PIMA ROAD THREE BASINS PROJECT

MISCELLANEOUS DESIGN MEMORANDUMS

PENTACORE
ARIZONA

Civil Engineering
Construction Administration
Land Surveying
GPS Surveys
Planning
ADA Consulting

MEMORANDUM

Date: April 7, 1998 **Project No.:** 5001.0007

To: Doug Cullinane

Company: City of Scottsdale

From: Christopher Hassert, P.E.

Subject: **Pima Road Three Basins Project**
Tract 21 Drainage
Design Memorandum

Comments

INTRODUCTION

This Design Memorandum focuses on the drainage for Tract 21. This parcel encompasses approximately 0.18 square miles in Section 36 of T4N, R4E and Section 31 of T4N, R5E. The location of Tract 21 in relation to the proposed Pima Freeway and other components of the Outer Loop Project is shown on Exhibit A at the end of this memorandum.

Coordination with other agencies and groups will be a critical component of the drainage design for the Tract 21 system. Specifically, close coordination with ADOT and their design engineer is necessary since the drainage system designed for Tract 21 will be shared by both the future Pima Freeway and Tract 21 occupants. Additionally, coordination with private utility companies will be required during the design of this system.

Assumptions and Constraints

Pentacore analyzed the hydrology for existing conditions in Tract 21. While the hydrology has been developed for existing conditions, the hydrology also simulates post-development, full buildout conditions. Per the City of Scottsdale, future development will be required to provide on-site retention that will control the post-development discharge rates from Tract 21. The post-development retention will ensure discharge rates from Tract 21 are comparable with the discharge rates under existing conditions. To model the existing conditions hydrology, the following was used:

- subbasin areas were determined from the digital mapping;
- parameters used for the intra-basin kinematic wave routing technique were obtained from the digital terrain model (DTM) including slope, length and area;
- roughness coefficients for the kinematic wave routing were estimated based on the characteristics of the terrain. A Manning's roughness value of 0.090 was used for overland flow and a value of 0.035 was used for routing within naturally formed washes (See Appendix A at the end of this memorandum);

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Other constraints and assumptions used for the hydrology include:

a) The area is divided into east (T21E) and west (T21W) subbasins which drain to a common concentration point at the southwest corner of Tract 21. T21W encompasses the west portion of Tract 21 and directly contributes runoff to the proposed drainage channel along the Pima Freeway. The northern boundary of Tract 21 consists of a spur dike that is coincidental with the proposed Union Hills Road extension. The spur dike is assumed to extend the full length of the northern boundary of Tract 21 to the intersection with the future Pima/Princess Drive extension. T21E encompasses the east portion of Tract 21. Flows from T21E join flows from T21W to produce the resultant peak flow at the southwester concentration point.

b) Approximately 0.04 square miles of area bordering the future extension of Pima/Princess Drive is omitted from T21E. Since this area naturally drains away from the Tract 21 concentration point under existing conditions, the hydrology model has been developed to assume this runoff will be conveyed southerly, under the proposed Princess/Pima Drive.

c) The SCS soil survey map for Maricopa County indicates the soils within Tract 21 are predominantly classified as hydrologic soil type "B". The hydrologic conditions for the drainage area are described as poor. The City's Design Standards Manual prescribes a soil loss SCS curve number of 77 for these conditions, however a value of 74 which slightly deviates from the prescribed value has been used to remain consistent with the current modeling used throughout the Pima Road Three Basins Project.

d) The channel configuration used for concentrated flow through the subbasins assumes the following:

   - unlined, trapezoidal channels with an 8-foot bottom invert and 2H:1V side slopes;
   - Manning's roughness coefficient of 0.035 used for natural desert washes;
   - the slope of the channels are estimated to be consistent with the natural grade slope as determined from the DTM for the area.

HEC-1 Modeling

The existing condition model presented in this study is developed for the 100-year/24-hour storm event. Using the assumptions and approximations previously described, the input and output for the HEC-1 model are shown in Appendix A at the end of this memorandum. The mapping used to delineate the modeling parameters for Tract 21 is shown on Exhibit B, "Tract 21 Hydrology" also at the end of this memorandum.

Hydrology Results and Design Values

The results from the hydrology model indicate the peak runoff flowrate at the southwest corner of Tract21, under existing conditions is 239 cfs. From Tract 21, two options are proposed for directing this 239 cfs downstream. Option 1 involves splitting the flow, whereby directing a portion of the runoff west under the Pima Freeway into the open drainage system for the Scottsdale Perimeter Center, and directing the remaining flow south along the east side of the Pima Freeway. Option 2 involves directing the entire flow south along the east side of the Pima Freeway The following section describes these options in greater detail.

HYDRAULICS

Tract 21 Hydraulics

Hydraulic analyses were conducted for channel sizing within Tract 21. Channels sized within Tract 21 were designed to carry the peak flow of 239 cfs at the downstream end of the system and taper progressively to carry a fraction (30%-60%) of the peak flow near the upstream end of the system where runoff into the channel is less substantial.

The profile for the proposed channel paralleling the freeway is shown in Exhibit C, "Tract 21 Channel Plan and Profile". The channel is divided into four segments labeled A-D. The individual segments lie between grade breaks which are necessary based on the existing topography. Segments A and B are proposed to be unlined as design velocities are maintained below 6 fps and the channel flow is subcritical. Segments C and D are proposed to be lined because both the slope and channel flows increase, causing velocities to exceed 9 fps and a supercritical flow regime. The hydraulic calculations and sketches showing proposed cross sections for channel segments A through D are presented in Appendix B.

Pentacore will consistently update the hydraulics of the system as the design proceeds to completion and make necessary adjustments throughout the design process to ensure the efficiency of the system and cost benefit to the City is optimized.

Discharge from Tract 21

Historically, flows from Tract 21 have proceeded west and were ultimately intercepted by the Bureau of Reclamation's (USBR) detention basin 3 located between Scottsdale Road and Pima Road. The proposed Outer Loop Freeway will intercept the runoff from being conveyed directly to detention basin 3. Alternatively, the runoff from Tract 21 may be easily conveyed to the USBR detention basin 4, which is located east of Pima Road and the Outer Loop Freeway. Because the contributing runoff area for Tract 21 is relatively small (0.18 square miles), the impacts to the downstream detention facilities are expected to be minimal, therefore conveying the runoff away from USBR's detention basin 3 can be investigated.

Once the peak flow generated by Tract 21 reaches the concentration point, it must be directed downstream. As mentioned previously in this memorandum, two viable options are proposed for directing Tract 21 runoff downstream. These options are discussed in detail below. A culvert sizing analysis has been performed for flows passing under the Pima/Princess Drive extension. This preliminary analysis indicates the entire 239 cfs can pass through a 6' X 8' CBC. The calculations supporting this sizing can be found in Appendix B at the end of this memorandum.

OPTION 1 - Splitting the flow between the Perimeter Center & the Freeway

This option involves directing a portion of the Tract 21 peak flow in a culvert pipe west under the Pima Freeway and into the existing open drainage system for the Scottsdale Perimeter Center. Pentacore performed a capacity study of the Scottsdale Perimeter Center drainage system based on the 1989 Collar, Williams, & White report to evaluate the potential magnitude of flow which may be routed to this system. The following points summarize the study.

1. The network of drainage channels and culverts are sized for the 100-yr/1-hr storm and all channels are designed with one foot of freeboard.

2. All components of the drainage system are designed for on-site runoff only.

3. "Culvert 4" in the Collar, Williams, & White report is the first culvert structure downstream of the Pima Freeway. This double 6'X 3' CBC is designed to carry 306 cfs with approximately 1.1' of freeboard below the adjacent roadway.

4. A culvert analysis performed by Pentacore for 400 cfs (94 cfs greater than the design flow) indicates a rise in headwater such that the channel is surcharged and the immediate surrounding area including the roadway is inundated by approximately 0.7 feet.

5. In order to prevent overtopping the channel banks and inundating the roadway, the flow directed to the Perimeter Center would be restricted to less than 94 cfs.

6. Increasing the size of "Culvert 4" would allow more flow to pass downstream, however each successive downstream culvert structure would, in turn, have to be enlarged. A more extensive analysis would be required to determine the degree of increase in size required for each specific culvert crossing. Results of the analysis for "culvert 4" are shown in Appendix B.

Advantages:

- Less concentrated flow; easier to convey along freeway
- Possibly reduces amount of R.O.W. required along freeway

Disadvantages:

- Additional maintenance required for flow splitting structure.
- Two culvert crossings required; 1 under freeway & 1 under Pima/Princess.
- Flows greater than 94 cfs may surcharge the Perimeter Center system at the first culvert crossing.

OPTION 2 - Directing the entire peak flow south along the Pima Freeway.

This option proposes directing the entire 239 cfs generated in Tract 21 along the east side of the Pima Freeway. Approximately the first 1000′ of channel south of Pima/Princess Drive is located on property owned and maintained by ADOT. Therefore, ADOT will be responsible for constructing the facilities necessary for conveying the Tract 21 flows within their property. South of the ADOT property to Bell Road, the proposed freeway grader ditch will have to be enlarged to accommodate the flows from Tract 21. Preliminary channel sizing calculations are shown in Appendix B. Given the existing topography along this alignment, both the channel in ADOT property and the channel south of the ADOT property to Bell road are recommended to be lined. This recommendation is based on the steep slopes producing velocities exceeding the maximum velocities recommended for naturally lined channels. In fact, slopes are steep enough in the channel reach from the ADOT property to Bell Road (approximately 1.3%) to require channel lining along the freeway regardless of the discharge from Tract 21.

Advantages:

- Entire flow contained within one system - less complicated maintenance.
- Avoid surcharging the drainage system for the Perimeter Center.
- Eliminate a culvert crossing under the freeway.

Disadvantages:

- Addendum to construction contract required to change channel configuration along freeway.

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APPENDIX A

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1****************************************                   ***************************************
 *                                      *                   *                                     *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *                   *   U.S. ARMY CORPS OF ENGINEERS      *
 *            SEPTEMBER 1990            *                   *   HYDROLOGIC ENGINEERING CENTER     *
 *             VERSION 4.0              *                   *        609 SECOND STREET            *
 *                                      *                   *     DAVIS, CALIFORNIA 95616         *
 * RUN DATE  04/07/1998  TIME  14:09:03 *                   *        (916) 756-1104               *
 *                                      *                   *                                     *
****************************************                    ***************************************
```

```
                    X     X  XXXXXXX   XXXXX           X
                    X     X  X        X     X         XX
                    X     X  X        X                X
                    XXXXXXX  XXXX     X        XXXXX   X
                    X     X  X        X                X
                    X     X  X        X     X          X
                    X     X  XXXXXXX   XXXXX          XXX
```

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THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL    LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM
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1                                          HEC-1 INPUT                                              PAGE  1

         LINE            ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

           1             ID        PENTACORE ARIZONA                   file T21.dat     4/07/98     CJH
           2             ID
           3             ID
           4             ID        TRACT 21 HEC-1 MODEL
           5             ID
           6             ID        DRAINAGE FROM TRACT 21 OUTLETTING TO THE PIMA FREEWAY/PRINCESS
           7             ID        BLVD. INTERSECTION.
           8             ID
                         *DIAGRAM
           9             IT       5                     300
          10             IO       3

          11             KK    T21E   BASIN
          12             KM         RUNOFF FROM TRACT 21-EAST
          13             BA   .0660
          14             PH                       .84    1.53    2.46    2.75    2.94    3.31    3.74    4.17
          15             LS              74
          16             UK     300   .0180    .090     100      17
          17             RK    2475   .0180    .035             TRAP       8       2

          18             KK    T21W   BASIN
          19             KM         RUNOFF FROM TRACT 21-WEST
          20             BA   .0730
          21             LS              74
          22             UK     300   .0175    .090     100      17
          23             RK    1882   .0128    .035             TRAP       8       2
          24             RK     990   .0030    .025             TRAP       8       2

          25             KK     T21
          26             KM         COMBINE T21E AND T21W
          27             HC       2
          28             ZZ
1
              SCHEMATIC DIAGRAM OF STREAM NETWORK
 INPUT
  LINE      (V) ROUTING          (--->) DIVERSION OR PUMP FLOW

   NO.      (.) CONNECTOR        (<---) RETURN OF DIVERTED OR PUMPED FLOW

   11          T21E
                .
                .
   18           .      T21W
                .        .
                .        .
   25          T21.........

 (***) RUNOFF ALSO COMPUTED AT THIS LOCATION
1****************************************                   ***************************************
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*                                         *                                          *                                     *
*  FLOOD HYDROGRAPH PACKAGE  (HEC-1)      *                                          *  U.S. ARMY CORPS OF ENGINEERS       *
*          SEPTEMBER 1990                 *                                          *  HYDROLOGIC ENGINEERING CENTER      *
*            VERSION 4.0                  *                                          *       609 SECOND STREET             *
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* RUN DATE  04/07/1998  TIME  14:09:03    *                                          *        (916) 756-1104               *
*                                         *                                          *                                     *
*******************************************                                          ***************************************



                              PENTACORE ARIZONA                   file T21.dat     4/07/98    CJH

                              TRACT 21 HEC-1 MODEL

                              DRAINAGE FROM TRACT 21 OUTLETTING TO THE PIMA FREEWAY/PRINCESS
                              BLVD. INTERSECTION.

 10 IO          OUTPUT CONTROL VARIABLES
                      IPRNT          3  PRINT CONTROL
                      IPLOT          0  PLOT CONTROL
                      QSCAL         0.  HYDROGRAPH PLOT SCALE

    IT          HYDROGRAPH TIME DATA
                       NMIN          5  MINUTES IN COMPUTATION INTERVAL
                      IDATE     1    0  STARTING DATE
                      ITIME       0000  STARTING TIME
                         NQ        300  NUMBER OF HYDROGRAPH ORDINATES
                     NDDATE     2    0  ENDING DATE
                     NDTIME       0055  ENDING TIME
                      ICENT         19  CENTURY MARK

                  COMPUTATION INTERVAL      .08 HOURS
                       TOTAL TIME BASE    24.92 HOURS

       ENGLISH UNITS
            DRAINAGE AREA           SQUARE MILES
            PRECIPITATION DEPTH     INCHES
            LENGTH, ELEVATION       FEET
            FLOW                    CUBIC FEET PER SECOND
            STORAGE VOLUME          ACRE-FEET
            SURFACE AREA            ACRES
            TEMPERATURE             DEGREES FAHRENHEIT


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


           **************
           *            *
 11 KK     *    T21E    *       BASIN
           *            *
           **************
                                RUNOFF FROM TRACT 21-EAST

            SUBBASIN RUNOFF DATA

 13 BA          SUBBASIN CHARACTERISTICS
                      TAREA        .07  SUBBASIN AREA

                PRECIPITATION DATA

 14 PH                                  DEPTHS FOR   0-PERCENT HYPOTHETICAL STORM
                ..... HYDRO-35 ......   ............... TP-40 ...............    ........... TP-49 ...........
                5-MIN  15-MIN  60-MIN    2-HR    3-HR    6-HR   12-HR   24-HR   2-DAY   4-DAY   7-DAY  10-DAY
                  .84    1.53    2.46    2.75    2.94    3.31    3.74    4.17     .00     .00     .00     .00

                                                    STORM AREA =       .07

 15 LS          SCS LOSS RATE
                      STRTL        .70  INITIAL ABSTRACTION
                     CRVNBR      74.00  CURVE NUMBER
                      RTIMP        .00  PERCENT IMPERVIOUS AREA

                KINEMATIC WAVE
 16 UK            OVERLAND-FLOW ELEMENT NO. 1
                          L       300.  OVERLAND FLOW LENGTH
                          S      .0180  SLOPE
                          N       .090  ROUGHNESS COEFFICIENT
                         PA      100.0  PERCENT OF SUBBASIN
                      DXMIN         17  MINIMUM NUMBER OF DX INTERVALS

                KINEMATIC WAVE
 17 RK            MAIN CHANNEL
                          L      2475.  CHANNEL LENGTH
```

```
                    S         .0180  SLOPE
                    N          .035  CHANNEL ROUGHNESS COEFFICIENT
                   CA           .07  CONTRIBUTING AREA
                SHAPE          TRAP  CHANNEL SHAPE
                   WD          8.00  BOTTOM WIDTH OR DIAMETER
                    Z          2.00  SIDE SLOPE
              NDXMIN              2  MINIMUM NUMBER OF DX INTERVALS
              RUPSTQ             NO  ROUTE UPSTREAM HYDROGRAPH

                                                    ***
                              COMPUTED KINEMATIC PARAMETERS
                                   VARIABLE TIME STEP
                                  (DT SHOWN IS A MINIMUM)

            ELEMENT   ALPHA       M         DT        DX      PEAK    TIME TO    VOLUME   MAXIMUM
                                                                       PEAK               CELERITY
                                          (MIN)      (FT)    (CFS)    (MIN)       (IN)     (FPS)

          PLANE1       2.22      1.67        .53     17.65   132.04    728.94     1.71       .59
          MAIN         1.74      1.42       1.56    825.00   125.82    731.82     1.70      9.65

CONTINUITY SUMMARY (AC-FT) - INFLOW= .0000E+00  EXCESS= .6061E+01 OUTFLOW= .6000E+01 BASIN STORAGE= .4753E-01 PERCENT ERROR=    .2

                                 INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL

          MAIN         1.74      1.42       5.00             119.75    730.00     1.70

     ***           ***            ***           ***            ***

                    HYDROGRAPH AT STATION     T21E

   TOTAL RAINFALL =     4.17, TOTAL LOSS =     2.45, TOTAL EXCESS =     1.72

  PEAK FLOW      TIME                         MAXIMUM AVERAGE FLOW
                                      6-HR        24-HR        72-HR      24.92-HR
+   (CFS)        (HR)
                             (CFS)
+     120.      12.17                  11.           3.           3.           3.
                           (INCHES)  1.508        1.704        1.704        1.704
                            (AC-FT)     5.           6.           6.           6.

                       CUMULATIVE AREA =       .07 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


              **************
              *            *
  18 KK       *    T21W    *        BASIN
              *            *
              **************
                                   RUNOFF FROM TRACT 21-WEST

                 SUBBASIN RUNOFF DATA

  20 BA            SUBBASIN CHARACTERISTICS
                         TAREA          .07  SUBBASIN AREA

                   PRECIPITATION DATA

  14 PH                                        DEPTHS FOR    0-PERCENT HYPOTHETICAL STORM
                         ..... HYDRO-35 ......    ............... TP-40 ...............   ........... TP-49 ...........
                   5-MIN  15-MIN  60-MIN    2-HR     3-HR     6-HR    12-HR    24-HR    2-DAY    4-DAY    7-DAY   10-DAY
                     .84    1.53    2.46    2.75     2.94     3.31     3.74     4.17      .00      .00      .00      .00

                                                          STORM AREA =       .07

  21 LS            SCS LOSS RATE
                         STRTL          .70  INITIAL ABSTRACTION
                        CRVNBR        74.00  CURVE NUMBER
                         RTIMP          .00  PERCENT IMPERVIOUS AREA

                   KINEMATIC WAVE
  22 UK              OVERLAND-FLOW ELEMENT NO. 1
                             L         300.  OVERLAND FLOW LENGTH
                             S        .0175  SLOPE
                             N         .090  ROUGHNESS COEFFICIENT
                            PA        100.0  PERCENT OF SUBBASIN
                         DXMIN           17  MINIMUM NUMBER OF DX INTERVALS

                   KINEMATIC WAVE
  23 RK              COLLECTOR CHANNEL
                             L        1882.  CHANNEL LENGTH
                             S        .0128  SLOPE
```

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                       N          .035  CHANNEL ROUGHNESS COEFFICIENT
                      CA           .00  CONTRIBUTING AREA
                   SHAPE          TRAP  CHANNEL SHAPE
                      WD          8.00  BOTTOM WIDTH OR DIAMETER
                       Z          2.00  SIDE SLOPE
                  NDXMIN             2  MINIMUM NUMBER OF DX INTERVALS
   24 RK        MAIN CHANNEL
                       L          990.  CHANNEL LENGTH
                       S         .0030  SLOPE
                       N          .025  CHANNEL ROUGHNESS COEFFICIENT
                      CA           .07  CONTRIBUTING AREA
                   SHAPE          TRAP  CHANNEL SHAPE
                      WD          8.00  BOTTOM WIDTH OR DIAMETER
                       Z          2.00  SIDE SLOPE
                  NDXMIN             2  MINIMUM NUMBER OF DX INTERVALS
                  RUPSTQ            NO  ROUTE UPSTREAM HYDROGRAPH

                                                      ***
                                    COMPUTED KINEMATIC PARAMETERS
                                          VARIABLE TIME STEP
                                        (DT SHOWN IS A MINIMUM)

              ELEMENT    ALPHA       M        DT        DX       PEAK    TIME TO    VOLUME    MAXIMUM
                                                                          PEAK                CELERITY
                                             (MIN)     (FT)     (CFS)     (MIN)      (IN)      (FPS)

              PLANE1      2.19     1.67       .52     17.65    145.31    729.21      1.71       .58
              COLLECTOR1  1.47     1.42      1.33    627.33    140.80    731.37      1.71      8.90
              MAIN        1.00     1.42       .82    330.00    137.26    733.00      1.70      6.73

 CONTINUITY SUMMARY (AC-FT) - INFLOW= .0000E+00  EXCESS= .6703E+01 OUTFLOW= .6635E+01 BASIN STORAGE= .5955E-01 PERCENT ERROR=    .1

                                       INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL

              MAIN        1.00     1.42      5.00              128.73    735.00      1.70

      ***          ***          ***          ***          ***

                         HYDROGRAPH AT STATION      T21W

    TOTAL RAINFALL =    4.17, TOTAL LOSS =    2.45, TOTAL EXCESS =    1.72

   PEAK FLOW     TIME                         MAXIMUM AVERAGE FLOW
                                    6-HR        24-HR       72-HR     24.92-HR
 +   (CFS)      (HR)
                          (CFS)
 +     129.     12.25                 12.          3.          3.          3.
                        (INCHES)    1.508       1.705       1.705       1.705
                         (AC-FT)       6.          7.          7.          7.

                        CUMULATIVE AREA =      .07 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

           **************
           *            *
   25 KK   *      T21   *
           *            *
           **************
                               COMBINE T21E AND T21W

   27 HC        HYDROGRAPH COMBINATION
                   ICOMP             2  NUMBER OF HYDROGRAPHS TO COMBINE
                                                      ***

      ***          ***          ***          ***          ***

                         HYDROGRAPH AT STATION       T21

   PEAK FLOW     TIME                         MAXIMUM AVERAGE FLOW
                                    6-HR        24-HR       72-HR     24.92-HR
 +   (CFS)      (HR)
                          (CFS)
 +     239.     12.17                 23.          6.          6.          6.
                        (INCHES)    1.508       1.705       1.705       1.705
                         (AC-FT)      11.         13.         13.         13.

