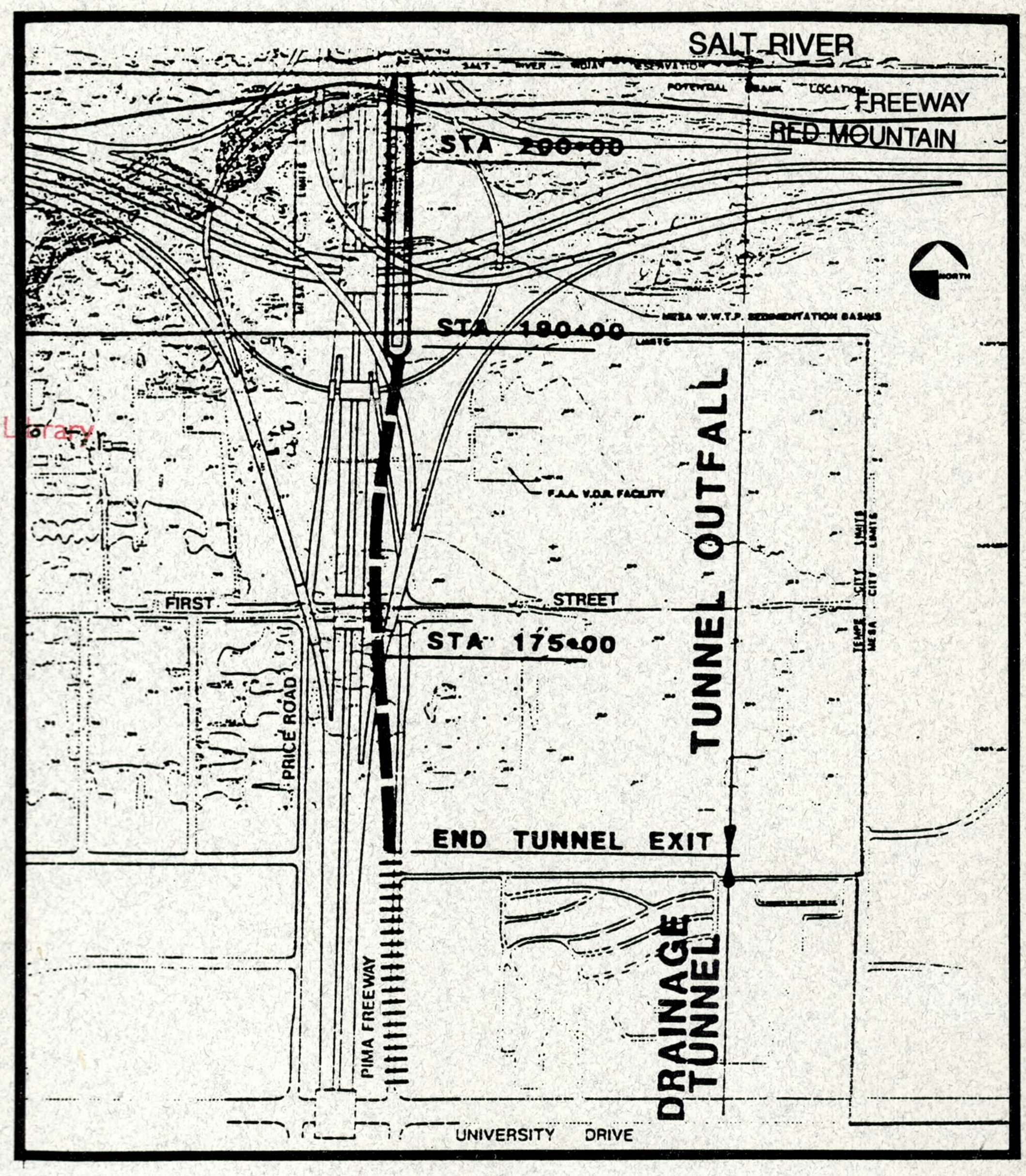

OUTER LOOP HIGHWAY SR 360 INTERCHANGE

VOLUME I

FINAL HYDRAULIC REPORT FOR PRICE ROAD TUNNEL SYSTEM

PREPARED FOR

ARIZONA DEPARTMENT OF TRANSPORTATION

AUGUST, 1989

HNTB

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OUTER LOOP HIGHWAY
SR 360 INTERCHANGE

FINAL HYDRAULIC REPORT
FOR
PRICE ROAD TUNNEL SYSTEM
VOLUME I

PREPARED FOR
ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION
AND
DELEUW CATHER COMPANY
(MANAGEMENT CONSULTANT)

AUGUST, 1989

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EXHIBITS

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I. Introduction

This report summarizes the hydraulic design of the Price Road Drainage Tunnel and its appurtenances for Arizona Department of Transportation (ADOT) and the Management Consultant - DeLeuw Cather Company (DCCO). The final design hydrology model is included as Appendix A (Volume II.) Hydrology model revisions and changes in assumptions made since the original hydrology model of October 1986 are documented herein. The overall hydraulic design of the tunnel system is documented by this report, as are the design of appurtenances not previously documented by design reports, such as Drop Structures A and B, the Carriage Lane Basin Inlet Structure and the Tunnel Outfall at the Salt River. The Intermediate Dropshafts and the Mesa Drain channel relocation/tunnel connection are appurtenances previously documented by design reports (see Section V, References). The effects of a recent change in the Salt River 10-year flood elevation upon the design hydraulics is summarized (see Section III-C-iii).

Also included in this report is a summary of Transient Hydraulic Analysis computer modeling of the tunnel system performed by Dr. Charles C. S. Song of the University of Minnesota, the Saint Anthony Falls Hydraulic Laboratory (SAFHL), and the Charles C. S. Song Company (see Section III-C).

The Price Road Tunnel Outfall Alternatives Hydraulic Analysis study results are contained within this report as well (see Section IV). The Outfall Alternatives study was required because of changes to the proposed alignment, the proposed configuration and the proposed conduit element sizes of the tunnel outfall that occurred subsequent to development of outfall plans by Howard Needles Tammen & Bergendoff (HNTB) in late 1988. Submitted in conjunction with this report will be a diskette for a personal computer, containing the U.S. Army Corps of Engineers HEC-2 computer models for the three outfall alternatives. These models may be adjusted by the consultant responsible for the final design, construction plans and construction documents preparation of the tunnel outfall, as long as water surface elevations and energy elevations at the upstream end of the model are not raised.

II. Hydrology Model

The design hydrographs for the tunnel system were developed by use of the U.S. Army Corps of Engineers computer program HEC-1 "Flood Hydrograph Package". The modeling was originally performed in 1986, and documented with two reports (Outer Loop Highway SR360 Interchange Hydrology Study Parts 1 and 2). The text from those reports is included in Appendix D (Volume II).

The result of the 1986 hydrologic modeling was 20 hydrographs at various key points along the Superstition Freeway and the Outer Loop/Price Road alignment. For the locations of the 20 hydrographs and the tabulated 50-year and 100-year peaks, see Exhibit 1. See Exhibit 2 for tables showing subarea characteristics and HEC-1 parameters, and a map showing the 100-year hydrograph peaks. Exhibit 3 includes a table showing the tunnel peak flows as generated by the 1986 HEC-1 hydrology model. All exhibits are included at the end of Volume I report, following Section V - References.

A. New or Altered Assumptions

There are 13 design assumptions listed in the HNTB October 1986 report "Outer Loop Highway SR360 Interchange Hydrology Study Part 2 - Inflow Hydrographs to Outer Loop Offsite Drainage System" (see Appendix D, Volume II). Of these assumptions, 1 through 8 and 10 through 12 are unchanged with the current configuration of the hydrology model. Assumptions 9 and 13 have been changed or modified as follows:

i. Originally, assumption 9 presumed that proposed forced flow conduit systems would deliver 100 cfs each from the cities of Chandler and Gilbert, directly into the Carriage Lane Basin. The timing and duration of these hydrographs were derived from preliminary information from the city of Chandler. The presumption that the pumped flows would be routed through the basin was consistent with earlier studies performed for the Flood Control District of Maricopa County(FCD) by Dibble Engineering.

There have been two changes affecting assumption 9. First, the city of Mesa requested that the pumped flow not be routed through the basin, but rather, be routed directly to the Carriage Lane Basin Outfall Pipe, downstream of the basin, with measures taken to prohibit direct backflow of the pumped discharges into the basin. The proposed system and the hydrology model were changed accordingly.

Second, the city of Gilbert withdrew from participating in the project. However, the decision was made to keep their 100 cfs of capacity in the design; therefore, no change was made to the model.

ii. Originally, assumption 13 presumed that onsite flows from within the Outer Loop corridor ROW would not be included in the hydrology model due to their very early and relatively minor peaks, and no data existed for incorporation.

As the design of the tunnel system progressed, information became available as to the locations and approximate peaks and shapes of hydrographs representing pump station flows from freeway ROW. Therefore, assumed hydrographs were added to the hydrology model for the proposed Guadalupe, Baseline, Price Road, SR360 Tunnel Ramps, Manhatton, Southern Avenue, Broadway and University pump stations. The addition of the 8 pump station hydrographs resulted in little, if any, increase to the tunnel ultimate design peak flow.

iii. Another assumption implicit in the original hydrology model was that there would be only two intermediate inflow dropshafts built along the tunnel between SR360 and Salt River. As the design of Section 12 progressed, the number of intermediate dropshafts was increased to 5. The hydrology model was subsequently modified accordingly.

iv. Finally, the original model was based upon the assumption that no flow would enter or be combined with the tunnel system north of University. Recently, this assumption was changed, in that it is now anticipated that offsite flows generated north of University and east of the tunnel will be drained into the surface conduit or channel that will be downstream of the siphon tunnel high point. The surface conduit or channel will be designed such that any increase in system peak flow due to this addition of flow will not be allowed to directly affect the siphon tunnel hydraulics. This additional flow was added to the hydrology model, in the following manner:

The hydrology models for the additional areas were received from Simons, Li and Associates (SLA). Exhibit 4 shows a map of the SLA area and Appendix B in Volume II presents the SLA model. The SLA rainfall distribution was replaced by the rainfall distribution used in the HNTB hydrology model (taken from the distribution used by Boyle Engineering for the original Section 12 hydrology model, at ADOT and DCCO's request.) The SLA models were then incorporated into the overall tunnel hydrology model.

The result of adding the SLA models was a slight increase in design discharge for the outfall channel downstream of the system high point.

B. Calibration

The HNTB hydrology model inflow hydrographs and the tunnel configuration were given to Dr. Song of the University of Minnesota, so that he could apply his dynamic hydraulic model in an analysis of potential transient hydraulic behavior of the tunnel system. Dr. Song's model resulted in higher peak discharges for the tunnel than those determined by the more simple routing techniques used in the HEC-1 hydrology model.

To accomplish the analysis of the effects of the offsite flow from south of University, the hydrology model was calibrated to produce approximately the same peaks as Dr. Song's model. This was accomplished by decreasing the calculation ordinate interval of the hydrology model from 15 minutes to 5 minutes. The 15 minute interval matched the ordinate interval used in the SLA hydrology models. The original hydrology model had to be run using the 15 minute interval due to the inclusion of the Carriage Lane detention basin and the detention basin/box culvert alternative to the tunnel system, requiring a 72 hour model run for confirming basin emptying within the specified period. The 72 hour run could only be accomplished by a 15 minute calculation interval, due to model limitations. Besides the changing the calculation interval, the roughness values for the routing routines were altered slightly.

The result of the calibration was that the tunnel peak discharge in the hydrology model increased to 2279 cfs, from an earlier peak of 2172 cfs. The original hydrology model, in the 1986 configuration, had a peak tunnel discharge of 2136 cfs.

When the offsite peaks from north of University were added to the model by the incorporation of the SLA models, the design peak discharge in the outfall increased to 2305 cfs and 2331 cfs at the two locations where it was assumed that the offsite flow would be added. (See Appendix A, Volume II).

III. System Hydraulics

A. Chronology of Alternatives

The configuration and alignment as originally proposed for the Price Road Tunnel by the October 1986 Alternatives Analysis study (See Section V, References) included an outfall at the Salt River along the Price Road alignment, two intermediate dropshafts, one drop structure in the southeast quadrant of the Superstition Interchange, an 18-foot diameter tunnel from the interchange to the river, a 102-inch diameter pipe connecting the Carriage Lane Basin to the drop structure at the interchange, a double 12′ by 12′ reinforced concrete box culvert (RCBC) riser and an energy dissipator at the river, and an inlet from the Carriage Lane Basin for combined Mesa, Chandler and Gilbert flows. See Exhibit 2 in the above referenced Alternatives Analysis report for the original profile. (Reference E).

As the original configuration and alignment were further developed towards final design, the following changes occurred:

i. An alignment change was considered in August 1987, such that the outfall would be located at Hayden Road, approximately 5,000 feet downstream of Price Road along the Salt River. The change was never made final, but was seriously enough considered that DCCO and ADOT personnel requested that ongoing hydraulic analysis by Dr. Song be limited only to Hayden Road outfall alternatives. The final decision in 1988 was to keep the outfall alignment at Price Road.

