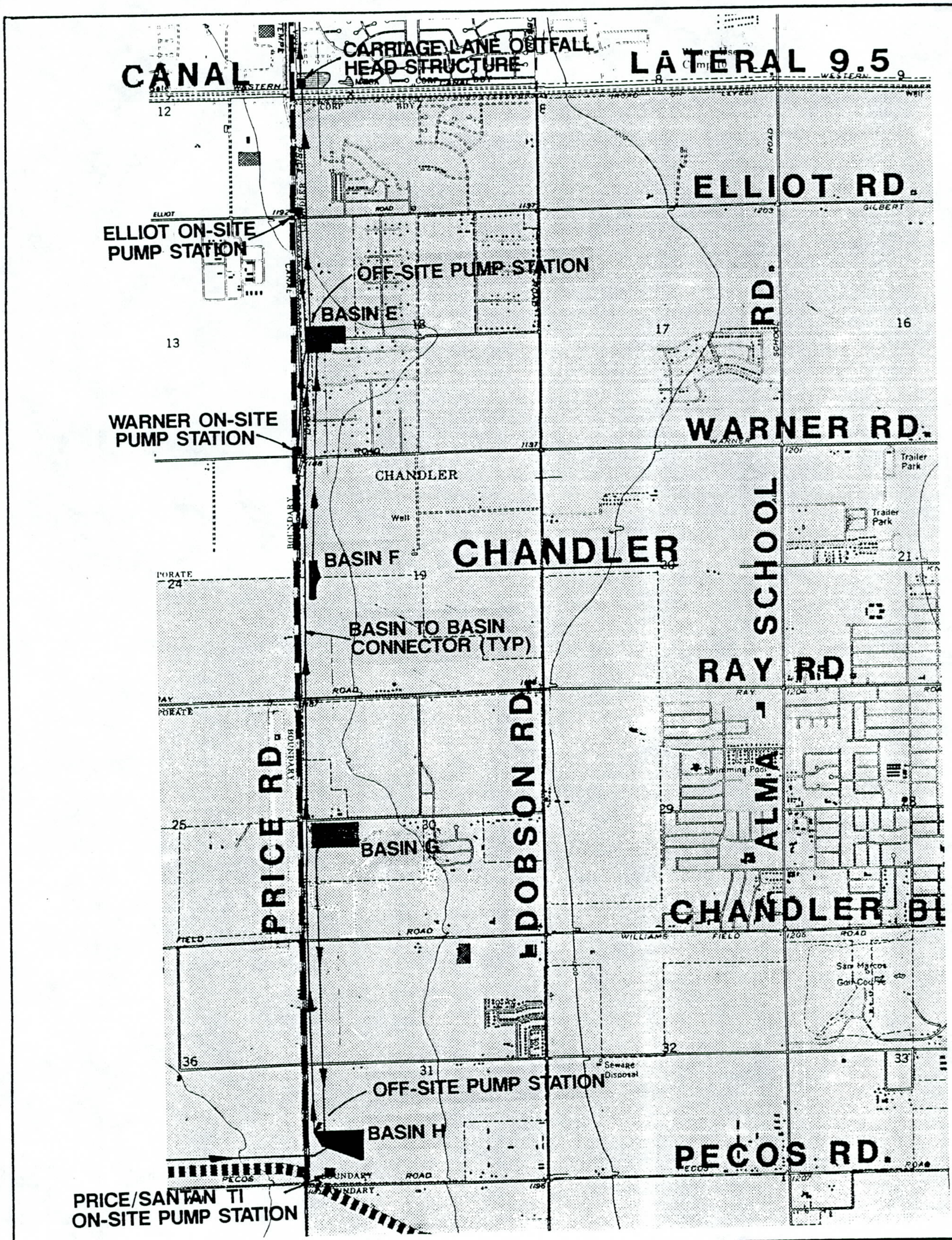
- Off-site flows entering the Price Expressway from the east are collected primarily at or near EFR intersections and delivered to Detention Basins E,F,G, and H in large diameter collector pipes buried under the EFR.

- Off-site flows entering the Santan Freeway are collected in concrete-lined channels running along the north side of the Freeway and discharged to Detention Basin B located at the Gila Drain crossing of the Freeway.

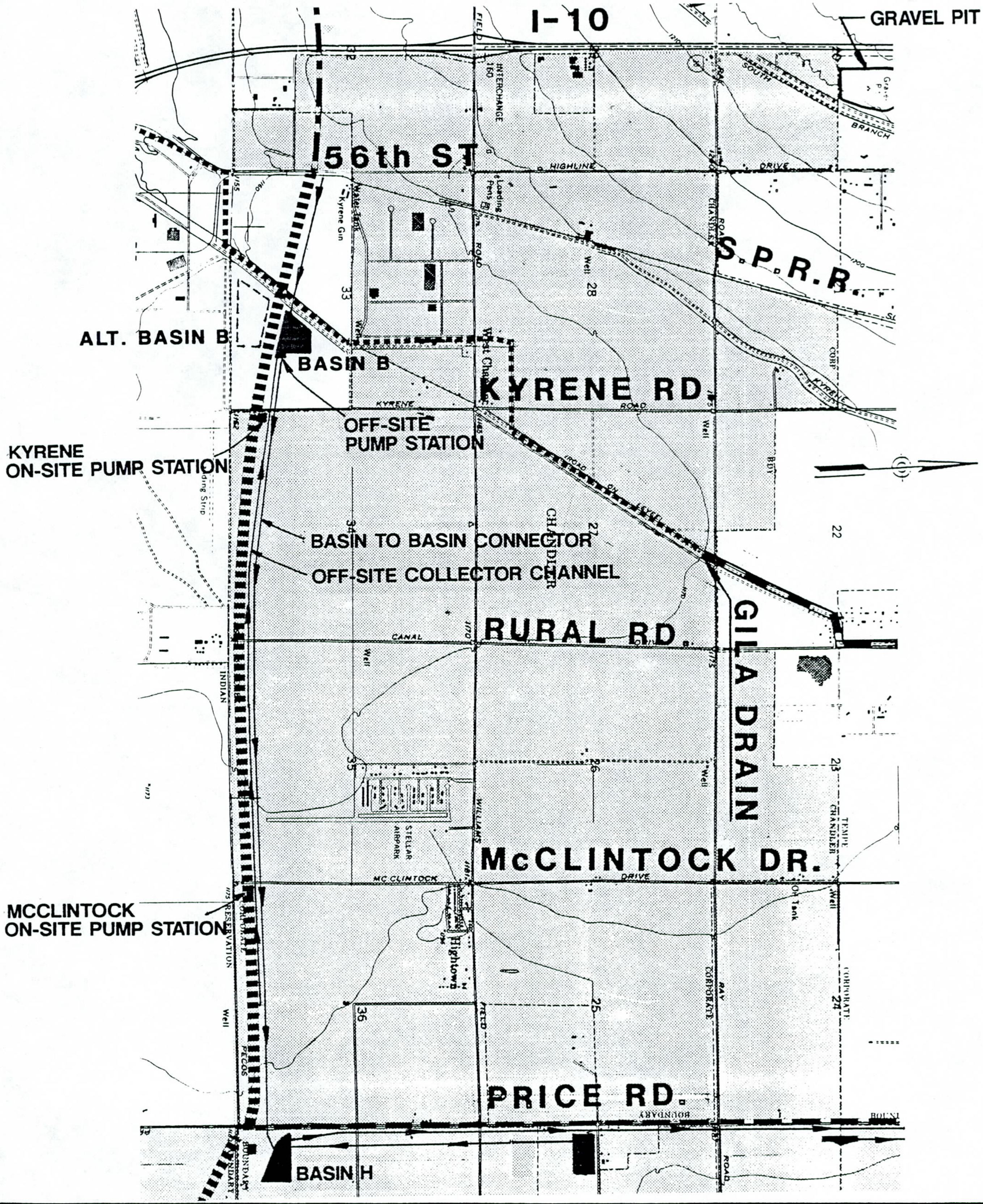
- On-site flows are collected in large diameter gravity sewers running along the east edge of Price Expressway and north edge of Santan Freeway. The on-site flows are concentrated at five low points in the mainline profile, where on-site pump stations are required.

- The five on-site pump stations are located at the Elliot and Warner Road crossings of Price Expressway; at the McClintock and Kyrene crossings of the Santan Freeway; and at the Price/Santan interchange. The on-site stations discharge to the off-site collector system. The stations have varying lengths of underground storage pipes incorporated into the on-site collector system to provide a storage buffer to reduce the required station capacity to 200 cfs for all five stations. The storage pipes allow these stations to be identical in basic design.

- The detention basins are integrated by basin-to-basin gravity outlet pipes and evacuation pump stations. The basins all evacuate north to the Carriage Lane Outfall (CLO) Head Structure, which is to be located at the intersection of Price Road and Western Canal. The pump station located at Basin B will evacuate this basin to Basin H. The Basin H pump station will, in turn, evacuate the combined volume to Basin E. The Basin E station will pump the entire combined stormwater volume to the CLO Head Structure. Also, Basin F will drain by gravity outlet to Basin E and Basin G likewise to Basin H.



CONTRACT 88-24 PRICE EXPRESSWAY GENERAL CONSULTANT TRACS NO. H-2222-01D	CONCEPT DRAINAGE REPORT FIGURE I.3 MAJOR DRAINAGE FACILITIES PRICE EXPRESSWAY	HDR



CONTRACT 88-24 PRICE EXPRESSWAY GENERAL CONSULTANT TRACS NO. H-2222-01D	CONCEPT DRAINAGE REPORT FIGURE I.4 MAJOR DRAINAGE FACILITIES SANTAN FREEWAY	HDR

Details of the concept drainage plan are shown in a set of plan sheets and basin site plans in Section VIII of this report. A separate set of half-size ("11x17") plan and profile drawings has also been submitted with this report.

The concept drainage plan does not include details such as pipes less than 36-inch in diameter, lateral or cross-drainage pipes, inlets, junction box details, appurtenances to pressure pipe (air release valves, etc), and any details beyond general layout and sizing of primary equipment in pump stations. These are details that will be assigned to various section designers as the final design is undertaken.

C. Carriage Lane Outfall Hydraulics

The outfall for this concept plan is the CLO and East Valley Tunnel which is presently under construction. The CLO itself is currently in the five-year plan for construction. The primary purpose of the Tunnel is to convey uncontrolled stormwater flows originating primarily in the Mesa vicinity to the Salt River. However, a small part of the system capacity is allocated to the cities of Mesa and Chandler, ADOT, and the Maricopa County Flood Control District (MCFCD) to discharge controlled stormwater flows into the CLO Head Structure. An interagency agreement between these parties, known as the Price Drain IGA, was drafted in 1988 to allocate allowable discharges and costs. A copy of this agreement is included in Appendix I.A.

The allocation of discharges directly influences the design approach for the Price/Santan system. Since the ADOT facilities for this system lie primarily within the city limits of Chandler and will be handling Chandler stormwater discharges, the combined CLO allocations for ADOT and Chandler have been used as a design basis.

The Price Drain IGA allocates allowable discharges into the CLO Head Structure during peak flow conditions, and also during off-peak conditions. The allocations during peak and off-peak conditions are listed in Table I.1.

Table I.1 Price Drain IGA Flow Allocations

Entity	Peak Flow Allocation CFS	Off-Peak Flow Allocation %	Proposed Off-Peak Flow Allocation CFS
Mesa	30	13.8	79
MCFCD	50	5.2	30
Chandler	100	10.3	59
ADOT	50	70.7	405
Total	230	100.0	573

The off-peak flow allocations are proposed at the present time. The DeLeuw Cather Company (DCCO) is the GEC for the CLO/Tunnel system, and a spreadsheet analysis of the CLO/Tunnel hydraulics was provided by DCCO for the purpose of determining off-peak flow allocations. HDR reviewed this calculation, made a minor revision, and recalculated the off-peak capacity as 573 cfs. For details, a copy of HDR's memo which proposes the off-peak flow allocations in Table I.1, along with both HDR and DCCO spreadsheets, is included in Appendix I.B. The Flood Control District of Maricopa County has reviewed the calculations and concur with the findings.

For the purpose of establishing the boundaries of this conceptual design, it was assumed that Chandler and ADOT have a combined peak flow allocation of 150 cfs and a combined off-peak allocation of 464 cfs into the CLO Head Structure. (An off-peak flow of 450 cfs was actually used in the design).

To determine the duration during the peak of the 100-year 24-hour design event when the flow into the CLO Head Structure would be restricted, the results of a transient flow analysis was used. The conclusions of this analysis are presented in a report by Howard Needles Tammen and Bergendoff (HNTB) entitled **Final Hydraulic Report for Price Road Tunnel System** (August, 1989). Excerpts of this report are included in Appendix I.C. An examination of Table 2 and text leads to the conclusion that peak flow conditions during which flows will be restricted to the 230 cfs total into the CLO Head Structure is approximately two to three hours. During this time period, HDR assumed for design purposes that the Chandler/ADOT combined input would be limited to 150 cfs for a three-hour time period and then increased linearly to the 450 cfs maximum off-peak discharge during a fourth hour. The ability to pump 450 cfs into the CLO/Tunnel system for most of the design storm significantly affects the design of the Price/Santan system as is discussed in Sections II and IV.

SECTION II

DRAINAGE AREA AND HYDROLOGY

A. Description of Drainage Area

The following is a summary of the results of the **Hydrology Study** (Dec, 1989) which considered off-site hydrology only.

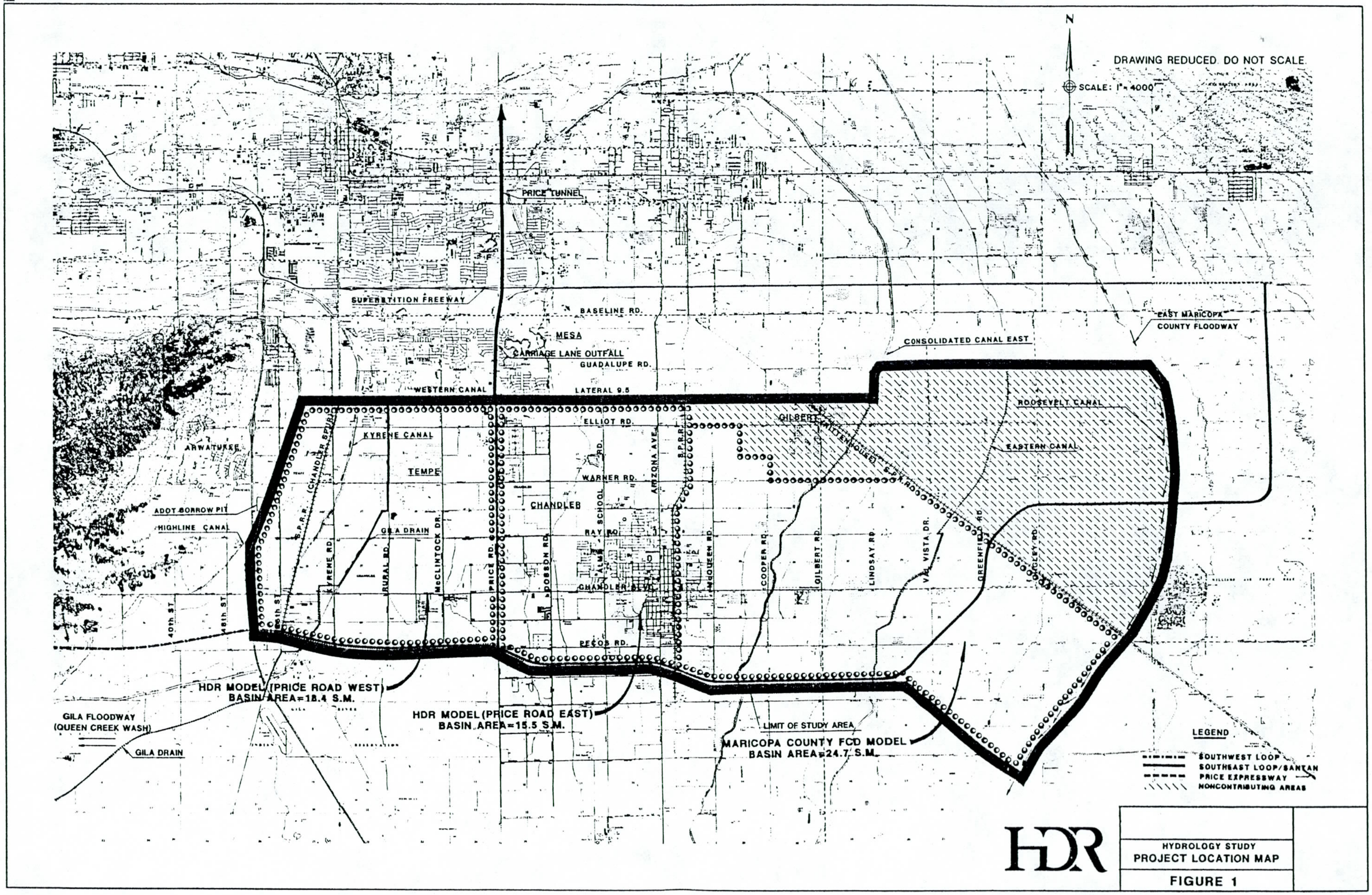
The drainage area boundaries for the off-site drainage analysis are the Western Canal/Lateral 9.5 on the north, the Santan Freeway alignment on the south, Interstate 10 on the west, and the Roosevelt Water Conservation District (RWCD) Canal on the east. The drainage area is shown on Figure 1 from the **Hydrology Study** (included here for convenience).

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. Off-site 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 off-site drainage for this section of Price Expressway has been the responsibility of the DeLeuw Cather Company (DCCO) and Howard, Needles, Tamman and Bergendoff (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. Considering the barriers, the effective combined drainage area contributing off-site 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.



HYDROLOGY STUDY
PROJECT LOCATION MAP

FIGURE 1

B. Hydrology Study (Dec. 1989) Review

B.1 Watershed Description

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.

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.

B.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.

B.3 Hydrologic Model

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. For this study, the stream network consisted of street surface flow paths, vestigial natural water courses, irrigation canals, and canal and railroad embankment barriers. 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.

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). The "balanced storm" procedure was used which creates a triangular shaped hydrograph from 5 and 15-minute and 1,2,3,6,12, and 24-hour rainfall depth.

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.

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.

Soil types were predominately Hydrologic Soil Group B and C types. Table II.1 is listing of SCS Runoff Curve Numbers for Antecedent Moisture II condition.

Table II.1 Runoff Curve Number (CN) Values

	Hydrologic Soil Group	
Cover Description	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

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.

C. Results

Peak discharges for the 100-yr 24-hr design event are shown at key locations for Price Expressway on Figure II.1, and for Santan Freeway on Figure II.2. The off-site peak discharges were taken from the **Hydrology Study** 5-minute interval HEC-1 runs, which generally produced somewhat higher peak flows than the 12-minute runs used to generate complete hydrographs for flow volume determination.

The 100-yr 24-hr design volumes from the 12-minute HEC-1 runs are listed in Table II.2 for Price Expressway and Table II.3 for Santan Freeway. The design hydrographs and volumes were used to size the detention basins and evacuation pump stations. The analysis of alternative systems is presented in detail in Section V, whereas details of the off-site system design are given in Section IV.

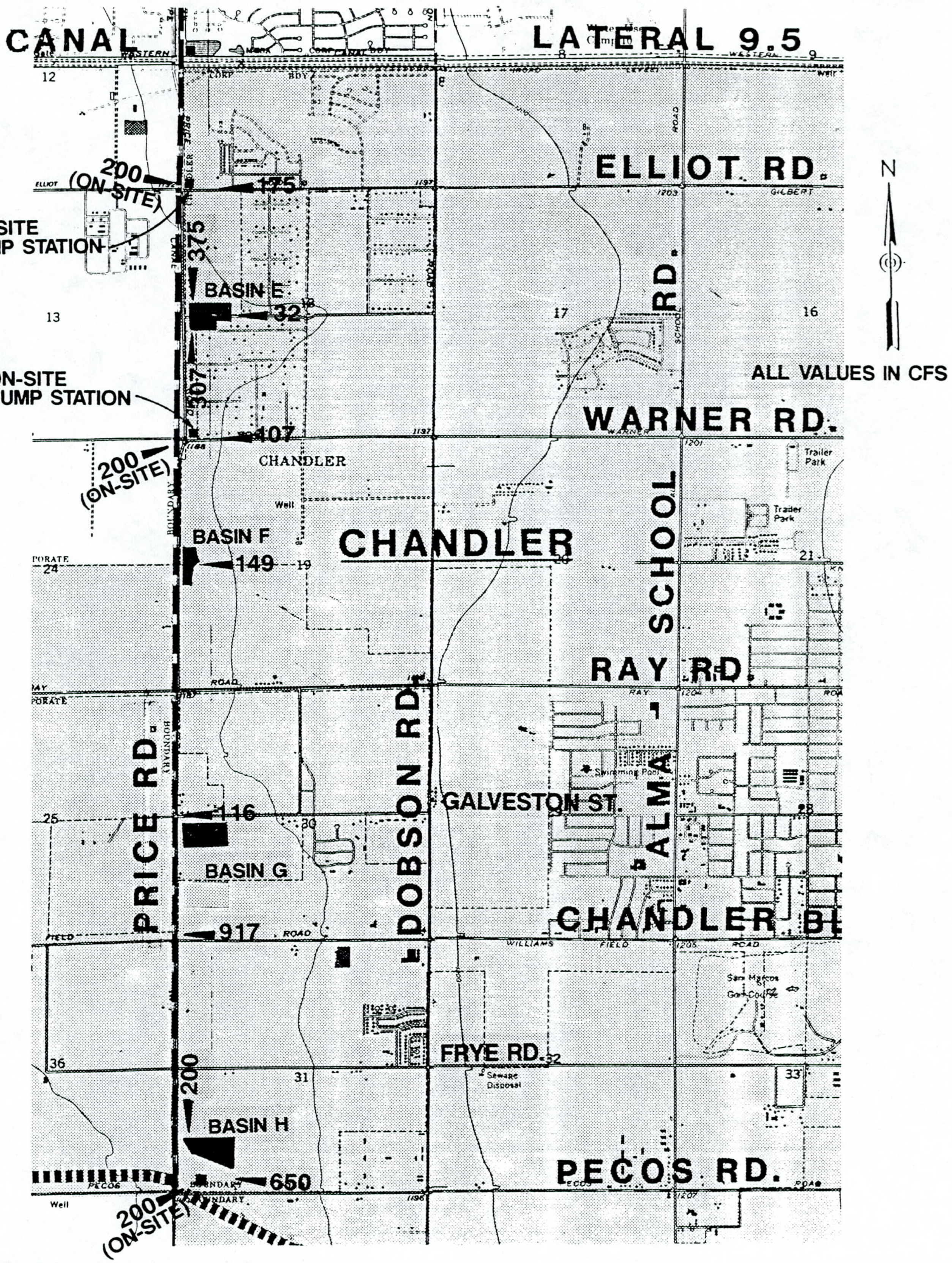
Figures II.1 and II.2 include 200 cfs flows noted as "on-site" flows. These are the design 50-yr 24-hr event discharges from the five on-site pump stations which discharge into the off-site collector system. The selection of the 200 cfs design discharge for all on-site pump stations is discussed in Section V, whereas details of the on-site collector system are presented in Section III.

Table II.2 Price Expressway Off-site Design Volumes

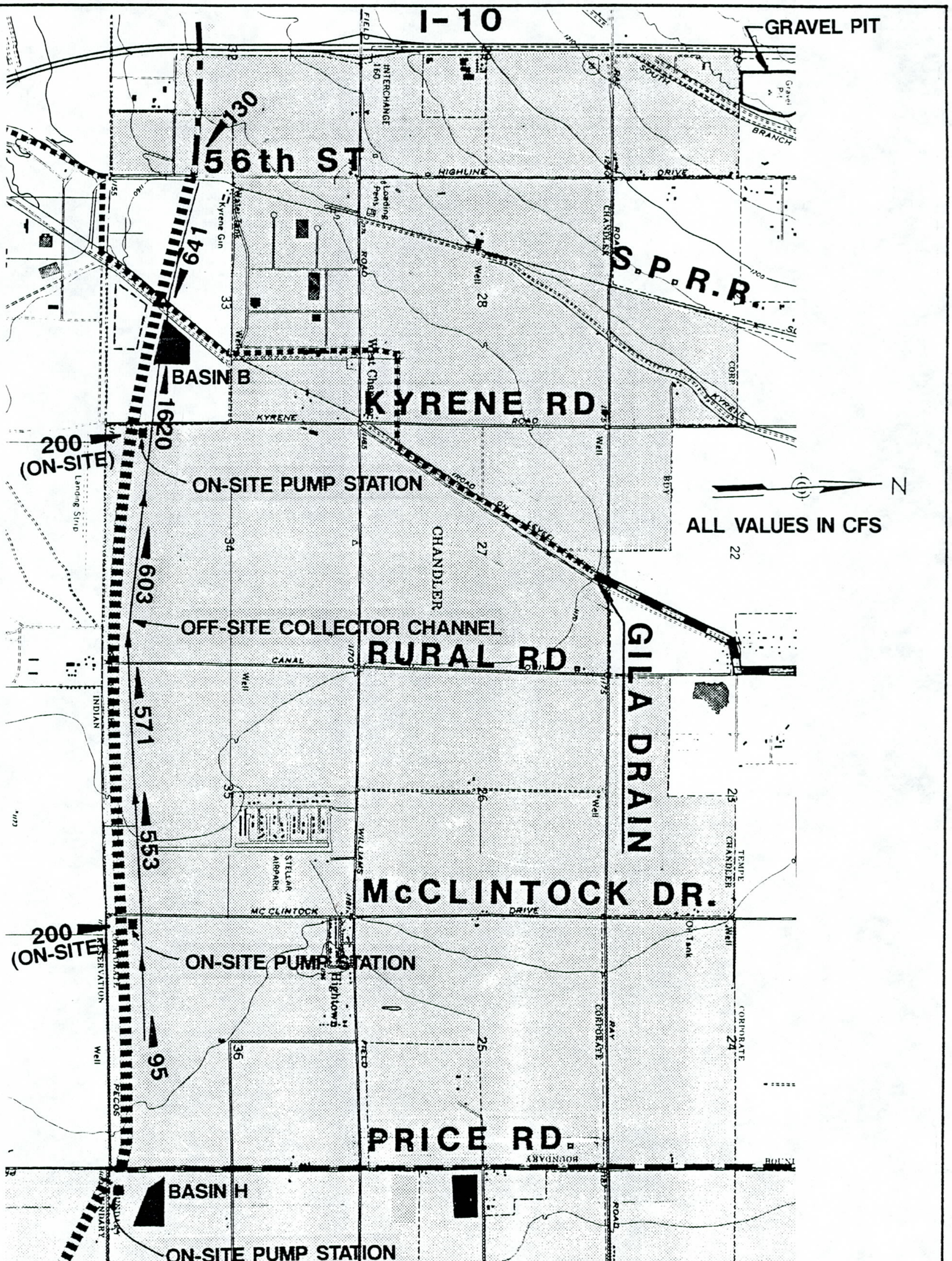
Location	Volume acre-feet
Western Canal to Warner	189
Warner to Ray	32
Ray to Chandler	216
Chandler to Santan Frwy.	732
Total	1169

Table II.3 Santan Freeway Off-site Design Volumes

Location	Volume acre-feet
Price to McClintock	81
McClintock to Gila Drain	391
Gila Drain to I-10	134
Total	606



CONTRACT 88-24 PRICE EXPRESSWAY GENERAL CONSULTANT TRACS NO. H-2222-01D	CONCEPT DRAINAGE REPORT FIGURE II.1 ON-SITE & OFF-SITE COLLECTOR DESIGN FLOWS - PRICE EXPRESSWAY	HDR



CONTRACT 88-24 PRICE EXPRESSWAY GENERAL CONSULTANT TRACS NO. H-2222-01D	CONCEPT DRAINAGE REPORT FIGURE II.2 ON-SITE & OFF-SITE COLLECTOR DESIGN FLOWS - SANTAN FREEWAY	HDR

SECTION III

ON-SITE DRAINAGE

A. General Description

The on-site drainage system consists of a series of large diameter storm sewers that collect and concentrate runoff from the mainline roadway section and the frontage roads. At low points in the depressed freeway sections, on-site pump stations will pump the ponded stormwater into the off-site collector system on the east side of Price Expressway and the north side of the Santan Freeway.