                        CUMULATIVE AREA =      .14 SQ MI
1
                                                   RUNOFF SUMMARY
                                            FLOW IN CUBIC FEET PER SECOND
```

TIME IN HOURS, AREA IN SQUARE MILES

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
+										
+	HYDROGRAPH AT	T21E	120.	12.17	11.	3.	3.	.07		
+	HYDROGRAPH AT	T21W	129.	12.25	12.	3.	3.	.07		
+	2 COMBINED AT	T21	239.	12.17	23.	6.	6.	.14		

1

SUMMARY OF KINEMATIC WAVE - MUSKINGUM-CUNGE ROUTING
(FLOW IS DIRECT RUNOFF WITHOUT BASE FLOW)

						INTERPOLATED TO COMPUTATION INTERVAL			
ISTAQ	ELEMENT	DT	PEAK	TIME TO PEAK	VOLUME	DT	PEAK	TIME TO PEAK	VOLUME
		(MIN)	(CFS)	(MIN)	(IN)	(MIN)	(CFS)	(MIN)	(IN)
T21E	MANE	1.56	125.82	731.82	1.70	5.00	119.75	730.00	1.70

CONTINUITY SUMMARY (AC-FT) - INFLOW= .0000E+00 EXCESS= .6061E+01 OUTFLOW= .6000E+01 BASIN STORAGE= .4753E-01 PERCENT ERROR= .2

T21W	MANE	.82	137.26	733.00	1.70	5.00	128.73	735.00	1.70

CONTINUITY SUMMARY (AC-FT) - INFLOW= .0000E+00 EXCESS= .6703E+01 OUTFLOW= .6635E+01 BASIN STORAGE= .5955E-01 PERCENT ERROR= .1

*** NORMAL END OF HEC-1 ***

3.4.4 Element Application

(1) **Overland Flow.** The overland flow element is a wide rectangular channel of unit width; so, referring to Figure 3.6, $\alpha = 1.486S^{\frac{1}{2}}/N$ and $m = 5/3$. Notice that Manning's n has been replaced by an overland flow roughness factor, N. Typical values of N are shown in Table 3.5. When applying Equations (3.43) and (3.46) to an overland flow element, the lateral inflow is rainfall excess (previously computed using methods described in Section 3.2) and the outflow is a flow per unit width.

An overland flow element is described by four parameters: a typical overland flow length, L, slope and roughness factor which are used to compute α, and the percent of the subbasin area represented by this element.

Two overland flow elements may be used for each subbasin. The total discharge, Q, from each element is computed as

$$Q = q * \frac{\text{AREA}}{L} \qquad (3.70)$$

Table 3.5

Resistance Factor for Overland Flow

Surface	**N value**	**Source**
Asphalt/Concrete*	0.05 - 0.15	a
Bare Packed Soil Free of Stone	0.10	c
Fallow - No Residue	0.008 - 0.012	b
Convential Tillage - No Residue	0.06 - 0.12	b
Convential Tillage - With Residue	0.16 - 0.22	b
Chisel Plow - No Residue	0.06 - 0.12	b
Chisel Plow - With Residue	0.10 - 0.16	b
Fall Disking - With Residue	0.30 - 0.50	b
No Till - No Residue	0.04 - 0.10	b
No Till (20-40 percent residue cover)	0.07 - 0.17	b
No Till (60-100 percent residue cover)	0.17 - 0.47	b
Sparse Rangeland with Debris:		
0 Percent Cover	0.09 - 0.34	b
20 Percent Cover	0.05 - 0.25	b
Sparse Vegetation	0.053 - 0.13 0.09	f
Short Grass Prairie	0.10 - 0.20	f
Poor Grass Cover On Moderately Rough	0.30	c
Bare Surface		
Light Turf	0.20	a
Average Grass Cover	0.4	c
Dense Turf	0.17 - 0.80	a,c,e,f
Dense Grass	0.17 - 0.30	d
Bermuda Grass	0.30 - 0.48	d
Dense Shrubbery and Forest Litter	0.4	a

Legend: a) Harley (1975), b) Engman (1986), c) Hathaway (1945), d) Palmer (1946), e) Ragan and Duru (1972), f) Woolhiser (1975). (See Hjemfelt, 1986)

*Asphalt/Concrete n value for open channel flow 0.01 - 0.016

Table 5.11
Manning's Roughness Coefficients*

Channel Material	Roughness Coefficient (n)		
	Minimum	Normal	Maximum
Corrugated metal	0.021	0.025	0.030
Concrete			
Trowel finish	0.011	0.013	0.015
Float finish	0.013	0.015	0.016
Unfinished	0.014	0.017	0.020
Shotcrete, good section	0.016	0.019	0.023
Shotcrete, wavy section	0.018	0.022	0.025
Asphalt (use maximum value when cars are present)	0.013	0.016	0.020
Soil Cement	0.018	0.020	0.025
Constructed channels with earth or sand bottom and sides of:			
Clean earth; straight	0.018	0.022	0.025
Earth with grass and weeds	0.020	0.025	0.030
Earth with trees and shrubs	0.024	0.032	0.040
Shotcrete	0.018	0.022	0.025
Soil Cement	0.022	0.025	0.028
Concrete	0.017	0.020	0.024
Dry rubble or riprap	0.023	0.032	0.036
Natural channels with sand bottom and sides of:			
Trees and shrubs	0.025	0.035	0.045
Rock	0.024	0.032	0.040
Natural channel with rock bottom	0.040	0.060	0.090
Overbank Floodplains			
Desert brush, normal density	0.040	0.060	0.080
Dense vegetation	0.070	0.100 0.09	0.160

* From: Simons, Li and Associates 1988. Adapted from Chow (1959) and Aldridge and Garret (1973).

APPENDIX B

TRACT 21 HYDRAULIC CALCULATIONS

UNLINED

Segment A
CHANNEL DESIGN WITH 2:1 AND 2:1 SIDESLOPES n= 0.025 natural

Bottom Width	Flow Depth	Flow Area	Wetted Perimeter	Hydraulic Radius	Channel Slope	Channel Capacity	Flow Velocity	Froude Number
4.00	1.46	10.1	10.5	0.96	0.01000	58.44	5.78	0.84

Segment B
CHANNEL DESIGN WITH 2:1 AND 2:1 SIDESLOPES n= 0.025 natural

Bottom Width	Flow Depth	Flow Area	Wetted Perimeter	Hydraulic Radius	Channel Slope	Channel Capacity	Flow Velocity	Froude Number
5.00	2.29	21.9	15.2	1.44	0.00570	125.20	5.71	0.66

LINED

Segment C
CHANNEL DESIGN WITH 2:1 AND 2:1 SIDESLOPES n= 0.021 shotcrete

Bottom Width	Flow Depth	Flow Area	Wetted Perimeter	Hydraulic Radius	Channel Slope	Channel Capacity	Flow Velocity	Froude Number
5.00	1.53	12.3	11.8	1.04	0.01970	125.80	10.20	1.45

Segment D
CHANNEL DESIGN WITH 2:1 AND 2:1 SIDESLOPES n= 0.021 shotcrete

Bottom Width	Flow Depth	Flow Area	Wetted Perimeter	Hydraulic Radius	Channel Slope	Channel Capacity	Flow Velocity	Froude Number
5.00	2.49	24.9	16.1	1.54	0.01050	239.61	9.64	1.08

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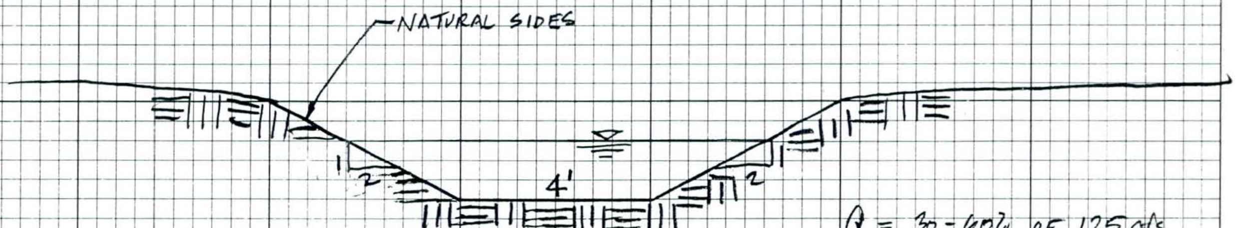
Job No. 5001.0007 Sheet ____ of ____

Project C.O.S. – OUTER LOOP DRAINAGE

Subject TRACT 21 DRAINAGE

Designed By CJA Date 4/2/98 Checked By ____ Date ____

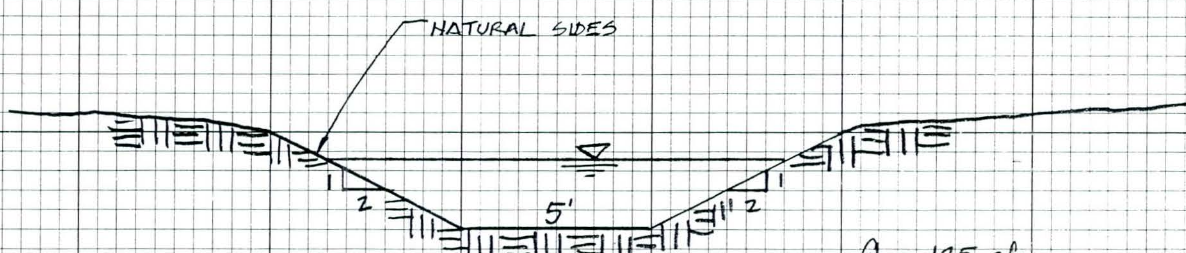
SEGMENT (A)



S = 1.0%
n = 0.025

Q = 30 - 60% OF 125 cfs
37.5 – 75 cfs

SEGMENT (B)



Q = 125 cfs

S = 0.57%
n = 0.021

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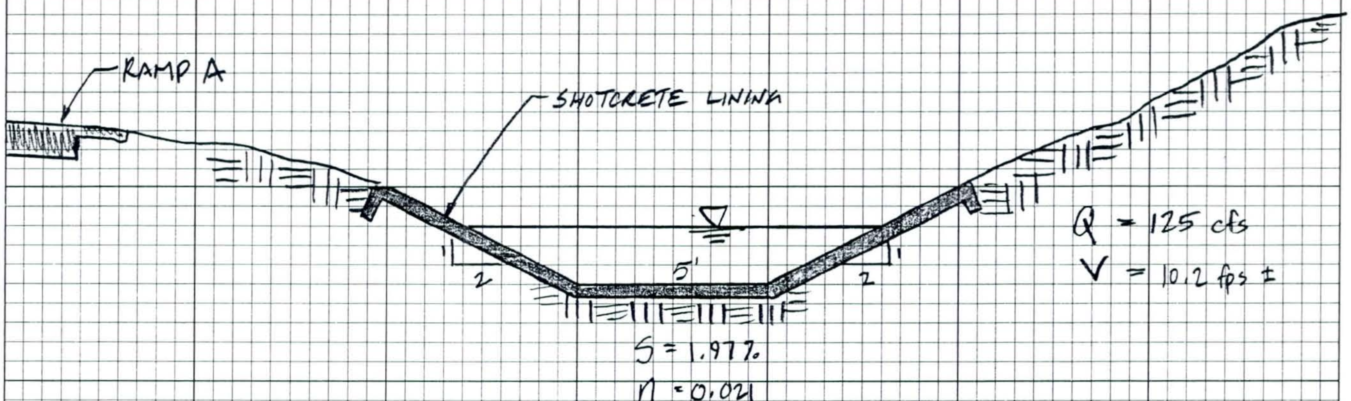
Job No. 5001.0007 Sheet ____ of ____

Project C.O.S. - OUTER LOOP DRAINAGE

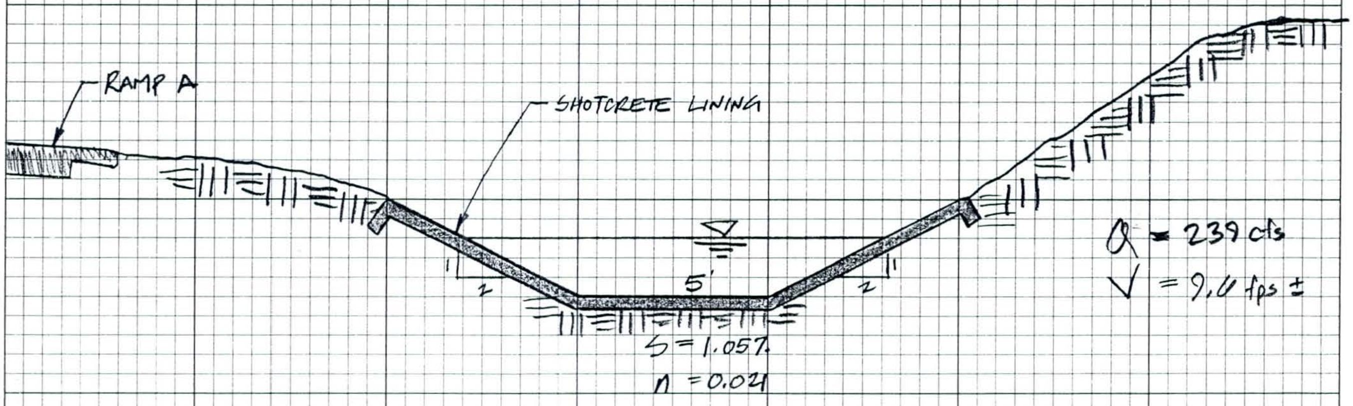
Subject TRACT 21 DRAINAGE

Designed By CJA Date 1/2/98 Checked By ________ Date ________

SEGMENT (C)



SEGMENT (D)



PROJECT: O.L.D. – TRACT 21

DESIGNER: CJH – PENTACORE

DATE: ____________

HYDROLOGIC AND CHANNEL INFORMATION

CULVERT 4 FROM C.W.W: PERIMETER CENTER DRAINAGE REPORT

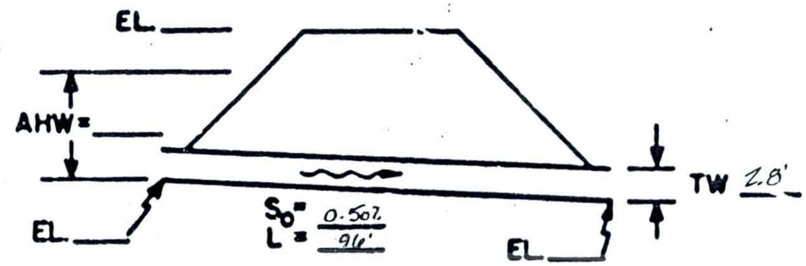
Q_1 = ______ TW_1 = 2.8'

Q_2 = ______ TW_2 = ______

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION : ____________



MEAN STREAM VELOCITY = ______

MAX. STREAM VELOCITY = ______

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q	SIZE	HEADWATER COMPUTATION: INLET CONT. $\frac{HW}{D}$	HW	OUTLET CONTROL $HW = H + h_o - LS_o$: K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW	CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
TRIAL (1) (2) 6'x3' WINWALLS & ROUNDED EDGES	400	(2) 6'x3'	2.21	6.63	0.2	2.6	*	*	2.8	2.8	0.48	4.92	6.63		—	

SUMMARY & RECOMMENDATIONS: L = 75'

$d_c = 0.315\sqrt[3]{(Q/B)^2}$

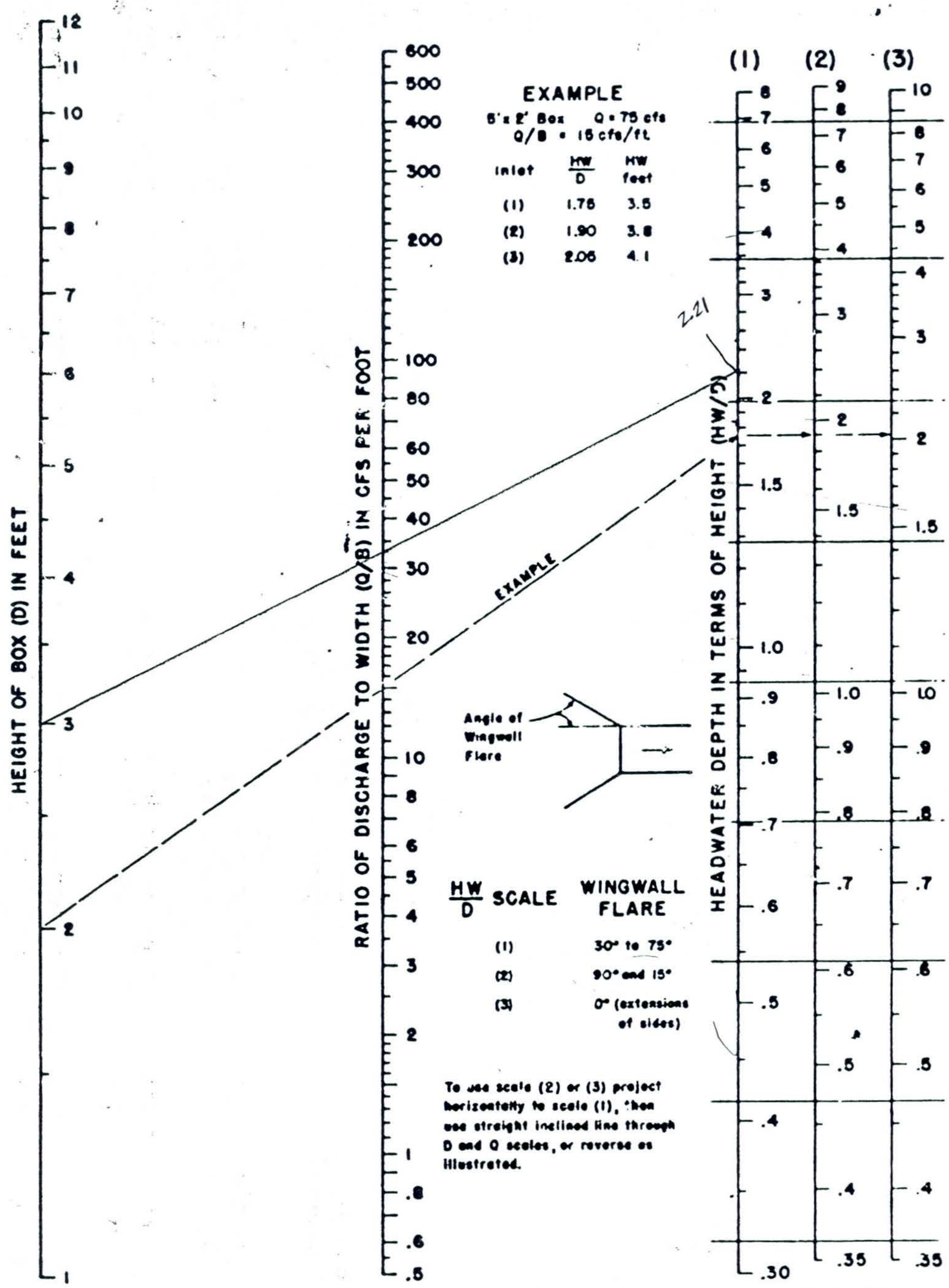
$= 0.315\sqrt{(400/12)^2} = 3.19'$ ⇐ * d_c CANNOT EXCEED D = 3' ∴ USE h_o = TW = 2.8'

SUMMARY: 400 cfs PRODUCES AN INLET CONTROL CONDITION WITH A HEADWATER WHICH EXCEEDS THE DEPTH OF THE CHANNEL.

5-18

Figure 7

CHART I



HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

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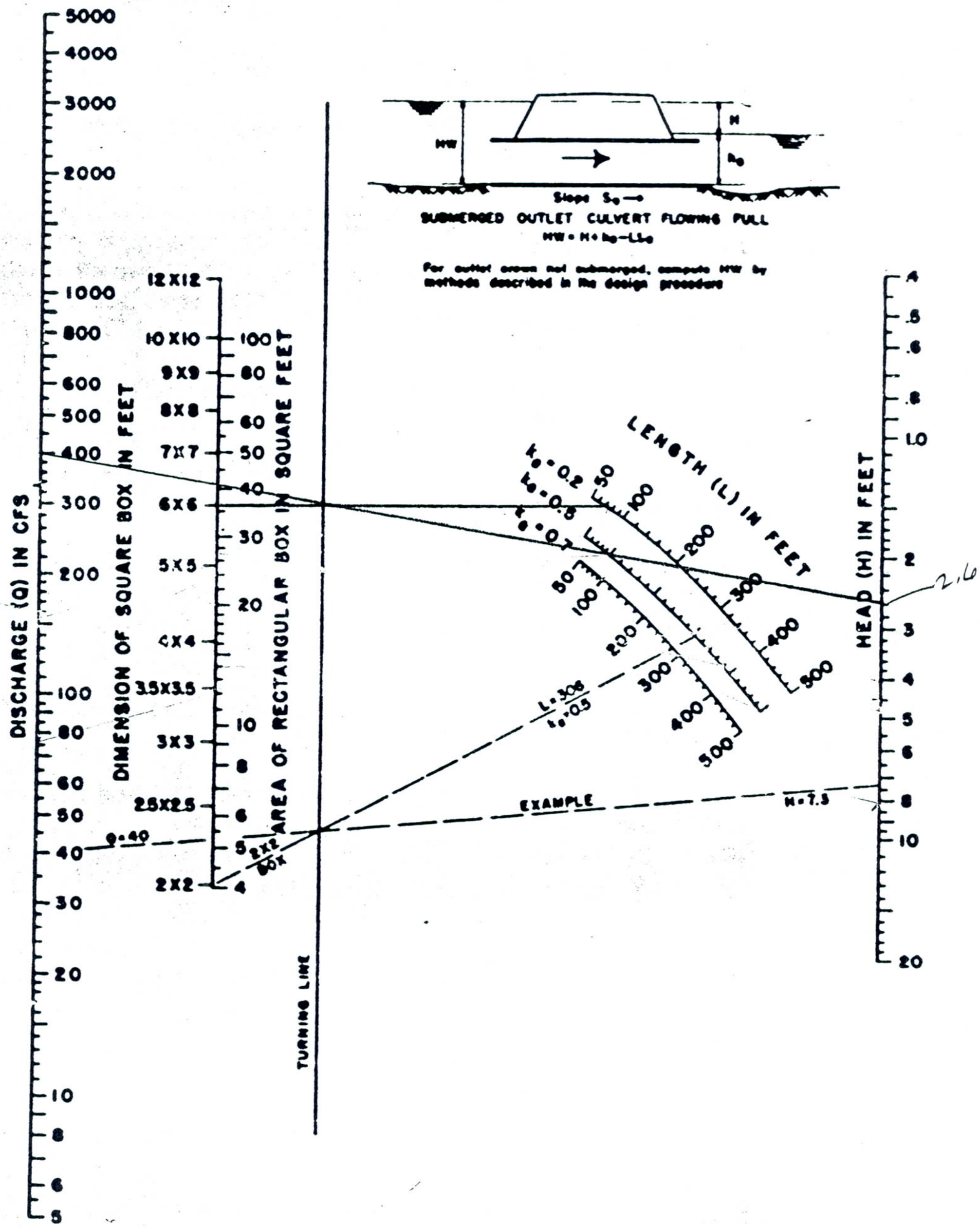
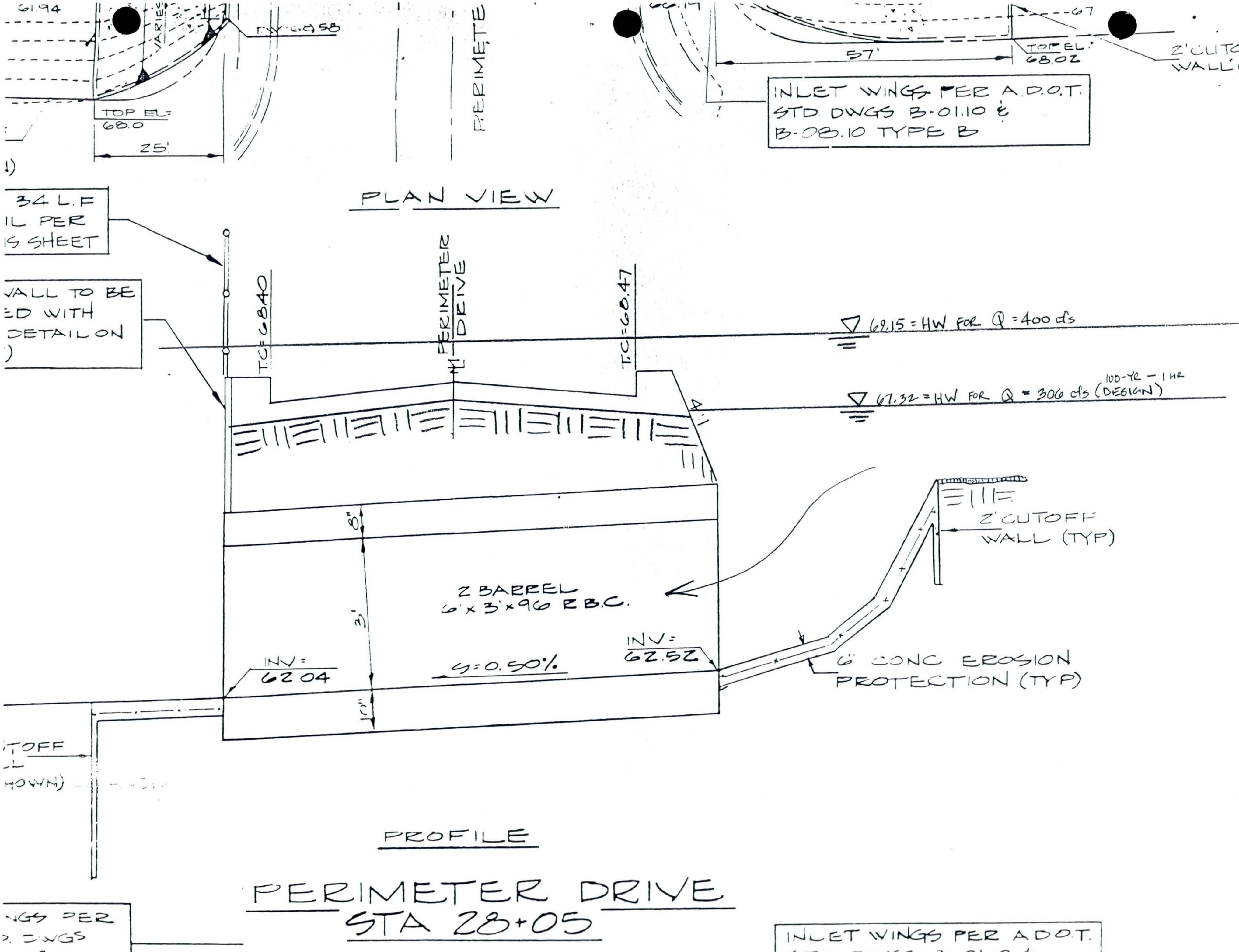
CHART 8



HEAD FOR
CONCRETE BOX CULVERTS
FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963



PERIMETER DRIVE STA 28+05

PROJECT: 5001.0007

DESIGNER: CJA

DATE: 4/

HYDROLOGIC AND CHANNEL INFORMATION

Q_1 = 239 cfs

Q_2 =

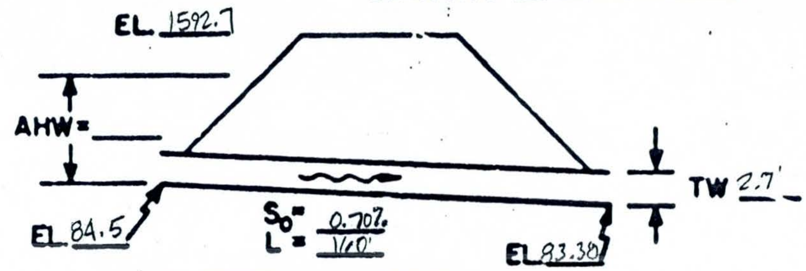
TW_1 = 2.7'

TW_2 =

(Q_1 = DESIGN DISCHARGE, SAY Q_{25}
Q_2 = CHECK DISCHARGE, SAY Q_{50} OR Q_{100})

SKETCH

STATION: PRINCESS / PIMA X-ING

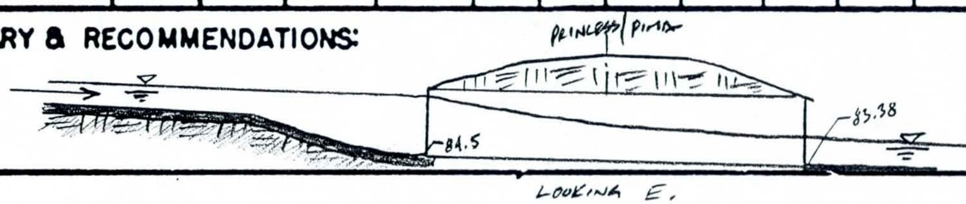