   The hydraulic design of the system was also affected by several changes of the tunnel length versus the length of surface conduits or open channels along the Price Road alignment. At various stages during design, the tunnel was shortened to avoid placing the tunnel construction shaft within the zone of influence of an aviation VOR, lengthened to avoid major utility conflicts, shortened to avoid untimely right-of-way acquisitions, and significantly shortened to allow flexibility in using either Hayden Road or Price Road outfall alignments.

   This last change, locating the tunnel low point and pump station near Fifth Street, reflects the tunnel as actually constructed, to be finished in early 1990. An interim condition will exist between the construction of the intermediate dropshafts associated with Outer Loop Section 12 and the construction of the tunnel outfall, the Mesa Drain inlet and the Carriage Lane Basin inlet. For a discussion of the interim condition, see Section III-B.

ii. The originally proposed two intermediate dropshafts were increased in number to five intermediate dropshafts, for savings in construction costs of the offsite drainage collection and interceptor systems between the Superstition Freeway and University Drive.

iii. The proposed Superstition Interchange drop structure, for the interception of the Mesa Drain flows and the conveyance of the Carriage Lane Basin outfall pipe flows to the tunnel, was split into two structures (Drop Structures A and B) in May 1987, due to inability of state-of-the-art drop structures to handle split flows at different alignment angles and significantly different elevations. Insufficient area existed to combine Mesa Drain and Carriage Lane Basin flows at common alignment and elevation.

iv. The 18-foot tunnel diameter has remained throughout the evolution of the tunnel design. However, the original hydraulic analysis performed by Dr. Song was to cover the possibility of either 18-foot or 21-foot diameter tunnels, due to the local availability of a 21-foot diameter tunnel machine.

v. The 102-inch diameter pipe from the Carriage Lane Basin was increased in size to a 108-inch diameter pipe in August 1987, to decrease overflow into the Carriage Lane Basin during the peak of the design event, and to increase off-peak flow capacity from the Carriage Lane area. The entire invert of the proposed Carriage Lane Basin outlet pipe was lowered twice. First, the pipe was lowered to drain the surveyed low elevation within basin. Originally, pipe flowlines had been set based upon City of Mesa as-built plans, evidently with a different datum than ADOT's. Subsequently, the pipe was lowered approximately six feet in the summer of 1988, to allow draining of the basin while pumped flows were using their portion of pipe capacity, and to allow interception of a small pipe inflowing to the Carriage Lane Basin from another City of Mesa detention basin to the east.

vi. The originally proposed double 12′ by 12′ RCBC riser was changed to consist partly of 18-foot diameter tunnel and partly of double 14′ by 14′ RCBC, for the 1988 tunnel outfall construction plans. Due to a failure to reach an agreement with the tunnel contractor to extend the tunnel currently under construction further northward, and due to the threat of hazardous waste along the proposed alignment of the tunnel extension, it is currently proposed to extend the tunnel system to just north of the 90-inch water line in First Street by a double 14′ by 13′ RCBC conduit and riser (with conveyance equivalent to tunnel conveyance). There, just north of First Street, will be the system high point and the slope break towards the river. The system extension, riser and energy dissipator will be discussed further in Section IV - Outfall Alternatives Hydraulic Analysis.

vii. The system as proposed in October 1986 was based upon the assumption that the pumped Chandler and Gilbert flows would be released into the Carriage Lane Basin, as was evidently assumed by earlier studies performed for the FCD. However, the City of Mesa requested that the pumped flows be kept separated from the basin and its natural contributing area's flows. Therefore, an inlet structure within the basin for the 108-inch pipe was proposed.

The structure was sized volumetrically by Dr. Song to contain the backsurge resulting from the most transiently active condition, the 100-year event inflowing to an empty tunnel. The primary outlet from the pond to the inlet structure (head structure) was established as a 3′ by 3′ orifice by Dr. Song, and modified to a 42-inch pipe by HNTB. The 42-inch pipe will be flap-gated within the inlet structure, to prevent backflow from the 108-inch pipe to the basin. The overflow elevation of the inlet structure was set at elevation 1189.00 feet, to allow approximately three feet of clearance below the elevation of the lowest adjacent dwellings. The freeboard is necessary should the 42-inch pipe become obstructed.

Lack of tunnel system flow capacity at the peak of the design storm versus the recently raised Salt River 10-year tailwater will result in a period of overflow from the 108-inch pipe and the inlet structure, into the basin. The risk to the basin and the adjacent area is less due to this overflow, than would be the risk due to using of less freeboard between the overflow elevation and the adjacent dwellings. This is evident because the overflow is approximately one to two percent of the total basin storage volume, and the peak basin water surface elevation has several feet of freeboard below the top of the inlet structure.

The above represents the changes that have occurred during the evolution of the tunnel system design, from the Alternatives Analysis to the present.

B. Interim Condition

An interim condition will exist for a limited time period between the construction of the intermediate dropshafts associated with Outer Loop Section 12 and the construction of the tunnel outfall, the connection to the Mesa Drain and the connection to Carriage Lane Basin. The intermediate dropshafts are required to be fully operational for the opening of Section 12 between University Drive and Southern Avenue. The Mesa Drain and Carriage Lane Basin connections are not required until the middle and late stages of the construction of Outer Loop Section 13 (the Superstition Interchange) scheduled 2 to 3 years subsequent to the opening of Section 12.

Due to the inflow to the tunnel being limited to flow from the intermediate dropshafts, the interim condition contributing drainage area to the tunnel will be relatively small. The interim drainage area will extend from the Superstition Freeway to University Drive, and lie between the Tempe Canal and Price Road.

During the interim condition, prior to the construction of the tunnel outfall, the tunnel will act as a detention storage structure. The final configuration of the tunnel system includes a pump station to be located at the tunnel low point, to empty the depressed tunnel occasionally for maintenance. This pump station will be used during the interim condition to drain the tunnel to the existing City of Tempe storm drain in Price Road. A sensor will only allow the tunnel pump station to operate while capacity exists within the Tempe pipe.

The tunnel will have the storage capacity to store nearly all runoff from a 10-year 24-hour storm from the above described limited drainage area. A peak of 26 cfs would occur near hour 23 of the storm. The tunnel would fill completely by hour 16 of a 50-year 24-hour storm, and an excess rate of 40 cfs would result. The 100-year 24-hour storm would cause the tunnel to fill at hour 14, with an excess of 289 cfs. The HEC-1 models are included in Appendix C, Volume II.

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C. Dr. Song's Studies

This subsection summarizes the results of hydraulic transient flow modeling for the Price Road Drain Tunnel conducted by the Charles C. S. Song Co. The purpose of Dr. Song's analysis was to simulate the design event(s) in the drainage tunnel system in order to ascertain whether transient flow conditions (backsurge) would occur. Additionally, he was to determine what measures, if any, were necessary to control potential transient flow conditions. Finally, he was to determine basic system design criteria such as total internal head and air release requirements, under design storm runoff conditions. (See Reference M).

The following describes the drainage system, the design and simulation conditions, the results of simulations, and conclusions and recommendations:

i. The Drainage System

These physical dimensions and configurations of the tunnel system used in the final simulation were determined through previous runs during the period of 1987 and 1988, or by HNTB prior to the start of Dr. Song's work.

a. Tunnel -- The drainage tunnel is 18 feet in diameter, 15,600 feet long, and has a slope of 0.0011 feet per foot (ft/ft).

b. Carriage Lane Basin Outfall Pipe -- The outfall pipe is a 108-inch in diameter and 10,500 feet long pipe having a slope of 0.00057 ft/ft.

c. Carriage Lane Basin Intake Structure -- The intake (head) structure consists of a 19-foot high control box having a cross-sectional area of 300 square feet with a 20-foot long weir. A 3 feet by 3 feet flap gate is also modeled to simulate the outflow from the basin to the outfall pipe.

d. Carriage Lane Basin -- A total surface area of 17.1 acres providing a volume of 157 acre-feet for the Carriage Lane Basin at elevation of 1192.00 feet is also included in the model.

e. Weirs at Tunnel Downstream End -- A 28-foot wide control section is used at the system high point (the slope break at top of the riser) with an invert of 1,157.00 feet. The weir at the river, just upstream of the energy dissipator, is set to an invert of 1,158.00 feet.

In Dr. Song's model, the whole length of the tunnel system including the drainage tunnel and the Carriage Lane Basin outfall pipe is divided into 87 elements, with a 300-foot increment for computational purpose. Exhibit 5 shows a schematic of the Dr. Song's model configuration. The 18-foot diameter drainage tunnel consists of the nodes from Sta. 1 to Sta. 3 and Sta. 40 to Sta. 90, while the outfall pipe consists of the nodes from Sta. 4 to Sta. 39.

The critical hydraulic elements in the tunnel system are listed in Table 1. It should be noted that the invert of the Carriage Lane Basin outfall pipe in the model is approximately six feet higher than the current design. Dr. Song and HNTB believe that this change will result in little or no effect on the overall results for the design conditions. Exhibit 6 presents the tunnel system configuration and the Price Road Drainage Tunnel System Profile is shown in Exhibit 7.

Table 1

Critical Hydraulic Elements
in The Tunnel System

Node No.	Structure Description	Invert	Top Elevation *
Sta. 1	Drop Structure A	1,120.00 ft	1,195.00 ft
Sta. 4	Carriage Lane Basin Head Structure	1,170.00 ft	1,189.00 ft
Sta. 37	Drop Structure B	1,164.36 ft	1,195.55 ft
Sta. 45	Dropshaft No. 5	1,118.00 ft	1,191.50 ft
Sta. 53	Dropshaft No. 4	1,115.37 ft	1,190.80 ft
Sta. 63	Dropshaft No. 3	1,112.07 ft	1,196.35 ft
Sta. 69	Dropshaft No. 2	1,110.09 ft	1,198.90 ft
Sta. 83	Dropshaft No. 1	1,105.47 ft	1,187.75 ft
Sta. 90	Tunnel Low Point	1,103.00 ft	1,182.00 ft

* Top elevation is obtained from HNTB's intermediate dropshafts site plan except at the Carriage Lane Basin head structure

ii. Design and Simulation Conditions

Prior to the series of simulation runs, it was decided that the tunnel system flow capacities would be designed for the 100-year tunnel inflow with a 10-year river flood. The river flood elevations used throughout Dr. Song's studies have been based upon preliminary results from Simons, Li and Associates' (SLA) modeling. Prior to 1989, extra elevation was added to the SLA elevation to account for the preliminary nature of their results, and an elevation of 1170.5 feet was used for the 10-year flood at Price Road. In February 1989, the 10-year flood elevation was raised to 1,172.12 feet by SLA. The study results of the effects due to the higher river flood elevation are included in Section III - C.iii.c.

HNTB provided Dr. Song with the tunnel inflow hydrographs at 11 locations. The origin of time in the mathematical model was set by Dr. Song at hour 11.75 of the hydrographs obtained from HNTB's HEC-1 results. Since Dr. Song's model only directly simulated the system upstream of the tunnel low point at Sta. 90, the head losses from the riser to the river are built into Dr. Song's computer program by internal, non-output documented calculations. The hydraulic control for the system was determined to be at the high point in the riser, therefore, Dr. Song used this location as the downstream control in his model. A head loss at the high point in the riser is calculated and added to the downstream end boundary condition, when there is outflow from the tunnel system. Dr. Song assumed a 21-foot wide weir and used the Francis submerged weir equation to calculate the flow rate.

Maximum outflows of 2,240 cfs and 1,150 cfs for the system resulted for the 100-year and 10-year design events versus 10-year and 100-year river floods, respectively, from Dr. Song's models. For the design condition of the 100-year tunnel inflows verse no flow in the river, a maximum outflow of 2,279 cfs was recorded from his model.

During the course of this transient flow modeling, the following simulation conditions were performed and documented in Dr. Song's reports:

Condition A -- 10-year tunnel inflow hydrographs with 100-year Salt River flood level at 1,173.00 feet at Hayden Road outfall (see Dr. Song's December, 1987 report).

Condition B -- 100-year tunnel inflow hydrographs with no flow in Salt River, with outfall at Price road (see Dr. Song's January and June, 1988 reports).

Condition C-- 100-year tunnel inflow hydrographs with 10-year Salt River flood level at 1,172.12 feet at Price road outfall (see Dr. Song's June and July, 1989 reports).

iii. Results of Simulations

The results of the simulations are organized into three parts based on the design conditions (Conditions A, B and C). Since Dr. Song has developed a total of nine (9) reports associated with the modeling of the tunnel system from May, 1987 to July, 1989, only the valid results are addressed herein.

a. Condition A -- 10-year tunnel inflow hydrographs with a 100-year Salt River flood level at 1,173.00 feet at Hayden Road.

This 1987 simulation run assumed the tunnel outfall was at Hayden Road and the system length was approximately 3,000 feet longer than for the current tunnel outfall at Price Road.