The on-site storm sewers for Price Expressway begin at Guadalupe Road on the north end and terminate at the Price/Santan interchange on the south end. There will be three on-site pump stations located at the Elliot and Warner Road bridge crossings, and at the Price/Santan interchange. The on-site sewers will also collect local runoff from the West Frontage Road. However, local runoff from the East Frontage Road (EFR) will be drained into the off-site runoff collection system under the EFR and combined with on-site pump station and off-site flows. The off-site collection system will convey these combined flows to a series of detention basins along Price Road. The off-site system is described in detail in Section IV.

The on-site storm sewer system for the Santan Freeway begins at the Price/Santan interchange on the east end and ends at the I-10/Santan interchange on the west end. Two on-site pump stations will be located at the Kyrene and McClintock crossings, respectively. A third station on the Santan is shown at Dobson Road, however, this station is not designed as it will not be needed until the Freeway is extended to the east beyond the current project limits. The station located at the Price/Santan interchange, as noted above, will also collect on-site stormwater from a portion of the Santan Freeway.

A separate study was conducted to determine the on-site pump station criteria. This study is presented in Section V. In general, the study determined that all five pump stations can be designed as identical stations with a peak capacity of 200 cfs. The difference in peak inflows is buffered by varying lengths of 96-inch diameter storage pipes placed upstream of the pump station intake and in the same approximate location parallel to the freeway alignment as the on-site storm sewers.

B. Hydrology

B.1 Design Criteria and Methodology

The conceptual design of on-site drainage systems was performed using guidelines of the ADOT Urban Highways **Design Procedures Manual (DPM)**, 1990 Edition. With the exception of portions of the East Frontage Road, where 100-year frequency design flows originating from off-site areas east of Price Road contribute to the system, the design stormwater peak flows generated in the on-site system are the product of runoff from areas within the project's right-of-way only. As a result, the drainage areas are generally small and the rational method (Q=CIA) was used to compute peak runoff flows. The rational method was developed for use in small urban areas, and the **DPM** recommends use be limited to watersheds of 80 acres or less.

The rational method requires three types of data for peak discharge computation. They are the runoff coefficient (C) which represents the ratio of runoff to rainfall; the rainfall intensity (I) which is the intensity of rainfall, in inches per hour, for a storm duration equal to time of concentration and; the drainage area (A) in acres.

For the on-site analysis two principal values of C were applied as recommended in the DPM:

Paved Surfaces	0.95
Highway Slopes (decomposed granite with 3:1 slope)	0.70

To minimize right-of-way requirements, the roadway design includes extensive use of retaining walls and maximum allowable side slopes in the project's interior. Therefore, the maximum 0.70 value of C for highway slopes was applied universally to all nonpaved areas within the project. Also, due to the conceptual drainage design being conducted concurrently with the roadway design, the highway slope designs were generally not finalized; therefore a 3:1 decomposed granite slope (worst case) was assumed.

Median areas on the mainline, which may be initially installed as bare earth were analyzed as being fully paved, in anticipation of ultimate pavement widening.

The rainfall intensity was determined using the standard ADOT method. A one hour precipitation depth was derived from precipitation maps in the ADOT hydrology manual, **Hydrologic Design for Highway Drainage in Arizona** (1969). Using this value, intensity values were selected by entering Rainfall Intensity Curves (**DPM**, Fig. 3.7-1) for a storm duration equal to the drainage area time of concentration. Due to the typically small drainage area size and therefore, the short amount of time expected for the entire area to contribute runoff, the estimated time of concentration rarely exceeded the minimum recommended value of 10 minutes.

The delineation of major on-site drainage areas was influenced by the rolling profiles of the depressed freeway sections, prevalent throughout much of the Price and Santan alignments. Vertical curve crest points set upper boundary limits and curve sag points set runoff concentration points. The major areas were then subdivided as needed to assist in developing the design of the major trunk sewer lines. The drainage areas, as shown on the Drainage Area Plan (see sheets DA-1 and DA-2 included in Section VIII) were approximated and are subject to adjustment during the final design process.

B.2 Storm Frequency and Precipitation Values

As directed in the DPM and by ADOT, the design frequency selected for use on on-site areas of depressed roadway is 50 years. Most sections of the Price/Santan alignments are depressed. Those portions not depressed and not directly contributing to a depressed concentration point shall be analyzed for the 10-year frequency storm by the section designer.

The precipitation values, as determined from ADOT **Hydrologic Design for Highway Drainage in Arizona** (1969) are as follows:

10-year	1-hour	1.50"
	6-hour	1.90"
	24-hour	2.30"
50-year	1-hour	2.10"
	6-hour	2.70"
	24-hour	3.05"

The majority of the on-site storm sewer pipes for this project, as illustrated in Section VIII and in the accompanying plan set, are based upon the 50-year design storm. Where off-site HEC-1 generated 100-year flows are combined with on-site 50-year flows (East Frontage Road), the off-site flows are very much higher than the on-site flows, and the off-site peaks occur much later than the on-site peaks. Therefore no adjustment in pipe sizes was required for the EFR on-site flows.

The right-of-way area north of Guadelupe Road to Baseline Road drains north to a mainline sag point at Baseline Road. Under a plan developed by Howard Needles Tammen & Bergendoff (HNTB), an on-site pump station is proposed at Baseline Road which would discharge into the Carriage Lane Outfall line (108-inch). The Price drainage system will tie into an existing on-site sewer extending south from Baseline Road. Off-site drainage contributing to the Price Expressway from the Western Canal to Baseline Road is being addressed by HNTB in a separate contract to design the CLO and East Valley Tunnel system, as discussed in Section I.

C. Gravity Drain System Design

C.1 Mainline System

On-site gravity storm drains are proposed to collect runoff from the frontage roads, ramps and mainline areas. The design of this system was based upon guidelines in the DPM. This closed conduit system will intercept surface runoff via a network of catch basins and lateral connector pipes, conveying and concentrating most of the on-site runoff to five mainline sag locations in the Price and Santan alignments:

1. Elliot Road
2. Warner Road
3. Price/Santan Interchange
4. McClintock Drive
5. Kyrene Road

Because of the depressed configuration of the two mainline profiles, the concentrations of runoff at these sag points have no positive outfall. Therefore, on-site stormwater pump stations at those locations are needed to lift the concentrated flows to an elevation where they may be discharged to the nearest surface outfall. A proposal to supplement these five pump stations with underground storage was examined, as described in Section V.

The peak inflows at these five locations range from 223 cfs at Kyrene Road to 385 cfs at the Price/Santan interchange. However, using 96-inch underground storage pipes, the peak discharge for each pump station will be the same (200 cfs). The input hydrographs for the storage volume calculations were developed using HEC-1 models. Program print-outs are included in Appendix III. These volumes were utilized in the on-site pump station design studies discussed in Chapter V. Underground storage was not analyzed at Kyrene Road due to the small difference between 223 cfs and the desired 200 cfs pumped discharge. On the Santan Freeway alignment, it is proposed to discharge the McClintock Drive and Kyrene Road pump flows (200 cfs) into an off-site collector channel which parallels the alignment on the north side. This channel gravity drains to Detention Basin B. At the Price/Santan interchange concentration point the 200 cfs flow will be pumped directly into Detention Basin H. The Elliot and Warner Road stations will pump into large diameter off-site/on-site storm drains installed underneath the East Frontage Road, where the storm drains can gravity flow to Detention Basin E. Since the 200 cfs flows are significant, these flows were added to the off-site flows to size these facilities.

Sufficiently elevated portions of the proposed Price Expressway and East Frontage Road are gravity drained directly into the nearest detention area. Elevated portions of the Santan Freeway may be gravity drained either to Basin B or to the paralleling off-site collector channel on the north side of the Freeway.

The storm drain facilities discussed here and shown on the plan sheets (Section VIII) and the separate Plan and Profile set represent only the major drainage structures. A limited effort was made to predict catch basin and small diameter (generally 36-inch or less) storm drain locations, although they are not shown. The overall collection system was conceptualized in order to obtain reasonable drainage boundaries, times of concentration and other input parameters necessary to the analysis. It should be noted that the storm drain trunk lines analyzed and shown in plan depict an average pipe size for the reach in which they are located. These sizes should be reanalyzed and may be revised during the detail design process.

C.2 East Frontage Road Combined System

The East Frontage Road of Price Expressway plays an important role in the overall drainage scheme for both the on-site and off-site drainage systems. This roadway is the primary location of the pressure discharge lines that run between off-site pump stations, the gravity lines that permit Basins F and G to evacuate to Basins E and H, and the collector drain lines. The collector drains will serve the double and sometimes, triple use of intercepting the 100-year off-site flows gathering along the Price Expressway east right-of-way, draining the frontage roads themselves, and in addition, serving as discharge lines for the Elliot and Warner Road on-site pump stations.

The number of storm drain lines of large diameter beneath the East Frontage Road may greatly impact utility relocation schemes. All future Design Consultants should be made aware of these potential impacts.

SECTION IV

OFF-SITE DRAINAGE

A. General Description

The off-site drainage system consists of a series of detention basins with associated collector pipes and channels, and evacuation pump stations along the east side of Price Expressway and the north side of Santan Freeway. As discussed in Section II, the off-site hydrology was developed in detail in a separate report (**Hydrology Study**, 1989). Design flows for the off-site collector system are shown in Figures II.1 and 2.

The basic concept along Price Expressway is to concentrate the 100-year 24-hour stormwater flows in large diameter collector pipes as the flow approaches the East Frontage Road. The collector pipes will be located under the EFR and will discharge the stormwater into four basins (E,F,G,H). Under the present plan as shown on the plan sheets, Basin F, a small basin, discharges to Basin E, and Basin G discharges to Basin H. An off-site pump station located at Basin H will evacuate the basin to Basin E. The Basin E off-site pump station will in turn pump the combined flow volume to the CLO Head Structure. From there the stormwater will flow by gravity through the Carriage Lane Outfall and East Valley Tunnel (Price) system to the Salt River.

The off-site collection system along the north side of the Santan consists of a concrete-lined trapezoidal channel flowing east to west to a large basin (Basin B) located at the intersection of the Gila Drain with the Freeway. There is also a smaller channel flowing west to east from the I-10/Santan interchange to Basin B. The pump station at Basin B will evacuate this basin to Basin H.

Several studies were performed to determine design criteria for the off-site system and to select an optimal arrangement of pump stations and basins. These were:

1. Determination of the Carriage Lane Outfall (CLO) hydraulics and the constraints imposed upon the Price/Santan drainage system by the Price Drain IGA discussed in Section I.

2. A study to select an optimal arrangement of basins and pump stations considering volume, distribution and evacuation time of basins versus number, size, and configuration of pump stations. The detailed study is presented in Section V.

3. A study of converting several basins into multi-use basins. This study is presented in this section (IV.B).

B. Detention Basin Requirements

B.1 Base Case

The location and size of the Price/Santan detention basins is determined by a number of complex and interrelated factors, including:

1. ADOT design criteria,
2. Carriage Lane Outfall (CLO) hydraulics,
3. Price Drain IGA,
4. design storm inflow hydrographs,
5. evacuation pump station capacities, and
6. multiple-use considerations.

The ADOT criteria establishes the basic configuration of basins. The **DPM** requires that the basin be sized to contain the 100-year, 24-hour runoff event and that the basin be evacuated within 36 hours after the 24-hour storm has passed. The **DPM** further specifies that the basin side slopes be no steeper than 3:1 (H:V) and that the depth not exceed 25 feet. However, to more easily maintain basin side-slopes, a 4:1 slope was used; and, because three of the basins are very large basins, a 30-foot maximum depth was used to minimize ROW requirements.

The CLO hydraulics and the IGA governing the proportion of peak and off-peak flows allocated to each of the participants was discussed in Section I.D. Reference documents are included in Appendix I. The basic constraints on basin size imposed by these factors are: 1) that ADOT and Chandler have a combined allowable discharge of 150 cfs into the CLO Head Structure during peak flow conditions in the CLO and East Valley Tunnel; and 2) that ADOT and Chandler have a combined allowable discharge of 81 percent of the maximum CLO off-peak discharge of 573 cfs, or 464 cfs, into the Head Structure. These constraints prompted an optimization study of pump stations and basins which is presented in detail in Section V.B. This study resulted in the selection of the off-peak maximum pump capacity of 450 cfs for the outlet pump station at Basin E.

The design inflow hydrographs to each basin were determined in the **Hydrology Study**. A separate HEC-1 model was assembled to simulate the inflow, outflow and routing functions of the system. The model routes the input hydrographs from the **Hydrology Study** and the on-site contributions from the five on-site pump stations through the five basins in the system. The final configuration includes the five basins (B,E,F,G,H), three pump stations (B,H,E) pumping basin-to-basin (B to H; H to E; E to CLO), and two outflow pipes connecting basins (F to E; G to H). Three basin evacuation scenarios were analyzed for maximum off-peak discharges into the CLO Head Structure of 250, 350, and 450 cfs, respectively. These scenarios were used to determine the selected configuration, as described in greater detail in Section V.B. The HEC-1 run to determine basin volume requirements for the selected 450 cfs option is included in Appendix III.C.

In assembling this HEC-1 basin model, several adjustments were made to more closely simulate conditions during the design event. First, peak pump discharges from each basin was proportioned approximately to the total combined inflow volume to the basin (i.e. B=150, H=340, and E=450 cfs, resp.). Second, the peak outflows were reduced proportionally during the CLO peak to reduce discharge to the IGA mandated 150 cfs into the CLO Head Structure. This occurs for approximately two to three hours. (See discussion in Section I.C). To compensate for limitations of HEC-1, the pumped outflow reduction during peak conditions was reflected by adding two "adjustment" hydrographs. The first hydrograph increases the net storage volume required during reduced outflow, and the second subtracts this volume back out with a "negative" hydrograph to obtain the correct inflow hydrograph to the next basin.

The basin volumes determined during the optimization study were then used to develop a site plan for each basin. Using the ADOT **DPM** criteria mentioned above and four feet of freeboard as requested by the ADOT project staff, a "base case" layout was developed for each basin. The "base case" design parameters, basin design criteria, site plans and sections for each basin are included in Section VIII. An alternate location for Basin B "Base Case" is shown south of the Santan Freeway on ROW that appears to have fairly restricted access.

Base case detention basins should have a simple basin design criteria perimeter landscape theme. The plantings will be used as a visual barrier to screen views into the basins from roads and surrounding developments. All plantings should be located in the 30 ft. perimeter area and on the side slopes no deeper than the freeboard level. No turf will be allowed and granite mulch will be used only on the side slopes.

B.2 Multiple-Use Basins

All of the basins can be configured as multiple-use basins, if desired. If basins are to be used for recreation, addition criteria must be applied which will increase the cost of the basins. A two-tiered basin is envisioned, with a deep section to handle storms up to the 10-yr. 24-hr. frequency and a shallower section for multi-use activities. In addition, multi-use area side-slopes are reduced to 6:1 (H:V) and bottom slopes are increased to one percent (from 0.5%) to provide for more rapid drainage and drying. A multi-use configuration is included for the Basins (B,G, and E) in Section VIII Plan Sheets. A multi-use option was not developed for Basin H because it receives flow from Basins B and G, on-site flows from about six miles of freeway, and also will receive overflows from Chandler's downtown detention basins. The cost to provide a multi-use tier which would not be inundated more frequently than once in ten years and large enough to be useful does not appear to be justified in this case. Basin F is small enough so that the base case shown is actually intended to be a multi-use basin.

Recreational opportunities are described in the following for each of Basins E,F,G, and B.

Basin E has many opportunities for recreational uses. There is enough room in the upper basin to provide two (2) softball/soccer fields or three (3) football/soccer fields. Also sport courts (i.e. basketball, volleyball) can be installed with the possibility of having more passive recreational uses of open green space for picnicking. Shade ramadas could also be used in the upper basin bottom. In addition, there should be enough space available above the freeboard storage to construct a parking lot, restrooms and a playground.

Basin F is not large enough to provide large play fields. Fitness trails, passive recreational uses, sport courts or picnicking are possible.

The upper basin of Basin G also has the room available to install two (2) softball/soccer fields or three (3) football/soccer fields. However, there is not enough open space to provide any further amenities if the ball fields are incorporated. Other than using the perimeter as a fitness trail, the site is limited. Another opportunity for the basin is a combination of sports courts and passive open space recreational activities, picnic areas, ramadas, playgrounds and open turf spaces. There is not enough area to provide any development above the freeboard level.

Basin B is the largest basin and therefore provides the highest level of recreational opportunities. There is enough area available to install a four (4) softball field complex possibly with a restroom/concession building. The remaining space could be used as a sport court area or for passive recreation. If softball fields are not constructed, the upper basin has multiple combinations of active and passive recreational opportunities. Many of the possibilities for incorporation into this site are listed in ADOT's **Landscape Design Guidelines for Urban Highways** under the Stormwater Detention Sites section.

The specific types of recreational facilities for each basin, if provided, should be coordinated with the local jurisdiction.

C. Collector System Design

C.1 Price Expressway Collector System

East of the Price Expressway, stormwater runoff reaches the proposed alignment primarily via surface city streets (as opposed to overland flow) in an east to west general flow direction. The 100-year 24-hour design runoff represents a significant volume requiring interception and collection by an off-site collector system to convey the runoff to proposed detention basins.

<u>Off-site Flows to Basin E</u>

The off-site hydrologic analysis indicated major concentrations of off-site surface flows contributing to Basin E from three primary directions. The estimated peak flow expected from Elliot Road, which lies north of the basin , is 175 cfs. The flow can be intercepted at or near the Elliot Road intersection with the East Frontage Road (EFR), which is the lowest point in the proposed Elliot Road profile east of the Price Expressway. It can be captured by a combination of catch basins in both the approach roadway and in the graded sump location. Low points in the East Frontage Road profile north and south from the intersection may also be utilized, as necessary, to intercept the westbound off-site runoff. This flow, combined with local frontage road runoff and pumped discharge (200 cfs) from the proposed on-site Elliot Road pump station will be piped beneath the East Frontage Road south to Basin E.

A minor peak runoff of 32 cfs is estimated to converge directly to Basin E near the half-mile point (Mesquite St.) between Elliot and Warner Roads. This can be discharged into Basin E either in a surface channel or by storm drain, as determined by the section designer.

Warner Road is the remaining source with a peak flow of 107 cfs. This flow can be intercepted at the EFR intersection in a manner similar to the Elliot Road off-site flow. A gravity drain under the EFR will convey this flow, local runoff and pumped discharge (200 cfs) from the proposed on-site Warner Road pump station north to Basin E.

Off-site Flow to Basin F

A 100-year off-site flow of 149 cfs is estimated at the half-mile point between Warner and Ray Roads. This peak flow is distributed between several streets in the proximity of Basin F, including Highland St. and Calle De Norte. These distributed flows can be discharged directly in Basin F, using inlets and storm sewers at sumps in the street developed by the section designer.

Minor overland flows are anticipated in the open areas along the Price alignment lying north and south of Basin F. It is proposed that unlined interceptor ditches be located parallel to and east of the East Frontage Road and graded to drain to Basin F. Minor runoff, primarily local street runoff only, is expected to be conveyed to the project via Ray Road. The EFR local system should be capable of absorbing this runoff without increase in storm drain size. The hydrology study assumed that off-site flows approaching on Ray Road were negligible. The road is somewhat higher than adjacent areas to the north and south.

Off-site Flows to Basin G

100-year off-site flows to Basin G will be conveyed by both Galveston Street and Chandler Boulevard. A series of catch basins at the Galveston intersection with the EFR will be required to intercept the 116 cfs estimated there. These catch basins can be connected, in a short run, directly to Basin G.

The 917 cfs design flow in Chandler Blvd. will be intercepted at Chandler Blvd.'s intersection with the EFR. The design flow should not be allowed to overflow to the west or south. A substantial installation of catch basins (probably several hundred linear feet), both on-grade and in sump locations, will be required to collect the 917 cfs. An estimated 2-8'x6' box culvert will convey the stormwater to Basin G, located 1900 feet north of Chandler Blvd. A junction box, which would serve as the outfall for the various catch basin connector pipes, is proposed beneath the EFR at the intersection. It is recommended that an adequate portion of the catch basins (curb inlets) be located on both sides of Chandler Blvd. east from the intersection, to the extent that the intersection is substantially clear of ponded water, based upon the judgement of the section designer, during more frequent rainfall events. EFR catch basins can also be used to collect the design flow and be connected directly to the box culvert which, in the conceptual drawings, is shown positioned to the inside (west) edge of the EFR. The Basin G gravity drain and the 96-inch pressure line from Basin H, adjacent to the box culvert, may be located closer to the mainline as required.

In addition, the City of Chandler may direct a 100 cfs peak evacuation flow from the downtown detention basins to either this basin or Basin H. Details can be found in **Stormwater Management Master Plan** (Camp, Dresser & McKee, Inc. Oct. 1986). The discharge planning for this should be coordinated with the City in final design.

Off-site Flows to Basin H

Flows to Basin H are anticipated to come from three primary directions. A surface runoff of 200 cfs will be concentrated at the Frye Road intersection with the EFR. This flow can be intercepted with a series of catch basins in the sumped intersection. A gravity storm drain, under the EFR, will convey this runoff south to Basin H.

Off-site flows of approximately 650 cfs are expected from the general direction of the existing Pecos Road alignment, which is severed by the proposed Santan Freeway path. This flow will be intercepted by the proposed off-site collector channel running parallel to and north of that alignment. This collector channel drains westerly to the basin.

The third potential source of off-site water is the evacuation 100 cfs line from several existing City of Chandler retention basins mentioned previously.

Data developed for the design of the Price off-site collector system is included in Appendix III.C.

C.2 Santan Freeway Collector System

Overland flows in the less developed area west of the Price Expressway travel in a general southwesterly direction. These flows, interrupted by the freeway alignment, can be intercepted by a concrete channel draining west, parallel to and on the north side of the Santan alignment. The size and capacity of the channel increases as it nears Basin B and the contributing drainage area grows. Off-site flows coming from areas west and north of Basin B will be intercepted by a similar channel draining easterly to the basin. The channel flows will be conveyed under the Santan cross-streets via box culverts.

The accumulated 100-year off-site flows arriving at Basin B from the east and west, respectively, are 1220 cfs and 641 cfs. The 1220 cfs design flow is supplemented by maximum 50-year design pumped discharges from on-site pump stations located at the Kyrene Road (200 cfs) and McClintock Drive (200 cfs) crossings. A summary of design data for the Santan off-site collector channel is also given in Appendix III.C.

The west limit of the Santan collector system is the future I-10/Santan interchange. The area between I-10 and the Southern Pacific Railroad (SPRR) spur, as shown on Figure II.B, contributes off-site runoff to Basin B. The gravel pit shown at the north end of this area is a detention basin for areas to the north and west (Ahwatukee), and is assumed to completely control runoff from these areas (see **Hydrology Study**). During a severe rainfall event, the local area runoff collecting in the northwest quadrant of 56th Street crossing of the future freeway likely goes in two directions under existing conditions. Runoff can flow south along the west side of 56th Street, and if the street is overtopped, it can also flow south in an existing tailwater ditch which discharges to the Gila Drain. Sufficiently deep flows can overflow railroad tracks (parallel to and east of 56th Street) and flow overland to the southeast, ultimately to be cutoff by the future Santan embankment. This study proposes that the off-site collection system channel begin on the east side of 56th Street and be sized for the full 641 cfs design flow concentrating at 56th Street. This will allow for several eventualities, including the possible abandonment of the tailwater ditch or the alteration of configuration and profile of 56th Street for the freeway overpass.

The drainage concept for the Santan Freeway east of the Price/Santan interchange was prepared by Dames and Moore as reported in **Southeast Loop Highway Drainage Design Concept**, (Sept., 1988). The concept consisted of an off-site collector channel paralleling the Santan alignment from the interchange east to the Superstition Freeway. The mainline profile alternates between depressed and nondepressed sections. Where depressed, occasional on-site pump stations would be required to evacuate sag points to the off-site collector channel. Small detention areas located on the proposed channel at road crossings would reduce peak flows and therefore, the channel size. All of the runoff collected in this system drains west to the Price/Santan Detention Basin H, discussed in this report. It is assumed that Basin H is sized to accept these future flows which will mainly be on-site flows of small volume and peak flow.