MEAN STREAM VELOCITY =

MAX. STREAM VELOCITY =

CULVERT DESCRIPTION (ENTRANCE TYPE)	Q (cfs)	SIZE	HEADWATER COMPUTATION: INLET CONT. $\frac{HW}{D}$	INLET CONT. HW	OUTLET CONTROL $HW = H + h_o - LS_o$: K_e	H	d_c	$\frac{d_c+D}{2}$	TW	h_o	LS_o	HW	CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
CBC w/ 45° WING-WALLS	239	6'x8'	0.95	5.7	0.2	0.6	2.96	4.48	2.7	4.48	1.12	3.96	5.7			

SUMMARY & RECOMMENDATIONS:



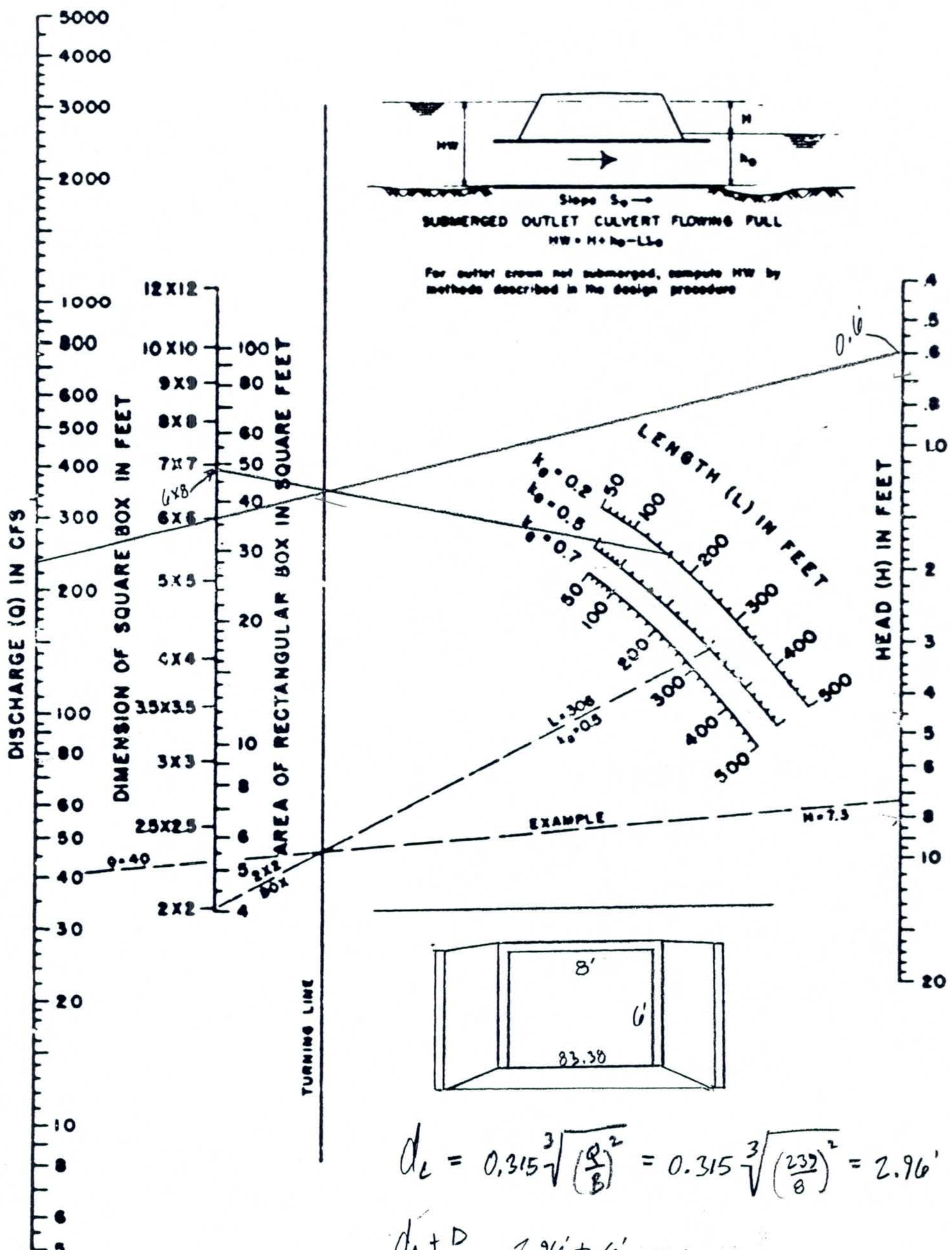
LOOKING E.

USE 6'X8' CBC w/ 45° WINGWALLS TO PASS 100-YR FLOWS UNDER PRINCESS / PIMA

5-18

Figure 7

CHART 8



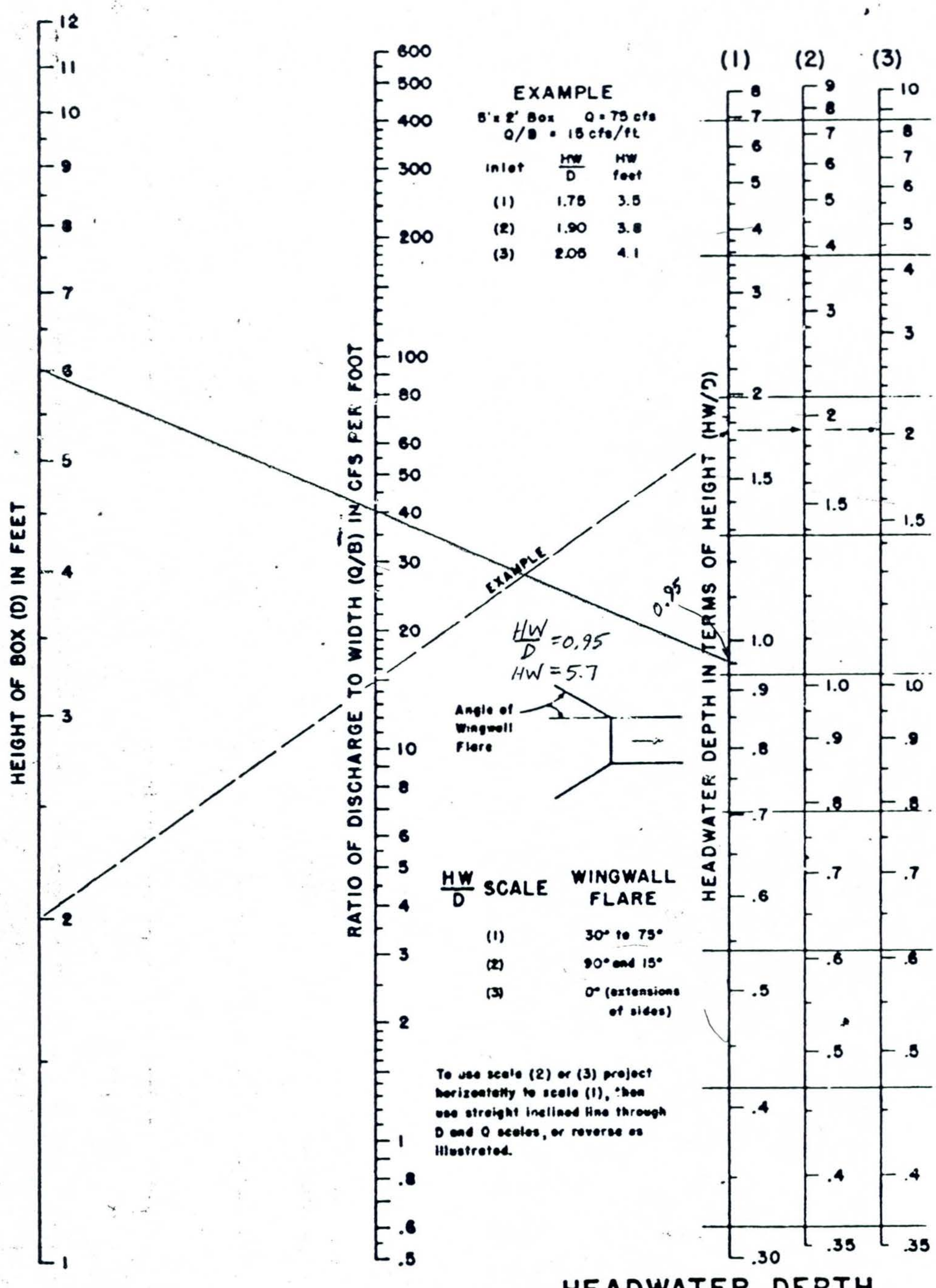
$$d_c = 0.315\sqrt[3]{\left(\frac{Q}{B}\right)^2} = 0.315\sqrt[3]{\left(\frac{239}{8}\right)^2} = 2.96'$$

$$\frac{d_c + D}{2} = \frac{2.96' + 6'}{2} = 4.48$$

HEAD FOR CONCRETE BOX CULVERTS FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

CHART I



HEADWATER DEPTH
FOR BOX CULVERTS
WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

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LINED

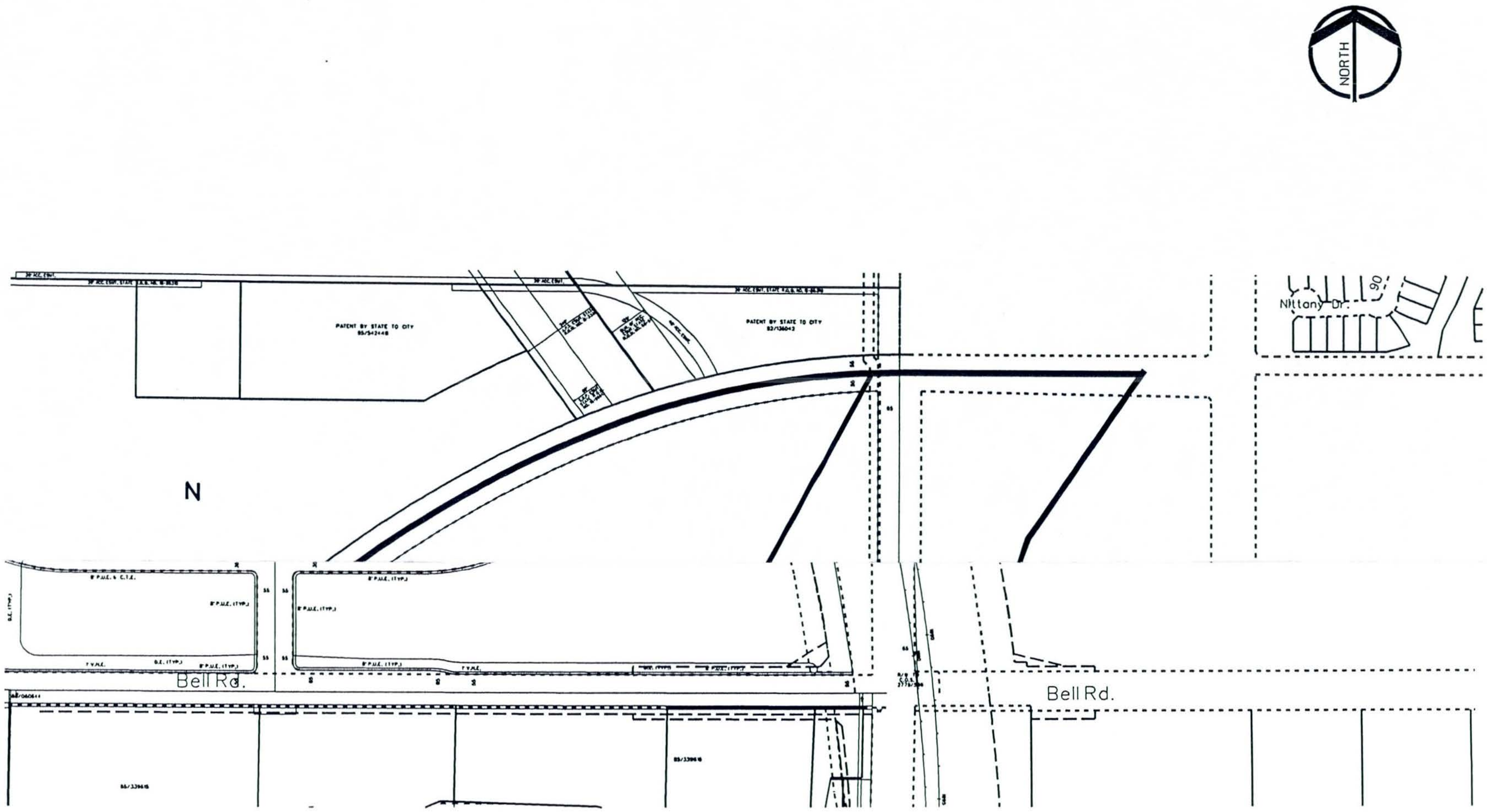
Segment A - Pima/Princess through ADOT Property Adjacent to Freeway
CHANNEL DESIGN WITH 2:1 AND 2:1 SIDESLOPES n= 0.021 shotcrete

Bottom Width	Flow Depth	Flow Area	Wetted Perimeter	Hydraulic Radius	Channel Slope	Channel Capacity	Flow Velocity	Froude Number
5.00	2.71	28.2	17.1	1.65	0.00740	239.17	8.47	0.91

LINED

Segment B - From ADOT Property to Bell Road Culverts
CHANNEL DESIGN WITH 2:1 AND 2:1 SIDESLOPES n= 0.021 shotcrete

Bottom Width	Flow Depth	Flow Area	Wetted Perimeter	Hydraulic Radius	Channel Slope	Channel Capacity	Flow Velocity	Froude Number
5.00	2.36	22.9	15.6	1.47	0.01300	239.17	10.43	1.20



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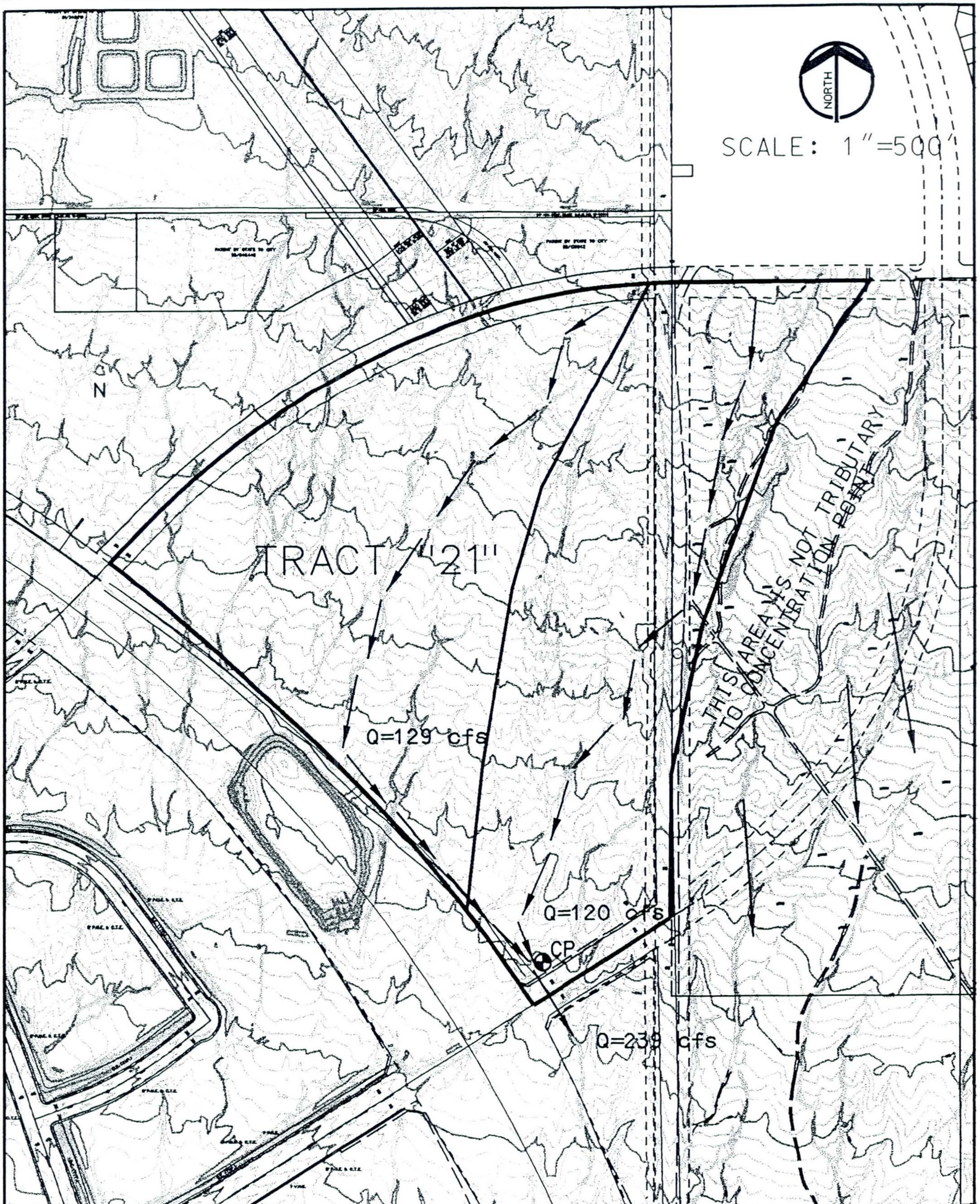
Land Surveying Planning ADA Consulting
2255 No. 44th St., SUITE 255 Phoenix, AZ 85008
TELEPHONE (602) 681-9272 FAX (602) 681-9339

EXHIBIT "A" - SITE MAP



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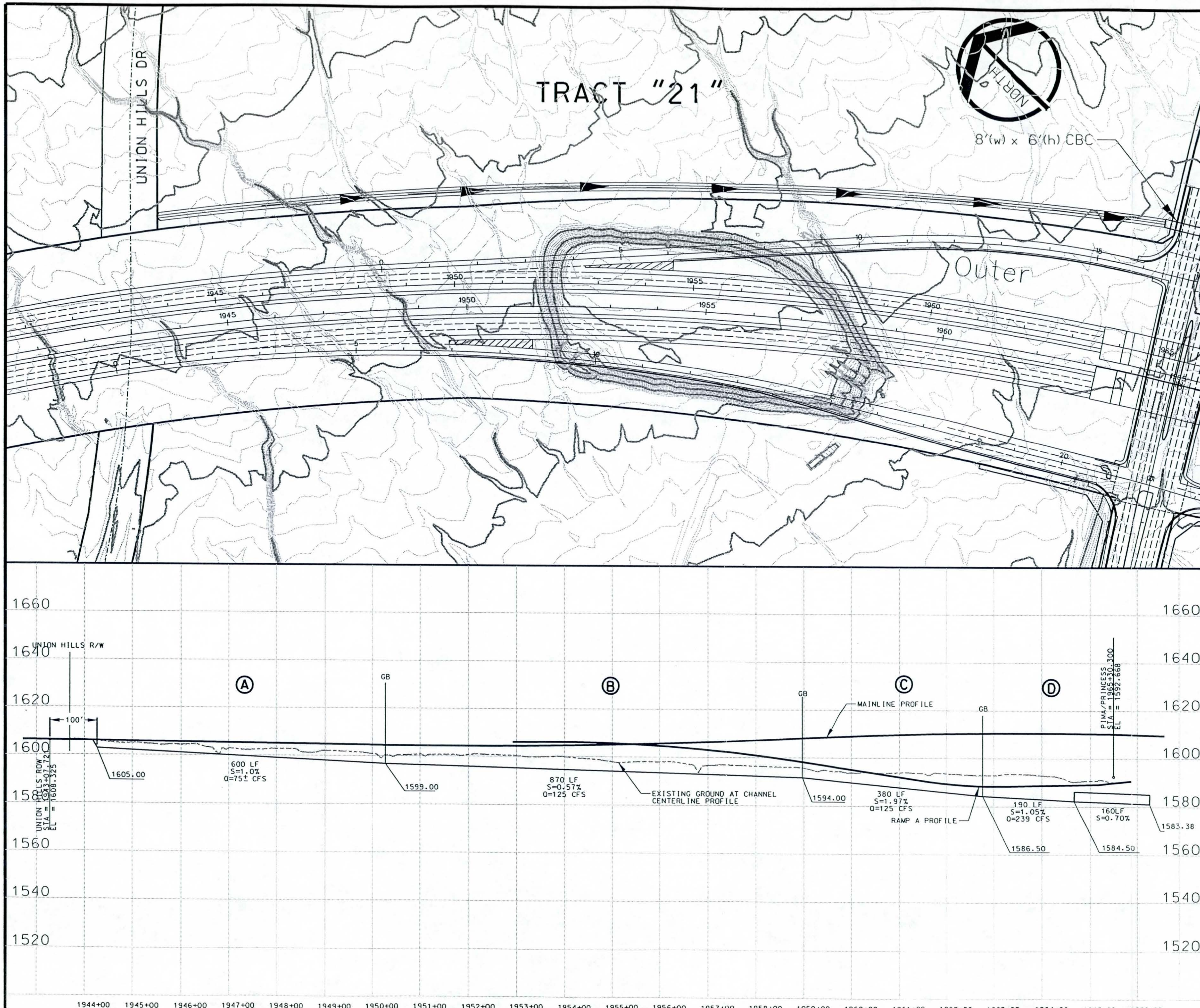
TELEPHONE (602) 681-9272 FAX (602) 681-9339

EXHIBIT "B" TRACT 21 HYDROLOGY

p:\0067\cadd\exhibts\tr21exbb.dgn Apr. 07, 1998 19:53:30



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2255 No. 44th St., SUITE 255 Phoenix, AZ 85008
TELEPHONE (602) 681-9272 FAX (602) 681-9339

REMOVAL NOTES

Description	Unit	Quan.

CONSTRUCTION NOTES:

EXHIBIT C

NOT FOR CONSTRUCTION

DATE	REVISION	BY

ENGINEER

TRANSPORTATION DEPARTMENT
TRANSPORTATION PLANNING
3939 CIVIC CENTER BLVD.
SCOTTSDALE, ARIZONA 85251

PROJECT TITLE: TRACT 21 CHANNEL PLAN AND PROFILE
10% CONCEPTUAL PHASE

SCALE HORIZ. 1"=200' VERT. 1"=20' | DESIGNED BY | DATE | DRAWN BY | AS-BUILT | BID NO. | PROJECT NO. 5001.0007 | SHT. OF

PENTACORE
ARIZONA

Civil Engineering
Construction Administration
Land Surveying
GPS Surveys
Planning
ADA Consulting

MEMORANDUM

Date: April 23, 1998 **Project No.:** 5001.0007

To: Doug Cullinane

Company: City of Scottsdale

From: Christopher Hassert, P.E.

Subject: **Pima Road Three Basins Project**
Hayden Road Storm Drain, 10% H&H Design Memorandum

Comments

PROJECT DESCRIPTION

This design memorandum focuses on the 10% design of the Hayden Road Storm Drain which serves as the outlet conduit for the Outer Loop Detention Basin. This system conveys the release outflows from the Outer Loop Basin to the Scottsdale TPC Desert Course. Based on discussions generated from the Value Engineering Analyses, the storm drain is designed to convey approximately 2000 cfs. This flow was provided by the City of Scottsdale as the upper limit of flow which can be accommodated by the TPC Desert Course without adverse effects to the course.

The modified HEC-1 hydrology model was used to develop the upstream invert elevation of the storm drain. As the hydrology model is further refined, the outlet invert elevation may change, which in turn will require adjustment of the storm drain profile.

The selected alignment and profile for the Hayden Road Storm Drain was based on existing and proposed utilities, available right-of-way, and existing topography. This alignment and profile is expected to vary as the hydrologic and hydraulic design is modified. Similarly, changes in utility information may precipitate changes in the design.

Pipe Material

An investigation performed by Dan Brauer concluded that Cast in Place (CIP) concrete pipe is suitable for portions of the Pima Road Storm Drain. Because of the similarities between the Pima Road system and the Hayden Road system, portions of the Hayden Road system are deemed suitable for CIP concrete pipe. The following assumptions are made for CIP concrete pipe.

2255 North 44th Street, Suite # 255 • Phoenix, Arizona 85008 • Tel. (602) 681-9272 • Fax (602) 681-9339

- CIP pipe is approximately 40% less expensive per linear foot for the pipe sizes, lengths, and depths anticipated for this storm drain system.
- CIP pipe can withstand internal pressures while operating at less than 15' of total head.
- A Manning's "n" value of 0.015 is assumed for pipe friction.
- CIP should not be used for the first segment exiting the Outer Loop Basin because of excessive head.

Selected Horizontal Alignment

The selected horizontal alignment of the Hayden Road Storm Drain predominantly runs within the east half of the Hayden Road right-of-way. Exhibit A shows the overall alignment of the storm drain. It is important to note that the alignment for the upstream portion of the storm drain shown is representative only. The actual location at which the pipe exits the basin will be designed by others, and the final outlet location and configuration has yet to be determined. The alignment of the downstream portion of the storm drain is expected to remain in the location shown in Exhibit A. However, an energy dissipator will be required at the downstream end of the system. This dissipator will be designed as the design process for the system progresses.

Plan & Profile

The plan and profile of the Hayden Road Storm Drain is shown in Exhibit A at the end of this memorandum. Attached notes reference specific parameters of the system such as pipe slope, grade breaks, existing ground elevation, and preliminary access manhole locations.

HYDROLOGY

The hydrology model developed for the Outer Loop Basin accounts for runoff generated by the area north of the proposed Pima Freeway and east of Scottsdale Road. The governing rainfall event for this model is the 100-yr, 24-hr storm. Modeling parameters accepted by the City of Scottsdale are used in the model including a maximum basin drain time of 36 hours which is an important function in sizing the Outer Loop Basin and corresponding outlet pipes.

HYDRAULICS

The hydraulics for the Hayden Road Storm Drain are governed by the maximum outflow proposed to be discharged from the Outer Loop Basin. Other factors such as existing utilities and existing topography also dictate pipe slopes and alignment which in turn affect the hydraulic modeling of the system. The upstream and downstream controls of the system are the basin headwater and the 100-year water surface elevation (tailwater) at the TPC golf course, respectively. These controls are anticipated to remain fixed throughout the design process, but may change as the project progresses. Specifically, the basin headwater may vary as the hydrology model for the basin is refined.

Storm Drain System Sizing

The pipe sizes for the Hayden Road Storm Drain are shown in Exhibit A. Haestad's StormCAD model was used to size the storm drain and develop the hydraulic and energy grade lines. The following assumptions and constraints were implemented.

Nevermind - I got confused

[redacted]

- CIP pipe with a Manning's "n" value of 0.015 was used. The first segment of pipe which undercrosses the Pima Freeway is assumed to be RGRCP with a Manning's "n" value of 0.013.