The modeling results indicated that no hydraulic transient problem exists, due to the tunnel being full of water at the start of the simulation. This will be true even with the current 100-year river flood level at 1,181.94 feet and the outfall at Price Road is used.

HNTB estimates the peak hydraulic grade line (HGL) at the Carriage Lane Basin head structure (at Sta. 4) may be high enough to cause overflow into the basin. This condition will be much closer to being the hydraulic control for the tunnel system due to the fact that the outfall location was moved upstream to Price Road, and a much greater differential exists between the 10-year and 100-year flood elevations there than at Hayden Road.

HNTB also calculates the HGL to be 1,179.80 feet at Sta. 4 for the condition of the 10-year tunnel inflows with a 10-year river flood elevation of 1,172.12 feet. This HGL could create backwater into some of lateral pipes connecting to the Carriage Lane Basin outfall pipe but would not endanger the neighborhood houses with flooding (minimum floor elevation at 1,192.00 feet).

b. Condition B — 100-year tunnel inflow hydrographs with no flow in Salt River.

The most severe surges were found to occur under this design condition. The Carriage Lane Basin with the head structure configuration described in the previous section was found to be an effective surge relief device. There is no overflow into the basin for this design condition. Dr. Song's simulation shows no tunnel outflow for the first 78 minutes from the start of the simulation, due to the tunnel storage. This storage volume is estimated to be 4,460,000 cubic feet (102 acre-feet). There also is no outflow from the Carriage Lane Basin to the Carriage Lane Basin outfall pipe for approximately 56 minutes during the simulation period, from minute 92 to minute 148, when the HGL in the head structure is higher than the water surface elevation in the basin. A maximum water surface elevation of 1181.50 feet is reached for this condition.

The maximum pressure head in the tunnel is estimated to be 100 feet, which consists of static pressure head, surge pressure head, and waterhammer pressure head. A maximum pressure head of 50 feet is estimated for the Carriage Lane Basin Outfall Pipe.

Three locations were established to be designed to allow air in the tunnel system to escape during the filling period. Drop Structure A, Drop Structure B, and the Carriage Lane Basin head structure should be designed to allow air outflow rates of 1,800 cfs, 600 cfs, and 600 cfs, respectively.

c. Condition C — 100-year tunnel inflow hydrographs with 10-year Salt River flood level at 1,172.12 feet.

These results present not only the hydraulic performance in the proposed tunnel system under the design condition but also show the effects of the increased 10-year river flood elevation from 1,170.50 feet to 1,172.12 feet. Maximum HGL at locations for the tunnel system are plotted in Exhibit 7.

The 1.6 feet increase in the 10-year river flood elevation appears to have the following effects (The comparison is made to Dr. Song's June, 1988 report):

1. There is no hydraulic transient problem in the tunnel system for this design condition for either river flood elevation, because the tunnel starts the design event full of water due to the river flood. Even with the original, lower 10-year river flood elevation, this condition produced the highest HGL at the upstream end, and resulted in a minor amount of overflow into the Carriage Lane Basin. The most significant effect of the higher flood elevation is to increase the overflow into the Carriage Lane Basin by overtopping the weir at the head structure. The overflow occurs because of the backwater effect, not by a backsurge resulting from transient conditions. The amount of the overflow increased from 0.53 acre-feet (as previously recorded) to 1.94 acre-feet. This overflow will last approximately 52 minutes with a maximum flow rate of 44.4 cfs.

2. Dr. Song assumed a submerged sharp-crested weir at the system high point in the riser and used the Francis submerged weir equation to calculate the outflow. HNTB felt that the weir in the riser is not a submerged sharp-crested weir and would not generate approximately two feet of additional head loss as Dr. Song predicted. However, Dr. Song's model did not consider the head losses taking place in the Drop Structure B, and his model showed adequate simulation results for the Carriage Lane Basin outfall pipe system. For the drainage tunnel, Dr. Song's results considered to be very conservative.

3. No flow will discharge into the Carriage Lane Basin outfall pipe from the Carriage Lane Basin for approximately 2.8 hours. The maximum pool elevation is at elevation 1,183.10 feet. This is 1.6 feet higher than the maximum pool elevation resulting for the 100-year event with no flow in the river. The rating curves for the Carriage Lane outfall pipe system is listed in Table 2.

Table 2

Rating Curve
for
The Carriage Lane Basin Outfall Pipe System

Time (min.)	HGL at Sta. 4 (ft)	Qt1 at Sta. 4 (cfs)	WSEL in Basin (ft)	Qout from Basin (cfs)	Qover to Basin (cfs)	HGL at Sta. 37 (ft)	Qt2 at Sta. 37 (cfs)
75.83	1,187.51	185	1,177.76	0	0	1,182.24	308
80.73	1,187.35	200	1,178.20	0	0	1,182.74	325
88.09	1,188.85	199	1,178.84	0	0	1,184.47	305
90.54	1,189.06	197	1,179.04	0	-2.4	1,184.81	299
95.45	1,189.14	191	1,179.43	0	-9.0	1,185.23	286
100.35	1,189.24	180	1,179.80	0	-19.9	1,185.72	269
105.26	1,189.34	167	1,180.15	0	-32.4	1,186.23	250
112.62	1,189.38	161	1,180.63	0	-38.9	1,186.23	242
119.97	1,189.41	156	1,181.04	0	-44.4	1,186.43	233
127.33	1,189.38	162	1,181.39	0	-38.5	1,186.06	238
139.59	1,189.17	189	1,181.83	0	-11.7	1,185.20	254
142.05	1,189.07	198	1,181.90	0	-2.9	1,184.95	258
144.50	1,188.55	204	1,181.96	0	0	1,184.61	260
186.19	1,182.85	201	1,182.82	0	0	1,179.21	239
188.64	1,182.85	203	1,182.85	3.4	0	1,179.03	241
196.00	1,182.81	216	1,182.94	15.6	0	1,178.58	255
210.72	1,182.48	233	1,183.04	32.5	0	1,177.69	274
235.24	1,181.88	248	1,183.09	47.7	0	1,176.52	291
250.00	1,181.60	253	1,183.08	52.7	0	1,176.02	298

NOTES:

Qt1: Total Flow at Sta. 4
Qout: Outflow from the Basin
Qover: Overflow to the Basin
Qt2: Total Flow at Sta. 37

iv. Conclusions and Recommendations

The conclusions for the hydraulic transient flow modeling are as follows:

a. There is no hydraulic transient problem in the tunnel system. The Carriage Lane Basin with head structure was found to be an effective surge relief device.

b. The most significant effect of the higher flood elevation is to cause overflow into the Carriage Lane Basin by overtopping the weir at the head structure. The overflow occurs by the backwater effect not by transient surge. The amount of the backflow is 1.94 acre-feet and will last approximately 52 minutes with a maximum flow rate of 44.4 cfs.

c. No flow will discharge into the Carriage Lane Basin outfall pipe from the Carriage Lane Basin for approximately 2.8 hours. The maximum pool elevation is at elevation 1,183.10 feet.

d. Since the maximum HGL in the Carriage Lane Basin outfall pipe is at 1,189.41 feet, it should not endanger the neighborhood houses (minimum floor elevation near 1,192.00 feet). Some of the pipes from adjacent detention pond should be carefully examined for backwater effects due to this high HGL.

e. There is no overflow from any of intermediate dropshafts according to Dr. Song's simulations.

Recommended design hydraulic parameters for the Price Road Drain Tunnel are described as follows:

a. The Carriage Lane Basin head structure should have at least 300 square feet of cross-sectional area with a 20-foot long weir at elevation of 1189.00 feet.

b. The drainage tunnel was hydraulically modeled based on the 10-year river flood elevation at 1172.12 feet. The river stream bed elevation must be kept below the elevation of 1,153.00 feet for the potential aggradation due to the future river work activities. Restudy or re-evaluation must be conducted if any of the above parameters has been changed.

c. We estimated that a river water surface elevation of 1,168.26 feet would not create any overflow into the Carriage Lane Basin, if any overflow into the basin is not acceptable to City of Mesa.

d. The maximum pressure head in the tunnel is estimated to be 100 feet, which consists of static pressure head, surge pressure head, and waterhammer pressure head. A maximum pressure head of 50 feet is estimated for the Carriage Lane Basin outfall pipe.

e. Three locations in the system were established to be designed for air relief. Drop Structure A, Drop Structure B, and the Carriage Lane Basin head structure should allow air outflow rates of 1,800 cfs, 600 cfs, and 600 cfs, respectively.

DIIGS/sc/3

D. Carriage Lane Basin Hydraulics

This subsection describes the results of hydraulic calculations of rating curves for the Carriage Lane Basin outfall pipe system, which includes a 108-inch outfall pipe, an intake (head) structure, and a 42-inch basin outlet pipe.

The results are presented in three cases as follows:

Case 1 : 200 cfs pumped flow and basin outflows with no flow in the Salt River (see Table 3)

Case 2 : 200 cfs pumped flow and 100-year tunnel inflows with a 10-year Salt River Flood Elevation of 1,172.12 feet (see Table 4)

Case 3 : 200 cfs pumped flow and basin outflows with 10-year Salt River flood elevation of 1,172.12 feet (see Table 5)

The purpose of these rating curve calculations was to estimate the available capacity in the outfall pipe and the basin outlet pipe for the above mentioned cases. Especially, it is to develop the stage/discharge relationship for the basin outlet pipe in order to demonstrate the basin be able to drain during off-peak condition. It should be noted that a constant 200 cfs pumped flow was assumed for all three cases.

i. Results of Calculations

Tables 3 to 5 present the rating curve calculations for the Carriage Lane Basin outfall pipe system. The results for Case 2 were obtained from the Dr. Song's last computer run made in July, 1989.

Table 3

Summary of Case 1 Rating Curve
for
The Carriage Lane Basin Outfall Pipe System

Case 1 : 200 cfs Pump Flow and Basin Outflows with No Flow in the Salt River

HGL at Head Structure (ft)	Q at Head Structure (cfs)	WSEL in Basin (ft)	Outflow from Basin (cfs)	HGL at Upstream Drop Structure B (ft)	Q at Upstream Drop Structure B (cfs)
1,169.82	230	1,170.34	30	1,164.21	230
1,170.00	242	1,171.00	42	1,164.39	242
1,170.36	259	1,172.36	59	1,164.75	259
1,170.63	272	1,173.63	72	1,165.02	272
1,170.99	284	1,174.99	84	1,165.38	284
1,172.70	288	1,177.13	88	1,167.09	288

Table 4

Summary of Case 2 Rating Curve
for
The Carriage Lane Basin Outfall Pipe System

Case 2 : 200 cfs Pumped Flow and 100-year Tunnel Inflows with
10-year Salt River Flood Elevation of 1,172.12 feet

HGL at Head Structure (ft)	Q at Head Structure (cfs)	WSEL in Basin (ft)	Outflow from Basin (cfs)	overflow to Basin (cfs)	HGL at Drop Structure (ft)	Q at Drop Structure (cfs)
1,187.51	185	1,177.76	0	0	1,182.24	308
1,187.35	200	1,178.20	0	0	1,182.74	325
1,188.85	199	1,178.84	0	0	1,184.47	305
1,189.06	197	1,179.04	0	-2.4	1,184.81	299
1,189.14	191	1,179.43	0	-9.0	1,185.23	286
1,189.24	180	1,179.80	0	-19.9	1,185.72	269
1,189.34	167	1,180.15	0	-32.4	1,186.23	250
1,189.38	161	1,180.63	0	-38.9	1,186.23	242
1,189.41	156	1,181.04	0	-44.4	1,186.43	233
1,189.38	162	1,181.39	0	-38.5	1,186.06	238
1,189.17	189	1,181.83	0	-11.7	1,185.20	254
1,189.07	198	1,181.90	0	-2.9	1,184.95	258
1,188.55	204	1,181.96	0	0	1,184.61	260
1,182.85	201	1,182.82	0	0	1,179.21	239
1,182.85	203	1,182.85	3.4	0	1,179.03	241
1,182.81	216	1,182.94	15.6	0	1,178.58	255
1,182.48	233	1,183.04	32.5	0	1,177.69	274
1,181.88	248	1,183.09	47.7	0	1,176.52	291
1,181.60	253	1,183.08	52.7	0	1,176.02	298

Table 5

Summary of Case 3 Rating Curve
for
The Carriage Lane Basin Outfall Pipe System

Case 3 : 200 cfs Pump Flow and Basin Outflows with 10-year Salt River Flood Elevation of 1,172.12 feet

HGL at Head Structure (ft)	Q at Head Structure (cfs)	WSEL in Basin (ft)	Outflow from Basin (cfs)	HGL at Upstream Drop Structure B (ft)	Q at Upstream Drop Structure B (cfs)
1,176.80	230	1,177.32	30	1,173.18	230
1,177.28	242	1,178.28	42	1,173.29	242
1,177.86	259	1,179.86	59	1,173.47	259
1,178.63	272	1,181.63	72	1,173.62	272
1,179.21	284	1,183.21	84	1,173.75	284

ii. Conclusions

The following conclusions are made to the Carriage Lane Basin Hydraulics:

a. The Carriage Lane Basin can be drained for all three cases. The 42-inch outlet pipe would have a lot more capacity than a constant 30 cfs, which was assumed in the HEC-1 model and the Dr. Song's models, during the off-peak period.

b. The overflow to the Carriage Lane Basin can be avoided by reducing the pumped flow during the peak flow period for the Case 2 condition. A sensor or telemeter can be installed in the Carriage Lane Basin head structure.

c. It seems that the pumped flow may be increased some what during a off-peak period without any impact to the basin if the whole tunnel system has been closely monitored.

d. The hydraulic design calculations for Drop Structures A and B are include in Appendix F, Volume II. The SAFHL drop structure study is Reference D. The hydraulic calculations for the Intermediate Dropshafts are Reference N. The Iowa Institute of Hydraulic Research heliciodal dropshaft report is Reference P.