D. Basin Outlet Design

The five detention basins are interconnected and evacuated with either gravity drain lines or pumped discharge pressure lines. Basins F and G gravity drain to Basins E and H, respectively. Basin B is pumped to Basin H and Basin H is pumped to Basin E. Basin E, the 'downstream' basin, discharges into the Carriage Lane Outfall at the Western Canal.

D.1 Basins F and G Gravity Outlets

The sizing of Basins F and G gravity drain lines are based upon their capacity to evacuate stored runoff volumes within a 36-hour period following the 100-year 24-hour storm.

The Basin F gravity line is proposed to be a 36-inch (n=0.012, length=4000 feet) diameter line set at a slope of 0.0013 ft/ft. At this slope the free-flow capacity is 26 cfs and capable of emptying the basin in 15 hours. In effect, this assumes a constant differential head between Basin F and Basin E of 5.2 feet which is equivalent to the elevation drop of the pipe between basins. Under a theoretical condition of one foot of hydraulic head difference between Basins F and E, the flow capacity drops to 11.5 cfs and evacuation time increases to 34 hours. Because Basins E and F are connected by a gravity line, the levels will tend to balance. Since Basin E is approximately three feet higher than Basin F, back-flow prevention is recommended for the Basin F outlet pipe. Generally, the basin water levels will draw down simultaneously and gradually over the 36-hour period, and the 36-inch outlet will allow Basin F to drain in 34 hours to Basin E at a sufficient rate, even under a minimal one foot available head difference.

The Basin G gravity line is proposed to be a 54-inch (n=0.012, length=6170 feet) diameter line at a slope of 0.00077 ft/ft. At this slope the capacity is 59 cfs and the basin evacuation time is 32 hours, although it requires a head difference of 5.2 feet between Basins G and H. Utilizing the full 36 hours allowed for evacuation, an average pipe flow of 52 cfs (head difference of 3.7 feet) would be adequate. A 54-inch pipe is recommended because it should closely meet the evacuation criteria. Basin G is only slightly higher than Basin H (1.6 feet), so back-flow prevention is not necessary.

The gravity drain inlets are located in the basin side slope and are proposed to be configured as a culvert entrance. The invert should be set at 0.5 feet above the basin bottom elevation, in order to mitigate excessive sediment transport to the downstream basins. Trash racks should be provided at the inlet ends. These lines are proposed to be installed beneath the East Frontage Road and due to their depth and function, they may not lend themselves to being combined with other on-site or off-site flows. A concrete lined low-flow channel should be incorporated into each basin (base case or multi-use) with capacity approximately equal to the design outflow.

D.2 Basins E, H and B Pressure Outlets

The basin discharge lines for Basins E, H and B are pressure lines exiting the off-site pump stations. The sizes of these lines are dependent upon pump sizes, pumping heads, line losses and run length. These factors were analyzed as part of the off-site pump station concept design (Section V) and are not specifically addressed in his section.

The considerations addressed here are grade and alignment of the pressure lines. Along the Price Expressway these lines are proposed to be located beneath the EFR, installed with minimum allowable cover to reduce trenching depths. Air relief valves may be required if profiles create high points in the lines. Along the Santan Freeway, the Basin B discharge to Basin H line has been located on the north side of the off-site collector channel where along mot of its alignment, installation is expected to be relatively conflict free and shallow. Erosion protection may be necessary where these pressure lines discharge into the receiving basin.

E. Extreme Event or System Failure Considerations

In the Price/Santan off-site drainage scenario, off-site flows combined with on-site flows are conveyed into the five stormwater detention storage areas, then evacuated to the Carriage Lane Outfall via a network of pump stations pumping basin-to-basin. The collection and evacuation systems under this scenario are vulnerable to four eventualities, any of which could lead to failure of the system and cause property damage and hazard to the public. Although these are extremely remote possibilities, emergency planning for them does not necessarily involve significant additional cost, as will be shown. Safety is already built into the system with the four feet of freeboard provided in the basins, so it would take an extreme storm event or extreme failure to result in flooding of areas adjacent to the freeway.

The four possibilities are: 1) during a major storm, full or partial mechanical failure of one or more off-site pump stations could result in detention pond overflow due to nonevacuation; 2) back-to-back major storms with the second occurring before the initial stored volume can be adequately evacuated from the detention basin; 3) a storm rainfall amount exceeding the system design storm, in this case a rainfall greater than the estimated 100-year 24-hour design storm; and 4) failure of the Carriage Lane Outfall and East Valley Tunnel system to perform as anticipated. If a basin overflows for any of these events, it is recommended that the excess water be conveyed into Price/Santan mainline pavement areas where the water can be retained until the system failure is corrected or the system has sufficient time to 'catch up' with the excess rainfall. This would necessarily result in the temporary closure of all or parts of Price/Santan system, but would prevent flooding of any residential or commercial areas in the vicinity.

For each basin an emergency overflow relief outlet shall be provided to discharge excess flows down into the Price/Santan depressed mainline areas rather than jeopardize adjacent private or public properties. It is proposed that the emergency overflow level be set at an elevation at least one foot below the four-foot freeboard elevation. Following are potential overflow schemes for Basins E, F, G, H and B.

For Basin E, an overflow spillway is proposed near Price mainline station 3234+00 which will allow overflows to pond in the East Frontage Road and upon attaining sufficient depth, spill to an access ramp which slopes down to the mainline.

At Basin F, overflow catch basins would be located near mainline station 3260+00 along the basin's west top of bank and excess flows discharged through them onto the surface of an interior ramp which leads down to the Price mainline.

At Basin G, which is adjacent to an elevated section of the Price mainline, excess flows will be forced out onto Chandler Boulevard and onto the East Frontage Road, which would be graded to convey the excess south to a point where the frontage road curb could be overtopped into a depressed mainline location, approximately station 3367+00. Refinement of the East Frontage Road profiles will be required to accomplish this.

An overflow spillway is proposed at Basin H near the northwest corner of the basin. In that location, excess flows would cascade down to interchange roadways and be temporarily retained there.

SECTION V

PUMP STATION DESIGN STUDIES

A. On-Site Pump Stations

There will be five on-site pump stations for the on-site drainage system considered in this report. Although the projected flows and capacities of the pump stations differ, they can be designed to be similar in configuration and individual pump capacity and power. The purpose of this section is to identify the optimal configuration.

To identify a cost effective design of the on-site pump stations, two alternatives were considered. The first alternative was to design a unique pump station for each site based on the peak flow into each station. The second alternative was to design the pump stations to have identical pumping equipment, wet wells, and buildings. The primary difference in the pump stations of the second alternative is in the amount of underground storage provided to attenuate the peak flows to keep the pump station influents equal.

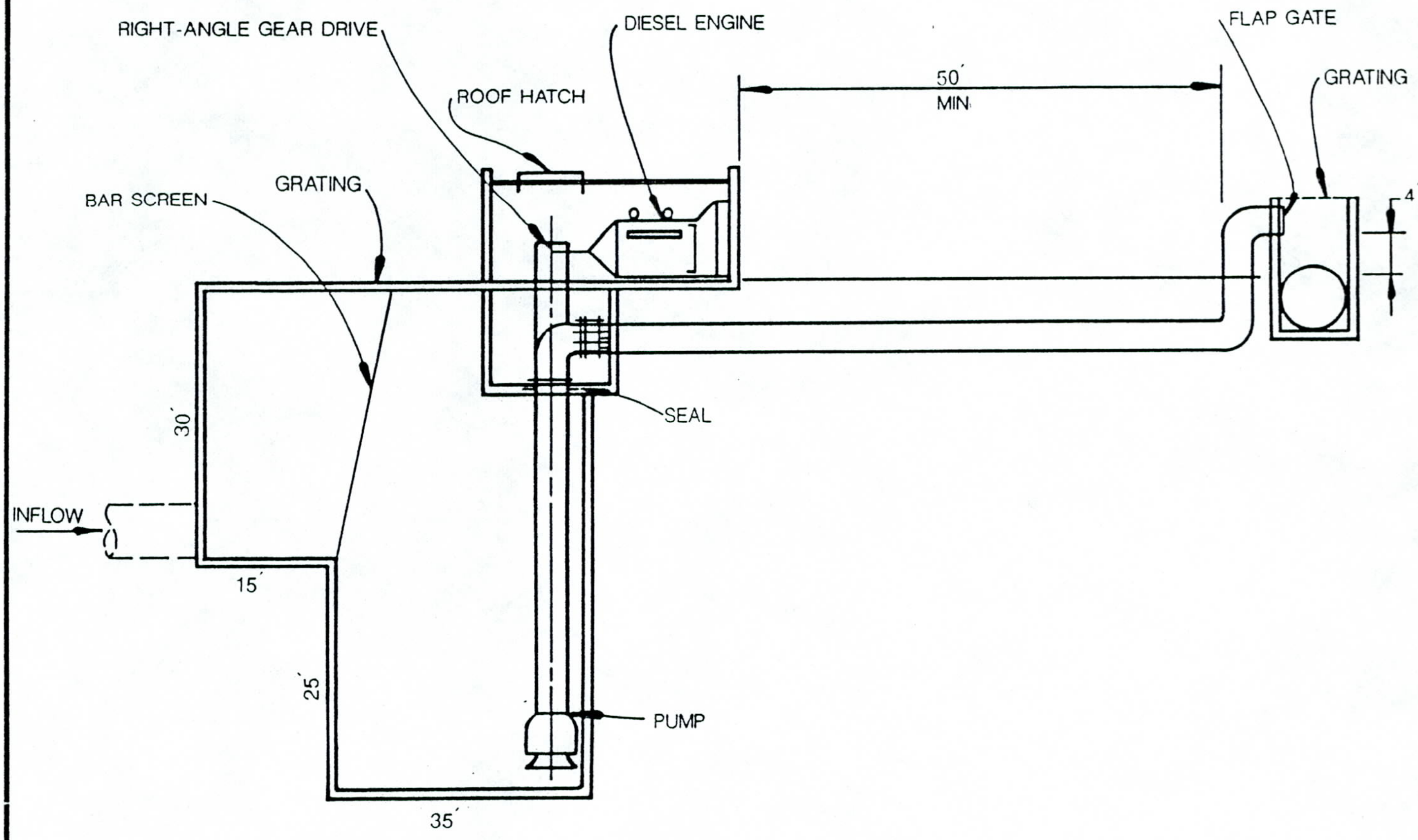
For the following reasons, HDR concludes that the most cost effective pump station system would be one designed for the second alternative. Identical pump stations would have the following advantages:

- They would be less costly to design
- There could be a significant cost savings if all pump stations were bid at the same time
- Pumping equipment could be interchangeable between pump stations (if they were all bid at the same time)
- Operation of the pump stations would be virtually identical to each other
- Long term maintenance is simplified

Simplicity of operation and maintenance is also considered to be an important factor, though not entirely quantifiable in terms of cost.

HDR reevaluated the pump station configurations presented in a previous report by Boyle Engineering Corporation, entitled **Storm Drainage Pump Station Study** (1986). An important consideration for these stations is the possibility of volatile fumes being ignited by the engine drives. There were essentially three "explosion resistant" designs proposed in the Boyle report. HDR has developed a modified version of the Type II design in that report which we believe will adequately address this concern, and also meet the cost effectiveness criteria mentioned above.

Figures V.1 and V.2 show a typical on-site pump station section and site plan, respectively. The wet well is vented at the top and cannot be accessed from the discharge gallery or engine room. The discharge gallery is sealed between the engine room and the wet well. The controls are housed in a separate air-conditioned room. The following features are also included to meet the cost-effectiveness criteria and the conditions expressed in an ADOT District #1 letter and



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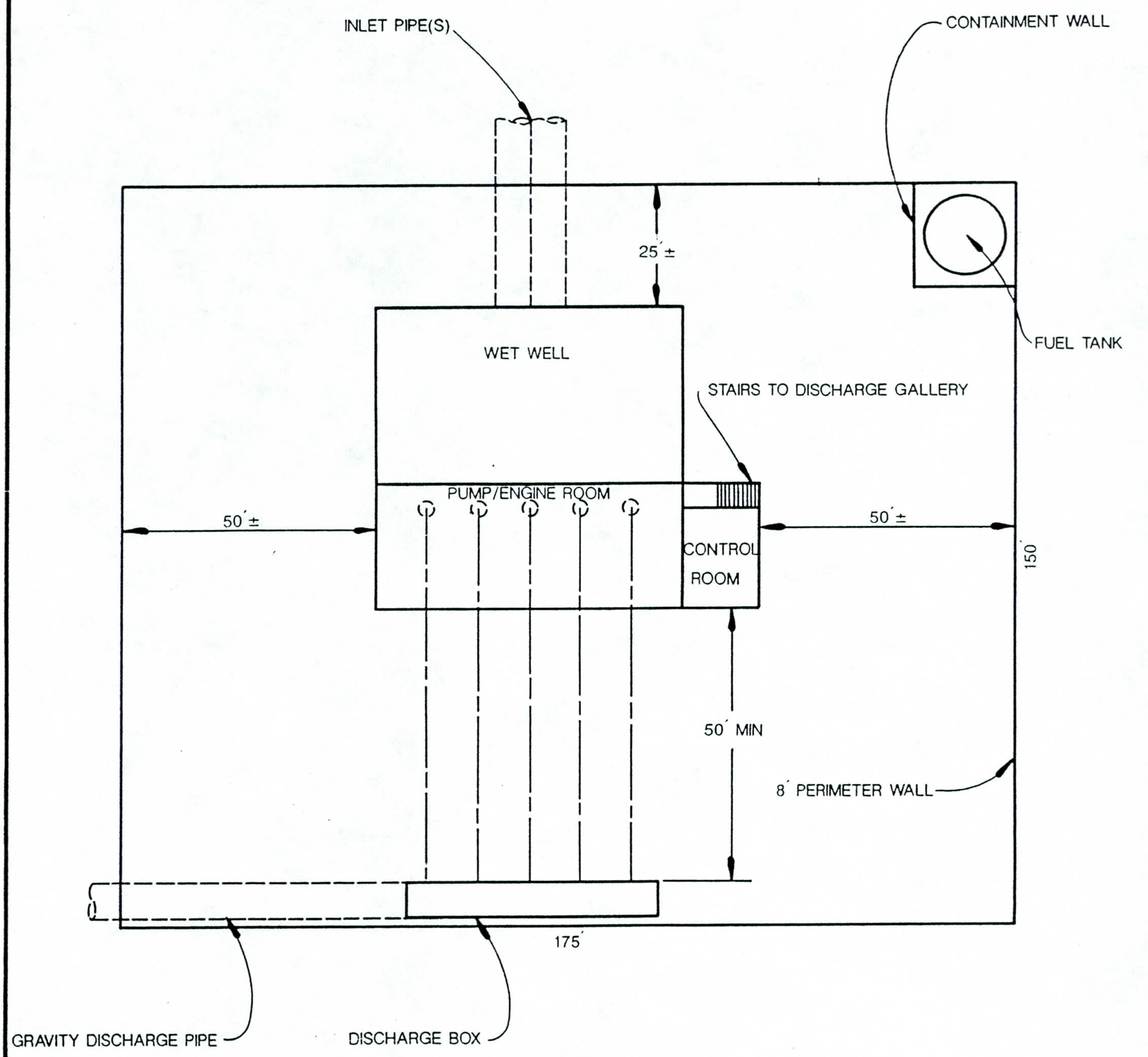
GENERAL CONSULTANT
TRACS NO. H-2222-O1D

CONCEPT DRAINAGE REPORT
FIGURE V.1
ON-SITE PUMP STATION TYP. SECTION

HDR



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TRACS NO H-2222-01D

CONCEPT DRAINAGE REPORT
FIGURE V.2
ON-SITE PUMP STATION SITE PLAN

HDR

DeLeuw Cather Company response with regard to Outer Loop (Pima Expressway) on-site pump stations. These are included in Appendix II.A.1.

Pump Station Design Criteria:

- Diesel Driven
- Right-angle drives/vertical turbine or mixed-flow pumps
- Fuel tanks above ground with 12 to 18 hours of operational capacity
- Industrial grade control system
- 8 foot high perimeter wall
- One standby unit

With the pump station shown on Figure V.1, the ADOT criteria is met along with the following additional criteria:

- Engines are on solid ground
- Engines can be removed by sliding them out, away from the pumps
- Pumps can be pulled through roof hatches
- Pumps discharge to a common header box
- Bar screen between inlet and wet well

This pump station could be sized for any number of pumps with the cost for the pump stations proportional to the number of pumps. The wet well is sized according to the **Hydraulic Institute Standards** [14th Edition, 1983] for pump stations. The sizes shown are such that the approach velocities are less than or equal to 2 fps. The dimensions of the pump station should not be any smaller than shown. However, to maintain a minimum pump cycle time, a commonly used formula for sizing the wet well is

$$V = \frac{\theta Q}{4}$$

where

θ = Minimum pump cycle time (20 minutes)
Q = Pump capacity in gpm
V = Volume in gallons

For cost estimating, the size of the wet well was checked using the sizes shown on Figure V.1 and the cycle time formula. If the volume based on the cycle time was larger, it was assumed that the width would be increased between the bar screens and the pumps. The cost of the pump station was then calculated based on 100 percent of the **Hydraulic Institute** recommended size plus the percent increase in size to maintain the cycle time. The following formula was used to calculate the total cost of the pump station:

46,619N+474,378+(100%+W%) * (41,881N+303,622)

where N = # of pumps (not including the standby pump)
W = percent increase in wet well volume required to maintain the minimum cycle time.

HDR investigated the economics of providing underground storage in large pipes and found that some storage can reduce the required size of pumps or the number of pumps required, and therefore, can reduce cost of the pump station. To derive the least costly combination of pumps versus storage, HDR developed the tables in Appendix A.2 and 3. The tables in Appendix II.A.2 compare the cost of providing from four to eight pumps at each location without storage. The tables and attached graphs show that the minimum cost is for a station with four or five pumps, plus one standby unit. However, in most cases the least cost number of pumps would result in very large pumps that may not be readily available and therefore, in effect, more costly than our estimate shows. The lack of competitive bids on cost is a factor that cannot be predicted or quantified, but is important in the overall selection process. In addition, each pump station would be somewhat unique and full operation and maintenance advantages of identical stations would not be achievable for this approach.

HDR reviewed catalogs from a number of pump manufacturers and concluded that an individual pump capacity of less than 24,000 gpm would provide good flexibility in operation and be available from at least five or six different manufacturers. A pump station with four pumps (plus one standby), each with a capacity of approximately 22,400 gpm would be capable of pumping approximately 90,000 gpm or 200 cfs. Establishing 24,000 gpm as the maximum size of pumps, HDR developed a second set of tables which consider the effect of underground storage upon cost. The storage pipes are 96-inch in diameter pipes intended to be placed in an approximate 1000-foot long section parallel to the mainline roadway. Thus, the on-site gravity storm sewer planned for that 1000-foot section would not be required, and each station is given a credit for eliminating a 1000-foot section of mainline gravity sewer.

The tables in Appendix II.A.3 show estimated costs for the pump stations with underground storage. Costs are developed for peak flows ranging from 100 to 250 cfs. Underground storage varies for each pump station and is reflected in the total cost. The graphs in Appendix II.A.3 compare the costs for the different flow rates evaluated.

Pump station costs are compared in Table V.1. The 200 cfs station (with storage) is the least or equivalent cost station for all but the Price/Santan station. Since the five stations can be made virtually identical if the 200 cfs alternate is selected, HDR recommends that the 200 cfs be adopted for the Price/Santan on-site pump stations. In general, the cost differences with or without storage capacity station were not significant. The characteristics of the recommended stations are presented in Table V.2. The required storage, stated in terms of length of 96-inch diameter pipe, is given for each station in Table V.2. Otherwise, the pump station configurations are identical except for minor differences due to site adaptation.

Table V.1 - Comparison of On-Site Pump Station Costs, $1000

Pump Station (Peak Inflow)	**Without Storage**	**With Storage (200 cfs/ Station)**	**Minimum Cost**
Elliot Rd. (287 cfs)	$2,354	$2,355	$2,354 w/o Storage
Warner Rd. (247 cfs)	2,143	2,115	2,115 w/Storage
Kyrene Rd. (212 cfs)	2,021	1,815	1,815 w/Storage
McClintock Dr. (295 cfs)	2,354	2,060	2,060 w/Storage
Price/Santan TI (385 cfs)	2,804	2,955	2,804 w/o Storage
Total	**$11,676**	**$11,300**	**$11,148**

It is also recommended that ADOT consider purchasing all of the on-site pump station equipment in a separate equipment purchase contract. The site civil works design can proceed knowing exact dimensions and requirements of the supplied equipment. The station can then be constructed and the equipment installed in a separate contract. This has the following potential advantages:

- Equipment purchased in quantity directly from a supplier is likely to be less costly.
- The civil works construction bids may be favorable because there are fewer contingencies and unknowns.

Table V.2 - Characteristics of Recommended On-site Pump Stations

Total Flow	200	cfs
Total Flow	89,760	gpm
Number of Pumps	4	plus 1 standby
Flow per Pump	22,440	gpm
Total Dynamic Head	40	feet
BHP per Pump	300	
Discharge Pipe Dia./in	30	inches
Discharge Pipe Velocity	10.2	fps
Cycle Time Wet Well Vol.	59,959	cu. ft.
HI Wet Well Vol.	70,125	cu. ft.
Flow with		
4 pumps running	89,700	gpm
3 " "	67,320	gpm
2 " "	44,880	gpm
1 " "	22,440	gpm
<u>Storage (Length of 96-inch Diameter Pipe)</u>		
Elliot Road	2,400	ft.
Warner Road	1,600	ft.
Kyrene Road	0	ft.
McClintock Road	1,200	ft.
Price/Santan TI	1,400	ft.

B. Off-Site Pump Stations

The purpose of the off-site pump stations is to pump water from three off-site detention basins, B, H, and E, to the Carriage Lane Outfall (CLO). Two system configurations were considered to accomplish this. The first system is a basin-to-basin configuration, i.e. Pump Station B would pump from Basin B to Basin H, Pump Station H would pump to Basin E, and Pump Station E would pump to the CLO. With this system, water from Basin B would be pumped three times and water from Basin H two times.

The second is a pressure system with a single pipeline from Basin B to the CLO. Each pump station would pump into the pipeline and would pump only the water collected off-site or from the local on-site pump stations. With this system, the only water that would be pumped more than once is the water from the on-site pump stations.

A cost analysis was conducted to select the optimal system considering cost as a major factor. Other factors such as operation, maintenance, interchangeability, and constructability were also considered in the selection process.

As previously discussed, ADOT and Chandler have a combined peak flow allocation of 150 cfs into the CLO Head Structure. However, it is possible that during off-peak periods, pumping greater quantities up to the maximum off-peak capacity of the CLO will allow ADOT to reduce the required detention basin sizes and to evacuate the basins more quickly. Three flow rates into the CLO were evaluated and applied to each system. The flow rates are: High, 450 cfs; Medium, 350 cfs; and, Low, 250 cfs. Table V.3 shows the projected outflow rates from each of the pump stations for the basin-to-basin and pressure systems respectively. The required detention basin volumes for each outflow rate were determined using the HEC-1 model developed in the **Hydrology Study** (1989).