- A rise in the HGL equal to the appropriate velocity head is added to the StormCAD HGL at the upstream end of the system. This is done because the StormCAD model does not account for this loss through the outlet headwall. See Appendix A for the output generated by the StormCAD model.
- 96" diameter RCP pipes are required for the first outlet segment of the system, since the use of 108" pipes would prevent the basin headwater from reaching an elevation of 1608 without compromising a maximum discharge rate of 1000 cfs per pipe. The remainder of the system is designed as dual 108" pipes.
- The current system design is intended to be a gravity flow system. However, certain segments within the system are flowing under pressure due to changes in the profile made to avoid existing major utilities. As the design progresses, options will be investigated to create a complete gravity flow system and avoid intermittent pressure flow areas.

Manhole Spacing & Sizing

Because of the large size of this storm drain (dual 108" pipes), the results of the hydraulic analysis by Dan Brauer recommends using 5' diameter RCP barrel sections attached to the pipe crowns to provide access to the system. Without additional inflow points in the system, larger junction structures offer no apparent advantages and impose more hydraulic losses from a design standpoint. Each 108" pipe is recommended to have it's own 5' diameter access manhole allowing each pipe to be entered independently of the other. The access manholes for each pipe are recommended to be located at approximately the same station.

Manhole spacing is proposed to be governed by the head differential between structures. Since a head of 15' (from the pipe springline) is deemed as the maximum head to be imposed on the CIP pipe, the spacing shall be regulated to limit total head at any one location to 15' maximum. In order to accomplish this requirement by access manhole spacing, each manhole rim is proposed to have an open grate lid, to serve as an air release and emergency pressure release for the system. The proposed locations for access manholes are shown in Exhibit A.

Peak Discharge from the Outer Loop Basin

As discussed during the Value Engineering phase of this project, a maximum discharge of 2000 cfs is based on the limited ability of the TPC golf course to accommodate flows greater than this amount. In fact, special care shall be required to design the outlet for 2000 cfs because of the narrow outlet channel, abrupt turn at the outlet, and extensive level of development of the existing golf course facilities.

Need to make some recommendations or include discussion regarding types of structures to be used. The magnitudes of flow and velocities are atypical to some of the standard design measures.

The specific outlet discharge for the system which corresponds to a basin HWL of 1608 is 1970 cfs based on the StormCAD model prepared by Pentacore. An output table from the StormCAD model is listed in Appendix A. This table lists hydraulic parameters of the system such as HGL, EGL, pipe slopes, pipe sizes, and flow velocities for the maximum discharge of 1970 cfs.

Outer Loop Basin Headwater - Upstream Control

The upstream control for the Hayden Road Storm Drain is the Outer Loop Basin headwater. At this point in the project, the design headwater is approximately 1608.0. Therefore the Hayden Road Storm Drain and in particular, the outlet pipes are sized based on a headwater at 1608.0. As mentioned above, the hydrology model and Outer Loop Basin design may be modified, and a headwater of 1608.0 may be adjusted. If the headwater elevation does in fact change, the hydraulics for the system will be modified accordingly.

Outer Loop Basin - Storm Drain Inlet Configuration

Various types of inlet configurations have been proposed by others to this point. However the most simple and cost effective configuration will be considered for this 10% H&H Memorandum. The chosen inlet configuration includes beveling the storm drain pipe ends and incorporating the basin sideslope into the headwall structure. The probable starting location for the storm drain inlet is east of Hayden Road. From there, the storm drain pipes can be directed on a curvilinear path to avoid the proposed abutments for ADOT's Hayden Road overpass, and proceed south along the east half of the Hayden Road right-of-way.

Peak Discharge from the Outer Loop Basin

As discussed during the Value Engineering phase of this project, a maximum discharge of 2000 cfs is based on the limited ability of the TPC golf course to accommodate flows greater than this amount. In fact, special care shall be required to design the outlet for 2000 cfs because of the narrow outlet channel, abrupt turn at the outlet, and extensive level of development of the existing golf course facilities.

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Outer Loop Basin Headwater - Upstream Control

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Outer Loop Basin - Storm Drain Inlet Configuration

Various types of inlet configurations have been proposed by others to this point. However the most simple and cost effective configuration will be considered for this 10% H&H Memorandum. The chosen inlet configuration includes beveling the storm drain pipe ends and incorporating the basin sideslope into the headwall structure. The probable starting location for the storm drain inlet is east of Hayden Road. From there, the storm drain pipes can be directed on a curvilinear path to avoid the proposed abutments for ADOT's Hayden Road overpass, and proceed south along the east half of the Hayden Road right-of-way.

Pima Road Three Basins Project
Hayden Road Storm Drain

04/17/98

Please provide reference to these ID's on the 11x17 plans

check these. should be $1604.99 + \frac{19.6^2}{64.4} = 1610.96$? (or [1602.59 + 0.0117(718)]).

Pipe	StormCAD Junction	Ground Up / Dn (ft)	HGL Up / Dn (ft)	EGL Up / Dn (ft)	Slope Energy/ onstruct. (ft/ft)	Discharge (cfs)	Pipe Diameter	Pipe Length (ft)	Velocity Avg (ft/s)
	in basin	HWL	1,607.97	1,607.97	----	----	----	----	----
P-1	I-1	1,612.00	1,604.99	1,607.97	0.0117	1,970.00	96 inch	718	19.6
	I-2	1,605.00	1,596.62	1,602.59	~~0.0039~~ 0.0047 per plans		dual pipes		
P-2	I-2	1,605.00	1,594.36	1,598.49	0.0159	1,970.00	108 inch	870	16.31
	I-3	1,594.00	1,581.27	1,585.40	0.0163		dual pipes		
P-3	I-3	1,594.00	1,580.14	1,584.25	0.0185	1,970.00	108 inch	545	16.27
	I-4	1,589.00	1,570.84	1,574.95	0.0350		dual pipes		
P-4	I-4	1,589.00	1,569.91	1,573.63	0.0083	1,970.00	108 inch	1,015.00	15.48
	I-5	1,574.00	1,561.50	1,565.22	0.0050		dual pipes		
P-5	I-5	1,574.00	1,560.57	1,564.29	0.0083	1,970.00	108 inch	140	15.48
	I-6	1,574.00	1,559.41	1,563.13	0.0330		dual pipes		
P-6	I-6	1,574.00	1,558.48	1,562.20	0.0083	1,970.00	108 inch	790	15.48
	I-7	1,563.00	1,551.93	1,555.65	0.0050		dual pipes		
P-7	I-7	1,563.00	1,551.00	1,554.72	0.0083	1,970.00	108 inch	600	15.48
	I-8	1,556.00	1,546.03	1,549.75	0.0149		dual pipes		
P-8	I-8	1,556.00	1,543.79	1,547.51	0.0083	1,970.00	108 inch	680	15.48
	I-9	1,548.00	1,538.16	1,541.88	0.0088		dual pipes		
P-9	I-9	1,548.00	1,535.92	1,539.64	0.0083	1,970.00	108 inch	640	15.48
	Outlet	1,536.00	1,530.62	1,534.34	0.0050		dual pipes		

Exhibit A shows 1531.0 which one is correct? Also, will an energy dissipation structure cause a greater impact on the tailwater condition?

Need to include summary of jct loss coefficients used if any. Seems like the losses at the "junctions" are excessive. Also, did you include an entrance loss at I-1

Pima Road Three Basins Project
Hayden Road Storm Drain

Please provide reference to these ID's on the 11x17 plans

Check these. should be $1604.99 + \frac{19.6^2}{64.4} = 1610.96$?
(or [1602.59 + 0.0117(718)]).