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IV. Outfall Alternatives Hydraulic Analysis

The tunnel outfall will connect from the end of the Price Road Drain Tunnel to the Salt River. The outfall was initially located to be west of Price Road at Hayden Road. Recently, the configuration of the Red Mountain interchange and the alignment of the East Papago were refined. The outfall is now moved approximately 1,000 feet upstream to the east at Price Road. The new alignment will be parallel to and east of the Price Road.

The purpose of this outfall alternatives hydraulic analysis is to establish and define the hydraulic control elements in the outfall reach based on the results of the Dr. Song's hydraulic transient flow modeling at the system low point (the end of the tunnel, at Dr. Song's Sta. 90).

The hydraulic elements include the location and elevation of the system high point, the riser size, and the configurations from the system high point to the river. The hydraulic information shall be adequate for the development of the outfall construction plans to be done by others.

Since the design of the final configurations of the Red Mountain interchange and the East Papago is just beginning the final outfall alignment will not be established in this report. Therefore, the exact length and location of the tunnel outfall except the system high point shall be defined during the development of the outfall construction plans.

Three alternatives are defined and analyzed for the tunnel outfall. Due to the constraints of the existing 90-inch water line at First (1st) Street, the outfall will consist of a box culvert riser to the system high point just north to the water line for all three alternatives. The riser is a double 14′ X 13′ RCBC. For an RCBC Manning's n value of 0.015, this size provides approximately the same hydraulic conveyance as an 18-foot diameter tunnel with an n value of 0.013. The system high point is set at the end of the riser at an elevation of 1,157.00 feet. The outfall configuration for each alternative will provide for maintenance access to the tunnel system.

A. Outfall Alternatives

Three alternatives were defined at the beginning of this outfall restudy. The horizontal alignment was also also established and is shown on Exhibit 8. Since this outfall restudy is to identify the critical hydraulic control parameters and key elements, the exact horizontal and vertical controls for the selected outfall alternative must be established during the final plan production.

All three alternatives will have the identical box culvert riser from the system low point (end of tunnel) to the system high point. These alternatives were selected to have either open channel or box culvert from the system high point to the river. The geometry of the box culvert or open channel was determined by using the conveyance equivalent to the 18-foot diameter tunnel. Exhibits 9 to 11 show the outfall and flood profiles for Alternatives 1 to 3, respectively.

The detailed description for each alternative is included as follows:

Alternative 1: The same double 14′ X 13′ RCBC will be continued to the river with a slope of 0.0003434 ft/ft. An energy dissipator is required at the outlet to prevent the scour erosion to the river and the damage to the outfall by major river floods. A 100-foot long approach channel with a 30-foot bottom rectangular section is proposed between the end of RCBC and the energy dissipator.

Alternative 2: This alternative is a modification of Alternative 1 with a concrete-lined open channel instead of a RCBC. This channel will have a trapezoidal section with a bottom width of 15 feet and side slopes of 1.5 horizontal to 1 vertical.

Alternative 3: This alternative is an interim condition with a unpaved temporary open channel, and the energy dissipator is not recommended for this alternative. The channel will have a 30-foot wide bottom, and side slopes of three to one (3H : 1V). The same channel slope is used as Alternatives 1 and 2.

B. Hydraulic Analysis

i. Hydrology

The 100-year discharge values used for the tunnel outfall were obtained from the HEC-1 results presented in Section III - Hydrology Model.

ii. Hydraulic Model

The hydraulic models were set up to simulate the subcritical profile computations for the outfall alternatives from the system low point through the riser, to the river. The hydraulic analysis was performed using U. S. Army Corps of Engineers' HEC-2 water surface profile computer program for the 100-year tunnel flood conditions, using a 10-year river flood as a starting water surface elevation for one profile, and the other profile assuming no flow in the river.

A starting water surface elevation of 1,172.12 feet was obtained from the SLA's study and used for the profile runs made with the river flood, while critical depth was assumed at the energy dissipator at the outlet to the river for the runs made with the no river flow condition.

The tunnel stationing system was used as the section identification number (Sec. No.) in the HEC-2 model. Therefore, Sec. No. 165.23 in the HEC-2 model represents the tunnel station 165+23.71.

Manning's roughness coefficients (Manning's n) for the outfall were n=0.013 for the 18-foot diameter tunnel, n=0.015 for the RCBC and the concrete-lined channel, and n=0.030 for the unpaved temporary channel.

The normal bridge method was adopted to simulate the box culvert and the tunnel by using the BT and GR cards. The X2 card was used to repeat the BT cards from the previous cross section. Cross-section elevations were repeated and modified to elevation data in GR cards based on the value entered in field 9 on the X1 card.

Contraction coefficient of 0.1 or 0.3 and expansion coefficient of 0.3 or 0.5 were used to estimate energy losses. The higher coefficients were adopted to the transition sections.

C. Results

There are several criteria for the selection of the outfall alternative besides the HGL control at the system low point. These criteria are initial construction cost and maintenance cost, and ease of maintenance access. Each alternative is presented for the most suitable combinations in its category. Following ~~are~~ is a discussion of the outfall alternatives studied, with comments, and a table with exhibits comparing the results (see Table 6 and Exhibits 9 to 11).

Alternative 1 shows the highest water surface elevation (1,174.88 feet) at the equivalent of Dr. Song's Sta. 90, for the three alternatives studied. Since a maximum HGL of 1,176.08 feet was resulted from Dr. Song's July, 1989 modeling, a minimum freeboard of 1.2 feet is therefore provided. However, due to the simplistic nature of the system configurations in Dr. Song's model, he tended to apply very conservative downstream boundary conditions, in part to compensate for not explicitly calculating head losses through Drop Structure B. The HEC-2 elevation of 1,174.88 resulted from a configuration that is hydraulically equivalent to the configuration modeled by Dr. Song, using RCBC with conveyance equivalent to the tunnel.

Therefore, a minimum one foot of freeboard is recommended between Dr. Song's HGL elevation and the HEC-2 elevation, to compensate for losses occurring in his model at the river, which are similar in magnitude to losses believed to be occurring further upstream.

The high flow velocities vary from 6.26 fps in the riser, to 9.06 fps in the tunnel for Alternative 1. A velocity of 13.6 fps was determined for the critical flow at the top of energy dissipator. The box culvert flows well above critical depth for both profile runs. Hence, critical flow does not occur at the grade break locations in the study reach except at the energy dissipator.

The HEC-2 results of Alternatives 2 and 3 were virtually identical for the profile run with the river flow condition. The box culvert and open channel both flow at a subcritical flow depth for the profile runs performed. For Alternative 2, a 17.5-foot high, 15-foot wide trapezoidal concrete-lined channel with side slopes of 1.5H:1V will provide a minimum freeboard of one foot. However, the temporary channel for Alternative 3 will have to be an unpaved, trapezoidal, 18-foot high channel with a 30-foot bottom width and three to one side slopes.

An effort was made to optimize the configurations of the temporary open channel. The reduction in channel bottom width from 30 feet to 15 feet caused a slight increase in the water surface elevation (0.15 feet increase) and led to a slightly higher flow velocities in the open channel. In order to keep the velocities in the unpaved channel under 5 fps for the most part of the channel reach, a channel width of 100 feet is needed. The 30-foot channel bottom reduced the potential erosion problems in part of the channel and to the river bank and bottom. The excavation volume necessary for construction is needed only approximately three percent higher than the 15-foot channel bottom.

For a steeper channel slope such as 0.00115 ft/ft, there is virtually no change on the maximum water surface elevation but critical flow depth occurred at the system high point. The intent was to try to lower the outlet elevation to alleviate the potential erosion impact to the river. The consequence of lowing the outlet by two (2) feet is to cause an undesirable critical flow condition at the system high point. Therefore, the 30-foot channel bottom was evaluated for the temporary channel of Alternative 3.

D. Recommendations

Alternative 2 with a double 14′ X 13′ box culvert riser, a 15-foot bottom, concrete-lined channel, and an energy dissipator is the recommended alternative. It is essential to keep the maximum water surface elevation at the system low point at least one foot below the elevation of 1,176 feet. The size of the box culvert riser and the elevation of 1,157.00 feet would not be changed unless a new transient flow model study is performed. However, the grades in the riser can be adjusted with the consideration for ease of maintenance access in mind. There will a little or no effect on the hydraulic performance for the selected outfall alternative if the box culvert must be extended a few hundred feet downstream.

No inlet nor dropshaft design shall be considered in the reach of the RCBC riser unless the new transient flow model is conducted to the tunnel system. The first inlet from the SLA distributing area shall be kept at least 200 feet from the system high point at the top of the riser, which is set at Sta. 180+00, approximately 1,476 feet downstream (north) of the sytem low point at the end of tunnel (Sta. 165+23.71).

The weir at top of the energy dissipator should be kept at 1,158.00 feet. Also the access ramp shall be designed to allow the maintenance access. The special surface treatment to the floors of RCBC and open channel shall be designed to increase the roughness for ease of maintenance access.

TABLE 6

SUMMARY OF HEC-2 RESULTS

SECTION I.D.	ELMIN (ST.)	Q (cfs)	ALT. NO.	W/10-YEAR RIVER FLOOD CWSEL (ft.)	W/10-YEAR RIVER FLOOD VCH (fps)	W/NO RIVER FLOW CWSEL (ft.)	W/NO RIVER FLOW VCH (fps)
204.75	1158.00	2331	1	1172.12	5.50	1163.71	13.61
			2	1172.12	5.50	1163.71	13.61
			3	1172.12	1.87	1160.98	10.83
204.74	1156.15	2331	1	1172.24	4.83	1165.76	8.09
			2	1172.24	4.83	1165.76	8.09
			3	N/A	N/A	N/A	N/A
203.00	1156.21	2331	1	1172.12	6.41	1165.81	8.68
			2	1172.43	3.65	1165.85	8.21
			3	1172.13	1.88	1162.92	6.92
185.00	1156.82	2305	1	1173.62	6.33	1167.90	7.43
			2	1172.57	3.79	1167.13	7.34
			3	1172.27	1.95	1165.68	4.60
180.00	1157.00	2279	1	1174.04	6.26	1168.36	7.17
			2	1172.42	6.26	1167.32	7.89
			3	1172.04	6.26	1165.61	9.46
178.00	1138.00	2279	1	1174.20	6.26	1168.72	6.26
			2	1172.58	6.26	1167.88	6.26
			3	1172.22	6.26	1166.65	6.26
169.00	1136.87	2279	1	1174.91	6.26	1169.43	6.26
			2	1173.29	6.26	1168.59	6.26
			3	1172.91	6.26	1167.35	6.26
165.23	1103.00	2279	1	1174.88	9.06	1169.40	9.06
			2	1173.26	9.06	1168.56	9.06
			3	1172.88	9.06	1167.32	9.06

DIIGS/sc/9/

V. References

A. "OUTER LOOP HIGHWAY SR 360 INTERCHANGE
HYDROLOGY STUDY PART 1
-Area Uncontrolled by City of Mesa Detention Ponds"
HNTB - September 1986

B. "OUTER LOOP HIGHWAY SR 360 INTERCHANGE
HYDROLOGY STUDY PART 1
-Area Uncontrolled by City of Mesa Detention Ponds
Appendix"
HNTB - September 1986

C. "OUTER LOOP HIGHWAY SR 360 INTERCHANGE
HYDROLOGY STUDY PART 2
-Inflow Hydrographs to Outer Loop Offsite Drainage System"
HNTB - October 1986

D. "OUTER LOOP HIGHWAY SR 360 INTERCHANGE
HYDROLOGY STUDY PART 2
-Inflow Hydrographs to Outer Loop Offsite Drainage System
Appendix"
HNTB - October 1986

E. "OUTER LOOP HIGHWAY SR 360 INTERCHANGE
ALTERNATIVES ANALYSIS
Offsite Drainage System and Outfall to Salt River"
HNTB - October 1986

F. "OUTER LOOP HIGHWAY SR 360 INTERCHANGE
ALTERNATIVES ANALYSIS
Offsite Drainage System and Outfall to Salt River
Appendix"
HNTB - October 1986