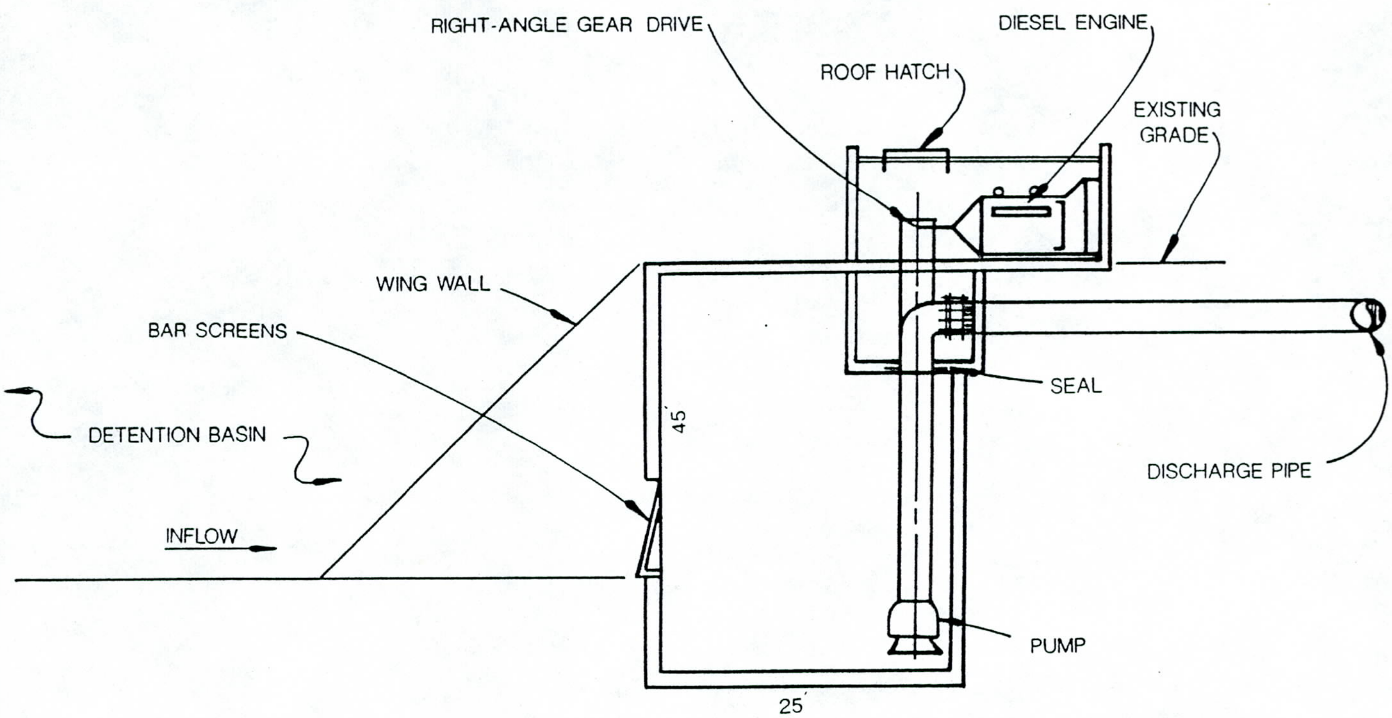
Table V.3 - Off-site Pump Station and Detention Basin Data

	Basin B			Basins G & H			Basins E & F		
Flow Rate	Outflow cfs	Volume ac-ft	Area ac	Outflow cfs	Volume ac-ft	Area ac	Outflow cfs	Volume ac-ft	Area ac
Basin-to-basin System									
High	150	570	25.6	340	549	30.3	450	231	15.2
Medium	120	604	26.0	260	625	33.0	350	248	16.0
Low	85	658	26.4	190	721	35.9	250	260	16.7
Pressure System									
High	150	570	25.6	225	549	30.3	75	231	15.2
Medium	120	604	26.0	180	625	33.0	60	248	16.0
Low	85	654	26.4	140	721	35.9	25	260	16.7

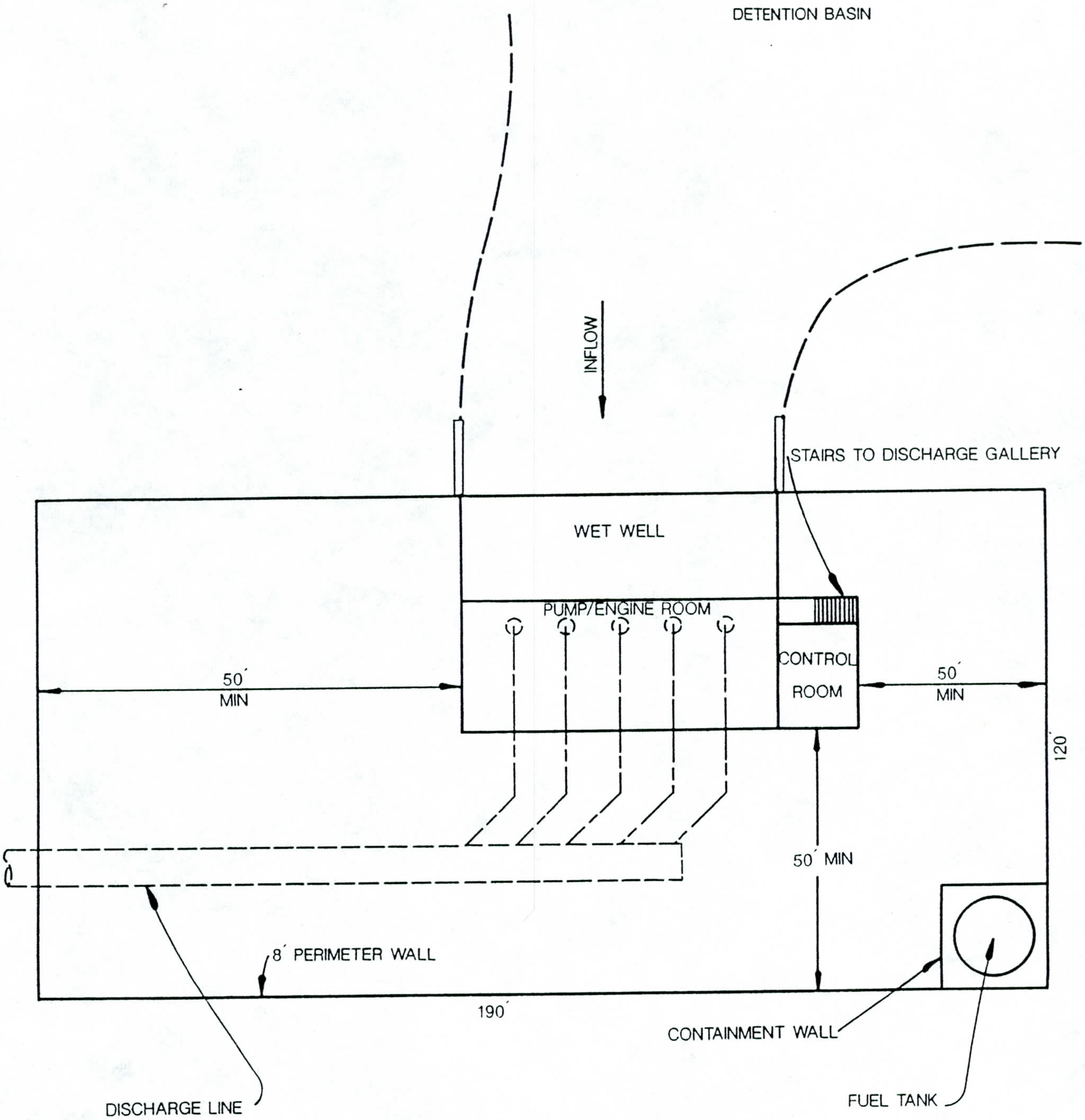
The first step in sizing the pump stations was to determine a workable floor plan and cross section. The plan considered most efficient and cost effective is shown on Figures V.3 and V.4. This plan will work for any size pump station, either off-site or on-site. For cost estimating, the following components in the building were considered:

- Building, including pump and control rooms
- Wet well
- Excavation and backfill
- Gratings, louvers, and hatches
- Controls
- Discharge piping
- Control valves
- Perimeter wall

The cost for these items in the pump station building is approximately $615,000 plus $100,000 per pump.



CONTRACT 88-24 PRICE EXPRESSWAY GENERAL CONSULTANT TRACS NO. H-2222-01D	**CONCEPT DRAINAGE REPORT** **FIGURE V.3** **OFF-SITE PUMP STATION TYP. SECTION**	HDR



CONTRACT 88-24
PRICE EXPRESSWAY
GENERAL CONSULTANT
TRACS NO H-2222-01D

CONCEPT DRAINAGE REPORT
FIGURE V.4
OFF-SITE PUMP STATION SITE PLAN

HDR

The wet well is sized according to Hydraulic Institute standards and the building is sized for the pumps to be located 10 feet apart, center-to-center, with about four to six feet between pumps. With the engines located adjacent to an access area, they can be removed easily if necessary. The pumps, due to their size and type, will have to be installed and/or removed through a roof hatch using a crane.

B.1 Basin-to-basin System

For a pump station to have the greatest flexibility, it needs to operate under a variety of conditions. Multiple pumps are required to achieve this end. Although the cost of the building (per pump) is consistent for the different flow rates, the cost of the pumps, engines, and fuel storage tanks varies for each scenario.

Pumps for this application are available from several manufacturers. HDR sought prices for mixed-flow or vertical turbine pumps that could pump up to 450 cfs with a head of approximately 70 feet. The prices received ranged from about $40,000 to over $300,000. The manufacturer of the most expensive pumps admits that the pump is expensive. Pumps that operate in this range will therefore cost approximately $150,000, including the right-angle gear drive and column. With centrifugal and turbine pumps, the flow rate is proportional to the speed in rpm of the pump, and is proportional to the square root of the head. The following equations show the relationship:

$$\frac{S_1}{S_2} = \frac{Q_1}{Q_2} = \frac{\sqrt{H_1}}{\sqrt{H_2}}$$

in which,

H = Head in feet
Q = Flow rate in gpm
S = Speed in rpm
Subscript 1 is for original conditions; subscript 2 is for altered conditions

The above relationships are known as the *Affinity Laws*. By applying the affinity laws, one model of pump is able to meet different flow conditions.

One of the problems in sizing the pumps for the different conditions is the fact that not all of the pumps will be operating at the same time. The pump stations must be designed to pump the maximum flow at the maximum head. However, the design flow is for a 100-year, 24-hr storm. Most of the pumping will result from the smaller storms that occur annually. Consider a pump station with six pumps at Basin E with a static head of 30 feet, pumping 450 cfs through a 72-inch diameter pipe of length 5,200 feet. To pump the total of 450 cfs (202,000 gpm), each pump would have to pump 34,000 gpm at a total dynamic head (TDH) of approximately 70 feet. However, if only one pump was needed, the TDH would be only 31 feet. On every pump curve reviewed, a change of 39 feet of head moved the flow completely off of the pump curve to the "right" and into a range where the Net Positive Suction Head (NPSH) is too high to operate.

Cavitation would result if operated in this range. Therefore, the pumps will have to be controlled to ensure that they operate within an acceptable range.

There are several alternative methods to control the flows and keep them on the curve. One method consists of a back-pressure device such as a ball valve. This method would require a pressure sensor in the line and would force the pumps to operate at the same TDH at all times.

Another method is to use a flow sensor in the discharge line that would signal the engine to slow down or speed up, depending on the head conditions. Thus, the pumps would operate within the curve at different flow/head combinations. There are other methods and the preferred system would be established in the detailed design process.

Cost estimate data for the basin-to-basin system, for each flow, with from three to seven pumps for Stations B and H, and from four to eight pumps for Station E, are given in Appendix II.B.1 The last table in Appendix II.B.1 is a summary of the costs, using six pumps for Station E, five for Station H, and four for Station B.

B.2 Pressure System

The pump sizes and TDH's for the pressure system pumps are completely different than the basin-to-basin system pumps. For Basin B, the flow will be the same and the TDH will double. At Basin H, the flow will decrease and the TDH will increase. At Basin E, the TDH will remain the same but the flow will be significantly less. Characteristics of the stations are given in Appendix II.B.2.

Mixed-flow pumps used for estimating the costs for the basin-to-basin system are not as available for pumping flows at the required higher heads unless staging is available. At Station B, two or more stages would be required to meet the pumping conditions. This requirement limits the number of pumps that are available because mixed-flow units are generally not available in more than two-stage configurations. Vertical turbine pumps, therefore, would be needed. A check of pump catalogs confirmed that there are four or more manufacturers who could provide vertical turbine pumps to meet the pumping requirements at all of the pumps stations. However, because of the potential to have solids and debris in the water, screens such as those used in wells, would be necessary to protect the pump impellers.

The problem with varying heads, as discussed previously, is even greater with the pressure system. For example, if all pump stations were pumping the maximum flow, the TDH at Station B would be approximately 320 feet. However, if Station B was pumping its maximum and the others were not pumping at all, the TDH at B would be only 265 feet. Therefore, controls to manage the flows and keep the pumps operating within the pump curve at minimum conditions would be needed, as for the basin-to-basin system.

The last table in Appendix II.B.2 is a comparison of pump station characteristics and costs for each of the three pump stations for the three flows (High, Medium, and Low) for the pressure system, using six pumps for each station. The number of pumps was chosen because of the operational flexibility. However, the number could be adjusted to provide any number of pumps per station.

B.3 Discussion and Recommendations

The following observations and conclusions can be drawn from this study:

1. The basin-to-basin system is nominally less costly than the pressure system.
2. The basin-to-basin system is operationally more flexible than the pressure system.
3. Pump stations cannot be identical for either system.
4. The pumps of either system will likely require speed or head control to avoid cavitation.
5. There is less than a five percent difference in the total system cost for the 250,350 and 450 cfs scenarios. Though slightly more costly, the 450 cfs system will evacuate the basins more rapidly. The average evacuation time following the end of the 24-hour design storm is 42 hours for the 450 cfs system compared to 80 hours for the 250 cfs system.

As a result of this analysis, the basin-to-basin system is the preferred system. Although the costs are not significantly lower for this system, there are operational advantages. Because the basins are interconnected and basin levels will be monitored from a remote location, basin storage can be maximized during a severe, but localized runoff event.

A disadvantage of this system is that the pump stations are interdependent. That is, if Station B or H operate, E must also operate, etc. This is not a significant disadvantage because four feet of freeboard in the basins provide substantial surcharge storage to prevent overflow should one or more pumps fail to operate. Furthermore, assuming the stations are properly maintained and periodically tested, the likelihood of more than one pump failing to operate is very small. Therefore standby pumping capacity, as required for on-site stations, is not considered to be necessary for off-site stations. The primary impact of pump outage is to extend the evacuation time.

The primary factors contributing to conclusion #5 above are, 1) that pump station costs do not rise in proportion to discharge, and 2) basin volumes did not decrease in proportion to the increase in evacuation rate. Given that a multiple-pump station is required for operational flexibility, the basic pump station for the 250 cfs scenario is already a large station. Providing a pump, engine, wet-well, and discharge pipe to nearly double the discharge to 450 cfs results in less than a 50 percent increase in total cost, which explains why the basin-to-basin pump station cost only increased from $12.9 million to $17.8 million (see Appendix II.B.1).

In general, the inflow rates to the basins greatly exceed the evacuation rates during the peak of the design storm hydrograph. The net inflow volume during the during the time when the inflow exceeds the outflow governs the basin size, and this time period is short. The time period during which the inflows exceed outflows ranges from about 4 to 10 hours. For example, increasing the CLO discharge from 250 to 450 cfs (i.e. 85 to 150 cfs for Basin B) for the four-hour peak inflow period for Basin B results in only a 88-acre-foot or 13 percent reduction in basin volume (See Table V.3). Therefore basin costs decreased only about 12 percent when the outflow was increased from 250 to 450 cfs.

However, increasing the discharge rate from 250 to 450 cfs results in an equally important reduction in the total evacuation time. Most jurisdictions in the area apply a 36-hour evacuation time for normally dry detention basins for design storms ranging from the 50-yr 24-hr(Gilbert) to 10-yr 2-hr(Phoenix pre-1985). The 450 cfs system will very nearly meet this criteria for the 100-yr 24-hr design storm, and will easily evacuate the system in less than 36 hours for storms of higher frequency and/ or shorter duration, such as the 50-yr 24-hr storm. Furthermore, the shorter evacuation time will make the system less vulnerable to being surcharged during back-to-back storms or extended low intensity storms. These advantages favor the 450 cfs system despite the minor (5 percent) increase in cost.

The 450 cfs maximum discharge is a condition imposed by the capacity of the Carriage Lane Outfall and East Valley Tunnel system and an intergovernmental agreement for sharing this capacity. The details of the Carriage Lane outfall hydraulics and the IGA were discussed in Section I and Appendix I.

For the reasons presented in the foregoing discussion, the basin-to-basin pumping system with a maximum discharge into the Carriage Lane Head Structure is the recommended system. Details of the recommended pump stations are presented in Table V.4. A typical pump station cross-section is shown in Figure V.3 and a typical site plan is shown in Figure V.4.
Except that standby capacity is not recommended for the off-site stations, the off-site stations are configured like the on-site stations (i.e. diesel engine-driven, right-angled direct drive, vertical turbine pumps, etc.). However, as can be seen in Table V.4, the off-site stations are not identical and vary in number of pumps, size of pumps, and other factors.

Table V.4 - Characteristics of Recommended Off-site Pump Stations

STATION	B	H	E
Total Flow, cfs	150	340	450
Total Flow, gpm	67,000	153,000	202,000
Pipe Diameter, inches	72	96	84
Outlet Pipe Length, feet	17,600	17,600	5,200
Static Head, feet	60	30	30
Head Loss, feet	20	20	20
Total Dynamic Head, feet	80	50	20
Total BHP @ 80% eff.	1,690	2,410	3,180
Outlet Pipe Velocity, fps	5	7	12
Required Wet Well Volume, Cu.ft.	44,750	102,191	134,919
PUMP DATA:			
Number of Pumps	4	5	6
Flow per Pump, gpm	16,750	30,600	33,670
BHP per Pump (80% eff.)	420	480	530
WWet Well Volume, cu. ft.	64,500	77,400	90,300
Needed Add'l Wet well Vol.,Cu.ft.	0	24,791	44,619
# Pumps Running	***************	Total Flow, gpm	**********
6			202,020
5		153,000	168,350
4	67,000	122,400	134,680
3	50,250	91,800	101,010
2	33,500	61,200	67,340
1	16,750	30,600	33,670
COST DATA			
Building/Wet Well	914,500	1,070,900	1 ,208,900
Fuel Storage Tank	75,600	108,000	143,100
Engines	240,000	300,000	360,000
Pumps	374,000	606,000	764,000
Subtotal	689,600	1,014,000	1,267,100
PUMP STATION TOTAL	$1,604,000	$2,085,000	$2,476,000
Pipe Unit Cost per ft.	$180	300	240
Total Pipe Cost	$3,168,000	5,280,000	1,248,000
SYSTEM TOTAL	$4,772,000	$7,365,000	$3,724,000

SECTION VI

SYSTEM MONITORING, CONTROL, AND TELEMETRY

The Price/Santan drainage system proposed herein is a complex interrelated system of eight pump stations and five detention basins. At the present time, the proposed Freeway Management System (see Appendix IV.A) includes a design guide for monitoring the operation of pump stations at a central location (ADOT District #1 Headquarters). The sensors that are to be provided in the pump station are listed in the **Freeway Management System Design Guide** (Kimley-Horn, Oct. 1989). The applicable portion of this guide is included in Appendix IV.

It is recommended that ADOT consider including remote control functions as well as monitoring functions for the management of the Price/Santan drainage system. In addition, there are facilities other than pump stations that should be monitored; namely, detention basins and junction structures such as the CLO Head Structure.

The design of the Price/Santan drainage system requires that discharges into the CLO Head Structure be reduced to 150 cfs during peak flow conditions. A level sensor should be provided at the CLO Head Structure which would monitor elevations and sound an alarm when elevations in the CLO are at or near peak design condition. This will alert District #1 headquarters that the evacuation pump station discharges (Pump Station E,H and B) need to be reduced to 150 cfs total into the CLO Head Structure. The manner in which the stations are to be feathered will depend upon basin levels since the pump stations are linked basin-to-basin. Therefore, basin levels should also be monitored.

Furthermore, in the event of extreme event conditions or a failure somewhere in the system, such as discussed in Section IV, the ability to control pump stations from the monitoring center would allow ADOT to provide rapid response to emergencies and reduce overall impacts.

For the purpose of the FMS system planning, HDR has provided ADOT with a list of pump stations and basins of the Price/Santan system. This list is included in Appendix IV. Outer Loop pump stations have been included in the list, however, there may be additional locations on the Outer Loop, such as associated with the East Valley Tunnel system that may be desired. These are presently under consideration by the Outer Loop Management Consultant.

Until the fiber-optic FMS system is installed, the remote monitoring to District #1 is being accomplished by the existing telemonitoring system.

SECTION VII

COST ESTIMATE

CLIENT	Arizona Department of Transportation	JOB NO.	173-39-44
PROJECT	Price Expressway	ESTIMATED	ETL
SECTION	Price Expressway	CHECKED	JZ
STATIONS	3095+22 to 3393+00	DATE	04-90
LENGTH	5.6 Miles	REVISED	06-90

Drainage Concepts

Price Expressway
(Baseline Road to Frye Road)

ITEM	DESCRIPTION	QUANTITY	UNIT	UNIT COST	TOTAL COST
1	Drainage (On-site):				
	On-site Pump Stations	2	EACH/LS	$1,900,000	$3,800,000
	(Gravity Storm Drain)				
	Pipe, Rein. Concrete, 36"	1,510	LF	$68	$102,600
	Pipe, Rein. Concrete, 42"	1,580	LF	$75	$118,500
	Pipe, Rein. Concrete, 60"	1,990	LF	$135	$268,600
	Pipe, Rein. Concrete, 66"	2,710	LF	$160	$433,600
	Pipe, Rein. Concrete, 72"	3,430	LF	$180	$617,400
	Pipe, Rein. Concrete, 78"	4,670	LF	$215	$1,004,000
	Pipe, Rein. Concrete, 84"	250	LF	$240	$60,000
	Pipe, Rein. Concrete, 96"	4,050	LF	$300	$1,215,000
	Special Junction Box	5	EACH	$10,000	$50,000
	Manholes	22	EACH	$2,500	$55,000
	Other Drainage Items	1	LS	$1,902,000	$1,902,000
				Subtotal (Item 1)	$9,626,700
2	Drainage (Off-site):				
	(Gravity Storm Drain)	0	LF	$0	$0
	Pipe, Rein. Concrete, 36"	4,200	LF	$68	$285,600
	Pipe, Rein. Concrete, 54"	6,560	LF	$115	$754,400
	Pipe, Rein. Concrete, 66"	1,320	LF	$160	$211,200
	Pipe, Rein. Concrete, 96"	2,240	LF	$300	$672,000
	Pipe, Rein. Concrete, 102"	2,680	LF	$420	$1,125,600
	(Pressure Pipe):				
	84" Dia.	5,330	LF	$284	$1,513,700
	96" Dia.	17,150	LF	$346	$5,933,900
	2-8'x 6' Box Culvert	1,910	LF	$450	$859,500
	Manholes	32	EACH	$2,000	$64,000
	Catch Basin, Type 4, Double	2	EACH	$2,000	$4,000
	Basin E Pump Station	1	EACH	$2,600,000	$2,600,000
	(Basin E)				
	Excavation	309,760	CY	$3	$929,200
	Land	570,636	SF	$5	$2,853,100
	(Basin F)				
	Excavation	66,147	CY	$3	$198,400
	Land	239,580	SF	$5	$1,197,900
	(Basin G)				
	Excavation	416,540	CY	$3	$1,249,600
	Land	715,360	SF	$5	$3,576,800
				Subtotal (Item 2)	$24,028,900
		SUB-TOTAL A (Items 1 -2)			$33,655,600
3	Other Misc. Items:	(15 % of Sub-Total A)			$5,048,300
		SUB-TOTAL B (Items 1 - 3):			$38,703,900
	Contingency & Engineering:	Construction (12% of Sub-Total B)			$4,644,400
		Estimated Construction Total			$43,348,300

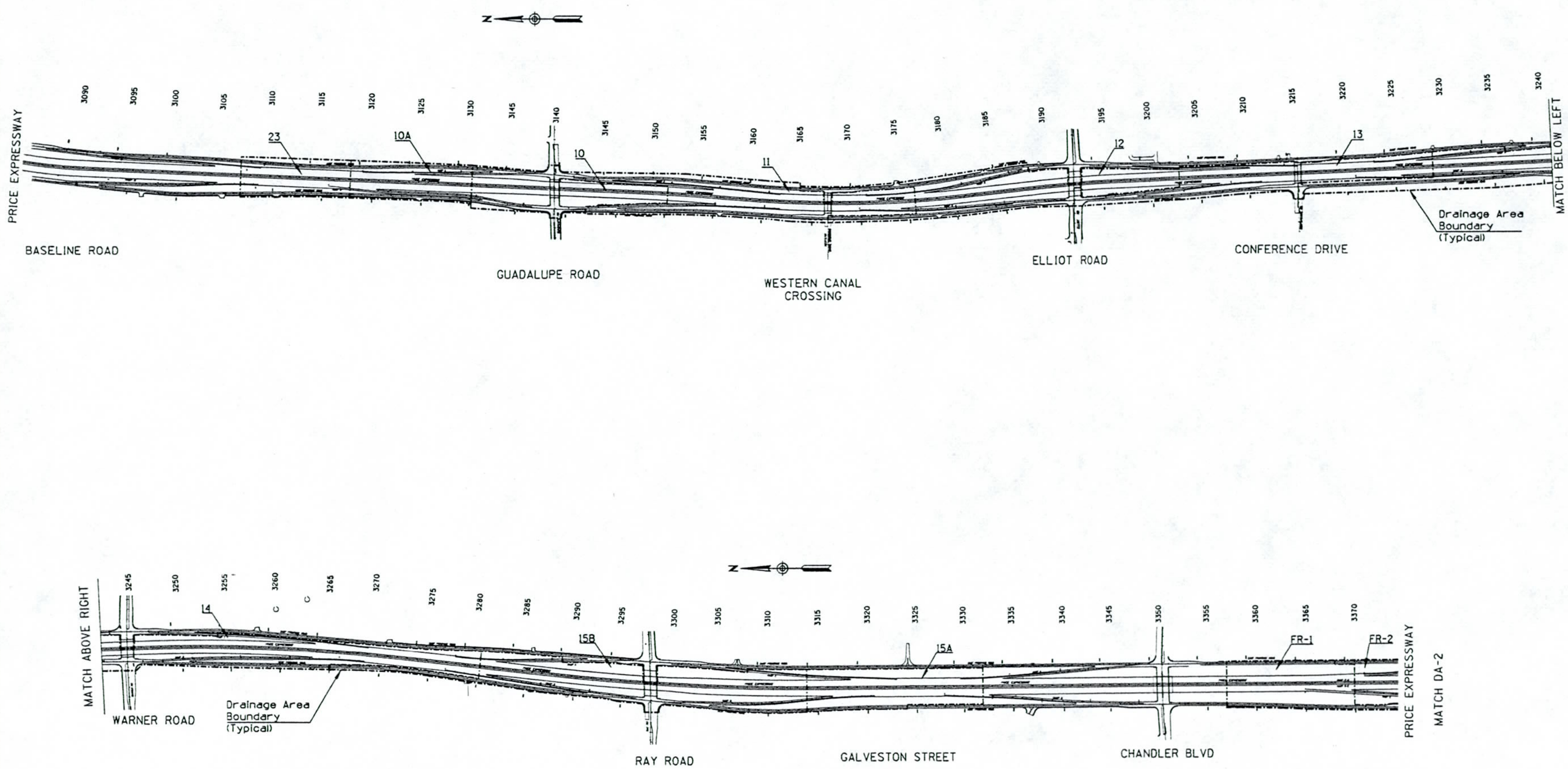
CLIENT	Arizona Department of Transportation	JOB NO.	173-39-44	Drainage Concepts
PROJECT	Price Expressway	ESTIMATED	ETL	
SECTION	Santan Freeway	CHECKED	JZ	Santan Freeway
STATIONS	1299+80 to 1570+00	DATE	04-90	(56th Street to Dobson Road)
LENGTH	5.1 Miles	REVISED	06-90	