Pipe	StormCAD Junction	Ground Up / Dn (ft)	HGL Up / Dn (ft)	EGL Up / Dn (ft)	Slope Energy/ onstruct. (ft/ft)	Discharge (cfs)	Pipe Diameter	Pipe Length (ft)	Velocity Avg (ft/s)
	in basin	HWL	1,607.97	1,607.97	----	----	----	----	----
P-1	I-1	1,612.00	1,604.99	1,607.97	0.0117	1,970.00	96 inch	718	19.6
	I-2	1,605.00	1,596.62	1,602.59	~~0.0039~~ 0.0047 per plans		dual pipes		
P-2	I-2	1,605.00	1,594.36	1,598.49	0.0159	1,970.00	108 inch	870	16.31
	I-3	1,594.00	1,581.27	1,585.40	0.0163		dual pipes		
P-3	I-3	1,594.00	1,580.14	1,584.25	0.0185	1,970.00	108 inch	545	16.27
	I-4	1,589.00	1,570.84	1,574.95	0.0350		dual pipes		
P-4	I-4	1,589.00	1,569.91	1,573.63	0.0083	1,970.00	108 inch	1,015.00	15.48
	I-5	1,574.00	1,561.50	1,565.22	0.0050		dual pipes		
P-5	I-5	1,574.00	1,560.57	1,564.29	0.0083	1,970.00	108 inch	140	15.48
	I-6	1,574.00	1,559.41	1,563.13	0.0330		dual pipes		
P-6	I-6	1,574.00	1,558.48	1,562.20	0.0083	1,970.00	108 inch	790	15.48
	I-7	1,563.00	1,551.93	1,555.65	0.0050		dual pipes		
P-7	I-7	1,563.00	1,551.00	1,554.72	0.0083	1,970.00	108 inch	600	15.48
	I-8	1,556.00	1,546.03	1,549.75	0.0149		dual pipes		
P-8	I-8	1,556.00	1,543.79	1,547.51	0.0083	1,970.00	108 inch	680	15.48
	I-9	1,548.00	1,538.16	1,541.88	0.0088		dual pipes		
P-9	I-9	1,548.00	1,535.92	1,539.64	0.0083	1,970.00	108 inch	640	15.48
	Outlet	1,536.00	1,530.62	1,534.34	0.0050		dual pipes		

Exhibit A shows 1531.0
which one is correct?
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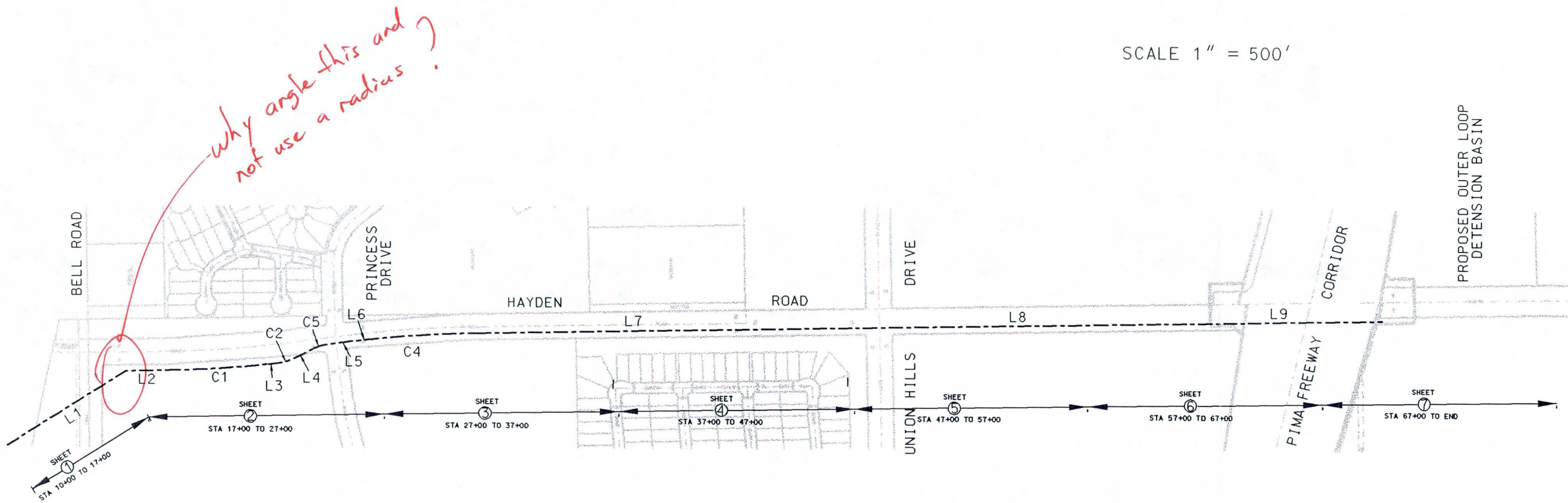
NORTH

SCALE 1" = 500'



LINE	DIRECTION	DISTANCE ft
L1	N30°57'16"W	600.00
L2	N00°01'19"E	177.84
L3	N05°08'27"W	43.74
L4	N24°54'07"W	104.25
L5	N06°03'07"W	37.71
L6	N05°59'28"W	45.60
L7	N00°01'55"E	1146.02
L8	N00°00'21"E	1652.42
L9	N00°00'22"W	1077.37

CURVE	LENGTH ft	DELTA	RADIUS ft
C1	430.68	05°09'43"	4780.26
C2	62.08	19°45'43"	180.00
C3	59.22	18°50'58"	180.00
C4	562.80	06°01'26"	5353.07

EXHIBIT A

NOT FOR CONSTRUCTION

DATE	REVISION	BY
ENGINEER		

TRANSPORTATION DEPARTMENT
TRANSPORTATION PLANNING
3939 CIVIC CENTER BLVD. SCOTTSDALE, ARIZONA 85251

PROJECT TITLE: HAYDEN ROAD STORM DRAIN

SCALE	DESIGNED BY	DATE 4/16/98	BID NO.	SHT.
	DRAWN BY	AS-BUILT	PROJECT NO. 5001.0007	1 OF 9

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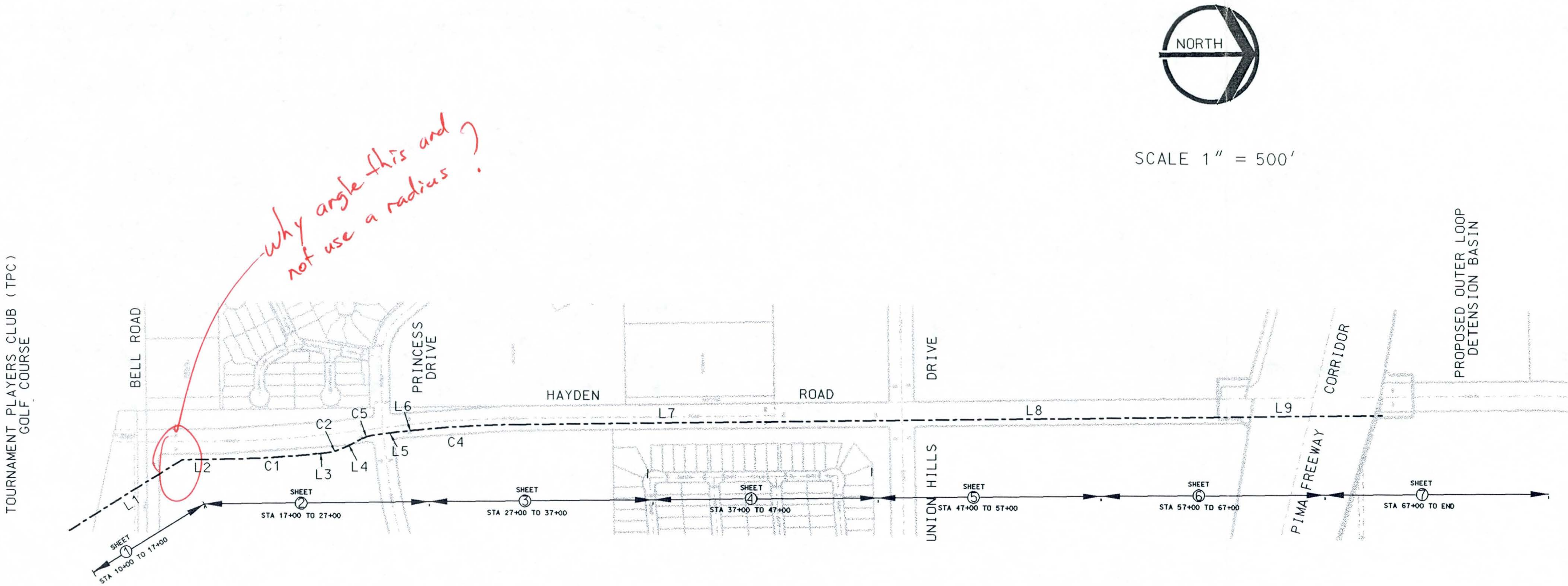

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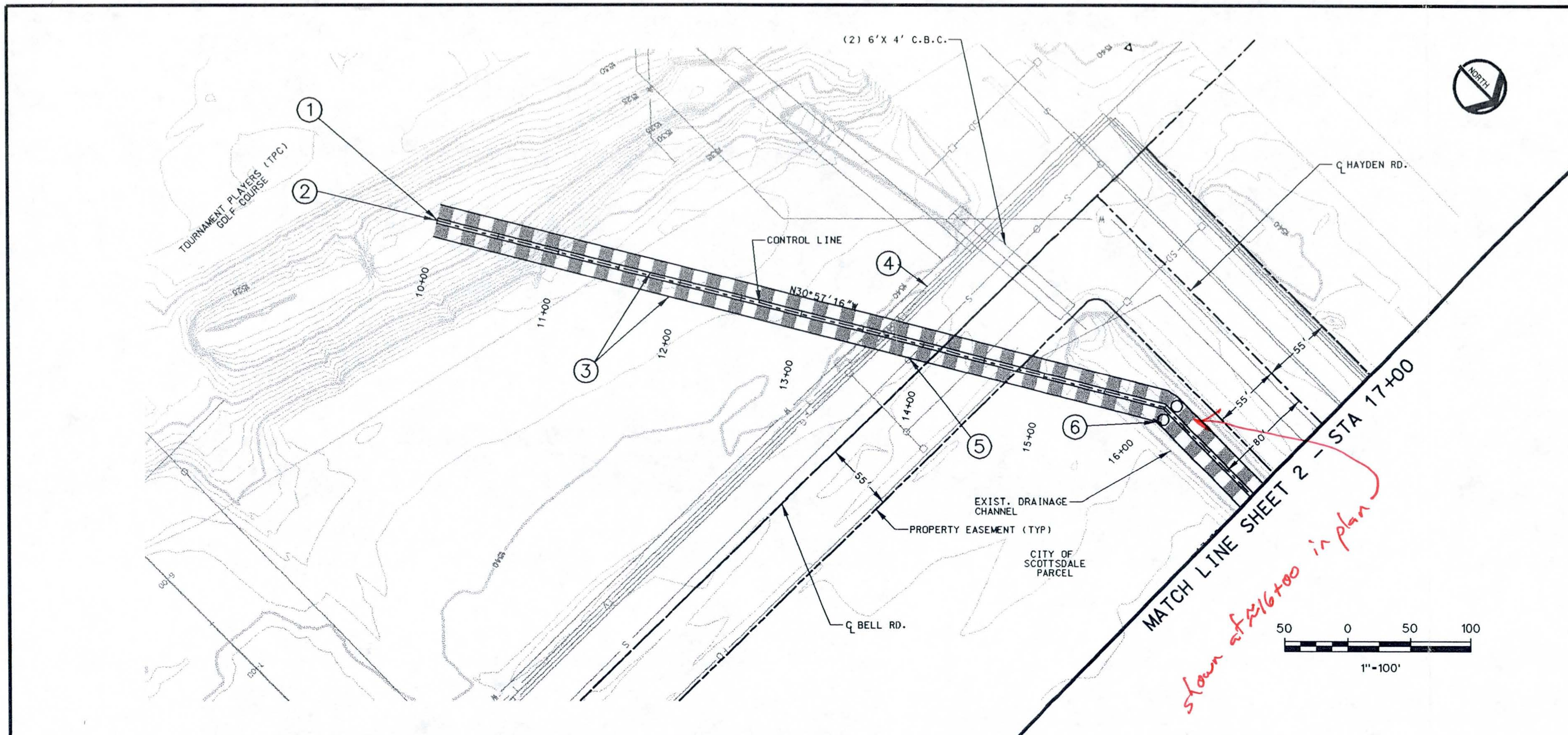
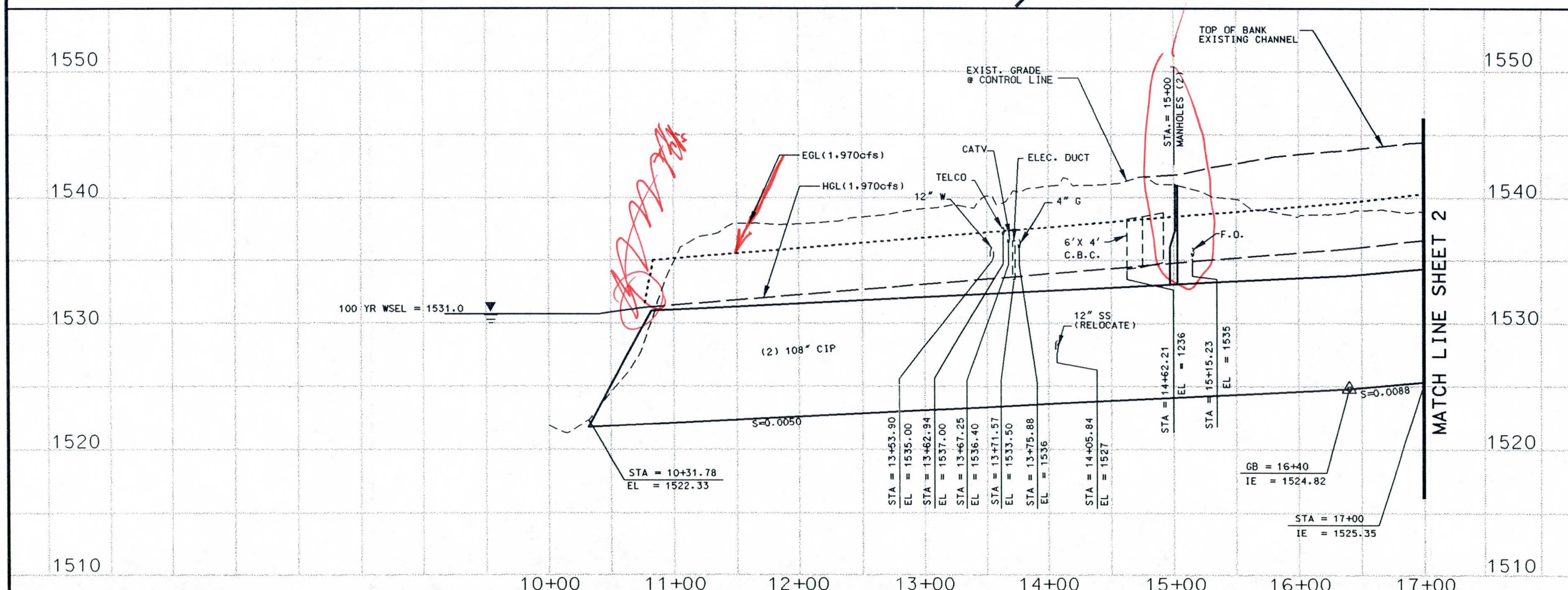
C4	562.80	06°01'26"	5353.07

EXHIBIT A

NOT FOR CONSTRUCTION

DATE	REVISION	BY
ENGINEER	TRANSPORTATION DEPARTMENT TRANSPORTATION PLANNING 3939 CIVIC CENTER BLVD. SCOTTSDALE, ARIZONA 85251	
PROJECT TITLE	HAYDEN ROAD STORM DRAIN	
SCALE	DESIGNED BY / DATE 4/16/98 / BID NO.	SHT.
	DRAWN BY AS-BUILT / PROJECT NO. 5001.0007	1 OF 9





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REMOVAL NOTES

☐ Description	Unit	Quan.

○ CONSTRUCTION NOTES:

1. INSTALL CONCRETE APRON AND ENERGY DISSIPATOR
2. INSTALL CONCRETE HEADWALL
3. INSTALL (2) 108" CIPP (SEE DETAIL A - SHEET 9)
4. EXISTING UTILITIES PROTECT IN PLACE (TYP)
5. RELOCATE 12" SANITARY SEWER SEE SHEET -
6. INSTALL (2) MANHOLES

EXHIBIT A

NOT FOR CONSTRUCTION

DATE	REVISION	BY

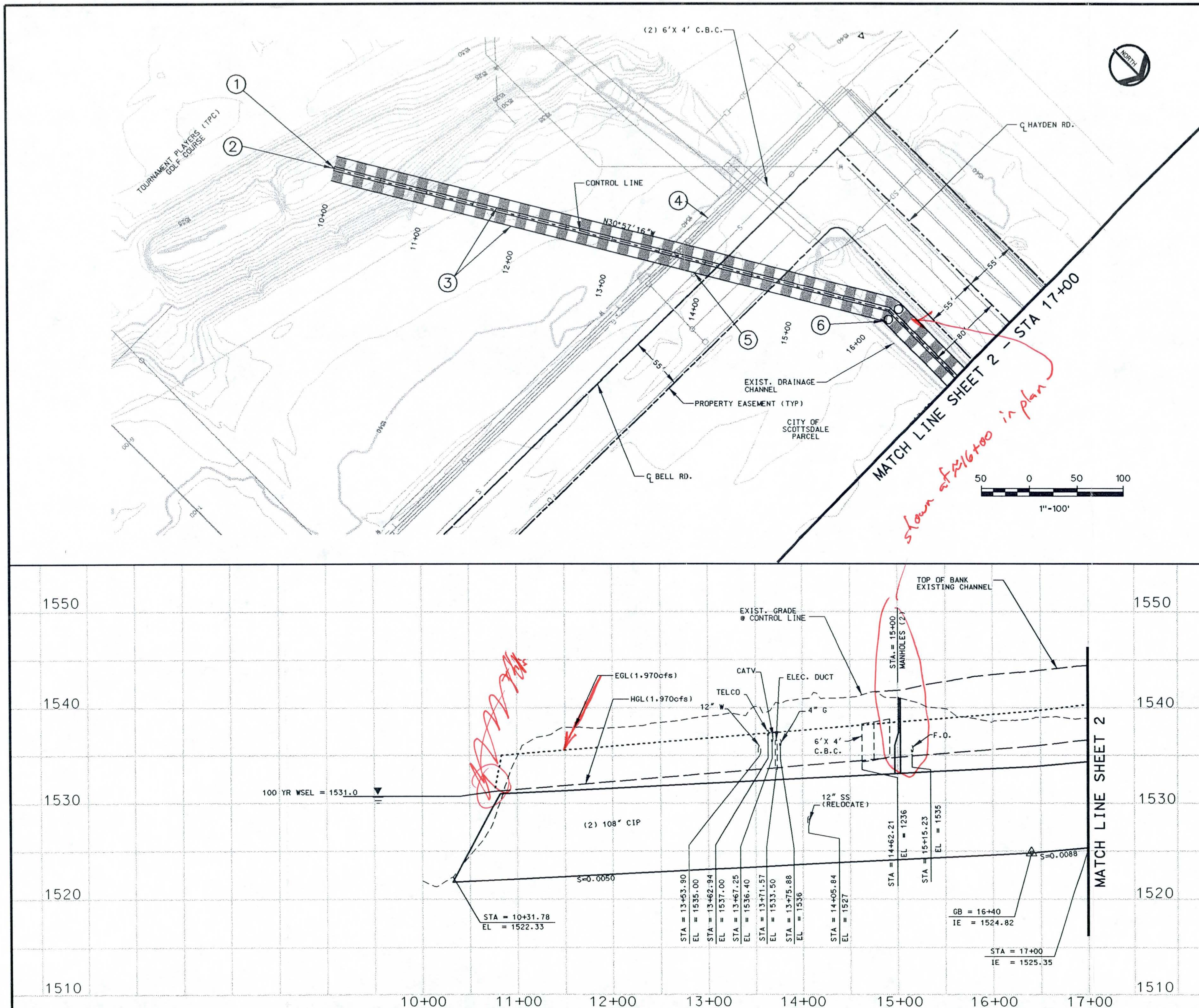
ENGINEER

CITY OF SCOTTSDALE ARIZONA

TRANSPORTATION DEPARTMENT
TRANSPORTATION PLANNING
3939 CIVIC CENTER BLVD.
SCOTTSDALE, ARIZONA 85251

PROJECT TITLE
HAYDEN ROAD STORM DRAIN
10% CONCEPTUAL PHASE

SCALE	DESIGNED BY	DATE	BID NO.	SHT.
HORIZ. 1"=100' VERT. 1"=10'	DRAWN BY	AS-BUILT 4/15/98	PROJECT NO. 5001.0007	2 OF 9



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REMOVAL NOTES

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EXHIBIT A

NOT FOR CONSTRUCTION

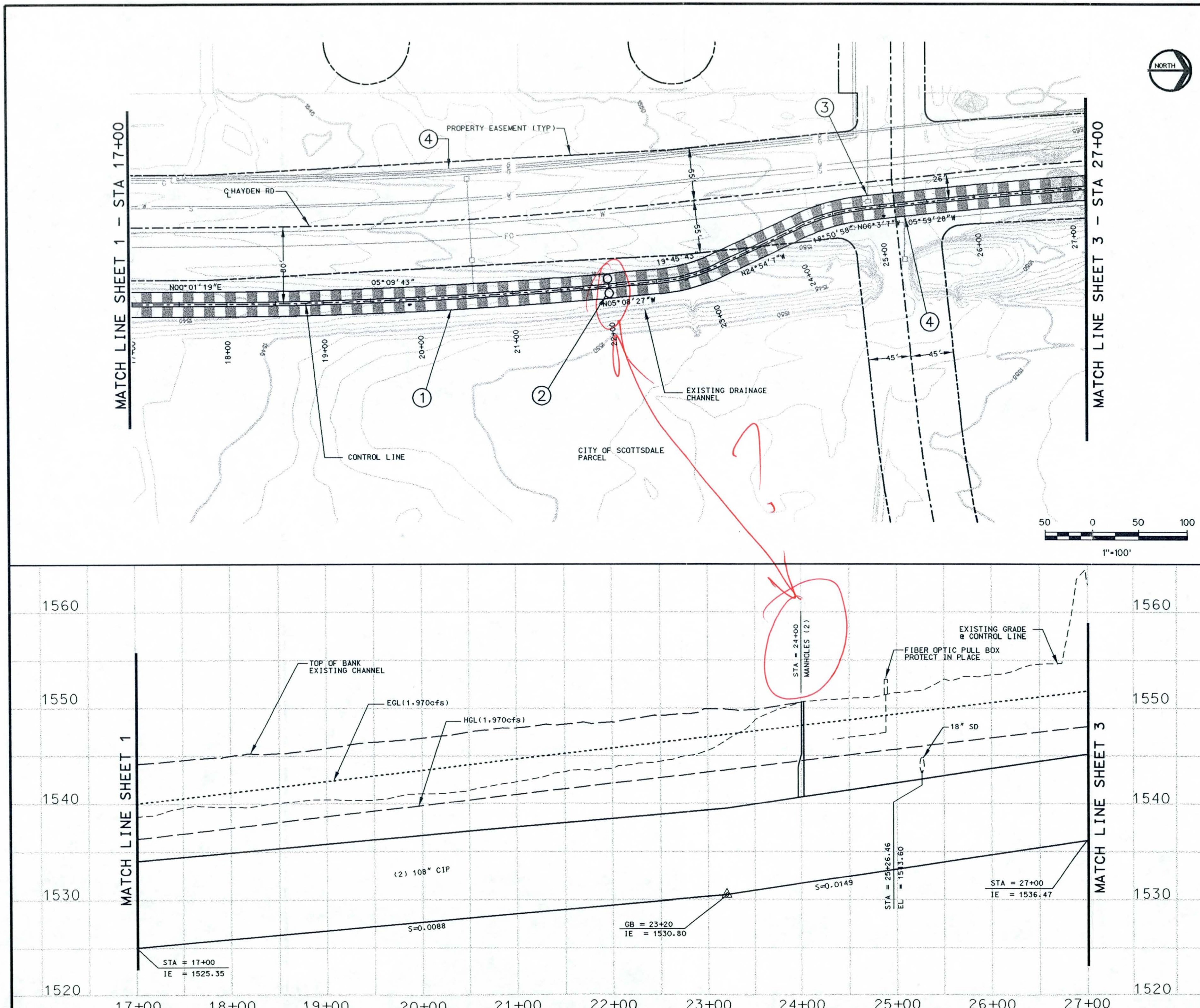
DATE	REVISION	BY

ENGINEER

TRANSPORTATION DEPARTMENT
TRANSPORTATION PLANNING
3939 CIVIC CENTER BLVD.
SCOTTSDALE, ARIZONA 85251

PROJECT TITLE: HAYDEN ROAD STORM DRAIN
10% CONCEPTUAL PHASE

SCALE	DESIGNED BY	DATE	BID NO.	SHT.
HORIZ. 1"-100' VERT. 1"-10'	DRAWN BY	4/15/98 AS-BUILT	PROJECT NO. 5001.0007	2 OF 9



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REMOVAL NOTES

☐ Description	Unit	Quan.

○ CONSTRUCTION NOTES:

1. INSTALL (2) 108" CIPP (SEE DETAIL A - SHEET 9)
2. INSTALL MANHOLES (2)
3. FIBER OPTIC LINE (PROTECT IN PLACE)
4. EXISTING UTILITIES PROTECT IN PLACE (TYP)

EXHIBIT A

NOT FOR CONSTRUCTION

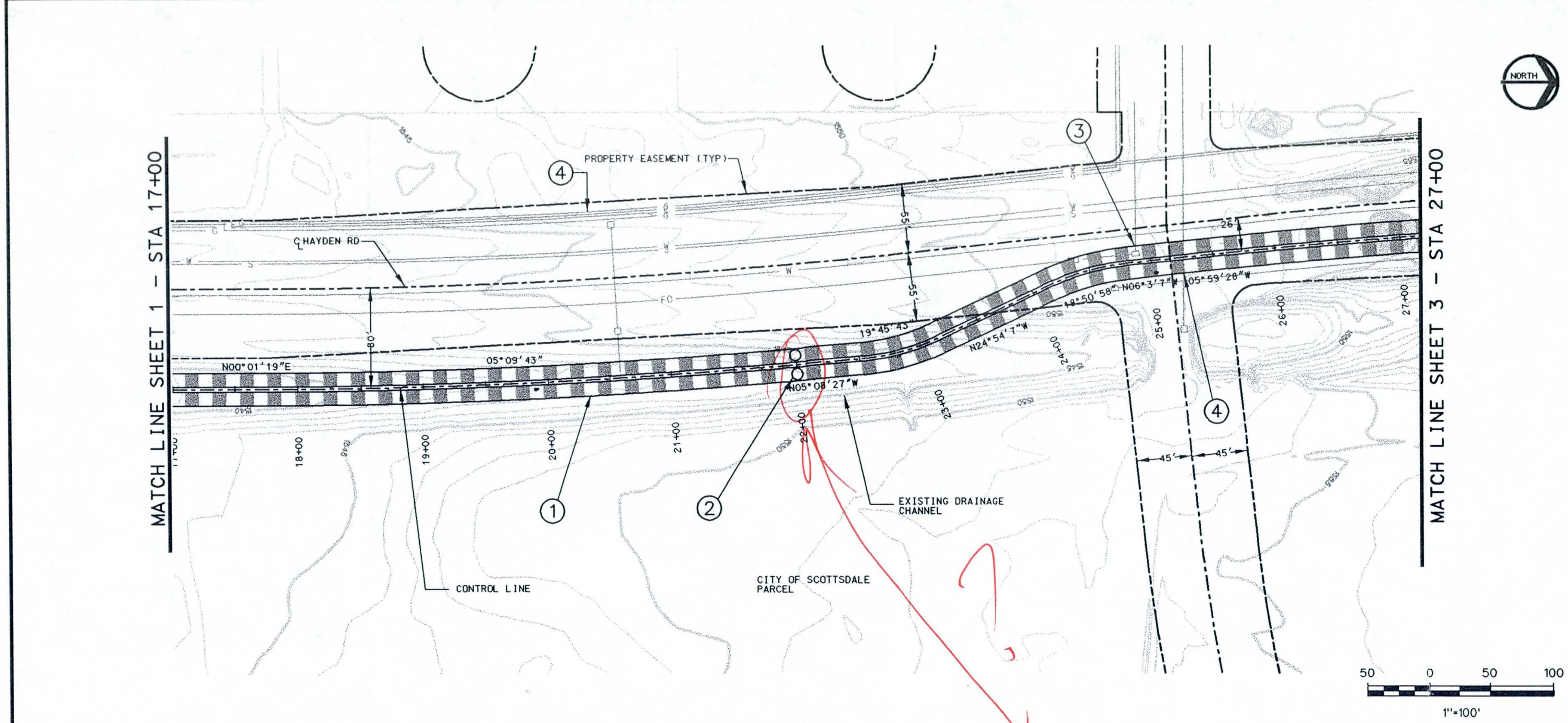
DATE	REVISION	BY

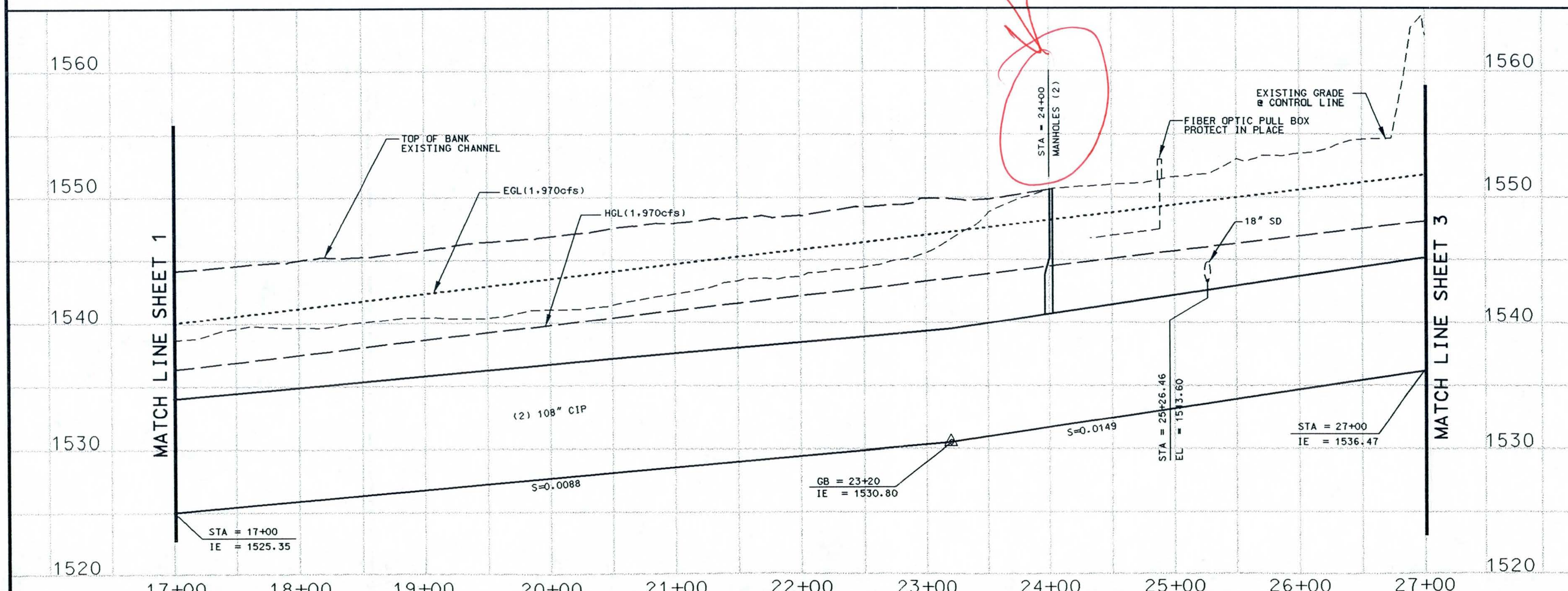
ENGINEER

TRANSPORTATION DEPARTMENT
TRANSPORTATION PLANNING
3939 CIVIC CENTER BLVD.
SCOTTSDALE, ARIZONA 85251

PROJECT TITLE
HAYDEN ROAD STORM DRAIN
10% CONCEPTUAL PHASE

SCALE	DESIGNED BY	DATE	BID NO.	SHT.
HORIZ. 1"=100'		4/15/98		
VERT. 1"=10'	DRAWN BY	AS-BUILT	PROJECT NO. 5001.0007	3 OF 9





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REMOVAL NOTES

☐ Description	Unit	Quan.

O CONSTRUCTION NOTES:

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EXHIBIT A

NOT FOR CONSTRUCTION

DATE	REVISION	BY

ENGINEER

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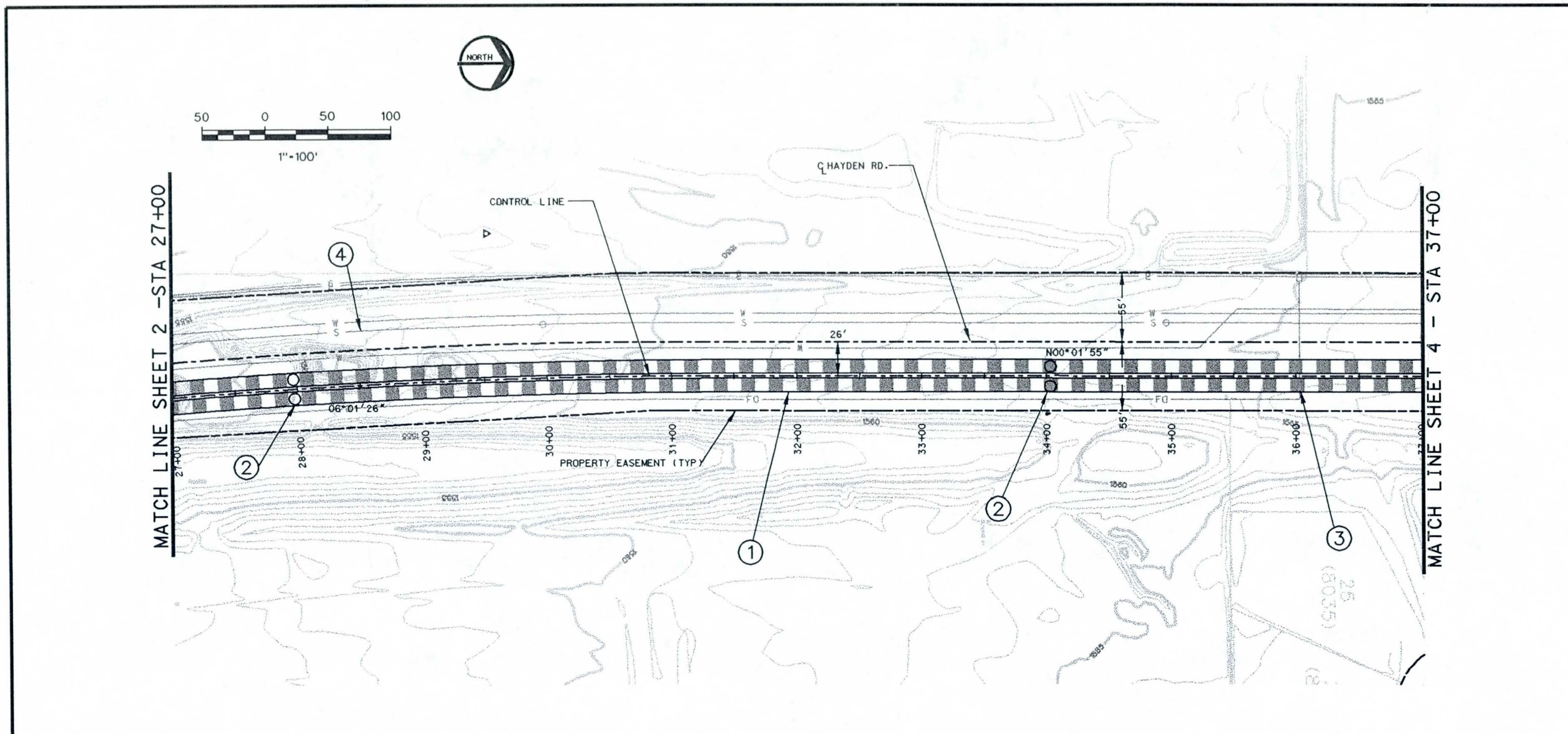
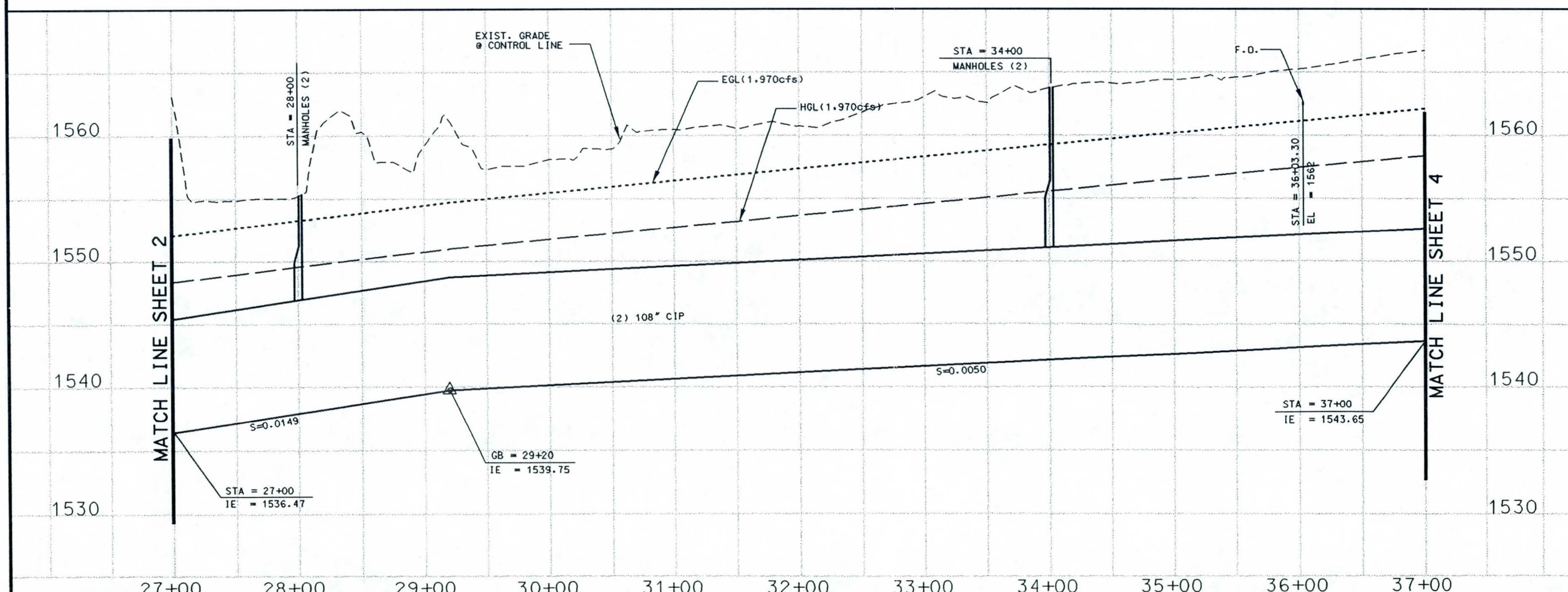
SCOTTSDALE, ARIZONA 85251

PROJECT TITLE: HAYDEN ROAD STORM DRAIN
10% CONCEPTUAL PHASE

SCALE	DESIGNED BY	DATE	BID NO.	SHT.
HORIZ. 1"=100' VERT. 1"=10'	DRAWN BY	4/15/98 AS-BUILT	PROJECT NO. 5001.0007	3 OF 9





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REMOVAL NOTES

☐ Description	Unit	Quan.

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1. INSTALL (2) 108" CIPP (SEE DETAIL A - SHEET 9)
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3. FIBER OPTIC LINE (PROTECT IN PLACE)
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EXHIBIT A

NOT FOR CONSTRUCTION

DATE	REVISION	BY

ENGINEER

CITY OF SCOTTSDALE ARIZONA

TRANSPORTATION DEPARTMENT

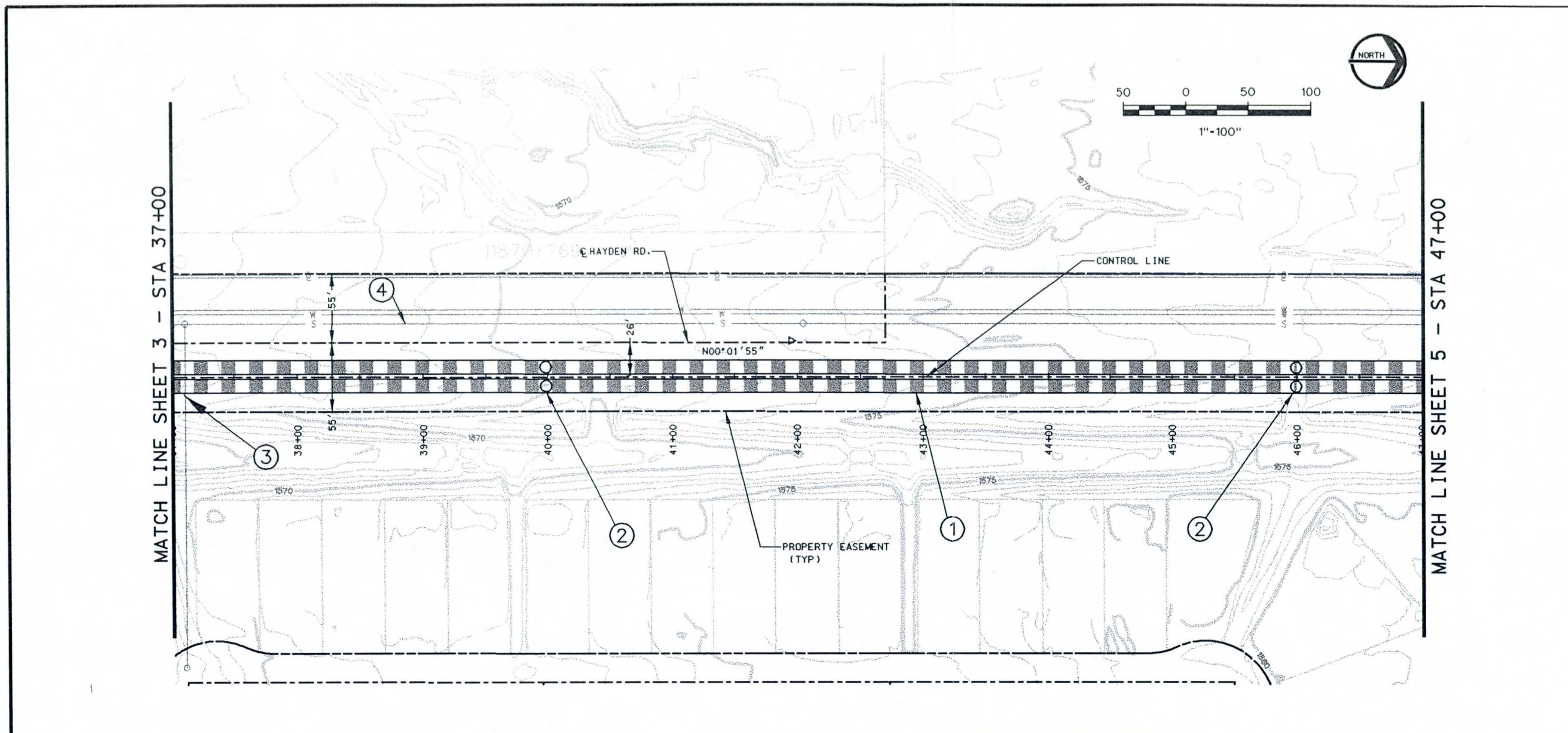
TRANSPORTATION PLANNING

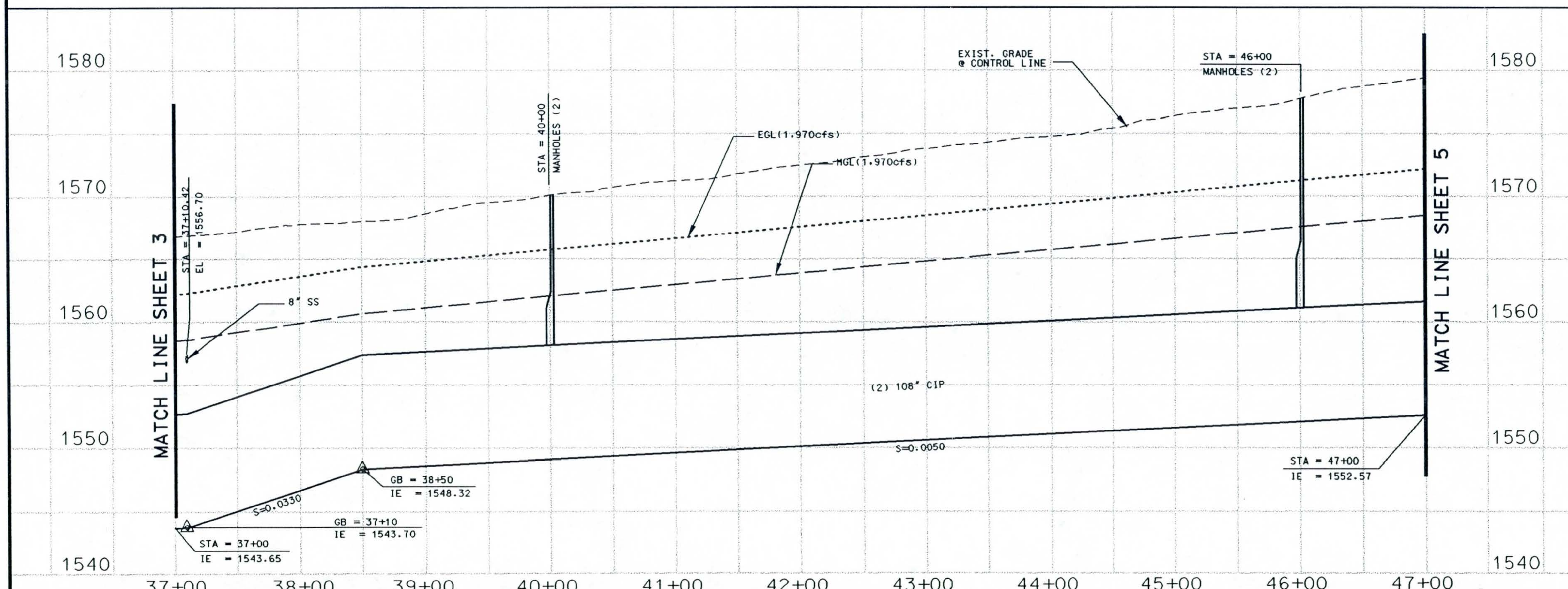
3939 CIVIC CENTER BLVD.
SCOTTSDALE, ARIZONA 85251

PROJECT TITLE

HAYDEN ROAD STORM DRAIN
10% CONCEPTUAL PHASE

SCALE	DESIGNED BY	DATE	BID NO.	SHT.
HORIZ. 1"=100' VERT. 1"=10'	DRAWN BY	AS-BUILT 4/15/98	PROJECT NO. 5001.0007	4 OF 9





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REMOVAL NOTES

☐ Description	Unit	Quan.

○ CONSTRUCTION NOTES:

1. INSTALL (2) 108" CIPP (SEE DETAIL A - SHEET 9)
2. INSTALL MANHOLES (2)
3. 8" SANITARY SEWER PROTECT IN PLACE
4. EXISTING UTILITIES PROTECT IN PLACE (TYP)

EXHIBIT A

NOT FOR CONSTRUCTION

DATE	REVISION	BY

ENGINEER

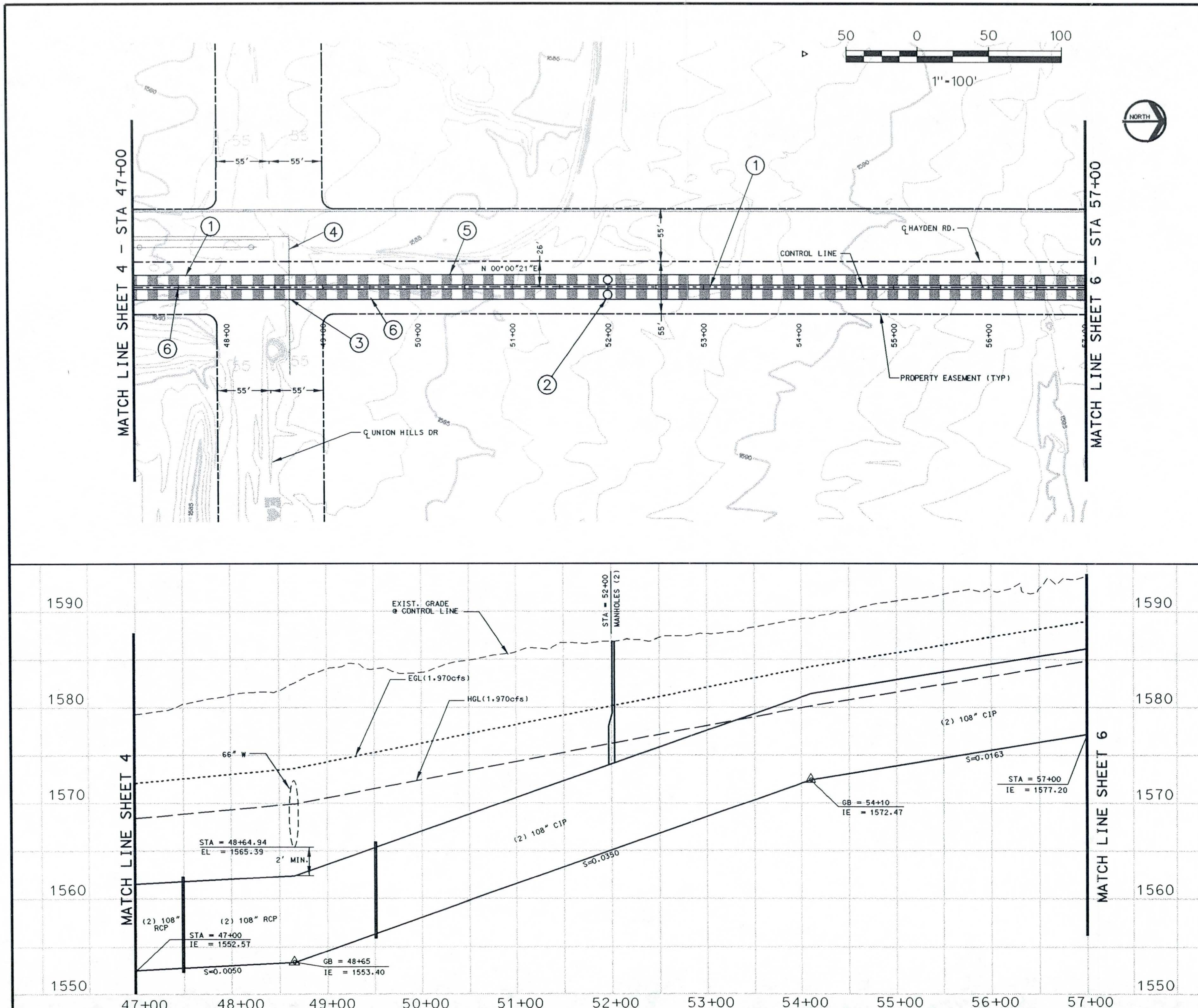
TRANSPORTATION DEPARTMENT

TRANSPORTATION PLANNING

3939 CIVIC CENTER BLVD.
SCOTTSDALE, ARIZONA 85251

PROJECT TITLE: HAYDEN ROAD STORM DRAIN
10% CONCEPTUAL PHASE

SCALE	DESIGNED BY	DATE	BID NO.	SHT.
HORIZ. 1"=100' VERT. 1"=10'	DRAWN BY	4/15/98 AS-BUILT	PROJECT NO. 5001.0007	5 OF 9



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REMOVAL NOTES

☐ Description	Unit	Quan.

○ CONSTRUCTION NOTES:

1. INSTALL (2) 108" CIPP (SEE DETAIL A –SHEET 9)
2. INSTALL MANHOLES (2)
3. 66" WATER MAIN PROTECT IN PLACE
4. EXISTING UTILITIES PROTECT IN PLACE (TYP)
5. STA 47+50– STA 49+50 INSTALL (2) 108" RCP (SECTION B SHEET 9)
6. INSTALL 108" PIPE COLLAR FOR CIP–RCP TRANSITION

EXHIBIT A

NOT FOR CONSTRUCTION

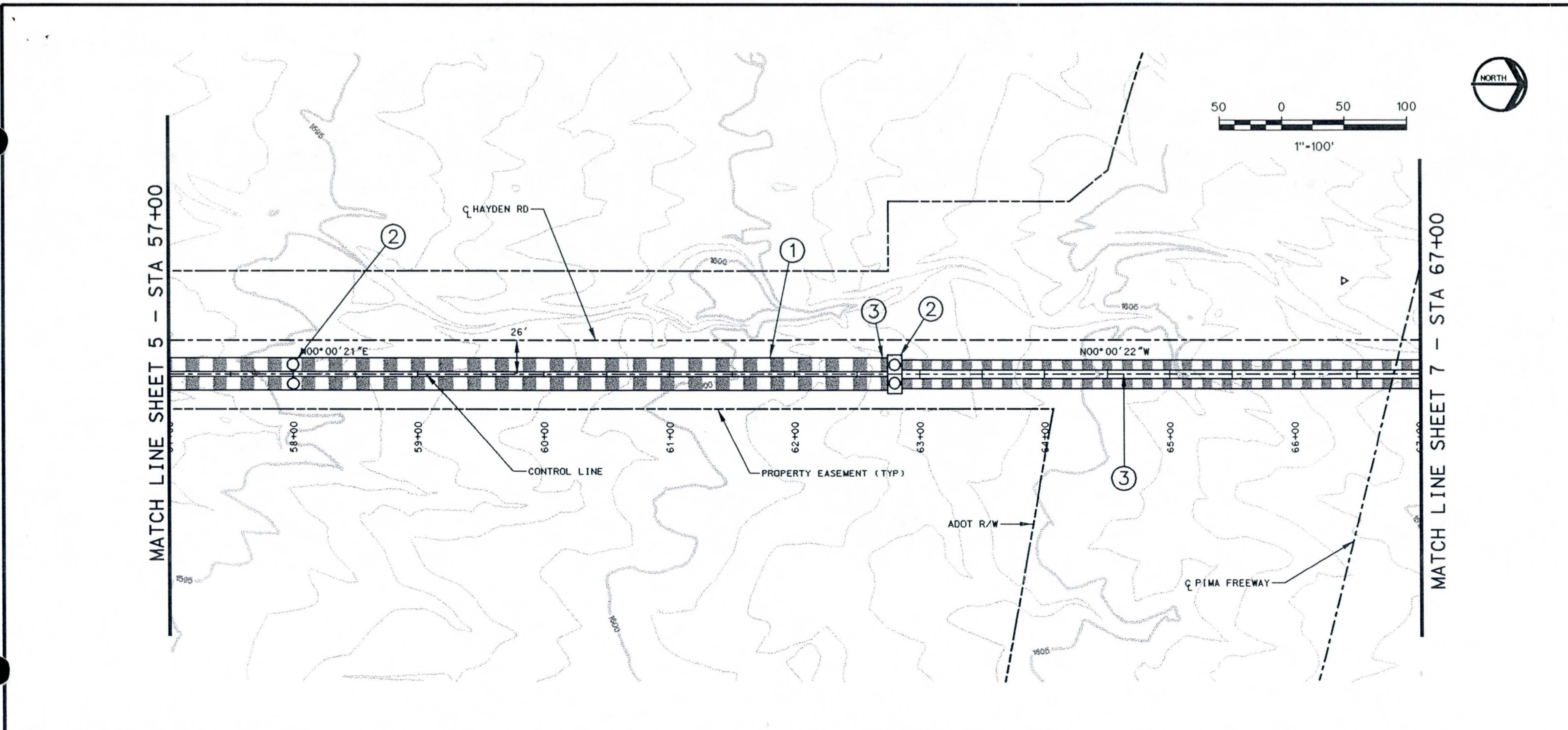
DATE	REVISION	BY

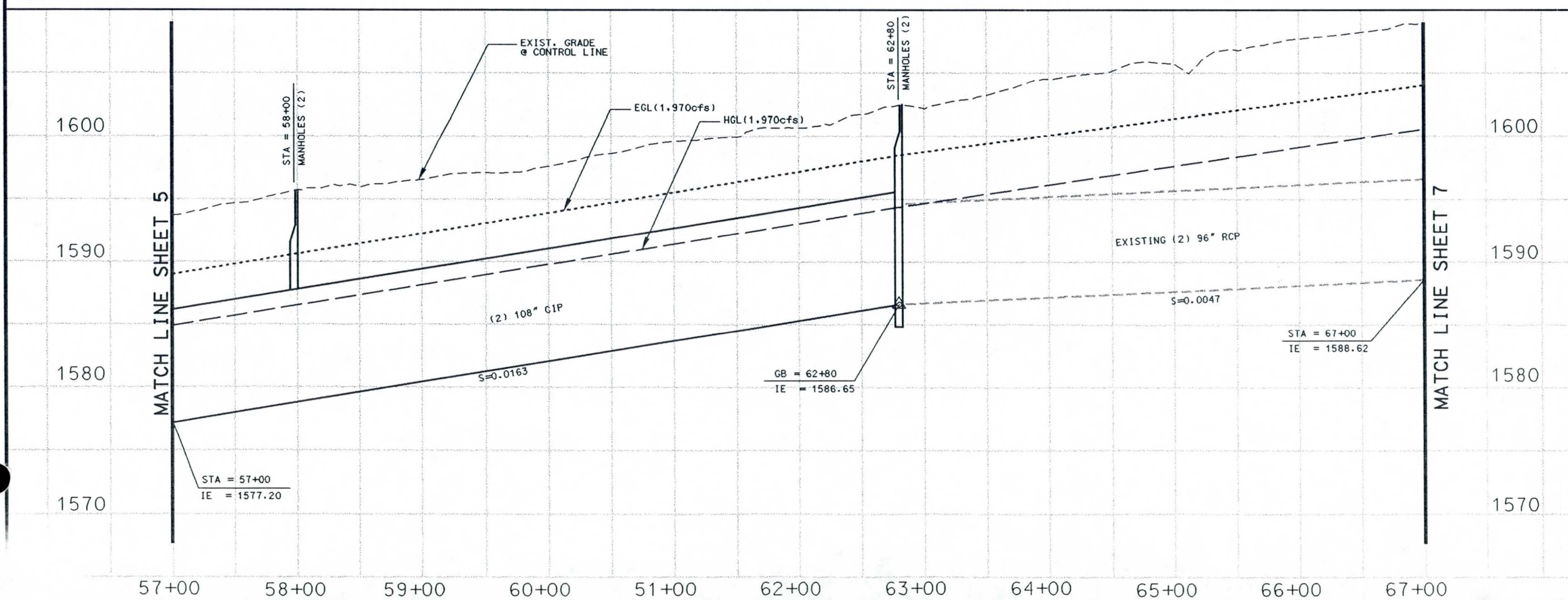
ENGINEER

TRANSPORTATION DEPARTMENT
TRANSPORTATION PLANNING
3939 CIVIC CENTER BLVD.
SCOTTSDALE, ARIZONA 85251

PROJECT TITLE: HAYDEN ROAD STORM DRAIN 10% CONCEPTUAL PHASE

SCALE	DESIGNED BY / DATE	BID NO.	SHT.
HORIZ. 1"=100' VERT. 1"=10'	4/15/98 DRAWN BY AS-BUILT	PROJECT NO. 5001.0007	6 OF 9





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REMOVAL NOTES

☐ Description	Unit	Quan.

○ CONSTRUCTION NOTES:

1. INSTALL (2) 108" CIPP (SEE DETAIL A - SHEET 9)
2. TIE-IN EXISTING MANHOLES WITH STUBOUTS
3. INSTALL 108" PIPE COLLAR

EXHIBIT A

NOT FOR CONSTRUCTION

DATE	REVISION	BY

ENGINEER

TRANSPORTATION DEPARTMENT