G. "Outer Loop Freeway
Superstition Freeway to the Salt River
Hydrology Report, Part A - Offsite Hydrology"
Boyle Engineering Corporation
May 1986

H. "Outer Loop Freeway
Superstition Freeway to the Salt River
Hydrology Report, Part A - Offsite Hydrology
Appendices B, C and D"
Boyle Engineering Corporation
May 1986

I. "Outer Loop Freeway
Superstition Freeway to the Salt River
Hydrology Report, Part B - Onsite Hydrology"
Boyle Engineering Corporation
August 1986, Revised October 1986

K. "Outer Loop Freeway
Superstition Freeway to the Salt River
Hydrology Report, Part B - Onsite Hydrology
Boyle Engineering Corporation
Appendices E and F"
August 1986, Revised October 1986

L. "Outer Loop Freeway
Superstition Freeway to the Salt River
Storm Drainage Pump Station Study"
Boyle Engineering Corporation
August 1986

M. Dr. Song's Reports and Computer Runs
1987-1989

N. "INTERMEDIATE DROPSHAFTS
Five Dropshafts and Tunnel Connections
Price Road Tunnel - Tempe Arizona
Design Report and Calculations"
HNTB
November 1988

O. "Phoenix-Casa Grande Highway
Stormwater Interceptor
Drop Structures Model Studies"
Warren Q. Dahlin and Joseph M. Wetzel
Saint Anthony Falls Hydraulic Laboratory
February, 1985

P. "Model Study of Price Road Tunnel
Heliciodal-Ramp Dropstructures"
Subhash C. Jain and John F. Kennedy
IIHR Limited Distribution Report No. 152
March 1989

Q. "Outer Loop Highway
Mesa Drain & Tempe Drain
Relocation Design Report"
HNTB - March 1989

R. "Superstition Freeway
Conceptual Study for Drainage
(Tempe Canal to RWCD Canal)"
Yost and Gardner Engineers
February 1975

S. "Chandler Stormwater Management
Master Plan - Phase 2
City Project #84-313
Working Paper #4"
Camp Dresser and McKee
May 1986

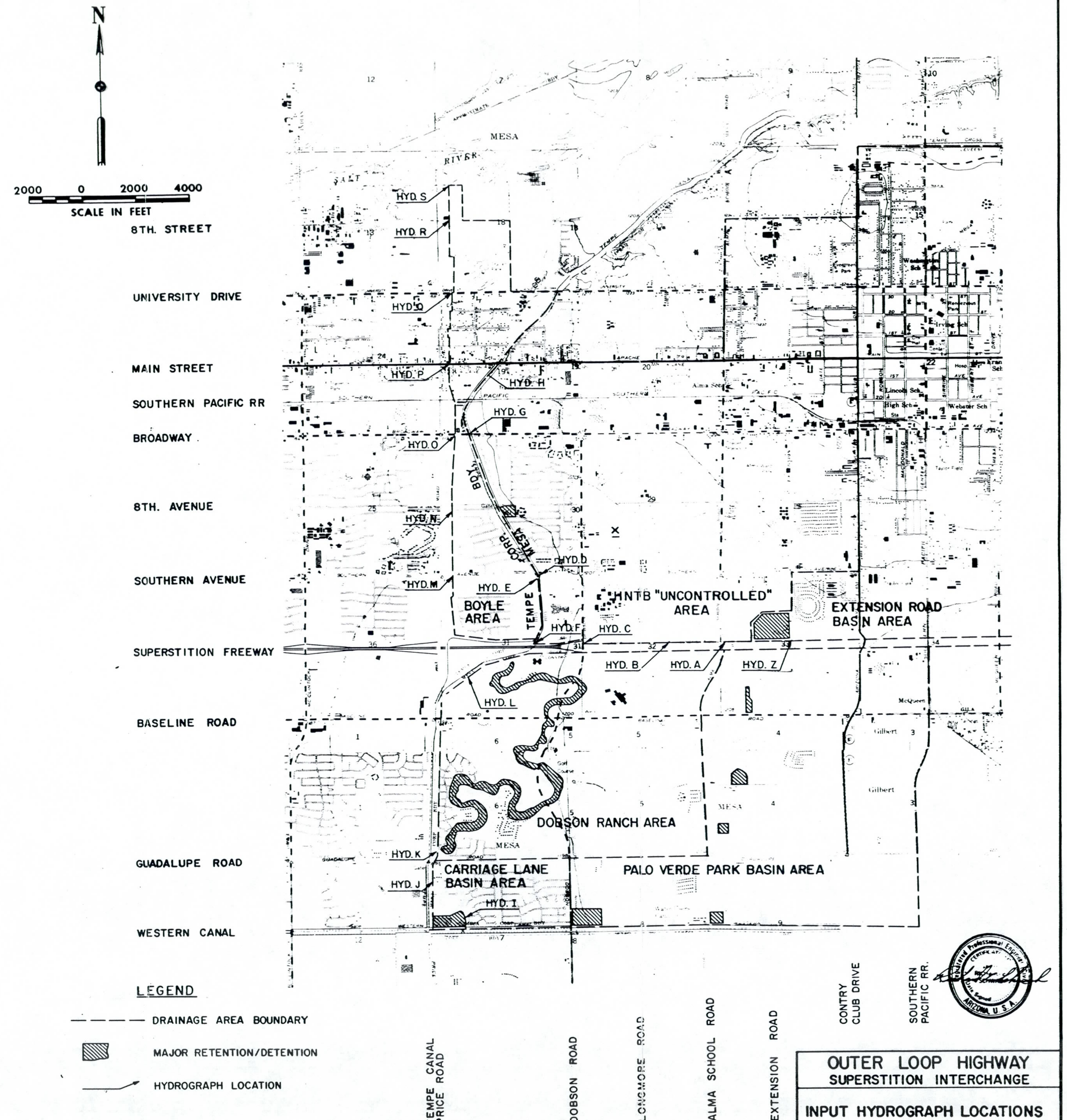
T. "Chandler Stormwater Management
Master Plan - Phase 2
City Project #84-313
Working Paper #5"
Camp Dresser and McKee
May 1986

50-YEAR INFLOW HYDROGRAPHS: PEAK FLOWS AND RUNOFF VOLUMES

Hydrograph	Location	Peak Flow	Time of Peak	Volume
A	Superstition Freeway and Alma School Road	228 cfs	13.25	62 AF
B	Superstition Freeway and Longmore Road	148 cfs	13.25	21 AF
C	Superstition Freeway and Dobson Road	335 cfs	13.50	50 AF
D	Southern Avenue and Tempe Canal	466 cfs	14.00	90 AF
E	Southern Avenue and Tempe Canal (pipe)	130 cfs	12.50	101 AF
F	Superstition Freeway and Tempe Canal	20 cfs	12.75	4 AF
G	Broadway Road and Tempe Canal (pipe)	129 cfs	13.25	30 AF
H	Between S.P.R.R. and Apache at Tempe Canal	27 cfs	16.75	7 AF
I	Carriage Lane Basin	474 cfs	13.00	775 AF
J	South of Guadalupe Road at Outer Loop (pipe)	40 cfs	12.25	8 AF
K	North of Guadalupe Road at Outer Loop (pipe)	42 cfs	21.50	163 AF
L	Proposed SR 360 I/C	90 cfs	12.75	10 AF
M	Southern Avenue and Outer Loop Corridor	138 cfs	12.75	17 AF
N	Balboa and Outer Loop	178 cfs	12.75	21 AF
O	Broadway and Outer Loop	83 cfs	12.75	9 AF
P	Apache and Outer Loop	45 cfs	12.75	5 AF
Q	University Drive and Outer Loop Corridor	186 cfs	12.75	23 AF
R	8th Street and Outer Loop Corridor	93 cfs	12.75	15 AF
S	1st Street and Outer Loop Corridor	14 cfs	12.50	1 AF
Z	Extension Road and Superstition Freeway	85 cfs	13.00	435 AF

100-YEAR INFLOW HYDROGRAPHS: PEAK FLOWS AND RUNOFF VOLUMES

Hydrograph	Location	Peak Flow	Time of Peak	Volume
A	Superstition Freeway and Alma School Road	264 cfs	13.25	78 AF
B	Superstition Freeway and Longmore Road	186 cfs	13.25	32 AF
C	Superstition Freeway and Dobson Road	412 cfs	13.50	70 AF
D	Southern Avenue and Tempe Canal	679 cfs	13.75	125 AF
E	Southern Avenue and Tempe Canal (pipe)	130 cfs	12.50	108 AF
F	Superstition Freeway and Tempe Canal	35 cfs	14.00	7 AF
G	Broadway Road and Tempe Canal (pipe)	130 cfs	13.00	39 AF
H	Between S.P.R.R. and Apache at Tempe Canal	61 cfs	16.00	22 AF
I	Carriage Lane Basin	546 CFS	13.00	839 AF
J	South of Guadalupe Road at Outer Loop (pipe)	40 cfs	12.25	8 AF
K	North of Guadalupe Road at Outer Loop (pipe)	53 cfs	21.50	199 AF
L	Proposed SR 360 I/C	110 cfs	12.75	12 AF
M	Southern Avenue and Outer Loop Corridor	170 cfs	12.75	20 AF
N	Balboa and Outer Loop	219 cfs	12.75	25 AF
O	Broadway and Outer Loop	102 cfs	12.75	11 AF
P	Apache and Outer Loop	56 cfs	12.75	6 AF
Q	University Drive and Outer Loop Corridor	232 cfs	12.75	28 AF
R	8th Street and Outer Loop Corridor	115 cfs	12.75	18 AF
S	1st Street and Outer Loop Corridor	18 CFS	12.50	1 AF
Z	Extension Road and Superstition Freeway	160 cfs	13.00	641 AF



OUTER LOOP HIGHWAY
SUPERSTITION INTERCHANGE

INPUT HYDROGRAPH LOCATIONS FOR HYDRAULIC MODEL

HNTB HOWARD NEEDLES TAMMEN & BERGENDOFF

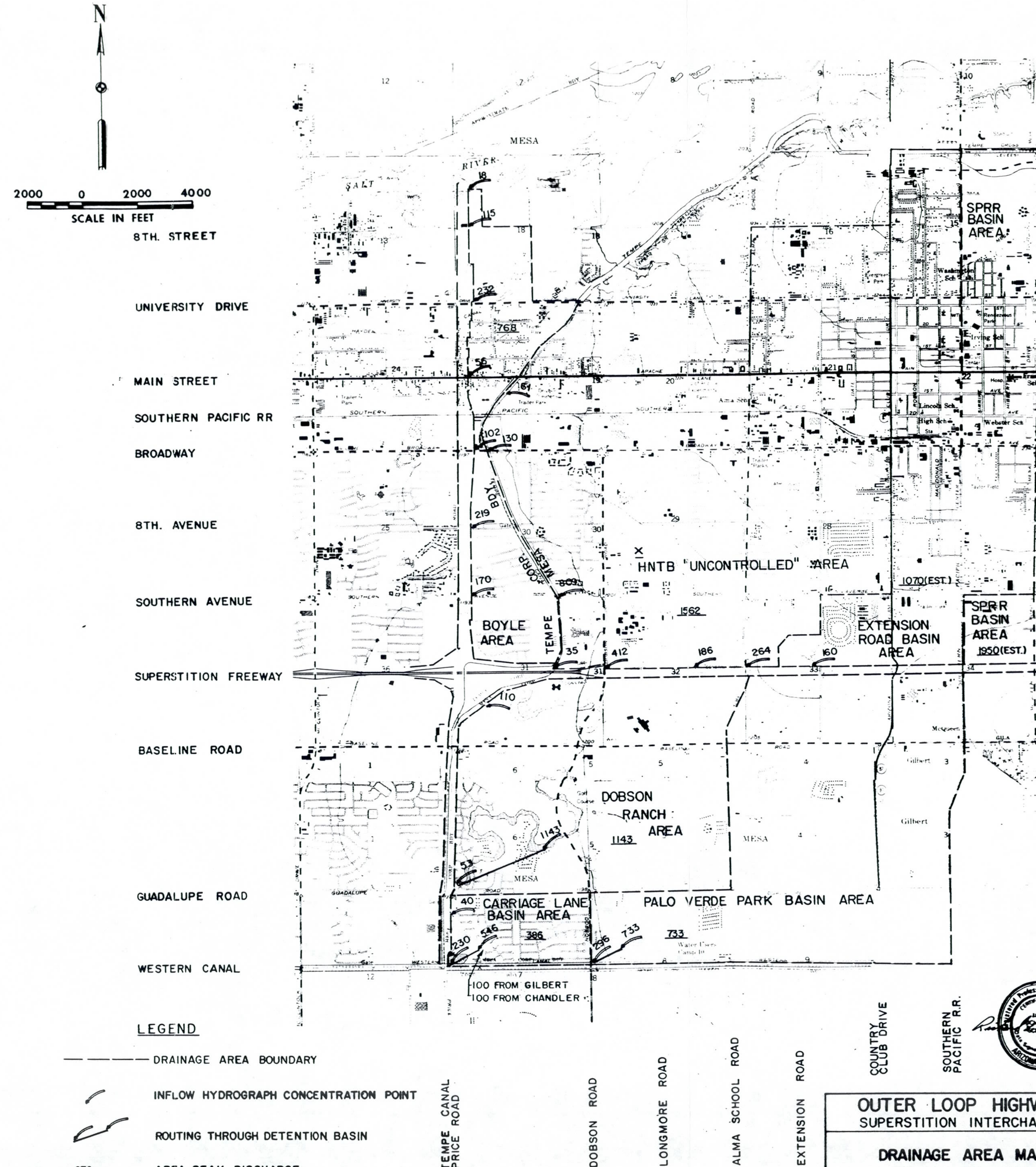
EXHIBIT 1

SUBAREA CHARACTERISTICS AND HEC-1 PARAMETERS

Subarea Number	Total Area (Sq. Mi.)	Directly Contributing Area (Sq. Mi.)	*Maximum Retention Volume (Acre-Feet)	Percent Impervious Area	SCS Curve Number	Hydraulic Length (Feet)	SCS Lag Time (Hours)
1	3.6	2.5	120	55	82	23,000	2.6
2	0.5	0.5	--	65	85	7800	0.9
3	2.9	2.9	--	65	85	18,500	2.1
4	0.1	0.1	--	65	85	9,000	0.5
5	See Boyle Engineering Corporation Report: "Outer Loop Freeway Superstition Freeway to the Salt River Hydrology Report Part A - Offsite Hydrology" May 1986						
6	See HNTB Report: "Outer Loop Highway SR360 Interchange Hydrology Study Part 1 - Area Uncontrolled by City of Mesa Detention Ponds" September 1986						
7	Not modeled with HEC-1; See Yost and Gardner Engineers Report: "Superstition Freeway Conceptual Study for Drainage (Tempe Canal to RWCD Canal)" February 1975						