ITEM	DESCRIPTION	QUANTITY	UNIT	UNIT COST	TOTAL COST
1	Drainage (On-site):				
	On-site Pump Stations	3.00	EACH/LS	$1,900,000	$5,700,000
	(Gravity Storm Drain)				
	Pipe, Rein. Concrete, 30"	1,000.00	LF	$50	$50,000
	Pipe, Rein. Concrete, 36"	4,150.00	LF	$68	$282,200
	Pipe, Rein. Concrete, 42"	1,660.00	LF	$75	$124,500
	Pipe, Rein. Concrete, 48"	3,600.00	LF	$85	$306,000
	Pipe, Rein. Concrete, 60"	600.00	LF	$135	$81,000
	Pipe, Rein. Concrete, 66"	180.00	LF	$160	$28,800
	Pipe, Rein. Concrete, 72"	1,090.00	LF	$180	$196,200
	Pipe, Rein. Concrete, 96"	5,600.00	LF	$300	$1,680,000
	Special Junction Box	6	EACH	$10,000	$60,000
	Manholes	24	EACH	$2,500	$60,000
	Other Drainage Items	1	LS	$1,076,000	$1,076,000
				Subtotal (Item 1)	$9,644,700
2	Drainage (Off-site):				
	(Pressure Pipe):				
	72" Dia.	17,470	LF	$244	$4,262,600
	Channel, Conc. Lined, 20'	1,920	SY	$22	$42,200
	Channel, Conc. Lined, 34'	21,607	SY	$22	$475,300
	Channel, Conc. Lined, 36'	54,137	SY	$22	$1,191,000
	Channel, Conc. Lined, 43'	5,967	SY	$22	$131,200
	Pipe, Rein. Concrete, 42"	310	LF	$75	$23,200
	Pipe, Rein. Concrete, 68"x 43"	1,450	LF	$115	$166,700
	2-10'x 4' Box Culvert	350	LF	$517	$180,900
	3-8'x 8' Box Culvert	270	LF	$675	$182,200
	3-10'x 5' Box Culvert	100	LF	$780	$78,000
	Basin B Pump Station	1	LS	$1,700,000	$1,700,000
	Basin H Pump Station	1	LS	$2,200,000	$2,200,000
	(Basin B)				
	Excavation	909,920	CY	$3	$2,729,700
	Land	1,154,340	SF	$5	$5,771,700
	(Basin H)				
	Excavation	563,053	CY	$3	$1,689,100
	Land	784,080	SF	$5	$3,920,400
				Subtotal (Item 2)	$24,744,200
		SUB-TOTAL A (Items 1 -2)			$34,388,900
3	Other Misc. Items:	(15 % of Sub-Total A)			$5,158,300
		SUB-TOTAL B (Items 1 - 3):			$39,547,200
	Contingency & Engineering:	Construction (12% of Sub-Total B)			$4,745,600
		Estimated Construction Total			$44,292,800
	Combined Drainage Construction Total (Price Expressway and Santan Freeway)				$87,641,100

SECTION VIII

PLAN SHEETS

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



	NAME	DATE
DESIGN		
DRAWN		
CHECKED		

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION

DRAINAGE AREA PLAN
STA 3085+00 TO STA 3374+00

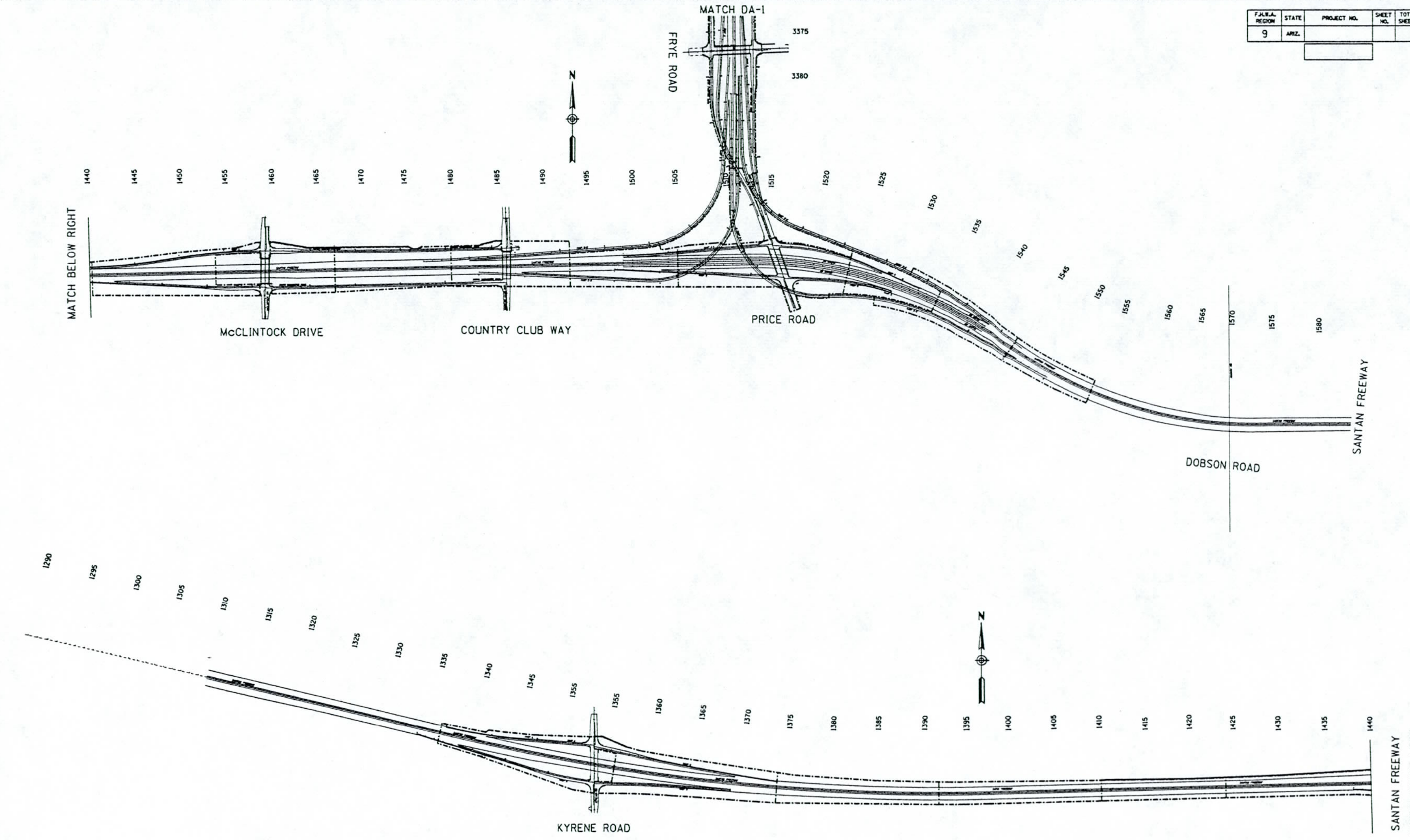
ROUTE | MILEPOST | LOCATION

DWG NO. DA-1 OF 2

TRACS NO.

DAPRT001

___ OF ___



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

	NAME	DATE
DESIGN		
DRAWN		
CHECKED		

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION

DRAINAGE AREA PLAN
STA 1290+00 TO STA 1570+00

ROUTE | MILEPOST | LOCATION

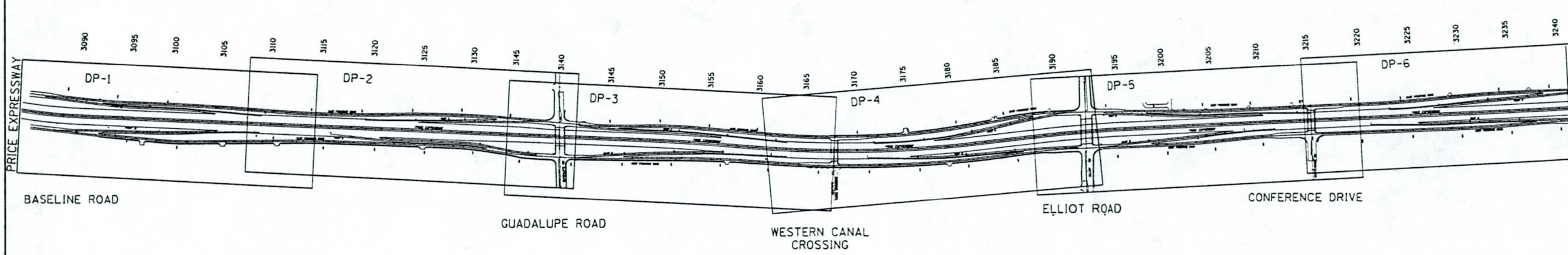
SHEET DA-2 OF 2

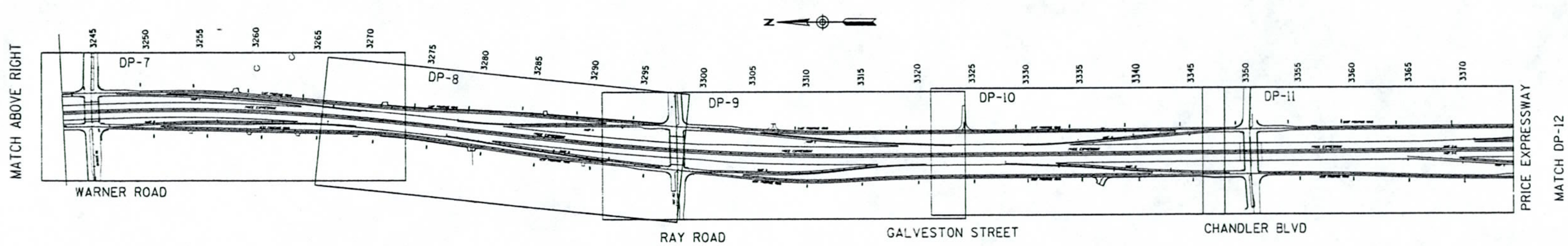
TRACS NO.

DASAM102

___ OF ___

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





	NAME	DATE
DESIGN		
DRAWN		
CHECKED		

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION

DRAINAGE PLAN KEY MAP
STA 3085+00 TO STA 3374+00

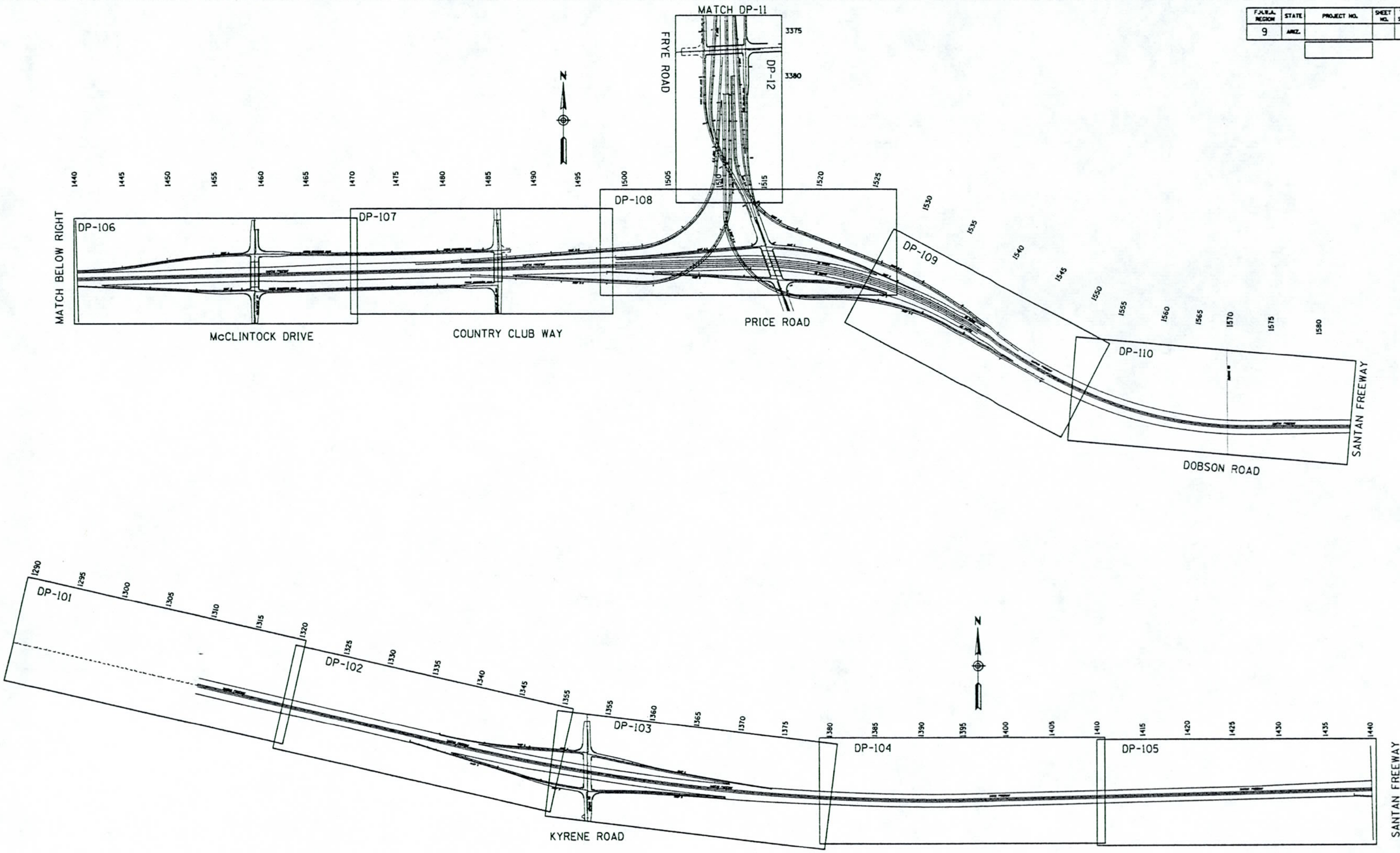
ROUTE | MILEPOST | LOCATION

DWG NO. 1 OF 2

SURVEY NO. | FINISHED PLANS | REVISIONS | LOCATION | DATE

TRACS NO. KYPR1101-102 ___ OF ___

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



	NAME	DATE
DESIGN		
DRAWN		
CHECKED		

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION

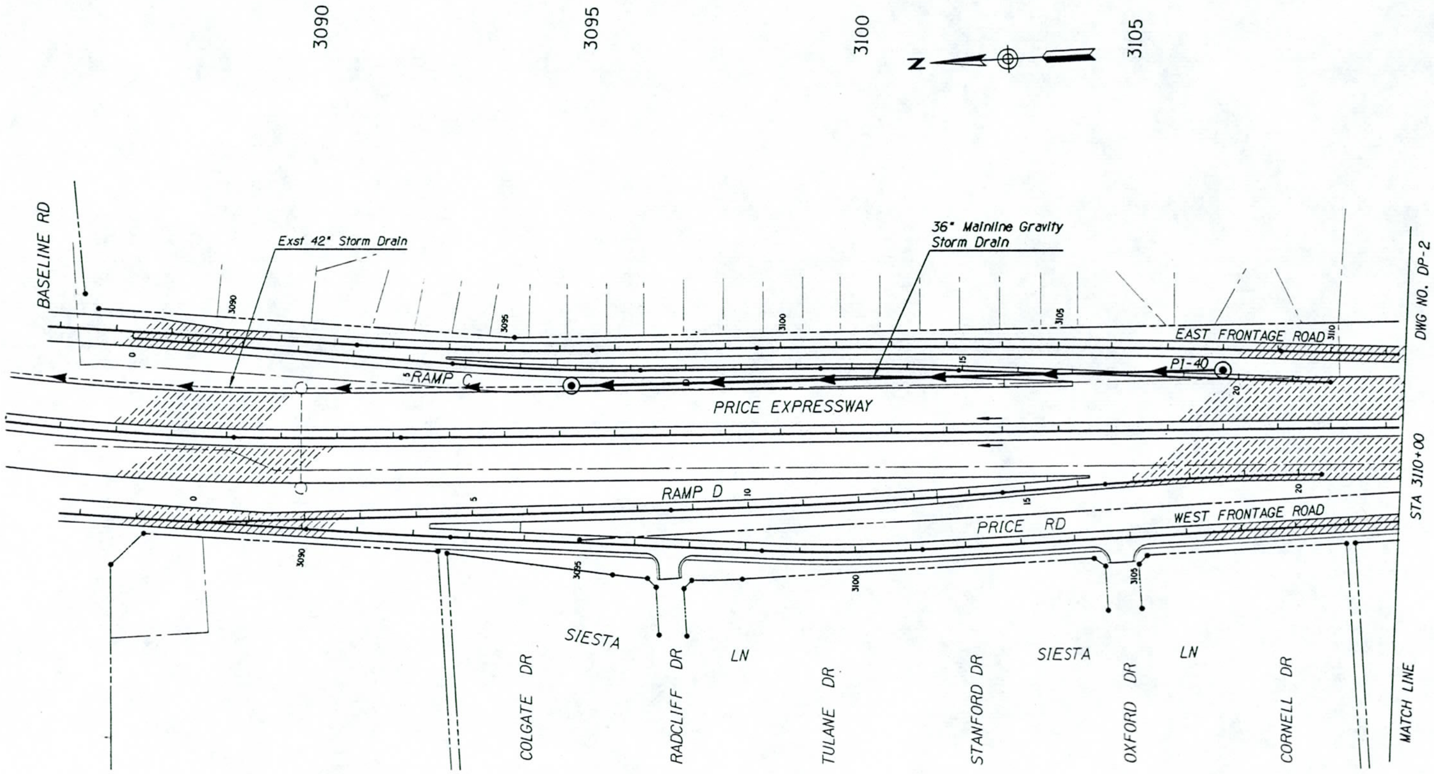
DRAINAGE PLAN KEY MAP
STA 1290+00 TO STA 1570+00

ROUTE | MILEPOST | LOCATION

DWG NO. 2 OF 2

TRACS NO. KYSAN105 ___ OF ___

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



SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

	NAME	DATE
DESIGN		
DRAWN		
CHECKED		

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION

DRAINAGE PLANS
STA 3085+00 TO STA 3110+00

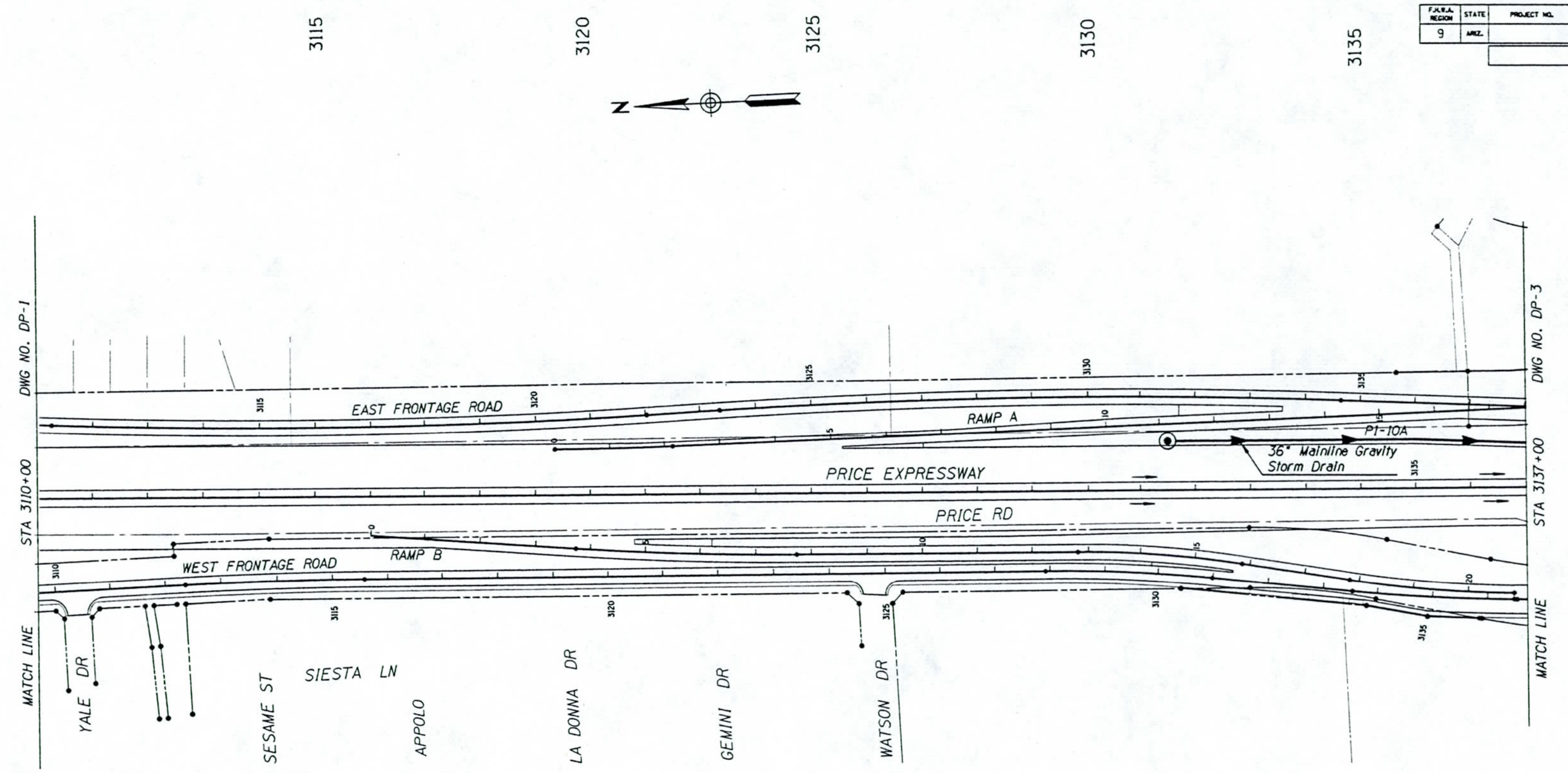
ROUTE | MILEPOST | LOCATION

DWG NO. DP-1 OF 24

----DGN=SPECIFICATION---- VIEW NAME:
----SYSTIME----

TRACS NO. DPPRIC01 ___ OF ___

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



	NAME	DATE
DESIGN		
DRAWN		
CHECKED		

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION

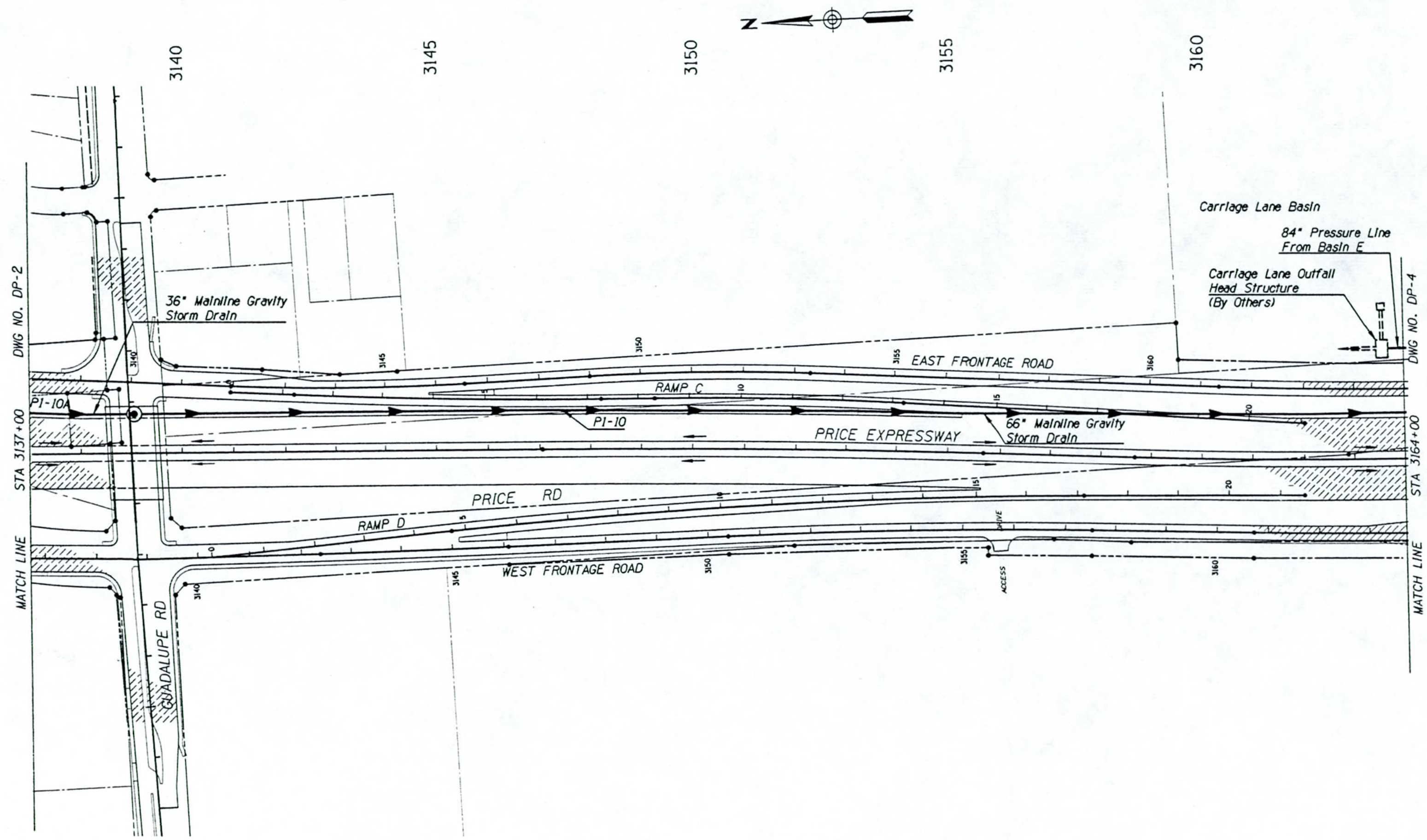
DRAINAGE PLANS
STA 3110+00 TO STA 3137+00