TRANSPORTATION PLANNING

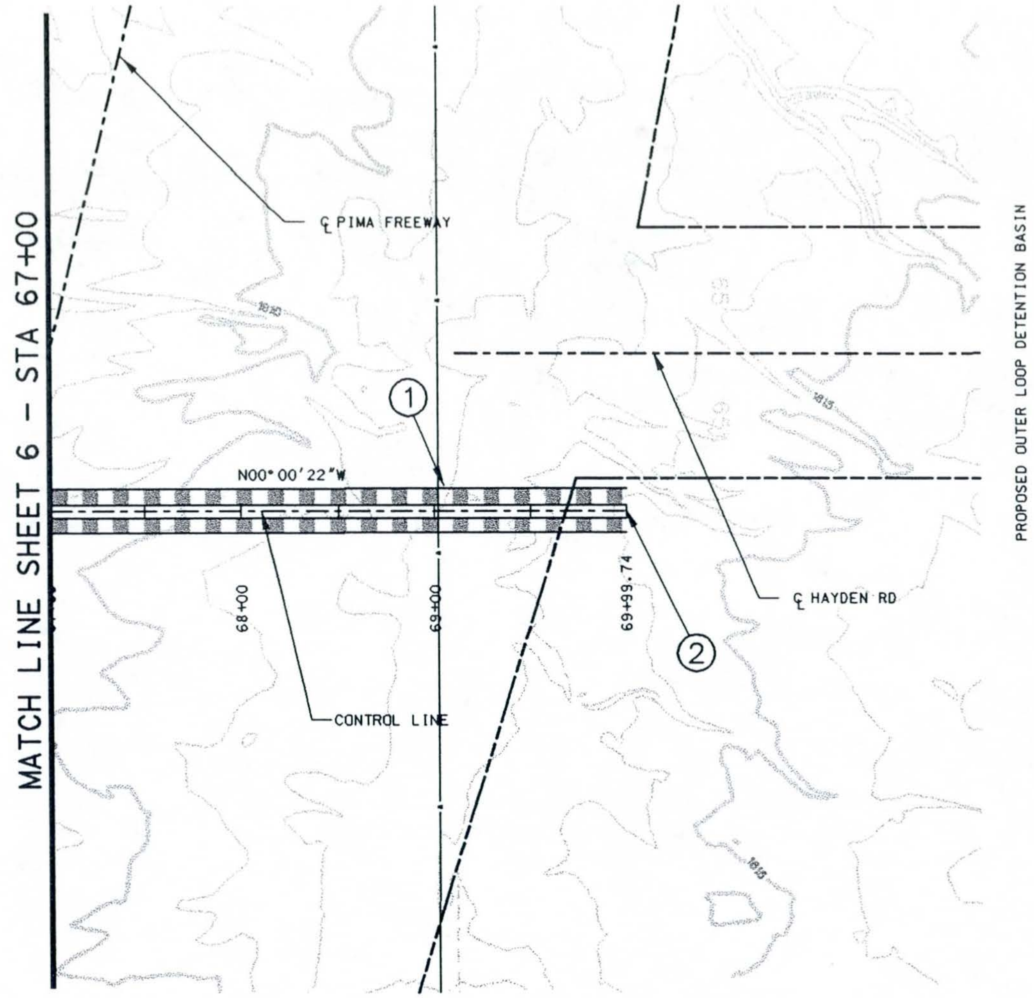
3939 CIVIC CENTER BLVD.
SCOTTSDALE, ARIZONA 85251

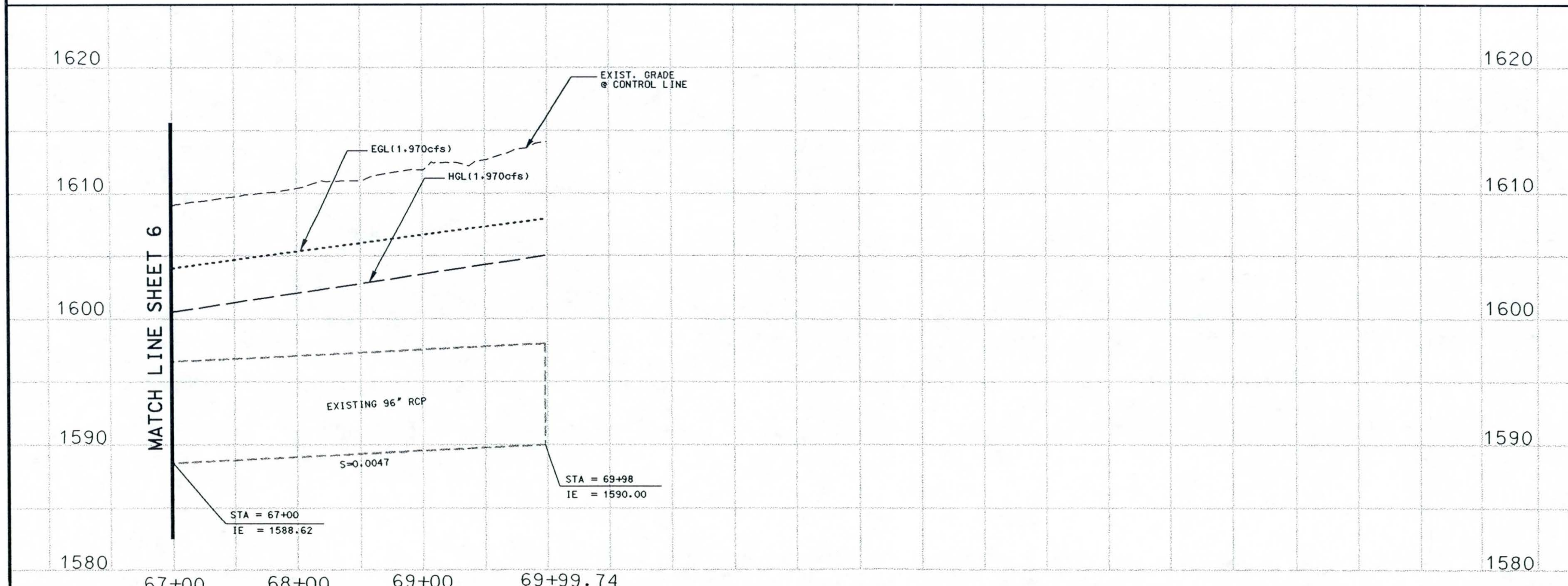
PROJECT TITLE

HAYDEN ROAD STORM DRAIN

10% CONCEPTUAL PHASE

SCALE	DESIGNED BY	DATE	BID NO.	SHT.
HORIZ. 1"=100' VERT. 1"=10'	DRAWN BY	4/15/98 AS-BUILT	PROJECT NO. 5001.0007	7 OF 9





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REMOVAL NOTES

☐ Description	Unit	Quan.

O CONSTRUCTION NOTES:

1. INSTALL (2) 96" RGRCP (SEE DETAIL A - SHEET 9)
2. BASIN OUTLET STRUCTURE (OUTLET BY OTHERS)

EXHIBIT A

NOT FOR CONSTRUCTION

DATE	REVISION	BY

ENGINEER

TRANSPORTATION DEPARTMENT
TRANSPORTATION PLANNING
3939 CIVIC CENTER BLVD.
SCOTTSDALE, ARIZONA 85251

PROJECT TITLE
HAYDEN ROAD STORM DRAIN
10% CONCEPTUAL PHASE

SCALE	DESIGNED BY	DATE	BID NO.	SHT.
HORIZ. 1"=100' VERT. 1"=10'	DRAWN BY	4/15/98 AS-BUILT	PROJECT NO. 5001.0007	8 OF 9

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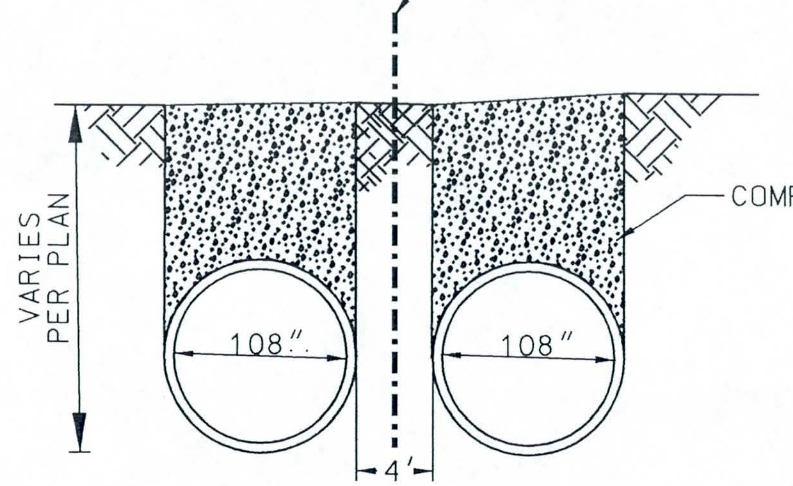
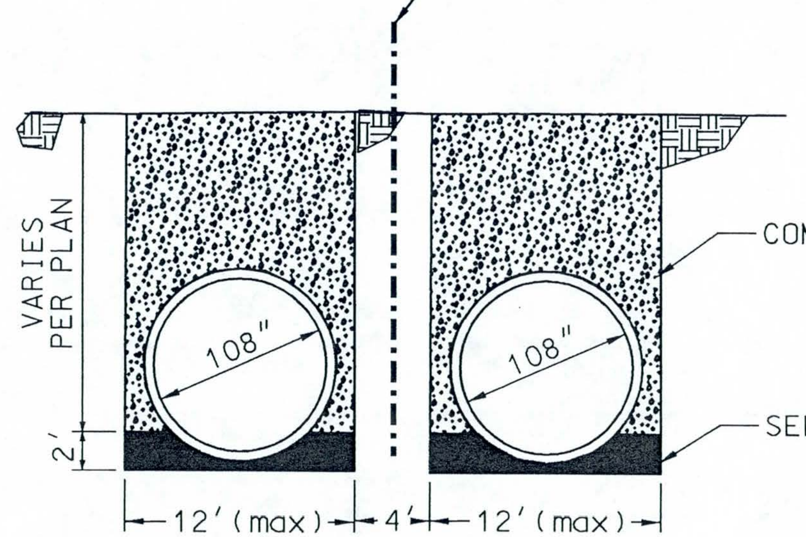


CAST-IN-PLACE (CIP) PIPE

DETAIL "A"



REINFORCED CONCRETE PIPE

DETAIL "B"

NOT FOR CONSTRUCTION

DATE	REVISION	BY
ENGINEER	TRANSPORTATION DEPARTMENT TRANSPORTATION PLANNING 3939 CIVIC CENTER BLVD. SCOTTSDALE, ARIZONA 85251	
PROJECT TITLE	HAYDEN ROAD STORM DRAIN 10% CONCEPTUAL PLANS	

SCALE	DESIGNED BY	DATE 4/16/98	BID NO.	SHT.
	DRAWN BY	AS-BUILT	PROJECT NO. 5001.0007	9 OF 9

Stantech
Consulting

Recd 6/17/98
WSO

Memo

To: File

From: George Sabol

Date: 16 June 1998

Reference: **PR3B – Outer Loop Basin Spillways**
FILE: 28900082

The basin spillways will direct and convey runoff from overland runoff north of the basin into the basin. The immediate area upgradient of the basin is presently natural, undeveloped land, and the drainage pattern is distributary flow. There is a fair degree of uncertainty as to the magnitude of flow reaching each spillway.

The spillway crest length is based on the estimated 100-yr, 6-hr peak discharge of each spillway divided by 2.5 cfs/ft of crest length. The spillway hydraulics are calculated for 5 cfs/ft to allow for hydrologic uncertainty and to provide for future drainage design options as the land north of the basin develops. The spillway hydrologic estimation will be provided in a separate memo.

Attached are the spillway hydraulic calculations. Empirical equations for the depth of flow and flow velocity at the toe of the spillway are not appropriate because of the relatively flat spillway slope (see Pages 1 and 2 of calculations). Those velocity/depth relations were estimated by Manning's equation (see Page 3). For q = 5 cfs/ft, the velocity at the spillway toe is about 10 fps. The hydraulic jump length is about 7.5 ft (see Pages 4 and 5). The depth of flow at the spillway crest is about 0.6 ft for 2.5 cfs/ft and about 0.9 ft for 5 cfs/ft. The spillway notch in the north bank of the basin should be no less than 1.5 ft to provide a minimum of 0.6 ft of freeboard. Topography and existing channel incisement may dictate greater spillway notch depths.

The riprap apron at the toe of spillway should extend a minimum of 7.5 ft out into the basin. Riprap is sized for 10 fps (see Page 7) and maximum size is 15 inches (use D_{50} = 8 inches).

George V. Sabol, PhD, PE
Senior Associate
Water Resources Division

Attachment

sci/p:\28900082\correspondence\memos\outer loop basin spillways, memo to file 6-15.doc

Project: PR3B Project Number: 28900082

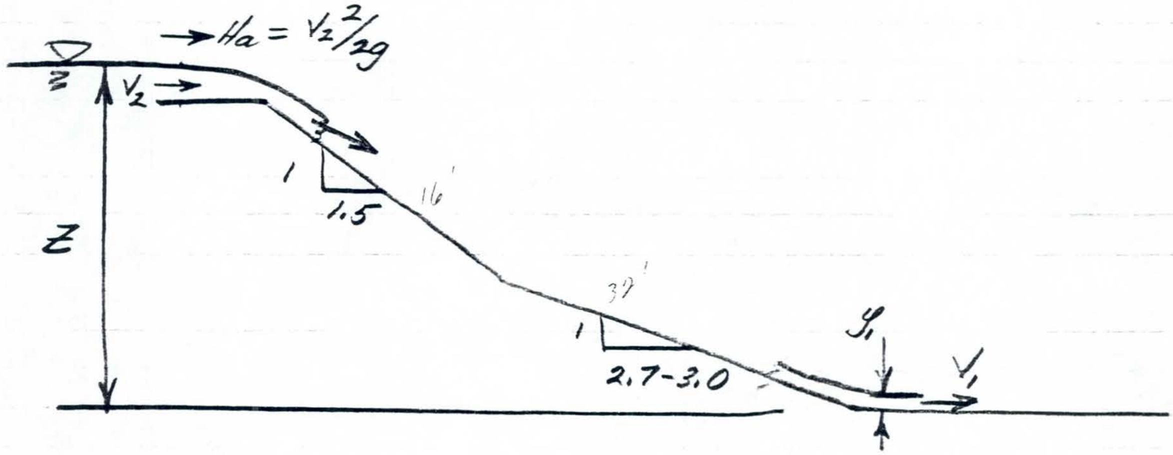
Notes: Scale:

Page 1 of Page(s)

Computed By: GVS Date: 20 May 93 Checked By: Date:

$$V_1 = \left(2g\left(Z + H_a - y_1\right)\right)^{1/2}$$

Ref. - Chow, pg. 382



maximum $Z = 25'$

assume $V_2 = 5$ fps

$H_a = 5^2/2g = .4$ ft

y_1 is small, let $y_1 = .1$ ft

$V_1 = (2g(25 + .4 - .1))^{1/2} = 40$ fps

The empirical curve for velocity (Chow, Fig. 14-15) has a V_1 velocity of about 35 fps at a $Z = 25'$. That is for a steeper & smoother spillway chute. Therefore, the velocity for the basin rundown spillways will be considerably lower than calculated.

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Since the drum gate acts as a weir, the discharge through the gate may be expressed as

$$Q = CLH_e^{1.5} \qquad (14\text{-}16)$$

where C is the coefficient of discharge, L is the length of the gate, and H_e is the total head. Laboratory investigations have shown that the flow over this type of gate can be completely defined by H_e, θ, C, the radius r of the gate, and the depth of approach. The depth of approach, however, has very little influence on the flow behavior when the approach depth, measured below the highest point of the gate, is equal to or greater than twice the head on the gate. This condition is well satisfied by most drum-gate installations, especially when the gate is in a raised position. Therefore, the coefficient C may be considered to be a function of H_e, θ, and r.

Bradley [27] has made a comprehensive study of the drum gate, using data obtained from 40 hydraulic models of existing drum-gate structures of various sizes and scales. The results of this study are shown by a family of curves (Fig. 14-14) where C is plotted against θ with the ratio H_e/r as a parameter. When $H_e/r = 0$, the gate becomes a straight inclined weir, and the corresponding dashed line in the family of curves is based on Bazin's data [12]. The curves extend downward to $\theta = -15°$. The discharge coefficients in the region between $\theta = -15°$ and the gate completely down can be obtained by graphical interpolation of the rating curves of the gate. The computation of the rating curve when the gate is completely down is the same as that for a spillway with an ungated crest (Art. 14-5).

14-10. Flow at the Toe of Overflow Spillways. The theoretical velocity of flow at the toe of an overflow spillway (Fig. 14-15) may be computed by

$$V_1 = \sqrt{2g(Z + H_a - y_1)} \qquad (14\text{-}19)$$

where Z is the fall, or vertical distance in ft from the upstream reservoir level to the floor at the toe; H_a is the upstream approach velocity head; and y_1 is the depth of flow at the toe. Owing to the energy loss involved in the flow over the spillway, the actual velocity is always less than the theoretical value. The magnitude of the actual velocity depends mainly on the head on the spillway crest, the fall, the slope of the spillway surface, and the spillway-surface roughness.[1] By reasoning and experiments it is shown that the deviation of the actual velocity from its theoretical value becomes larger when the head is smaller and the fall is greater.

On the basis of experience, theoretical analysis, and a limited amount of experimental information obtained from prototype tests on Shasta and Grand Coulee dams, the U.S. Bureau of Reclamation [29] has studied the

[1] See [28] for further information.

Ref- Chow, 1959

relationship between the actual velocity and a theoretical value.[1] From the results of this study, a chart (Fig. 14-15) was prepared to show the actual velocity at the toe of spillways under various heads, falls, slopes from 1 on 0.6 to 1 on 0.8, and the condition of average surface roughness. It is felt that this chart is sufficiently accurate for preliminary-design

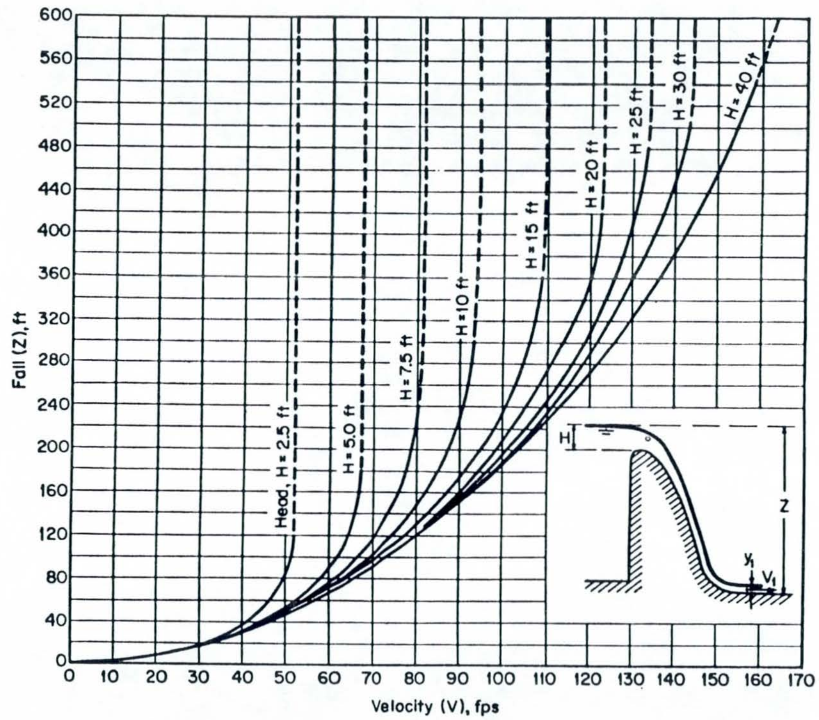

FIG. 14-15. Curves for determination of velocity at the toe of spillways with slopes 1 on 0.6 to 0.8.

purposes, although it can be refined by additional experimental information which may become available in the future.

Experiments by Bauer [30] indicate that friction losses in accelerating the flow down the face of a spillway may be considerably less than the normal friction loss in flow with well-developed turbulence. Therefore, the friction loss is not significant on steep slopes, but it would become important if the slope were small. For this reason, the chart in Fig.