*Not including detention specifically modeled in HEC-1

Subarea 1 - Palo Verde Park Basin Area
Subarea 2 = Carriage Lane Basin Area
Subarea 3 = Dobson Ranch Area
Subarea 4 = Offsite Area Adjacent to Corridor, South of Superstition Freeway
Subarea 5 = Boyle Area
Subarea 6 = HNTB "Uncontrolled" Area
Subarea 7 = Extension Road Basin Area



LEGEND

DRAINAGE AREA BOUNDARY

INFLOW HYDROGRAPH CONCENTRATION POINT

ROUTING THROUGH DETENTION BASIN

678 AREA PEAK DISCHARGE

678(EST.) ESTIMATED FROM YOST AND GARDNER 50-YEAR PEAK DISCHARGE

OUTER LOOP HIGHWAY
SUPERSTITION INTERCHANGE

DRAINAGE AREA MAP
RUNOFF SUMMARY
100-YEAR DISCHARGES

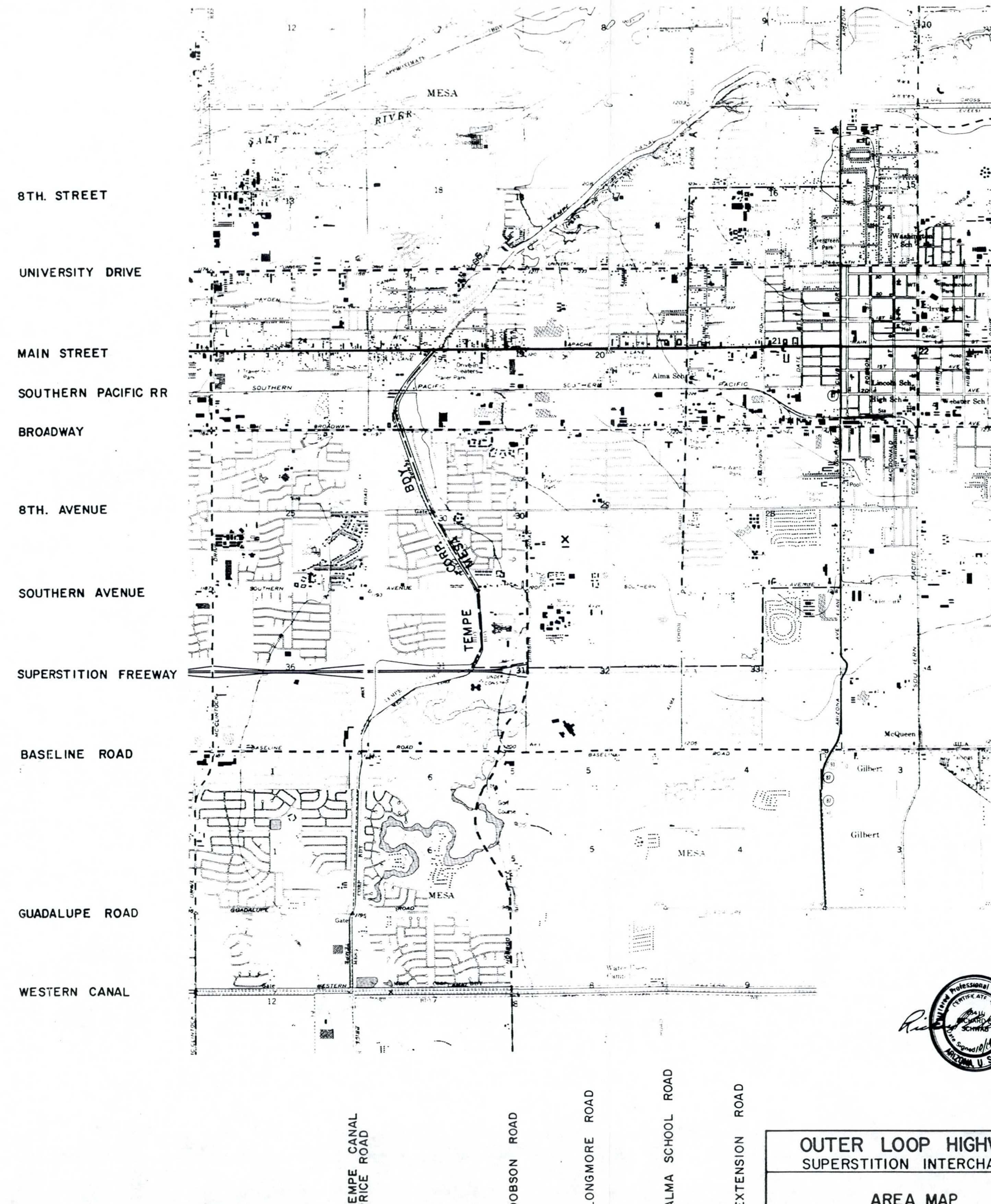
HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

EXHIBIT 2

TUNNEL ALTERNATIVE
100-YEAR DESIGN HYDROGRAPHS: PEAK FLOWS AND RUNOFF VOLUMES

Hydrograph	Location	Peak Flow	Time of Peak
50	Superstition Freeway and Mesa Drain	1705 cfs	13.83
60	Carriage Lane Basin Outflow	230 cfs	13.00
70	South of Guadalupe Road at Outer Loop	275 cfs	13.25
80	North of Guadalupe Road at Outer Loop	304 cfs	14.83
1	Proposed SR 360 I/C	2004 cfs	13.83
2	Balboa and Outer Loop	2077 cfs	14.00
3	University Drive and Outer Loop Corridor	2279 cfs	14.08

NOTE: Hydrograph numbering is from HEC-1 computer model, listed in Appendix A.