ROUTE | MILEPOST | LOCATION

DWG NO. DP-2 OF 24

TRACS NO. | DPPRIC02 | ___ OF ___

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



	NAME	DATE
DESIGN		
DRAWN		
CHECKED		

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION

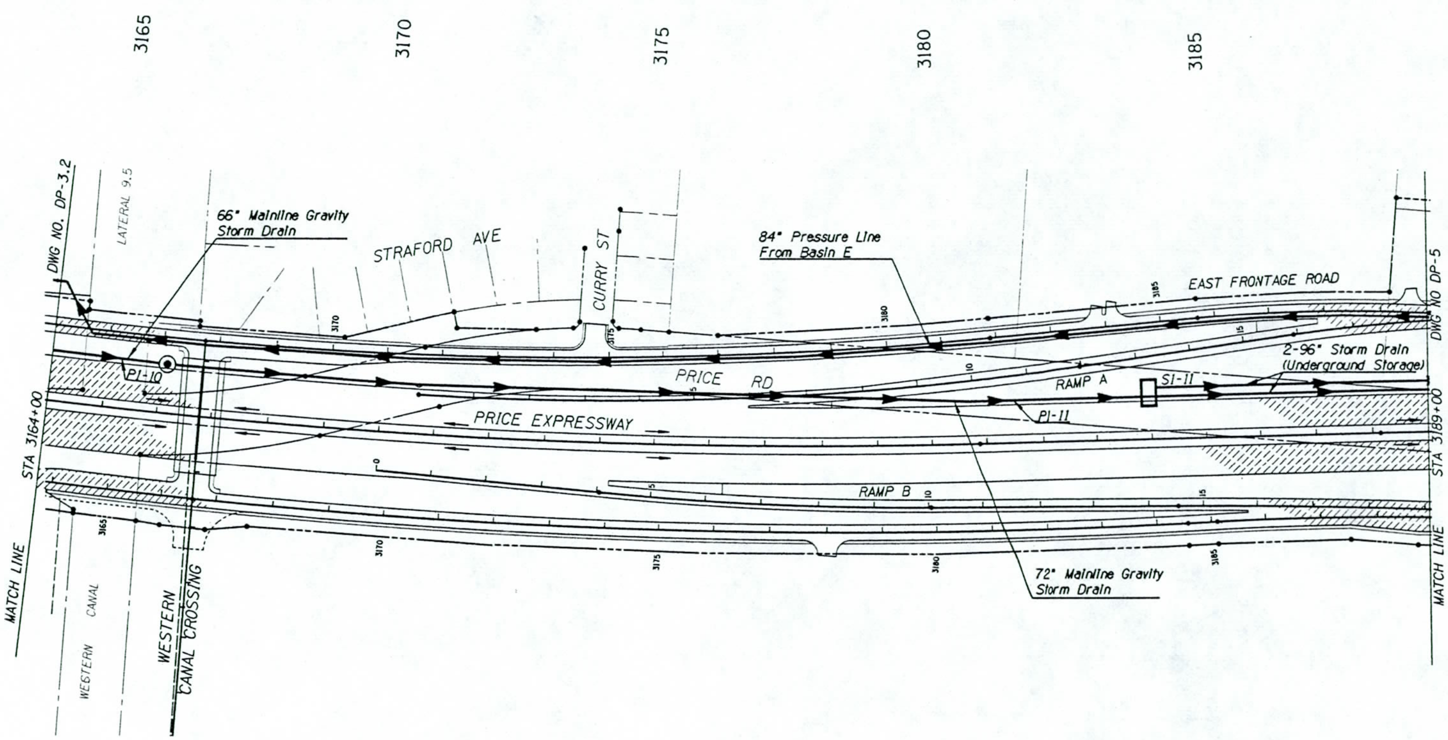
DRAINAGE PLANS
STA 3137+00 TO 3164+00

ROUTE | MILEPOST | LOCATION

DWG NO. DP-3 OF 24

TRACS NO. | DPPRIC03 | ___ OF ___

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



	NAME	DATE
DESIGN		
DRAWN		
CHECKED		

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION

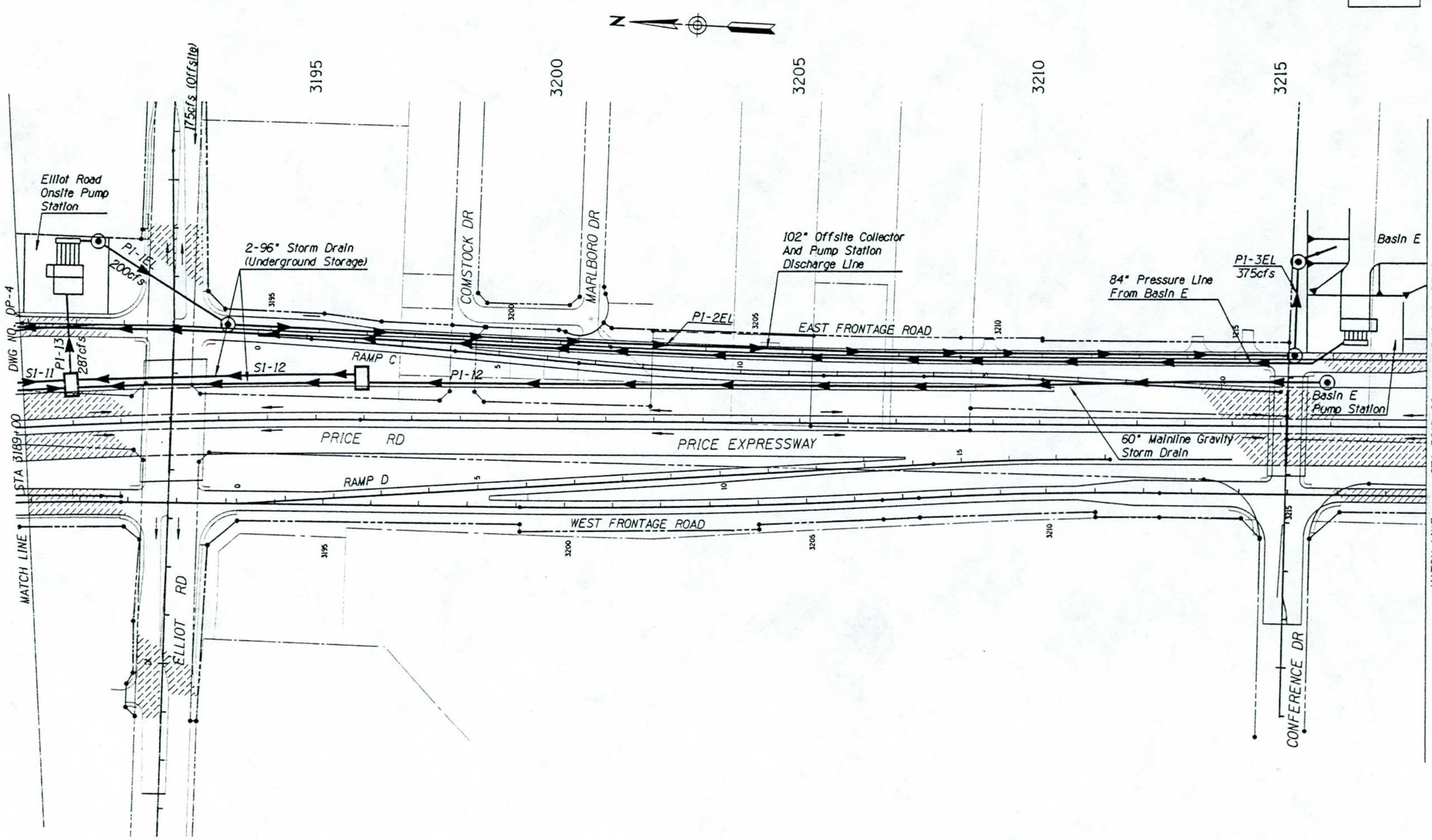
DRAINAGE PLANS
STA 3164+00 TO STA 3189+00

ROUTE | MILEPOST | LOCATION

DWG NO. DP-4 OF 24

TRACS NO. DPPRIC04 ____ OF ____

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



	NAME	DATE
DESIGN		
DRAWN		
CHECKED		

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION

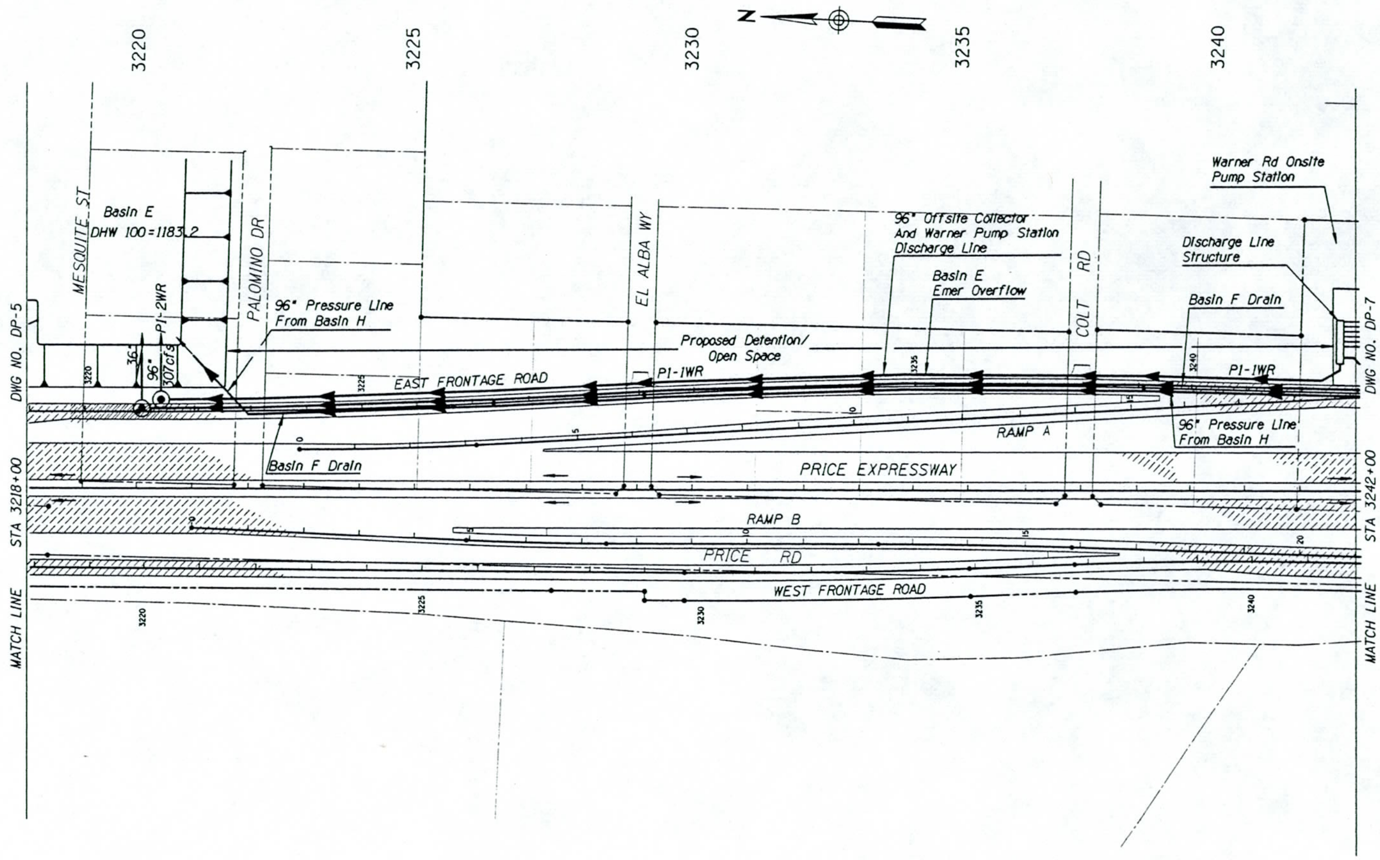
DRAINAGE PLANS
STA 3189+00 TO STA 3218+00

ROUTE | MILEPOST | LOCATION

SHEET DP-5 OF 24

TRACS NO. DPPRIC05 ___ OF ___

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



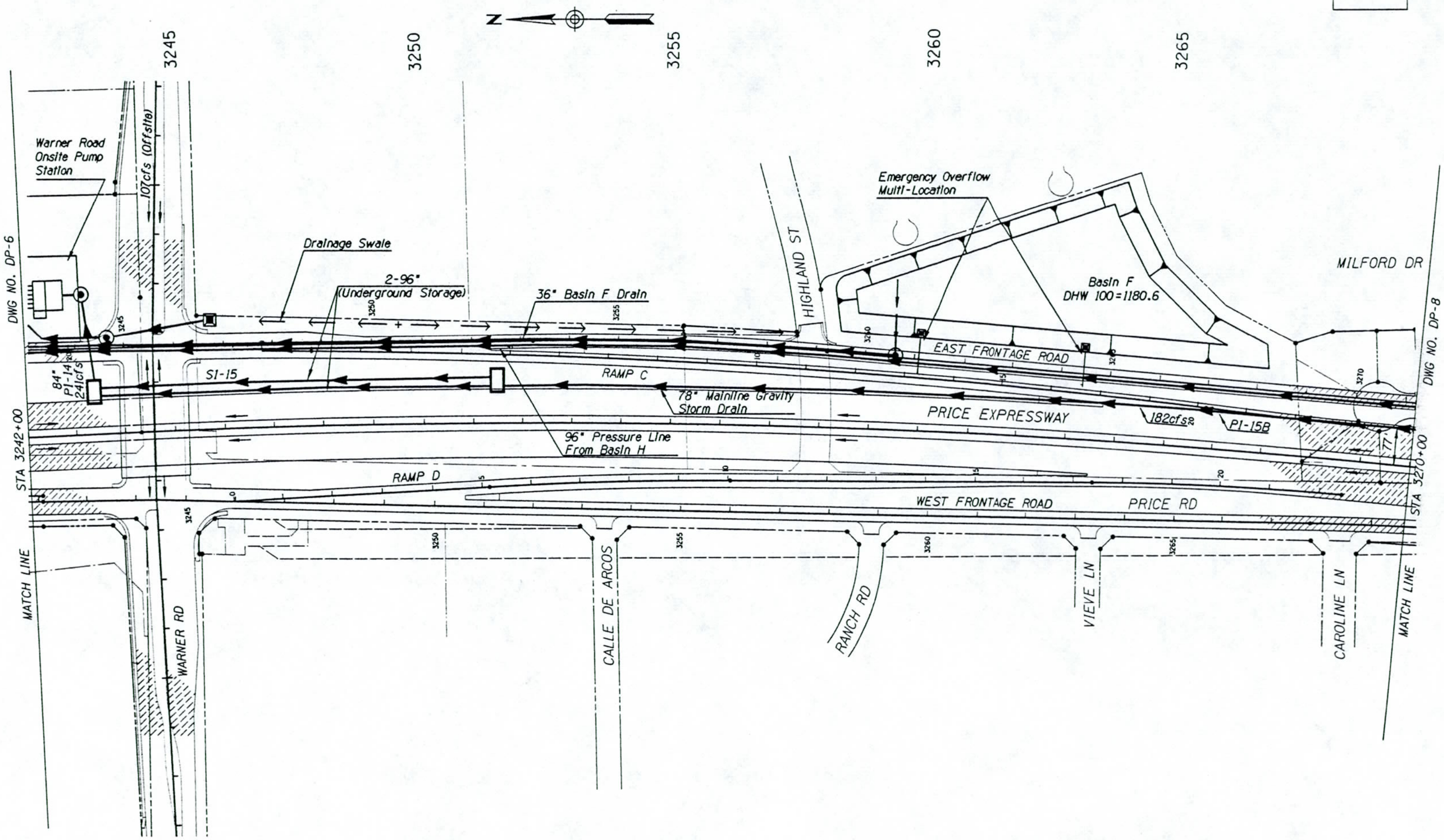
SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

	NAME	DATE	ARIZONA DEPARTMENT OF TRANSPORTATION HIGHWAYS DIVISION
DESIGN			
DRAWN			DRAINAGE PLANS
CHECKED			STA 3218+00 TO STA 3242+00
ROUTE	MILEPOST		LOCATION

DWG NO. DP-6 OF 24

-----DGN-SPECIFICATION----- VIEW NAME:
-----SYSTIME-----

TRACS NO. DPPRIC06 ___ OF ___



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

	NAME	DATE
DESIGN		
DRAWN		
CHECKED		

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION

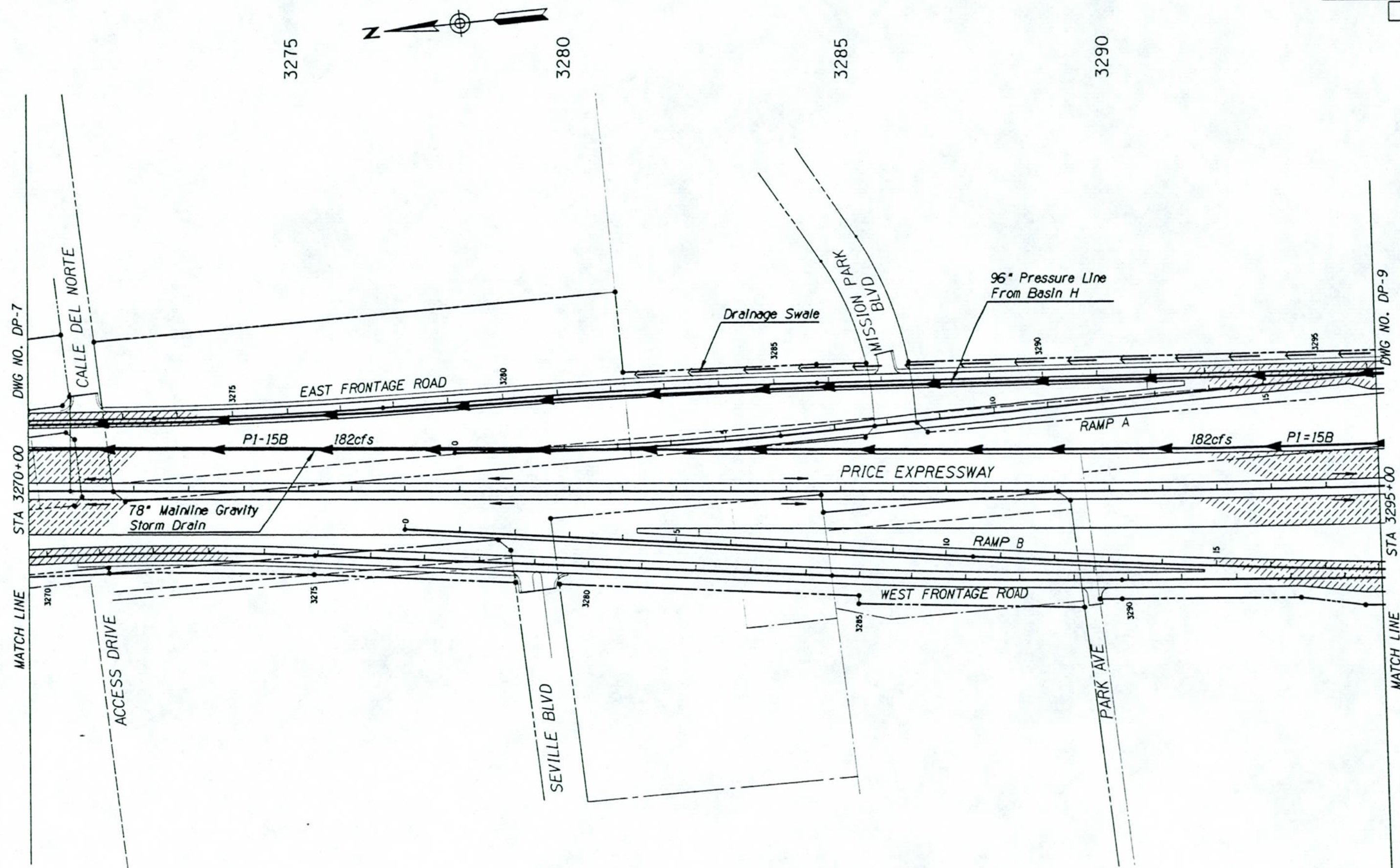
DRAINAGE PLANS
STA 3242+00 TO STA 3270+00

ROUTE | MILEPOST | LOCATION

SHEET DP-7 OF 24

TRACS NO. DPPRIC07 ___ OF ___

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



SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

	NAME	DATE
DESIGN		
DRAWN		
CHECKED		

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION

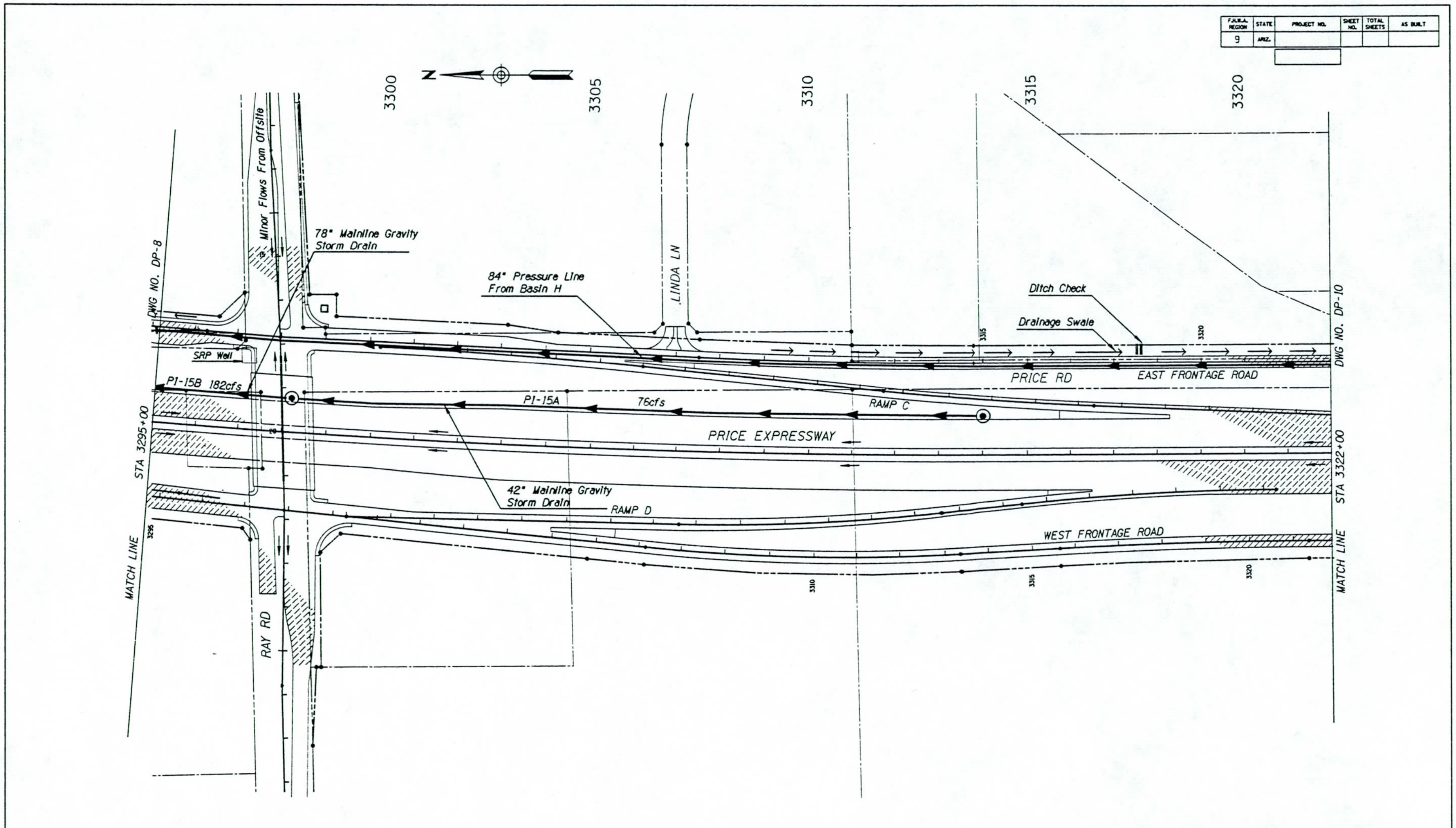
DRAINAGE PLANS
STA 3270+00 TO STA 3295+00

ROUTE | MILEPOST | LOCATION

DWG NO. DP-8 OF 24

-----DGN-SPECIFICATION----- VIEW NAME:
-----SYSTIME-----

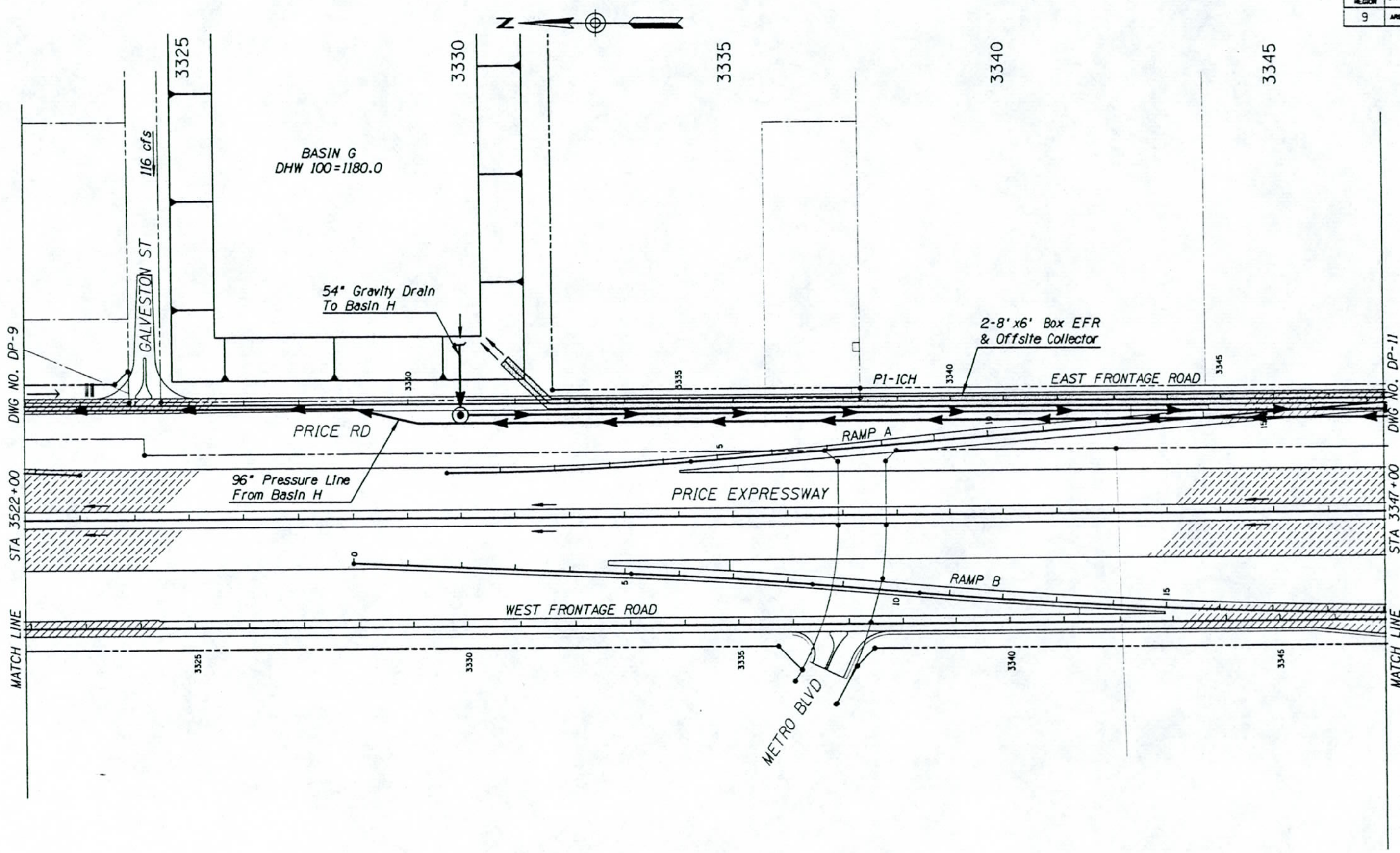
TRACS NO. DPPRIC08 ___ OF ___



	NAME	DATE	
DESIGN			ARIZONA DEPARTMENT OF TRANSPORTATION HIGHWAYS DIVISION
DRAWN			
CHECKED			DRAINAGE PLANS STA 3295+00 TO STA 3322+00
ROUTE	MILEPOST	LOCATION	DWG NO. DP-9 OF 24