[1] The theoretical velocity defined by the Bureau is $V_1 = \sqrt{2g(Z - 0.5H)}$.

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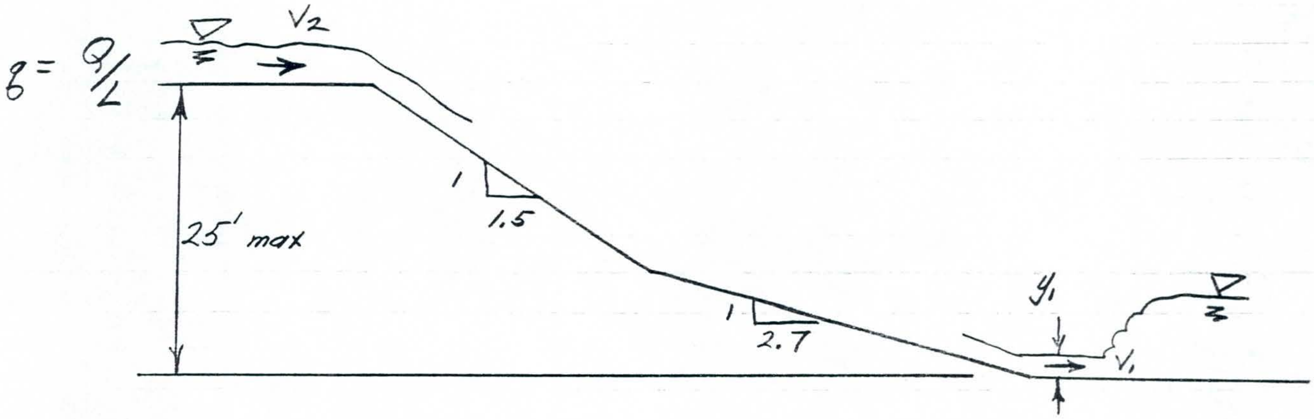
L = spillway length



Spillway design criteria:

1. $L = Q/2.5$ cfs/ft

   where Q = 100-yr, 6-hr peak discharge

2. spillway hydraulics must be adequate for $q = 5$ cfs/ft to account for discharge uncertainty and to provide flexibility for future drainage design

Analysis:

$$q = 5 = V_1 \cdot y_1$$

$$V_1 = \frac{1.49}{n} y_1^{2/3} S^{1/2}$$

$S = .37$

$n = 0.05$ for rough shotcrete, shallow flow and air entrainment on spillway face

$$V_1 = \frac{1.49}{.05} y_1^{2/3} .37^{1/2} = 18 y_1^{2/3}$$

y_1 ft	.1	.25	.33	.4	.45	.5
V_1 fps	3.8	7.1	8.6	9.7	10.5	11.3
q cfs/ft	.4	1.8	2.8	3.9	4.7	5.7

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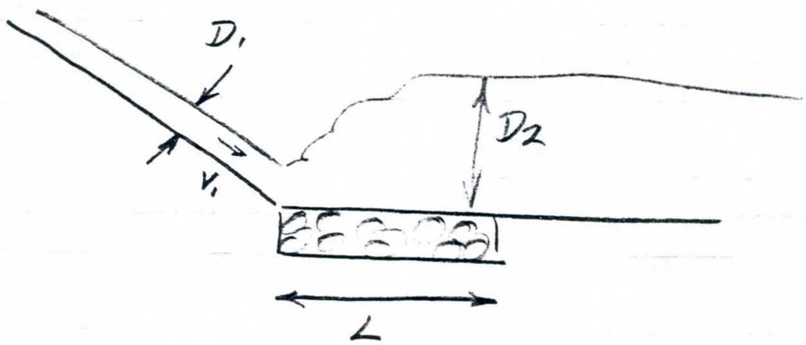
Hydraulic Jump Length



$D_1 = .45'$

$V_1 = 10$ fps

$$F_1 = \frac{V_1}{\sqrt{gD_1}} = 2.6$$

$$D_2/D_1 = \tfrac{1}{2}\left(\sqrt{1+8F_1^2} - 1\right)$$

$$= 3.25$$

$D_2 = 1.5'$

$L/D_2 \approx 5$

$L = 7.5'$

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Ref. - USBR Engineering Monograph No. 25

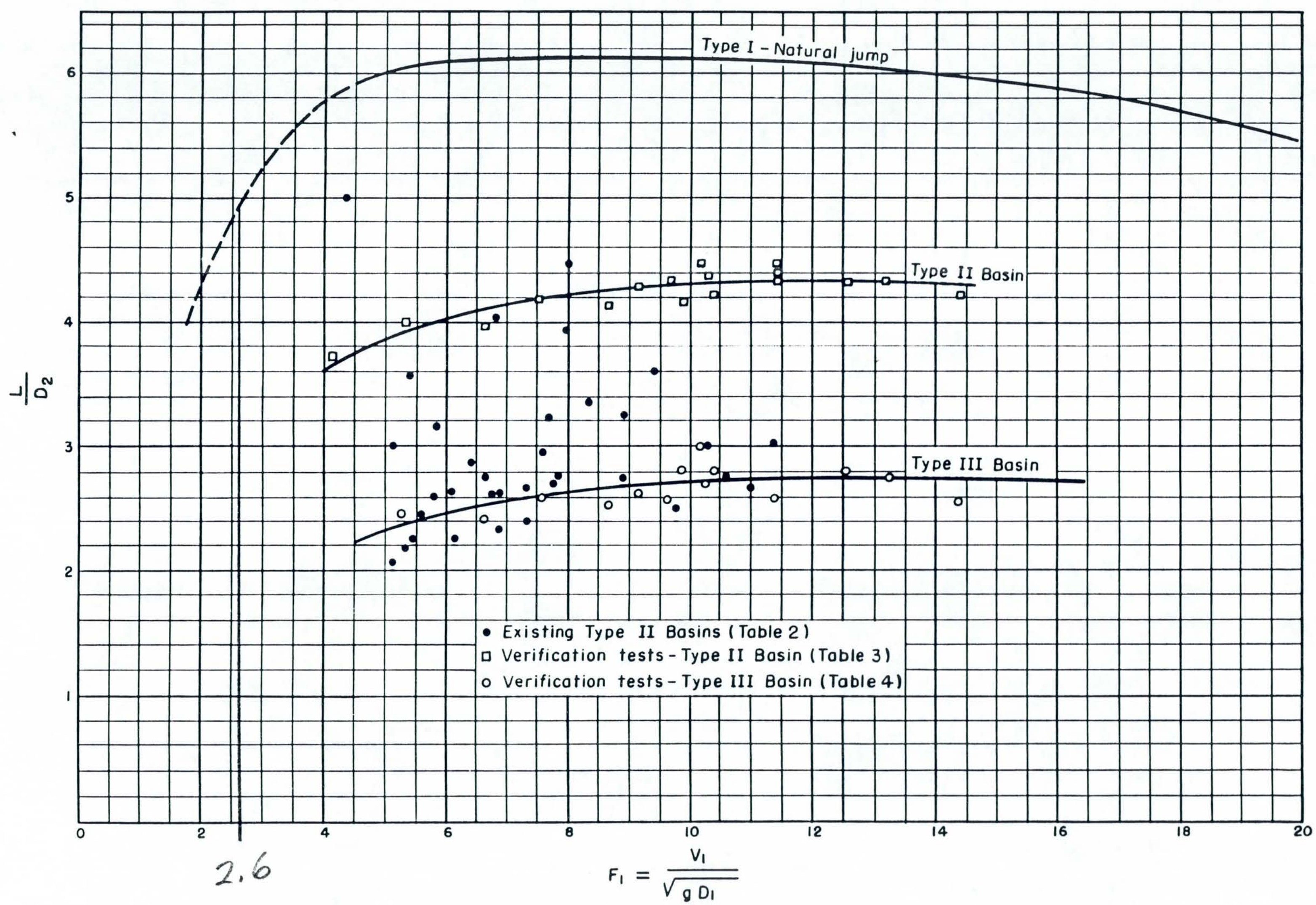

FIGURE 12.—*Length of jump on horizontal floor (Basins I, II, and III).*

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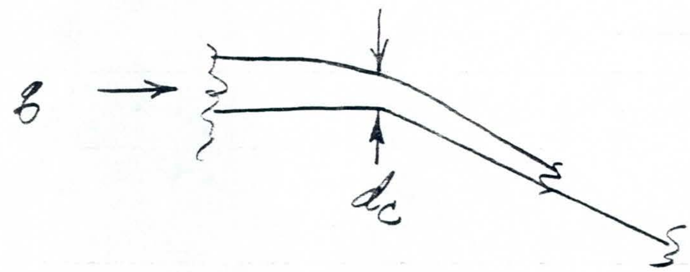
Depth of flow @ spillway crest



$$d_c = \left(\frac{q^2}{g}\right)^{1/3}$$

For $q = 2.5$ cfs/ft
$d_c = 0.6$ ft

For $q = 5$ cfs/ft
$d_c = 0.9$ ft

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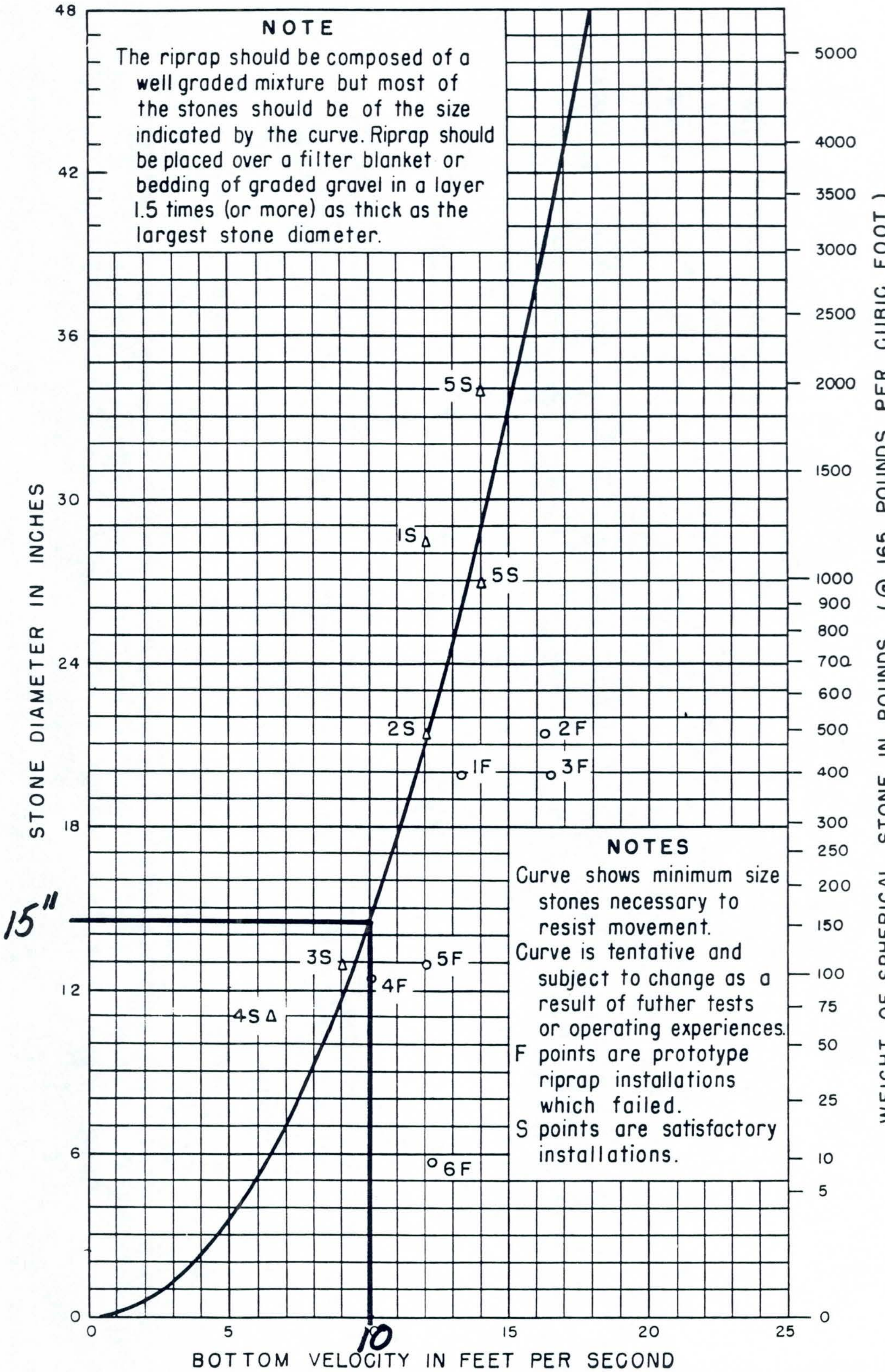

FIGURE 165.—*Curve to determine maximum stone size in riprap mixture.*

Ref. - USBR Engineering Monograph No. 25, 1963

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MITTAL

To: Mark Landsiedel, COS
Doug Cullinane, COS
Collis Lovely, COS
John Rodriguez, FCDMC
Scott Ogden, FCDMC
Marty Bressor, Pentacore
Chuck Gopperton, Stantech
Carlos Carriaga, Stantech

See Revised Report Dated 13 November

From: George V. Sabol

Date: 23 June 1998

Reference: **PR3B – PIMA FREEWAY BASIN SCOUR REPORT**
FILE: 28900082

A memorandum on Basin Scour Analysis and Toe-Down is provided for your review.

George V. Sabol, PhD, PE
Senior Associate
Water Resources Division

Attachment

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Memo

Stantech Consulting

To: File

From: George Sabol

Date: 22 June 1998

Reference: **PR3B – Pima Freeway Basin**
Basin Scour Analysis and Toe-Down
FILE: 28900082

Basin Configuration and Operation

The Pima Freeway Basin is a linear detention basin that extends from near Scottsdale Road on the west to the Union Hills Drive interchange with the Pima Freeway on the east. The basin is on the north side of the Pima Freeway and essentially parallel to it throughout that approximately 8,500 ft length. Benches with tree planters are located on the north bank where the basin depth exceeds 15 ft. The side-slope of the basin is 1V:1.5H on the south bank. On the north side, the side-slope is 1V:1.5H along the far west end where no tree planters are contained. The slope is also 1V:1.5H below each tree planter bench. The slope is about 1V:2.7H at rundown spillways that are between tree planters. The shotcrete lined side-slope extends 1 foot above the maximum water surface elevation (100-yr, 24-hr storm MWSE of approximately 1,608 ft).

At Hayden Road, the eastern and western portions of the basin are connected by a flow equalizer culvert structure. During certain basin inflow conditions, inflow to the eastern portion of the basin will pass through the equalizer culvert where flow will be detained in the western portion of the basin. Upon flow recession, detained water in the western portion of the basin will flow back through the equalizer culvert to the eastern portion of the basin. The outlet to the basin is double-barrel, 108 inch diameter concrete conduits that connect to the Hayden Road Conduits. Outflow passes through that outlet and is discharged to the USBR basin at the TPC Desert Golf Course on the east side of Hayden Road.

The basin is sized to route the 100-yr, 24-hr design storm and all lesser magnitude runoff events while maintaining a MWSE of 1,608 ft or less. The major source of inflow to the basin is the Pima Road Conduits at the far eastern end of the basin. Runoff from north of the basin, including the Grayhawk development, enters the basin through rundown spillways located at existing flow paths along the north side of the basin. The design peak discharges to the basin are summarized as follows:

Location of Basin Inflow	Peak Inflow, in cfs (100-yr, 24-hour storm)
Pima Road Conduits	3,571
Total Spillway Inflow in eastern basin	2,119
Total Spillway Inflow in western basin	1,060

The basin has a bottom width of 60 ft with invert slopes of 0.1% and a basin invert low elevation of 1,590 ft at the outlet works near Hayden Road. The bed of the basin will be unlined and landscaped with non-irrigated, native plants. As flow enters the basin and prior to the onset of detention ponding within the basin, the basin will function as a channel conveying potentially large discharges throughout major lengths of that basin. Scour potential is the greatest at the toe of each spillway and longitudinally along the length of the basin adjacent to the shotcrete lining, and scour protection along that lining is required to protect the bank lining from underscour and potential failure of that lining.

The most critical scour condition would be sustained inflow to the basin from an event of less than 100-yr, 24-hr magnitude wherein the basin would function more as a channel with little impoundment storage to cause a tailwater condition that would diminish the flow velocity. Actual inflow conditions that would affect tailwater conditions, flow velocity and flow depth are too complex and varied to fully anticipate. For this reason, scour potential is estimated for a range of basin flow from 1,000 to 3,000 cfs. Based on those results, reasonably prudent scour depths are estimated for which basin lining toe-down depth and scour protection are provided in the basin design.

Basin Hydraulics

Hydraulics (discharge-depth-velocity) relations for channel flow conditions are provided in Appendix A. Those relations are based on open channel flow with no backwater due to impoundment. Flow velocities exceed 3 fps for discharges exceeding about 550 cfs. Flow velocities of 5 fps can be expected for channel discharges exceeding 2,400 cfs. Considering the material comprising the unlined bed of the basin, the basin invert is susceptible to erosion and local scour.

Basin Bed-Material Size Gradation

Two sources of material size gradation data are available for the Pima Freeway Basin:

1. Geotechnical Investigation Report, Desert Greenbelt Phase 1 Channels, Pima Road and CAP Canal, Scottsdale, Arizona: AGRA Earth & Environmental, 25 August 1995.

2. Geotechnical Engineering Report, Freeway Basin and Outlet Conduit, Pima Three Basins Project : Ricker, Atkinson, McBee & Associates, Inc., 27 May 1998, and Supplement No. 1, 18 June 1998.

One size gradation sample is available for use from (1). It is identified as DB-1 @ 20-21.5 ft (see Appendix B for data). Two sets of size gradation data are available from (2), (see Appendix B for data). Percent retained on the #4 sieve and percent passing the #200 sieve are provided for 10 sample locations at various depths. That data are shown in Table 1 for the sample depths most closely representing the basin bed material. Two additional size gradation data are provided for sample locations 9 and 10, both at depths of 18 to 22 feet. Those gradation data are provided in Table 2.

TABLE 1

Basin bed-material size gradation data from RAM

Sample No.	Sample Depth, in feet	% Retained #4 Sieve (6.35 mm)	% Passing #200 sieve (0.127 mm)