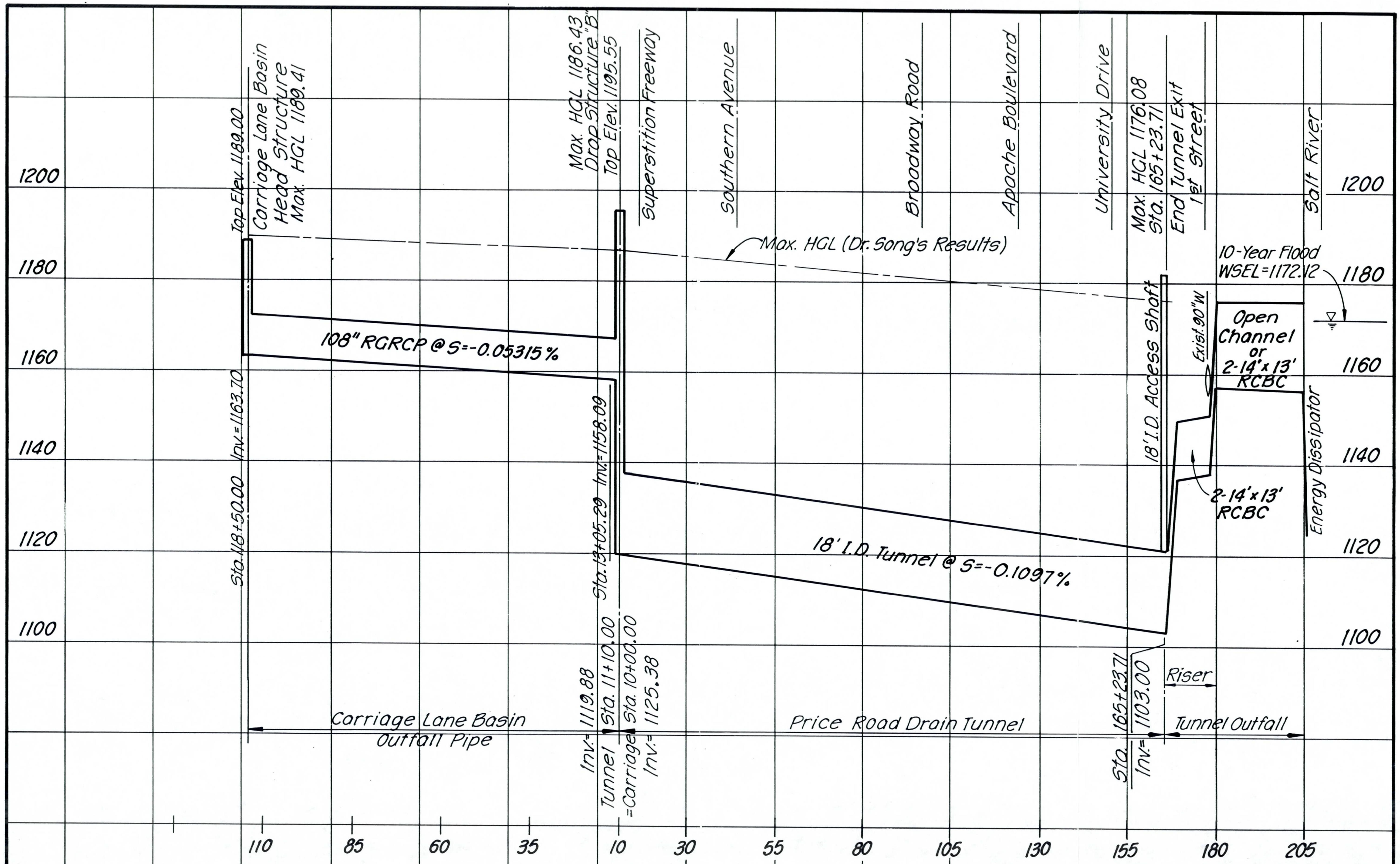


OUTER LOOP HIGHWAY
SUPERSTITION INTERCHANGE

AREA MAP
FOR
HYDRAULIC MODEL

HNTB
HOWARD NEEDLES TAMMEN & BERGENDOFF

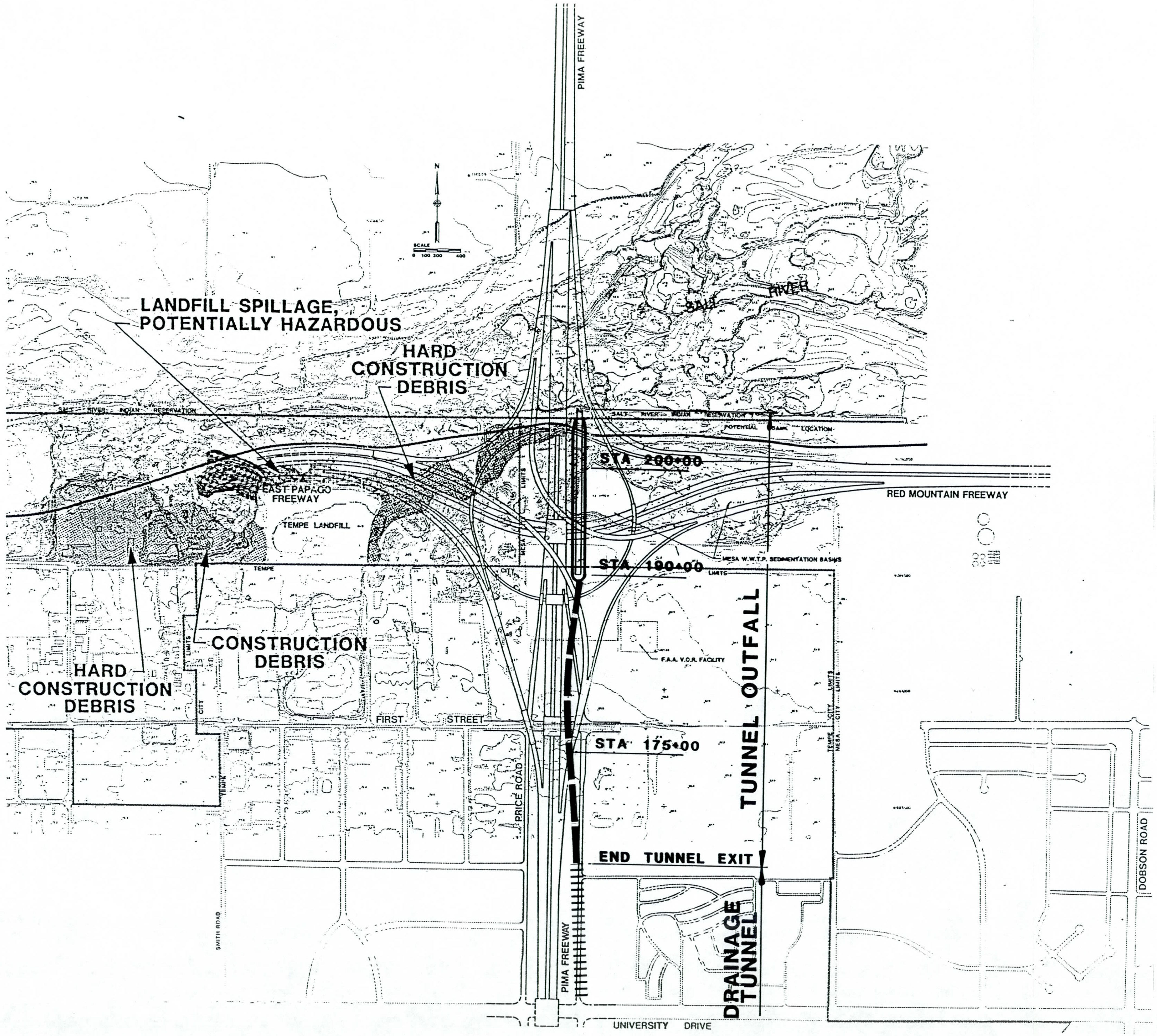
EXHIBIT 3



Price Road Drain Tunnel System Profile

EXHIBIT 7

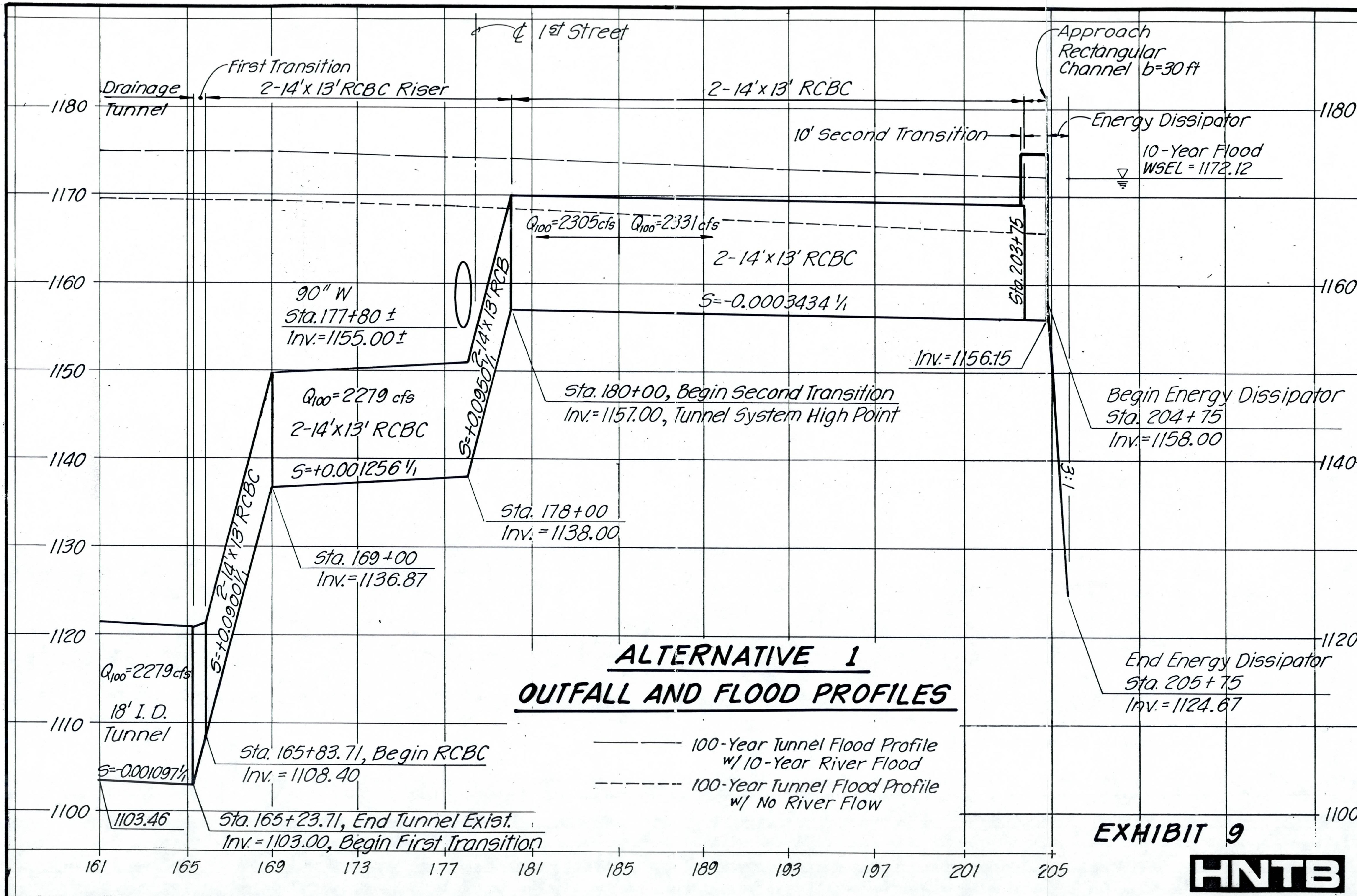
HNTB



TUNNEL OUTFALL ALIGNMENT

EXHIBIT 8

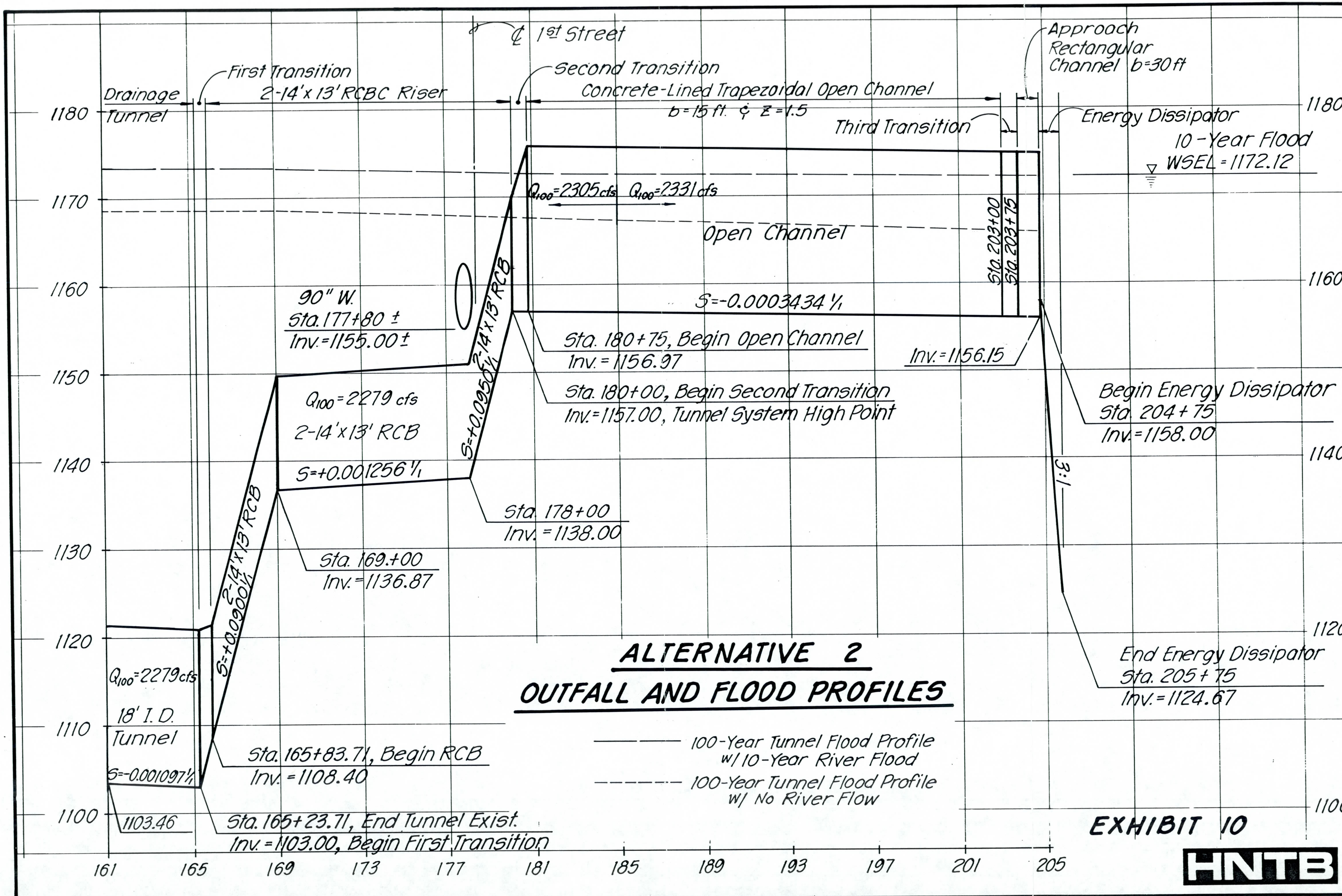
HNTB



ALTERNATIVE 1
OUTFALL AND FLOOD PROFILES

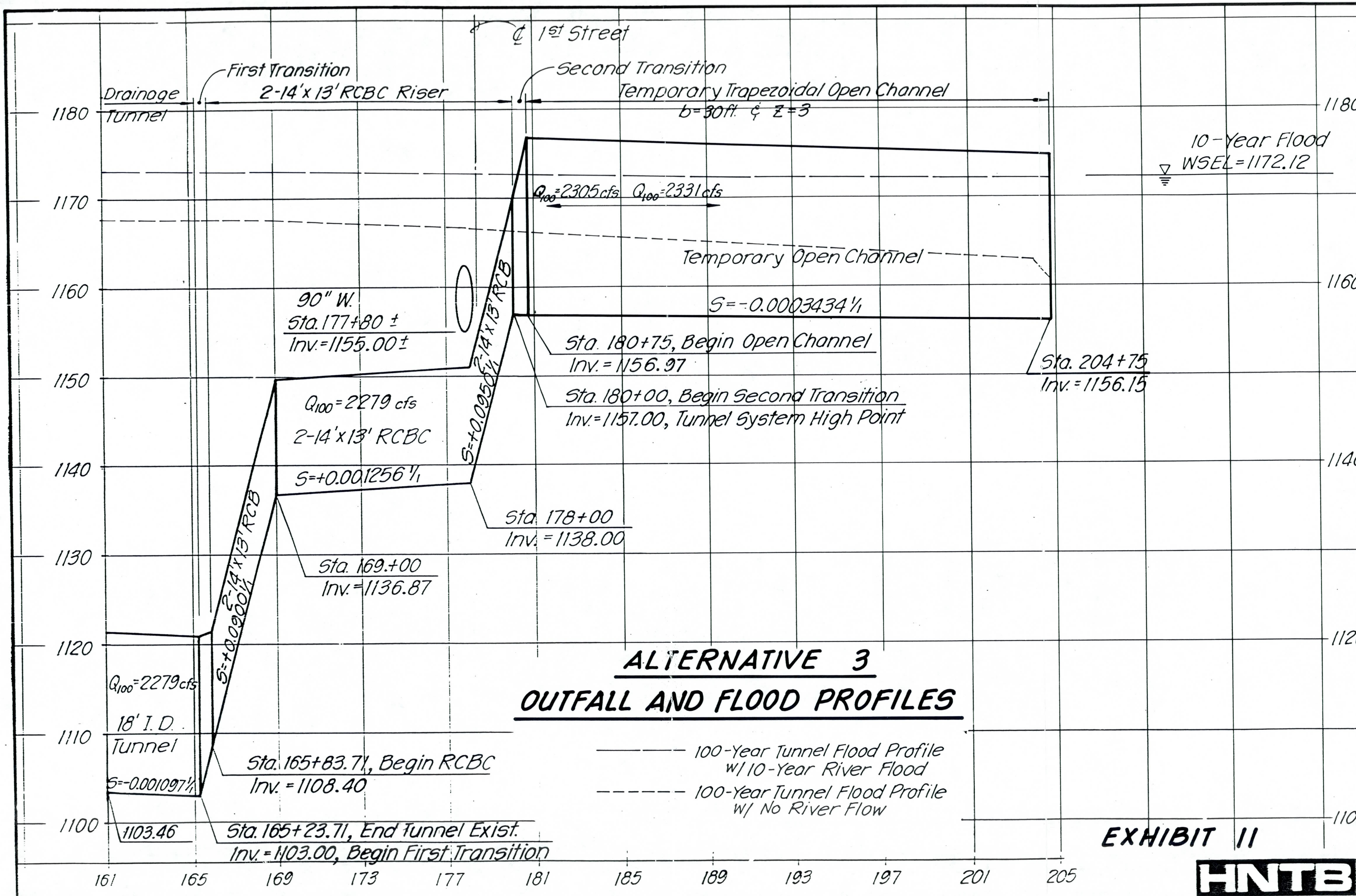