TRACS NO. DPPRIC09 ___OF___

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



	NAME	DATE
DESIGN		
DRAWN		
CHECKED		

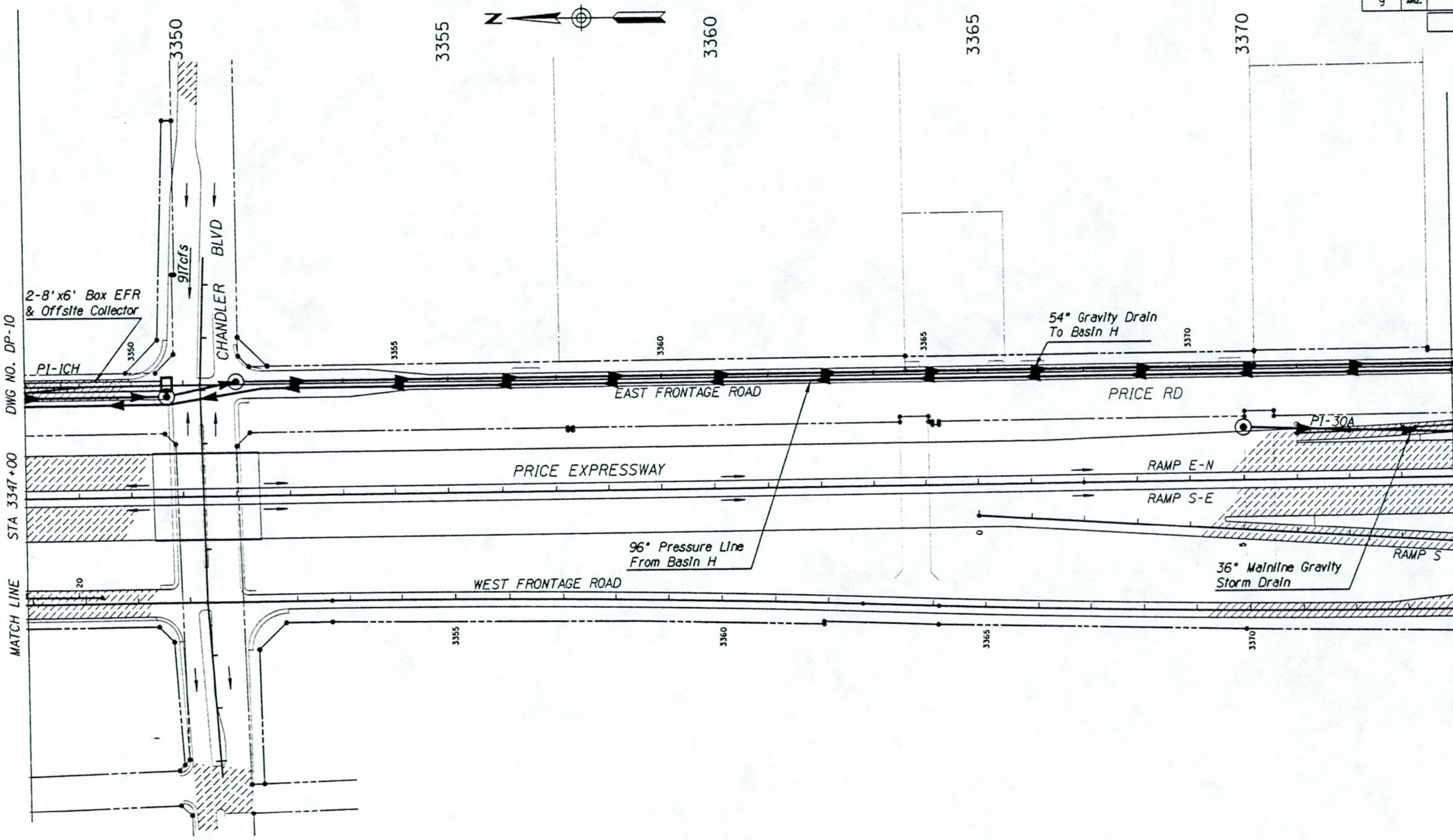
ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION

DRAINAGE PLANS
STA 3322+00 TO STA 3347+00

ROUTE | MILEPOST | LOCATION

DWG NO. DP-10 OF 24

TRACS NO. | DPSANTIO | ___ OF___



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

	NAME	DATE
DESIGN		
DRAWN		
CHECKED		

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION

DRAINAGE PLANS
STA 3347+00 TO STA 3374+00

ROUTE MILEPOST LOCATION

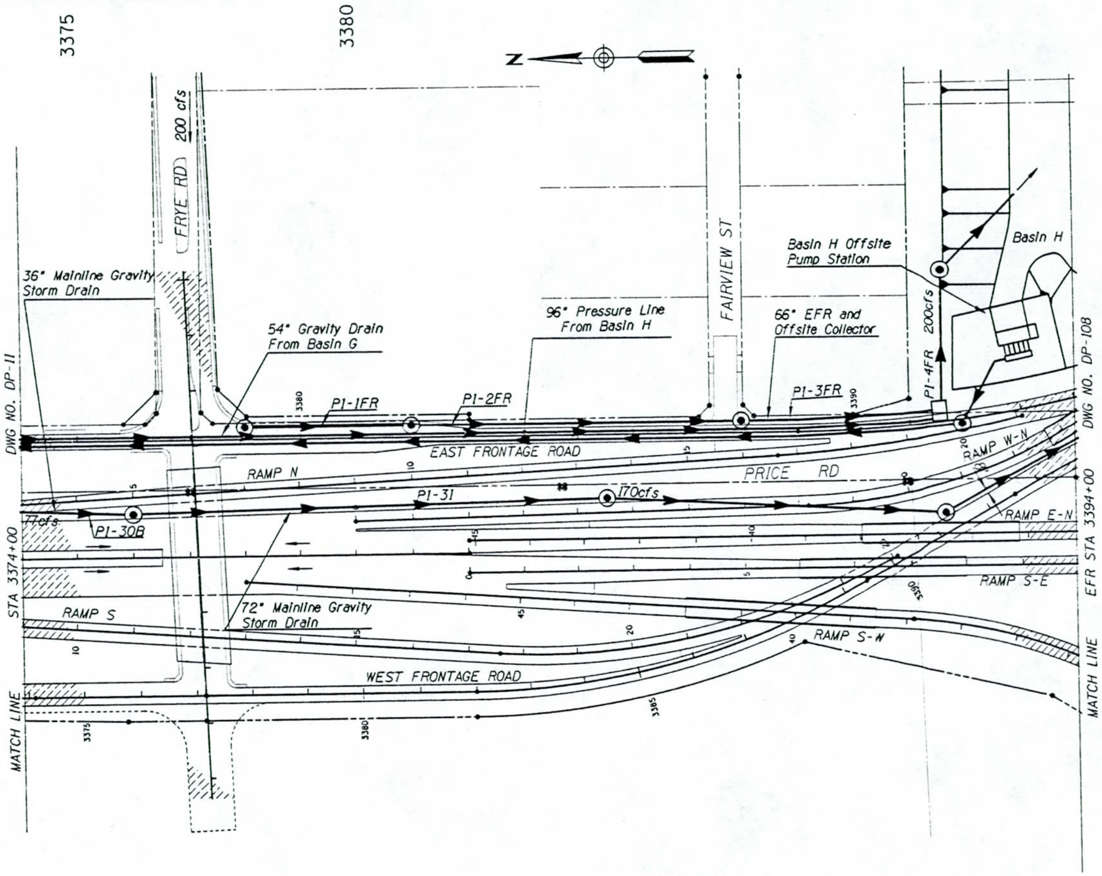
DWG NO. DP-11 OF 24

TRACS NO. DPSANT11 ___ OF ___

SURVEY NO. | FINISHED PLANS | REVISIONS | LOCATION | DATE

-----DGN-SPECIFICATION----- VIEW NAME:
-----SYSTIME-----

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



SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

	NAME	DATE
DESIGN		
DRAWN		
CHECKED		

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION

DRAINAGE PLANS
STA 3374+00 TO INTERCHANGE

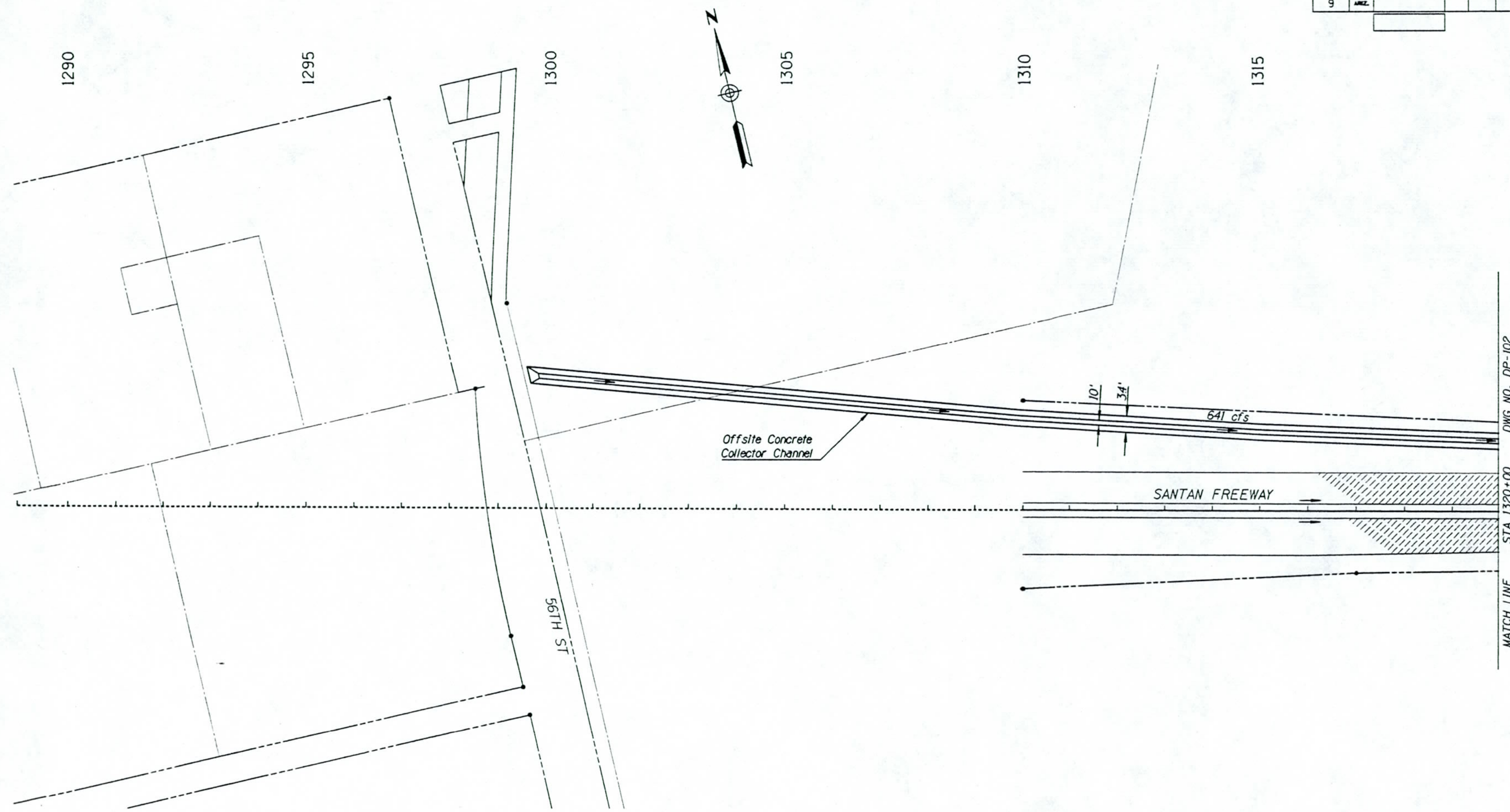
ROUTE | MILEPOST | LOCATION

DWG NO. DP-12 OF 24

-----OGN-SPECIFICATION----- VIEW NAME:
-----SYSTIME-----

TRACS NO. DPSANT12 ___ OF ___

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



	NAME	DATE
DESIGN		
DRAWN		
CHECKED		

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION

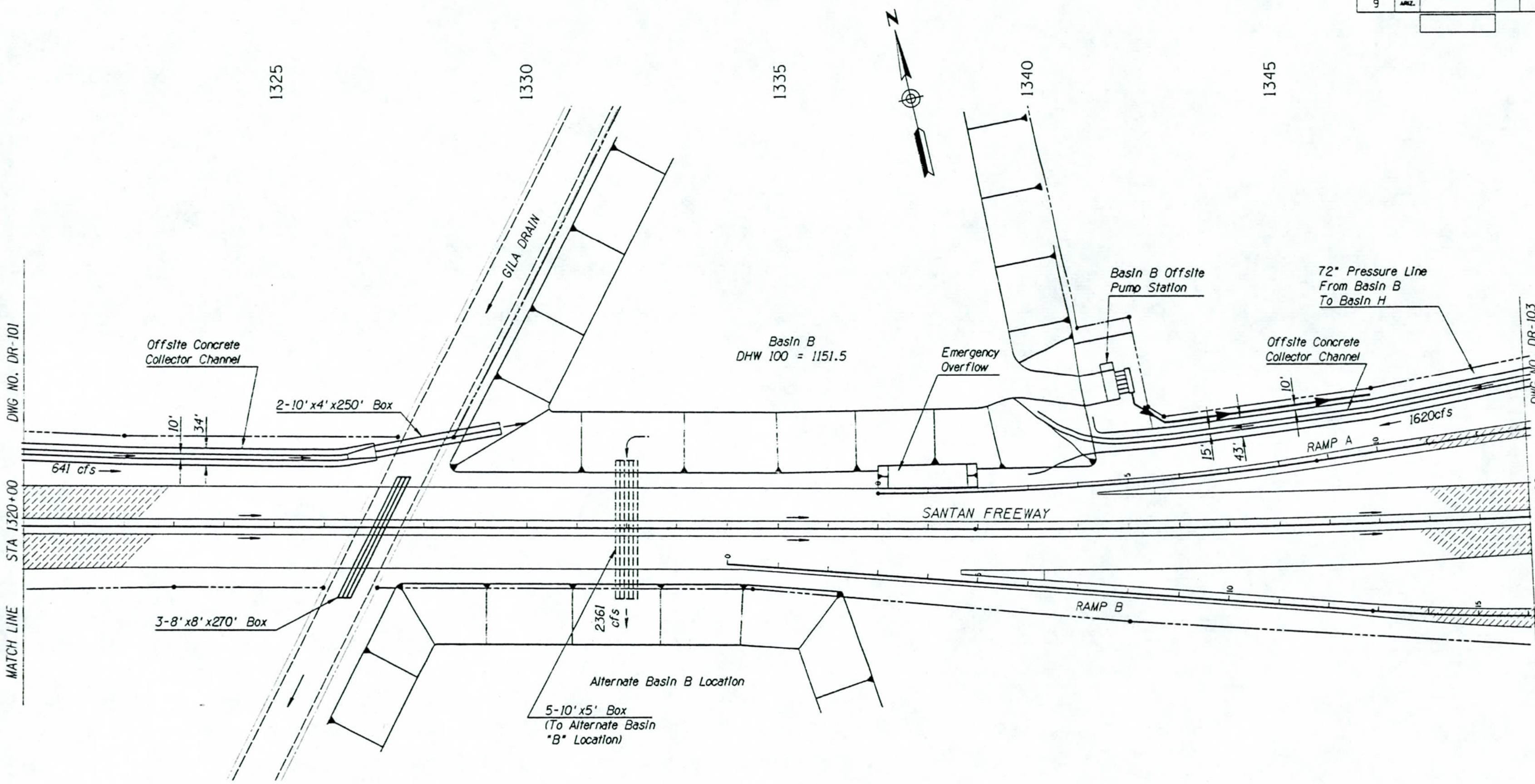
DRAINAGE PLANS
STA 1290+00 TO 1320+00

ROUTE | MILEPOST | LOCATION

DWG NO.DP-101 OF 24

TRACS NO. DPSAN101 ___ OF ___

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



SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

	NAME	DATE
DESIGN		
DRAWN		
CHECKED		

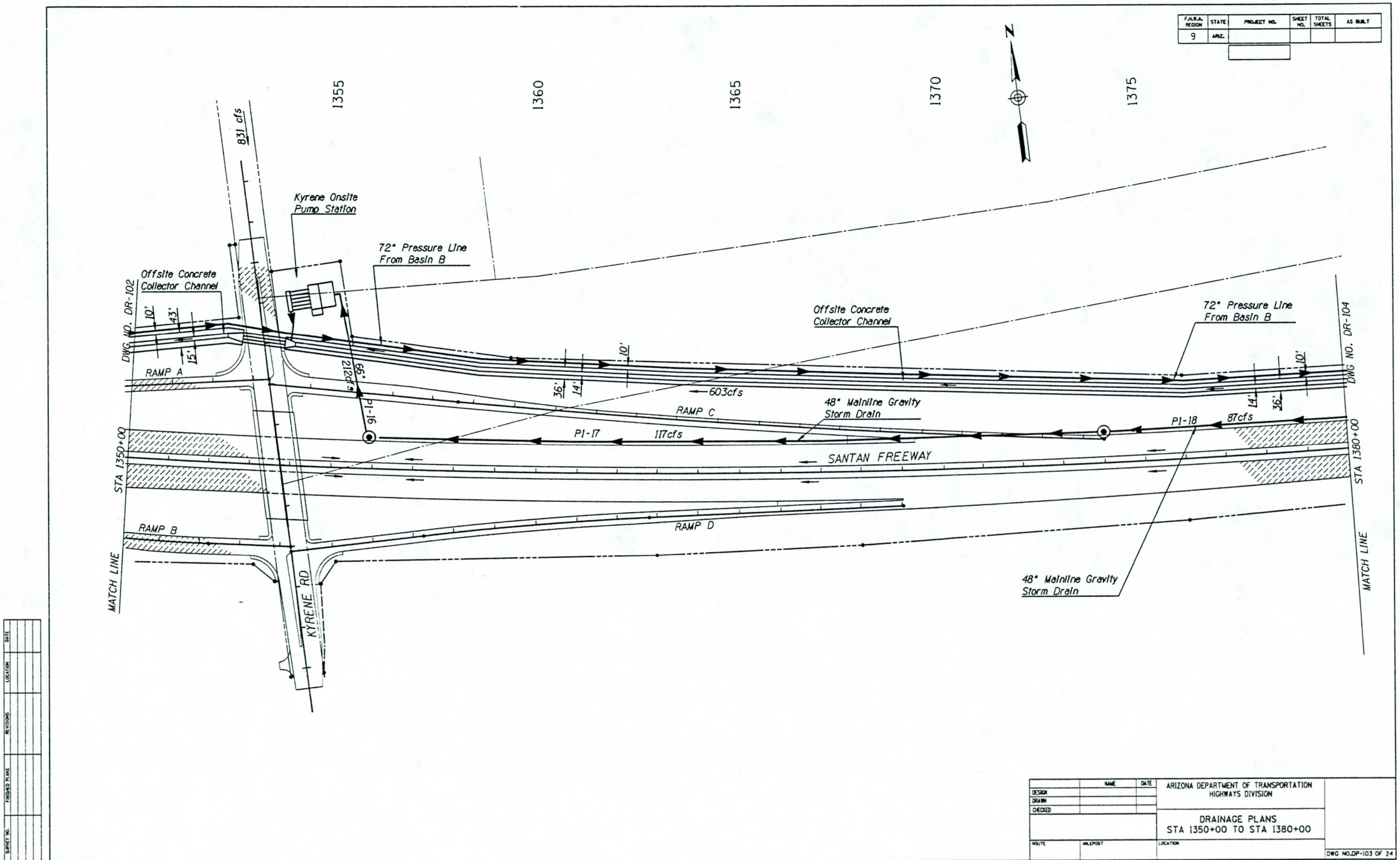
ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION

DRAINAGE PLANS
STA 1320+00 TO STA 1350+00

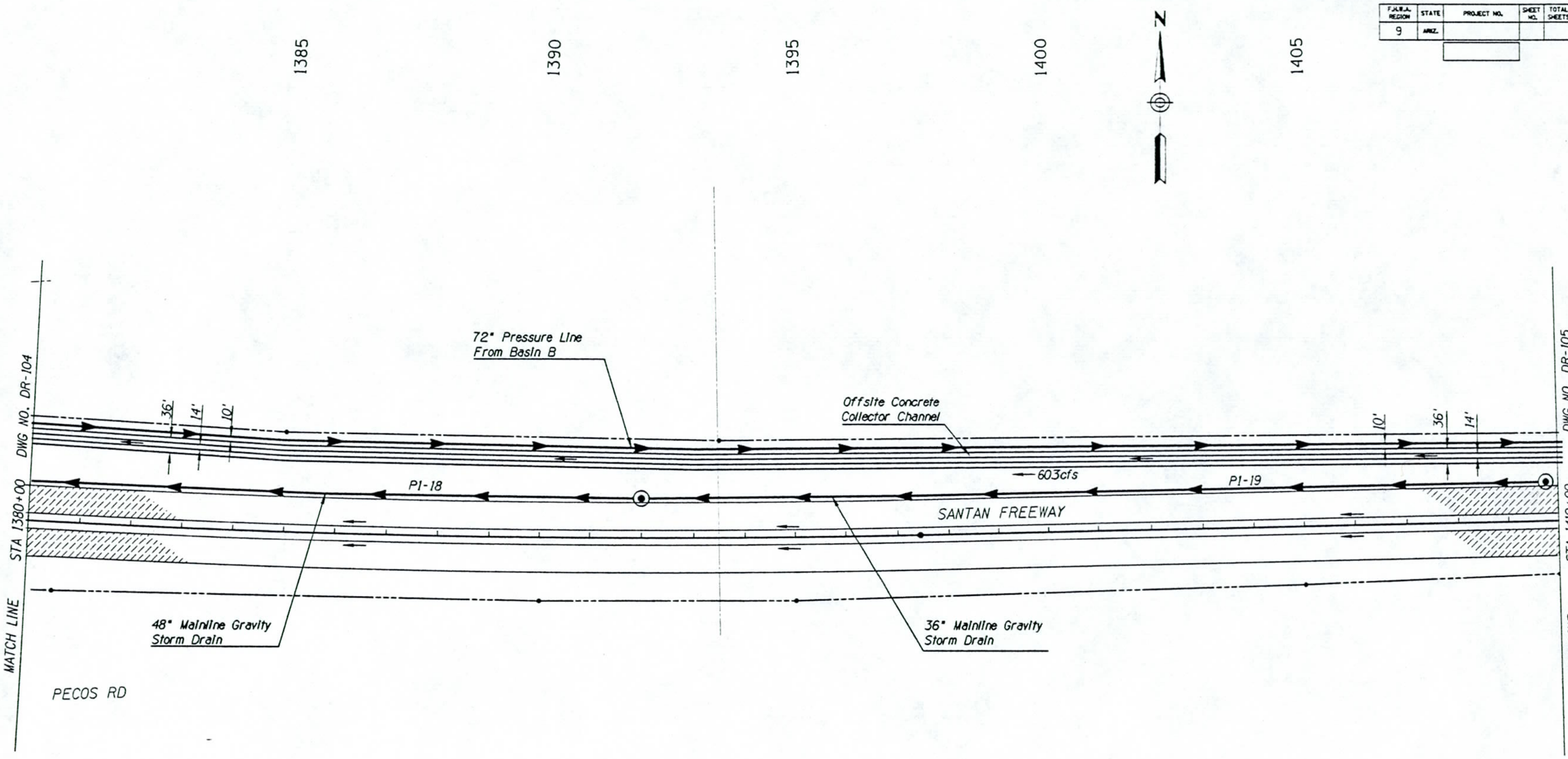
ROUTE | MILEPOST | LOCATION

DWG NO.DP-102 OF 24

TRACS NO. DPSAN102 ___ OF ___



TRACS NO. DPSAN103 ___ OF ___



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

SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

	NAME	DATE
DESIGN		
DRAWN		
CHECKED		

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION

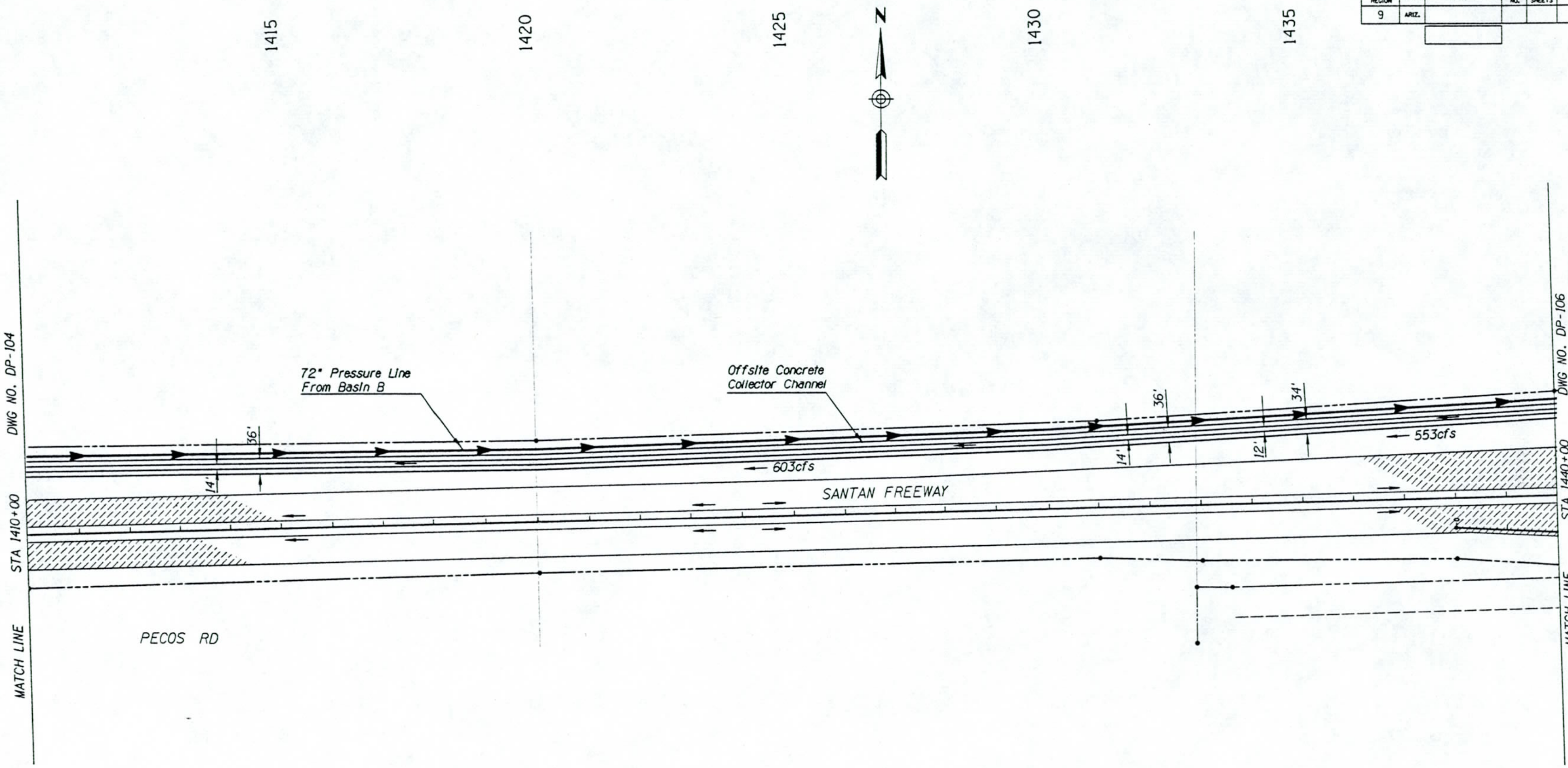
DRAINAGE PLANS
STA 1380+00 TO STA 1410+00