EXHIBIT 9

HNTB



ALTERNATIVE 2
OUTFALL AND FLOOD PROFILES

EXHIBIT 10

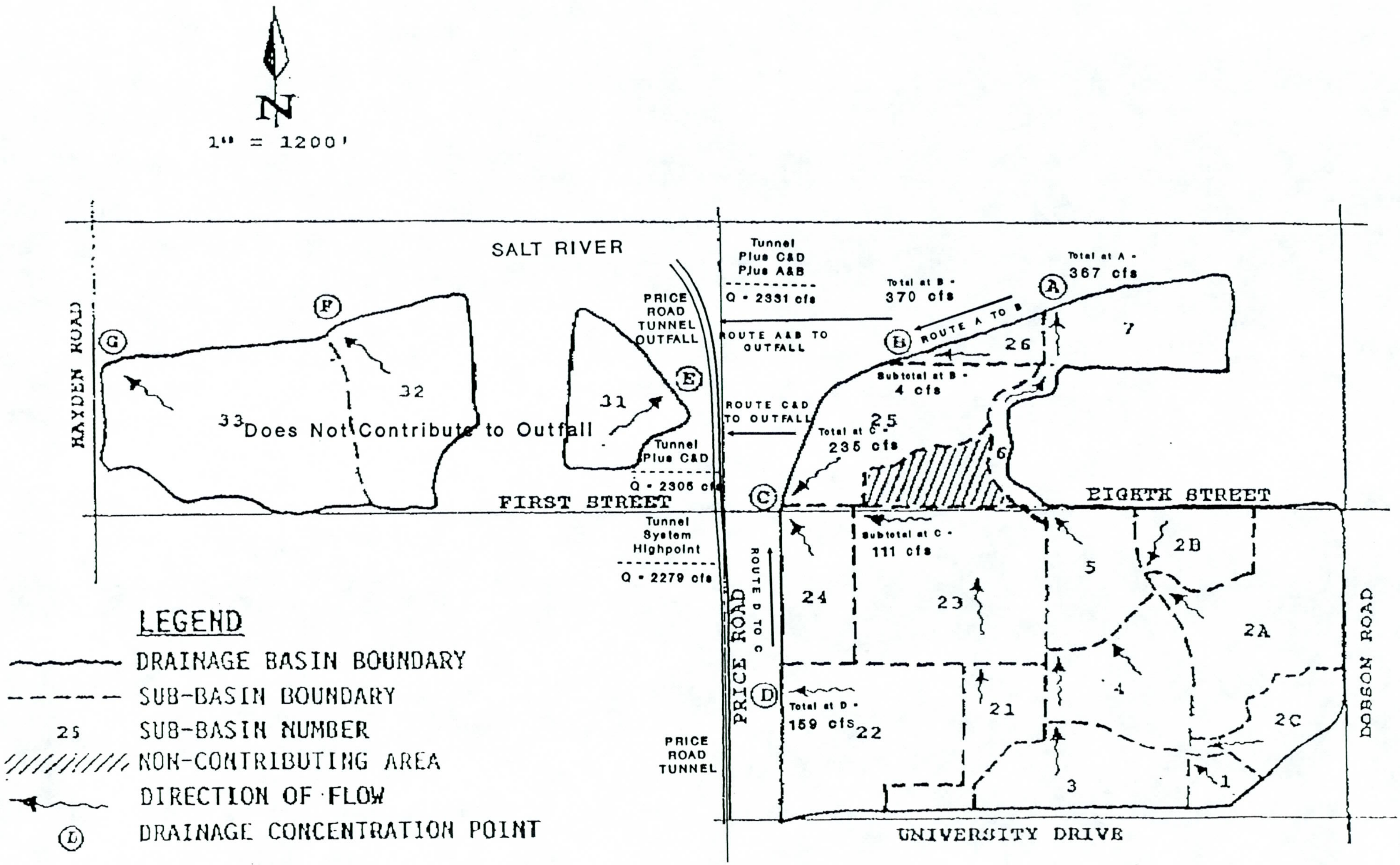


ALTERNATIVE 3

OUTFALL AND FLOOD PROFILES

EXHIBIT 11

HNTB



SCHEMATIC DIAGRAM OF THE WATERSHED

EXHIBIT 4

Updated by HNTB July 1989

sla SIMONS, LI & ASSOCIATES, INC.

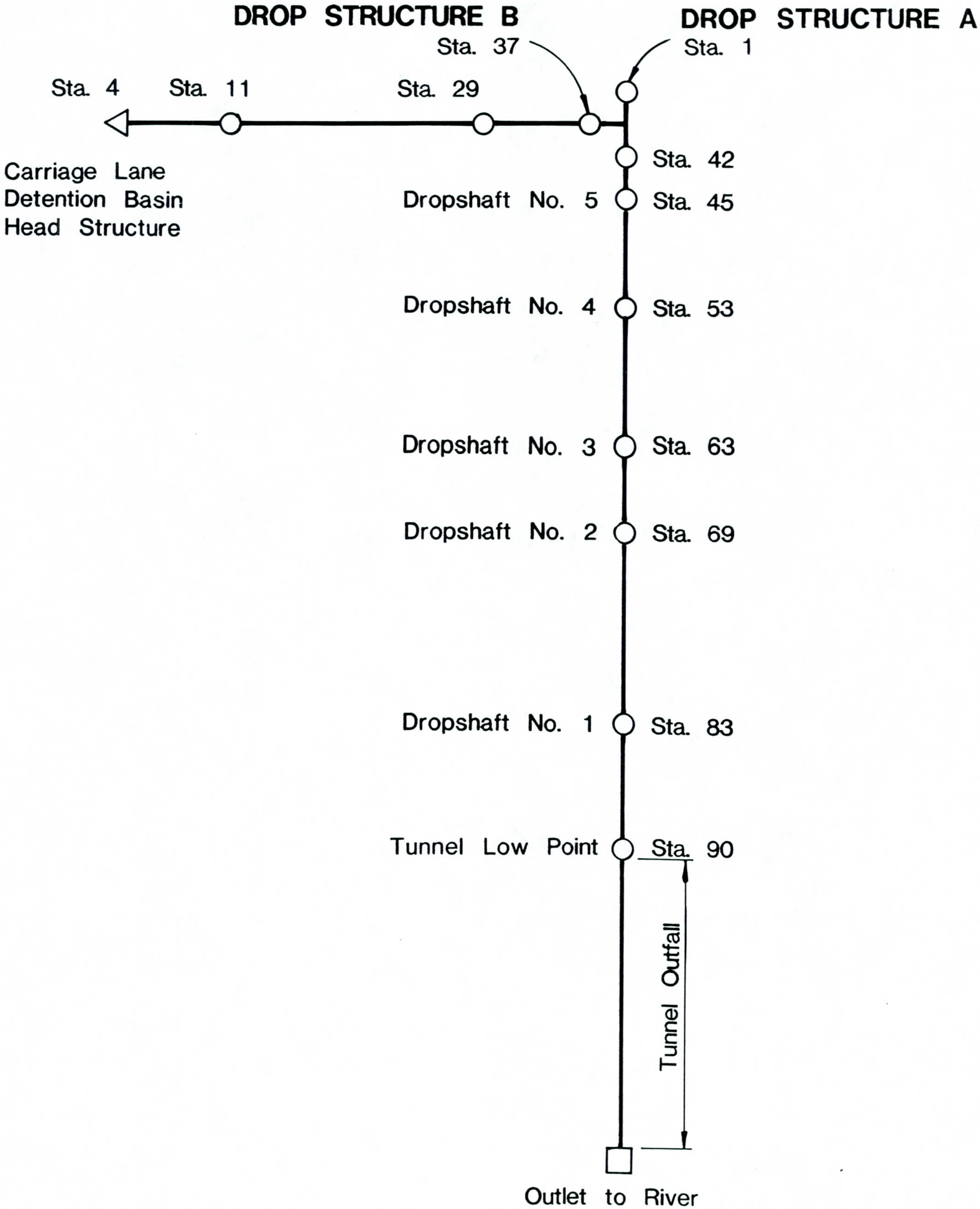


EXHIBIT 5
DR. SONG'S MODEL CONFIGURATION

HNTB

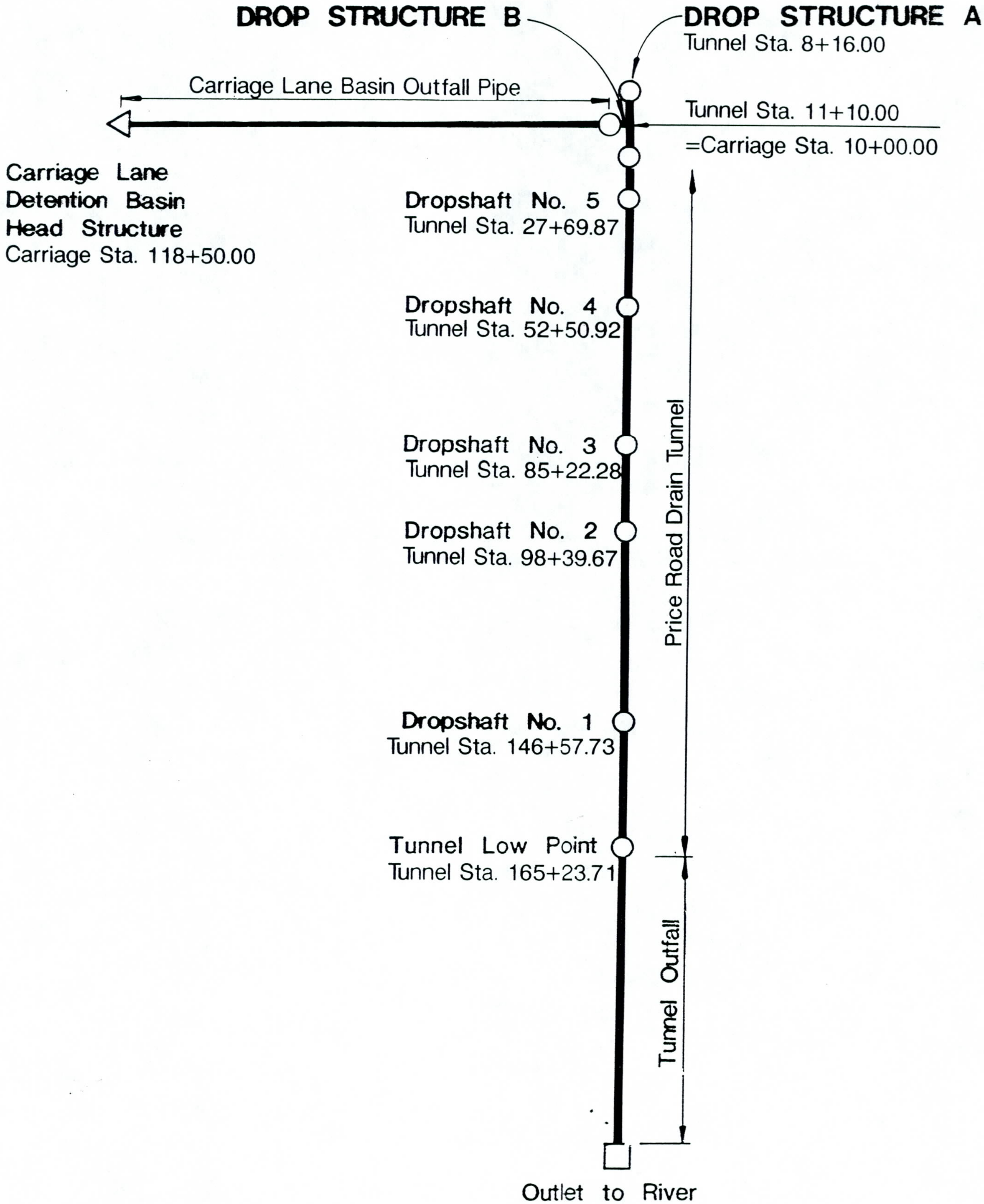


EXHIBIT 6

TUNNEL STYSTEM CONFIGURATION

HNTB