ROUTE | MILEPOST | LOCATION

DWG NO.DP-104 OF 24

-----DGN=SPECIFICATION-----
-----SYSTIME-----
VIEW NAME:

TRACS NO. | DPSAN104 | ___ OF ___



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

SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

	NAME	DATE
DESIGN		
DRAWN		
CHECKED		

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION

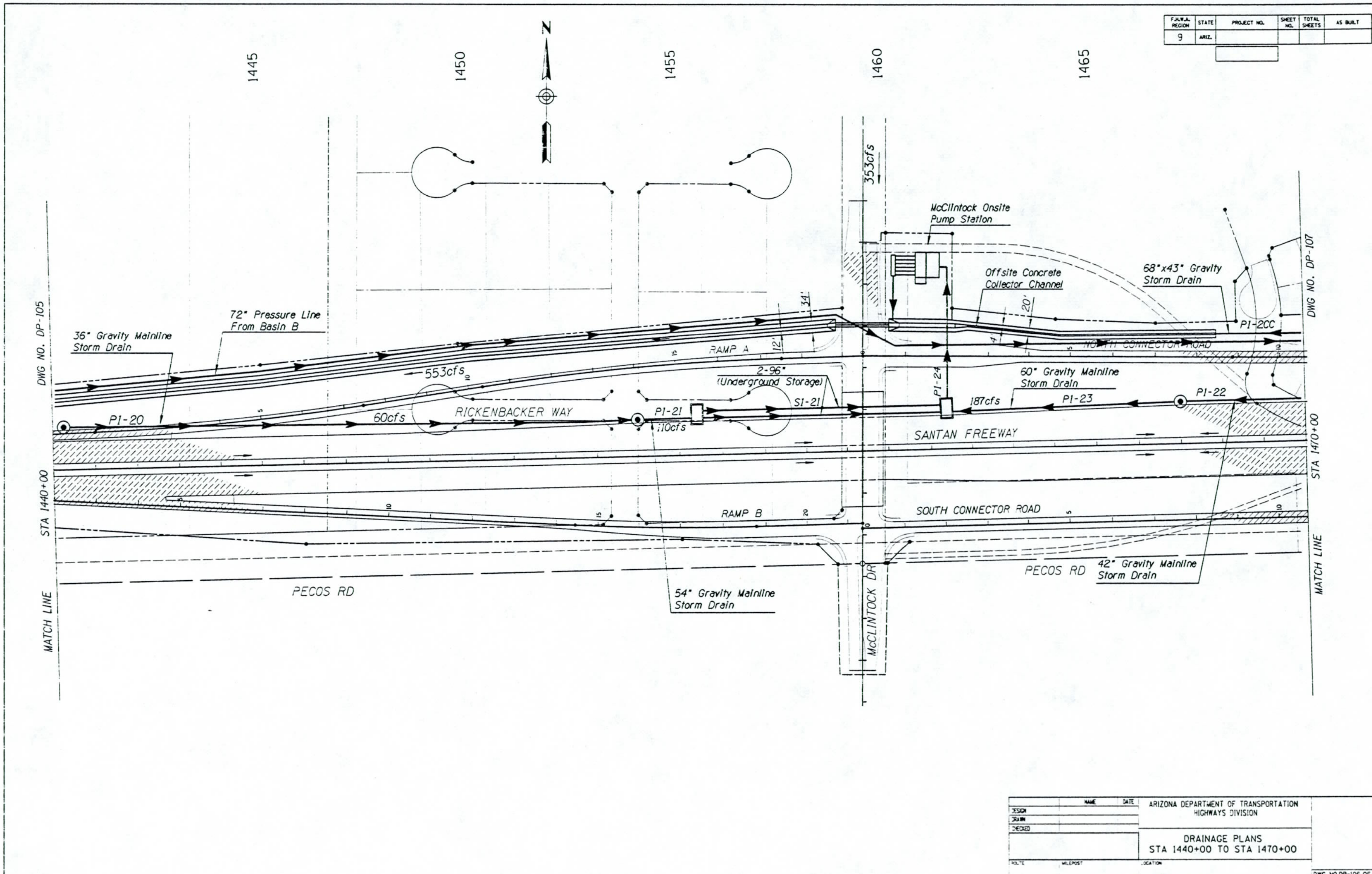
DRAINAGE PLANS
STA 1410+00 TO STA 1440+00

ROUTE | MILEPOST | LOCATION

DWG NO.DP-105 OF 24

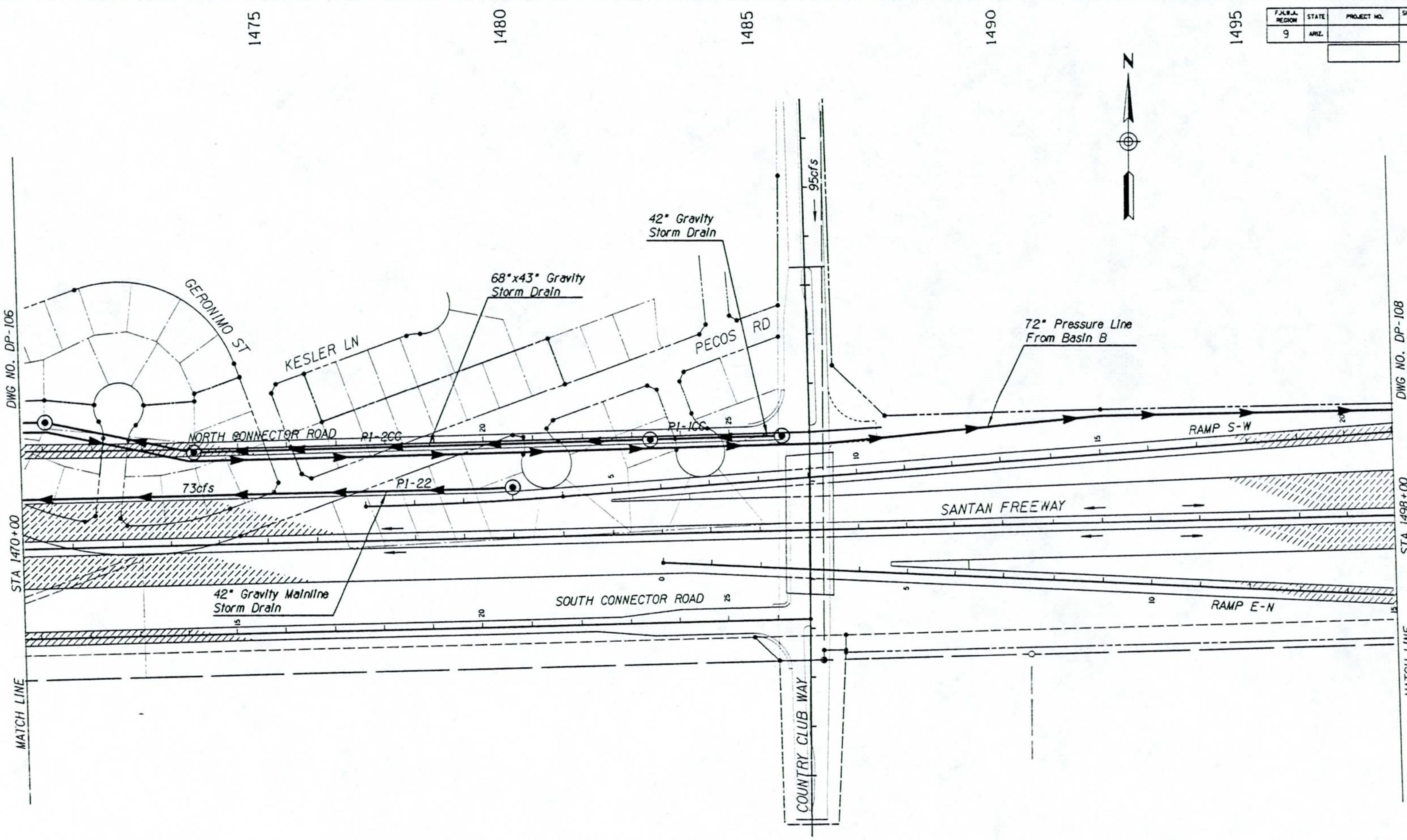
OGN=SPECIFICATION= VIEW NAME:
SYSTIME

TRACS NO. DPSAN105 ___ OF ___



TRACS NO. DPSAN106 ___ OF ___

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

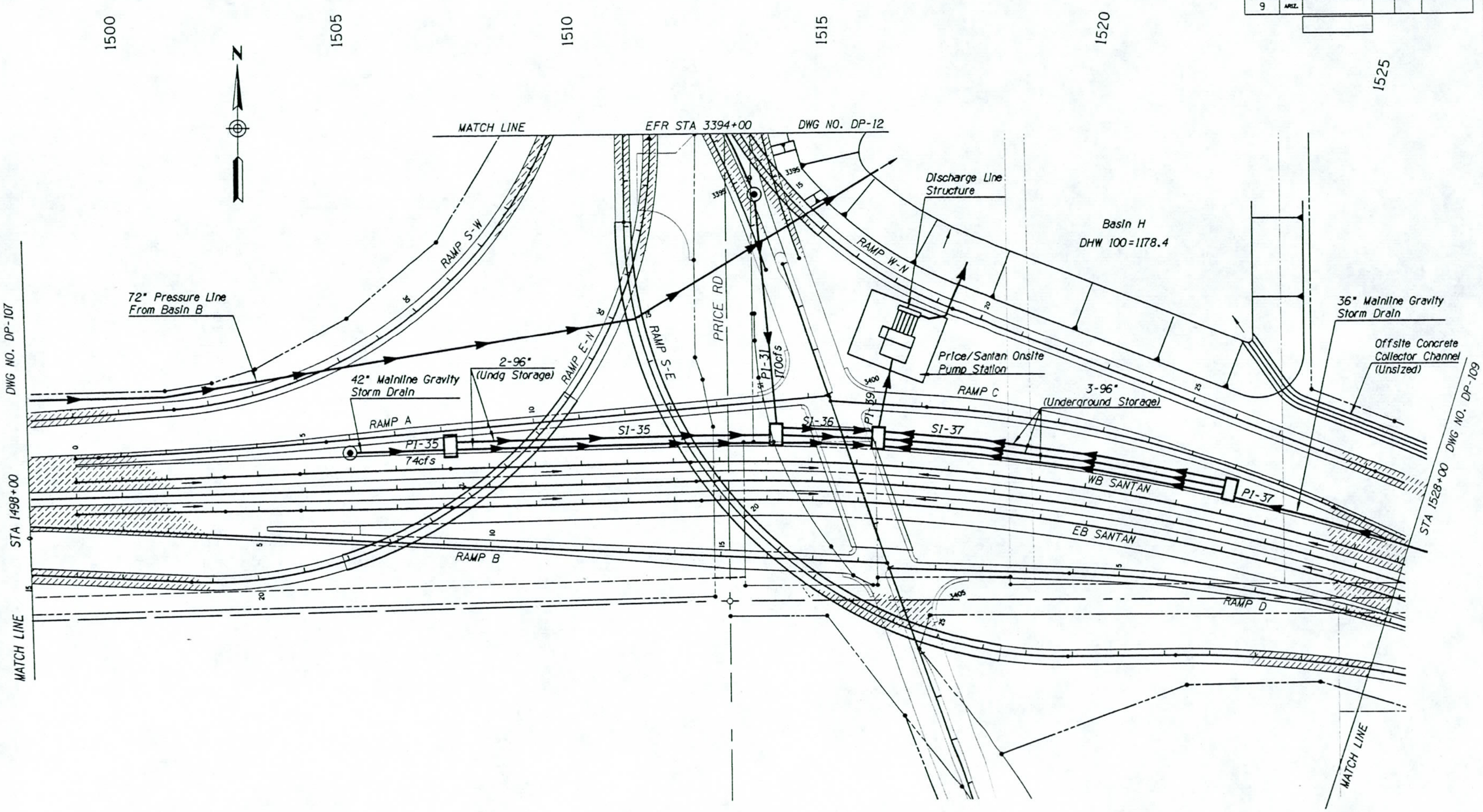


SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

	NAME	DATE	
DESIGN			ARIZONA DEPARTMENT OF TRANSPORTATION HIGHWAYS DIVISION
DRAWN			
CHECKED			DRAINAGE PLANS STA 1470+00 TO STA 1498+00
ROUTE	MILEPOST	LOCATION	DWG NO.DP-107 OF 24

---OGH-SPECIFICATION--- VIEW NAME:
---SYSTIME---

TRACS NO. DPSAN107 ____ OF ____



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

	NAME	DATE
DESIGN		
DRAWN		
CHECKED		

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION

DRAINAGE PLANS
STA 1498+00 TO STA 1528+00

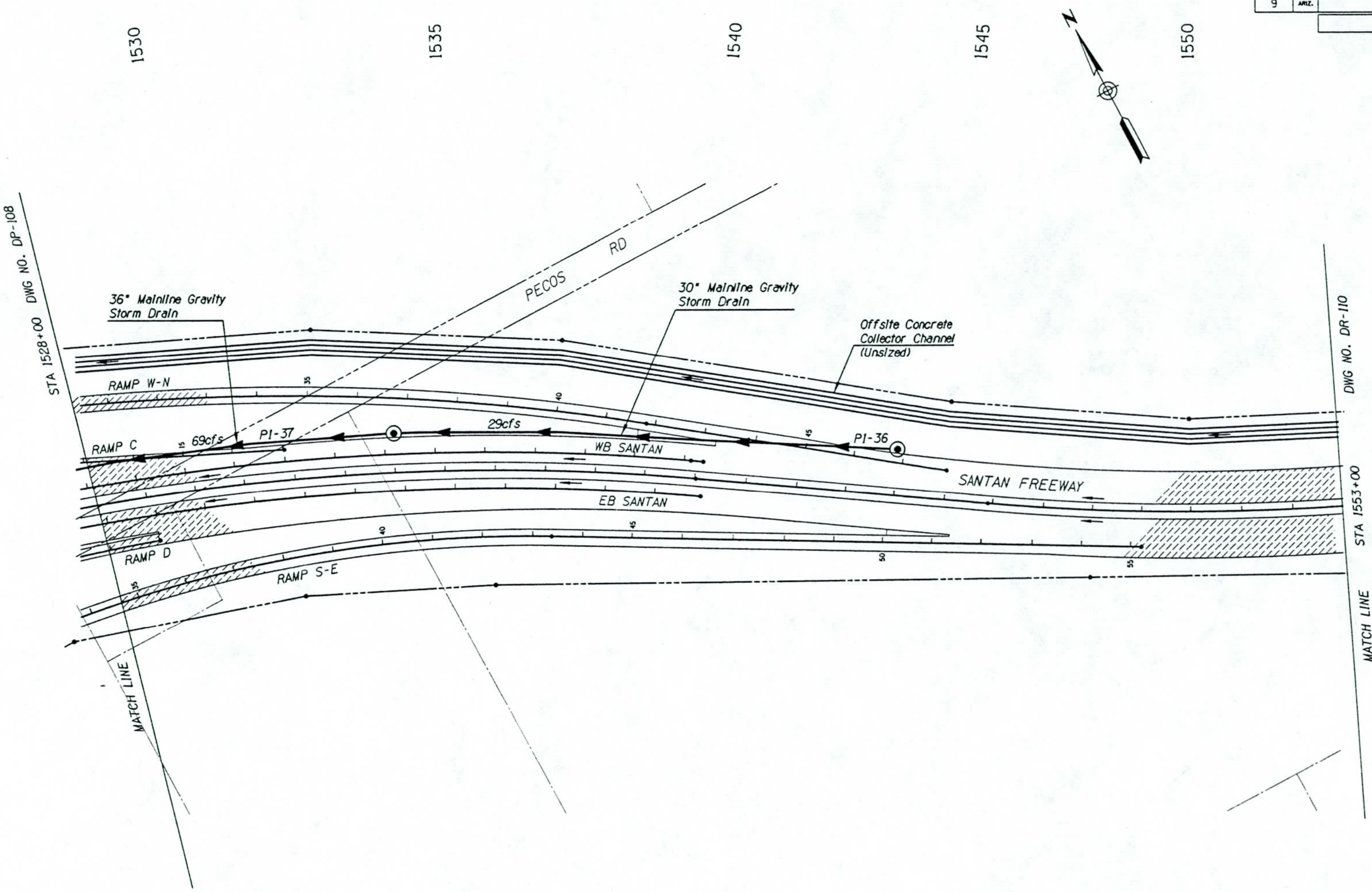
ROUTE | MILEPOST | LOCATION

SHEETDP-108OF 24

SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

TRACS NO. DPSAN108 ___ OF ___

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



SURVEY NO.	FINISHED PLANS	REVISIONS	LOCATION	DATE

	NAME	DATE
DESIGN		
DRAWN		
CHECKED		

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION

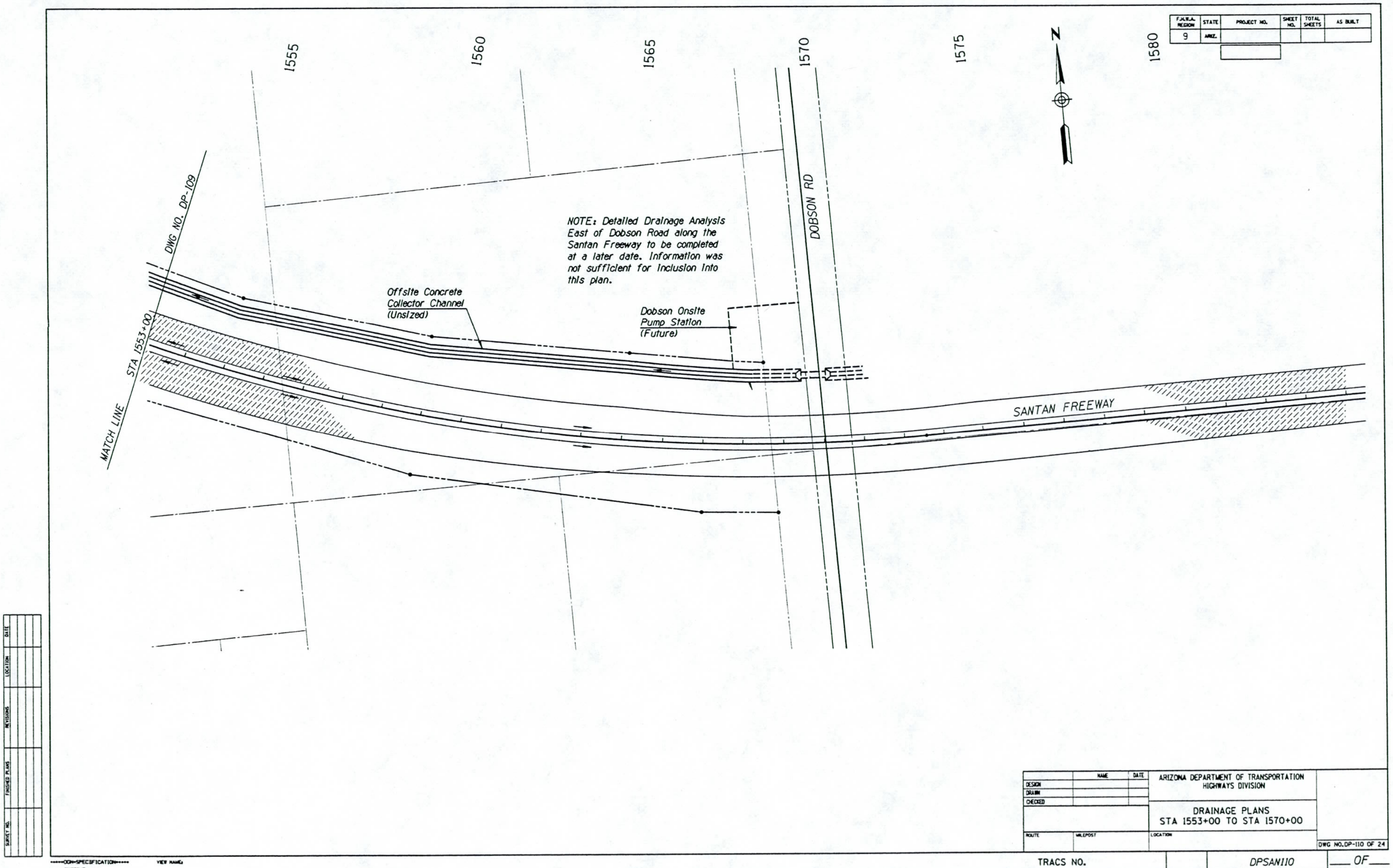
DRAINAGE PLANS
STA 1528+00 TO STA 1553+00

ROUTE | MILEPOST | LOCATION

DWG NO.DP-109 OF 24

-----DGN-SPECIFICATION----- VIEW NAME:
-----SYSTIME-----

TRACS NO. | | DPSAN109 | ___ OF ___


1555
1560
1565
1570
1575
1580
N
DWG NO. DP-109
MATCH LINE
STA 1553+00
DOBSON RD
NOTE: Detailed Drainage Analysis East of Dobson Road along the Santan Freeway to be completed at a later date. Information was not sufficient for inclusion into this plan.
Offsite Concrete Collector Channel (Unsized)
Dobson Onsite Pump Station (Future)
SANTAN FREEWAY
F.H.W.A. REGION
STATE
PROJECT NO.
SHEET NO.
TOTAL SHEETS
AS BUILT
9
ARIZ.
NAME
DATE
DESIGN
DRAWN
CHECKED
ROUTE
MILEPOST
LOCATION
ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION
DRAINAGE PLANS
STA 1553+00 TO STA 1570+00
DWG NO.DP-110 OF 24
TRACS NO.
DPSAN110
OF

REFERENCES

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2. Arizona Department of Transportation, **Hydrologic Design for Highway Drainage in Arizona**, March, 1969.

3. Arizona Department of Transportation, Urban Highways Section, **Landscape Design Guidelines for Urban Highways**, May 1988.

4. Boyle Engineering Corporation, **Storm Drainage Pump Station Study for Outer Loop Freeway**, August, 1986.

5. Camp Dresser McKee, **Stormwater Management Master Plan for City of Chandler**, October, 1986.

6. Dames and Moore, **Drainage Concepts, Southeast Loop Highway (Santan Freeway)**, Technical Memorandum No. 2, Arizona Department of Transportation, July 19, 1988.

7. HDR Engineering, Inc., **Hydrology Study for Price Expressway**, Arizona Department of Transportation, December, 1989.

8. Howard Needles Tammen & Bergendoff, **Final Hydraulic Report, Price Road Tunnel**, Arizona Department of Transportation, August, 1989.

9. Kimley-Horn & Associates, **Freeway Management System Design Guide**, Arizona Department of Transportation, October, 1989.

10. United States Army Hydrologic Engineering Center, **HEC-1 Flood Hydrograph Package**, August 2, 1988 Version.

11. Hydraulic Institute **Hydraulic Institute Standards**, 14th Edition, 1983.