(1)	(2)	(3)	(4)
1	10-15	22	15
2	10-15	3	63
3	15-20	15	20
4	10-15	9	34
5	15-20	14	31
6	10-15	13	23
7	15-20	13	29
8	10-15	16	23
9	15-20	14	30
10	10-15	16	22

The ten RAM (Table 1) samples show fairly consistent size gradation except Sample No. 2, which shows exceptionally fine material. Disregarding Sample No. 2 as nonrepresentative, the averages of column (3) and (4) are 15% and 25%, respectively. The size gradation data are presented in Table 2. Notice that the average for the nine RAM samples agrees favorably with the one AGRA sample. Notice that the two samples from RAM for sample locations No. 9 and No. 10 (Table 2) are for soil that is finer than is represented by either the one AGRA sample or the average of the other nine RAM samples. The AGRA size gradation data is used in the scour analysis as generally representing the basin bed material. Also notice that 67% of the material is sand or finer with about 24% in the silt and clay size fraction. Virtually all of the material is smaller than fine gravel. Clearly, this material is susceptible to erosion and scour even under moderate flow depths and velocities. It is also noted that the RAM Sample No. 2 indicates that zones of extremely fine sand, silt and clay may be exposed in the basin excavation.

TABLE 2

Basin bed-material size gradation

Sieve	Sieve Size, in mm	% Finer			
		AGRA[1]	RAM		
(1)	(2)	(3)	No. 9	No. 10	Avg[2]
200	0.075	24	43	33	25
100	0.150	29	49	38	---
50	0.300	34	---	---	---
40	0.425	37	57	47	---
30	0.600	41	---	---	---
16	1.18	54	69	62	---
10	2.00	67	---	---	---
8	2.36	71	81	77	---
4	4.75	92	93	93	---
0.25”	6.35	97	---	---	85
0.375”	9.5	100	100	100	---

Notes: 1 – From Reference (1)
2 – Average of nine samples from Reference (2)

Scour Analysis

Scour analyses are performed for discharges of 1,000, 2,000 and 3,000 cfs (see Appendix C). Results are presented in Table 3. Reference material for the scour equations and procedure are provided in Appendix D.

TABLE 3

Scour depth estimation

Scour Components	Discharge, in cfs		
	1,000	2,000	3,000
	Depth of Scour, in feet		
(1)	(2)	(3)	(4)
Local Scour[1]	1.76	2.88	3.80
Low-Flow Incisement[2]	1.0	1.0	1.0
Anti-Dune Scour	0.19	0.30	0.39
Total Scour	2.95	4.18	5.20

Notes: 1 – Local scour is average of scour depth estimated by four methods (see Appendix C)
2 – Assumed depth

Reasonable estimates of scour depth (particularly adjacent to the shotcrete bank lining estimated at 3 to more than 5 feet.

Bank Lining Toe-Down and Scour Protection

Based on the estimated scour depth of the native material that is likely to comprise the bed of the basin, the recommended toe-down of the shotcrete bank lining is 3.5 feet, and that would apply to all areas of the basin lining. On the south? ~~north~~ slope, riprap as shown in Figure 1 will be installed along the entire length of the basin. The riprap, as shown in Figure 1, provides scour protection if local scour along the riprap should exceed 3.5 feet. In that case, the loose riprap will tumble into the scour hole thus armoring that bank against scour migration toward the bank lining. It is recommended that the same toe-down and riprap scour protection be provided along the ~~south~~ North? bank lining due to its minimal cost and beneficial scour protection that it provides. That riprap also (D_{50} = 8 inch) provides an apron and additional scour protection for the spillways and any uncontrolled runoff that could pass over the north slope basin lining. At the spillways, the riprap will extend to the surface and the one foot of native material backfill will not be used.

This scour analysis is based on limited size gradation analyses and assumptions. Actual size gradation of that basin bed material, and its variability over that approximately 8,500 feet of basin length, will not be known until that basin is excavated. Based on that basin excavation, and appropriate size gradation analyses, it may be necessary to refine these scour analyses and scour protection facilities.

Basin Maintenance

The basin is to be inspected annually and after each significant runoff event. Scour holes must be backfilled with riprap or other competent material. Low-flow incisement must be monitored and corrective measures taken to avoid flow concentration along the bank lining.

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Senior Associate
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Attachment

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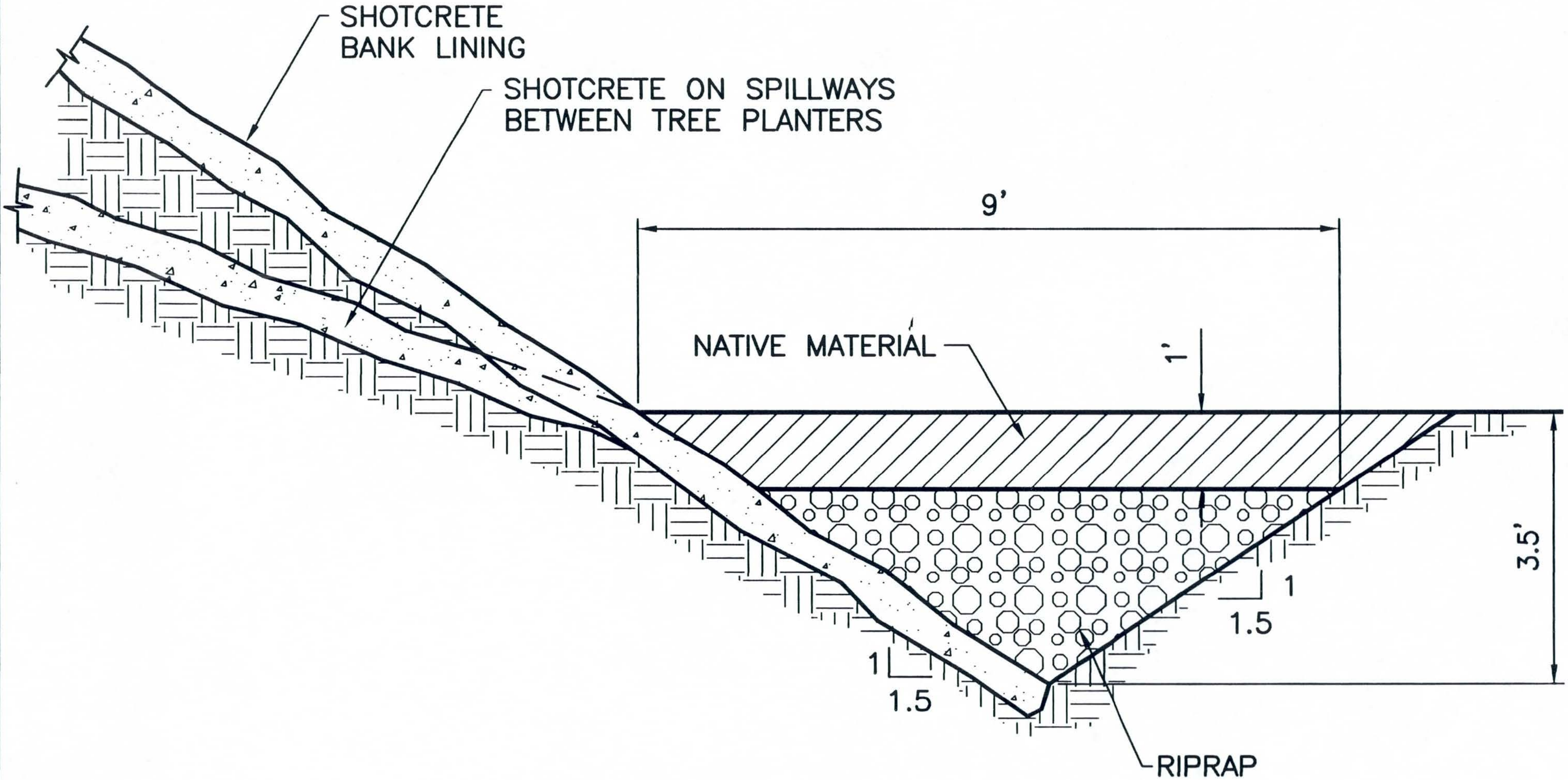

DETAIL —

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FIGURE 1

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Appendix A

Basin Flow Rating Table
Rating Table for Trapezoidal Channel

Project Description	
Project File	p:\28900051\flow master\outer loop basin\flowdept.fm2
Worksheet	channel flow depth
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Channel Depth

Constant Data		
Mannings Coefficient	0.030	
Channel Slope	0.001000	ft/ft
Left Side Slope	1.500000	H : V
Right Side Slope	1.500000	H : V
Bottom Width	60.00	ft

Input Data	Minimum	Maximum	Increment
Discharge	100.00	5,500.00	100.00 cfs

Rating Table

Discharge (cfs)	Depth (ft)	Velocity (ft/s)
100.00	1.04	1.57
200.00	1.57	2.04
300.00	2.00	2.38
400.00	2.37	2.65
500.00	2.71	2.88
600.00	3.02	3.08
700.00	3.31	3.25
800.00	3.59	3.41
900.00	3.85	3.56
1,000.00	4.09	3.69
1,100.00	4.33	3.82
1,200.00	4.56	3.94
1,300.00	4.78	4.05
1,400.00	4.99	4.15
1,500.00	5.20	4.25
1,600.00	5.40	4.35
1,700.00	5.60	4.44
1,800.00	5.79	4.53
1,900.00	5.97	4.61
2,000.00	6.16	4.69
2,100.00	6.33	4.77
2,200.00	6.51	4.84

Basin Flow Rating Table
Rating Table for Trapezoidal Channel

Rating Table

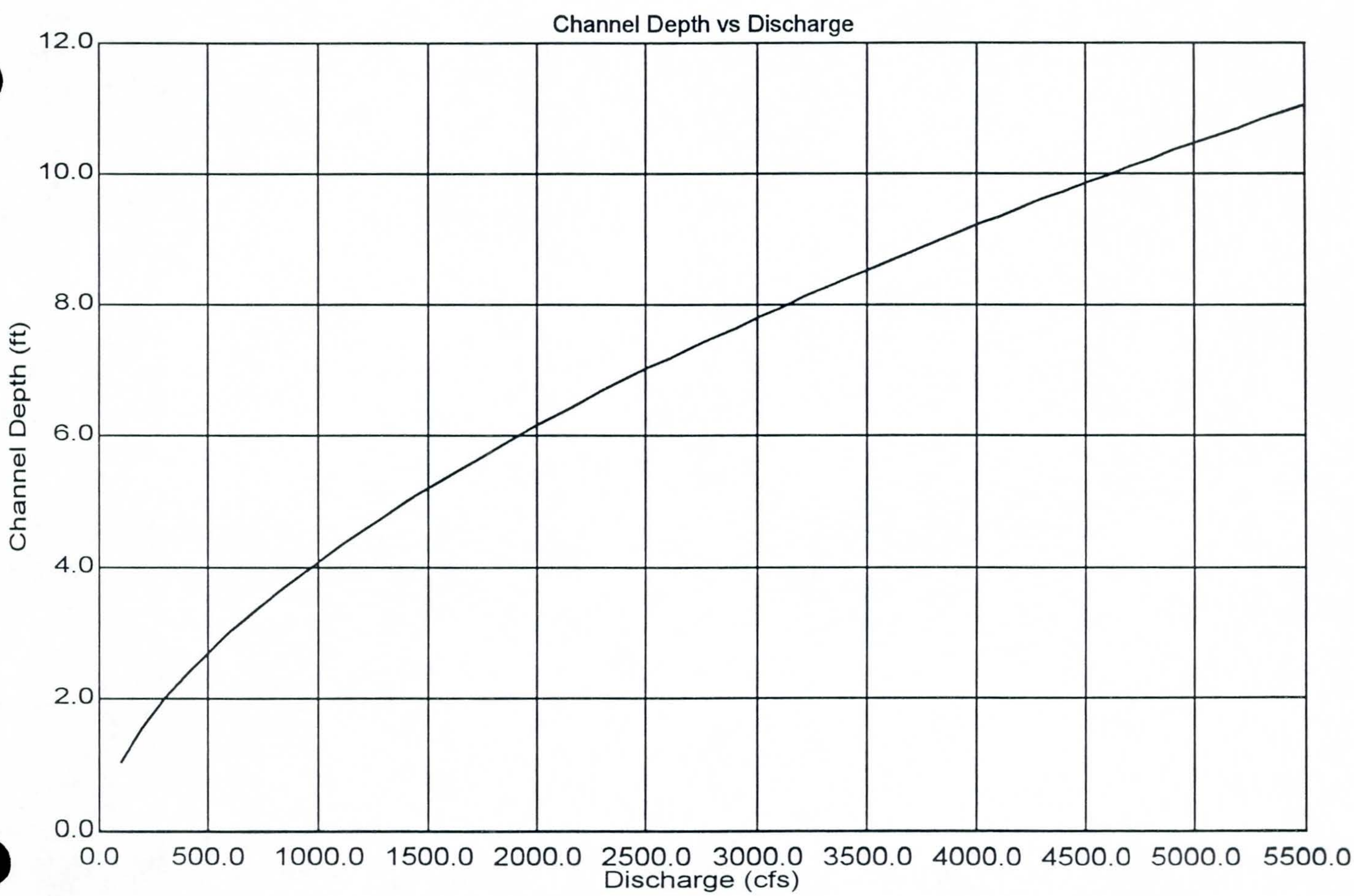
Discharge (cfs)	Depth (ft)	Velocity (ft/s)
2,300.00	6.68	4.92
2,400.00	6.85	4.99
2,500.00	7.01	5.06
2,600.00	7.17	5.12
2,700.00	7.33	5.19
2,800.00	7.49	5.25
2,900.00	7.64	5.31
3,000.00	7.80	5.37
3,100.00	7.95	5.43
3,200.00	8.09	5.48
3,300.00	8.24	5.54
3,400.00	8.38	5.59
3,500.00	8.52	5.64
3,600.00	8.66	5.69
3,700.00	8.80	5.74
3,800.00	8.94	5.79
3,900.00	9.07	5.84
4,000.00	9.20	5.89
4,100.00	9.34	5.93
4,200.00	9.47	5.98
4,300.00	9.59	6.03
4,400.00	9.72	6.07
4,500.00	9.85	6.11
4,600.00	9.97	6.15
4,700.00	10.09	6.20
4,800.00	10.22	6.24
4,900.00	10.34	6.28
5,000.00	10.46	6.32
5,100.00	10.58	6.36
5,200.00	10.69	6.40
5,300.00	10.81	6.43
5,400.00	10.93	6.47
5,500.00	11.04	6.51

Basin Normal Depth
Plotted Curves for Trapezoidal Channel

Project Description	
Project File	p:\28900051\flow master\outer loop basin\flowdept.fm2
Worksheet	channel flow depth
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Channel Depth

Constant Data		
Mannings Coefficient	0.030	
Channel Slope	0.001000 ft/ft	
Left Side Slope	1.500000 H : V	
Right Side Slope	1.500000 H : V	
Bottom Width	60.00	ft

Input Data	Minimum	Maximum	Increment
Discharge	100.00	5,500.00	100.00 cfs



Basin Flow at 500 CFS
Worksheet for Trapezoidal Channel

Project Description	
Project File	p:\28900051\flow master\outer loop basin\flowdept.fm2
Worksheet	channel flow depth
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Channel Depth

Input Data		
Mannings Coefficient	0.030	
Channel Slope	0.001000	ft/ft
Left Side Slope	1.500000	H : V
Right Side Slope	1.500000	H : V
Bottom Width	60.00	ft
Discharge	500.00	cfs

Results		
Depth	2.71	ft
Flow Area	173.76	ft²
Wetted Perimeter	69.78	ft
Top Width	68.14	ft
Critical Depth	1.28	ft
Critical Slope	0.012405	ft/ft
Velocity	2.88	ft/s
Velocity Head	0.13	ft
Specific Energy	2.84	ft
Froude Number	0.32	
Flow is subcritical.		

Basin Flow at 1000 CFS
Worksheet for Trapezoidal Channel

Project Description	
Project File	p:\28900051\flow master\outer loop basin\flowdept.fm2
Worksheet	channel flow depth
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Channel Depth

Input Data		
Mannings Coefficient	0.030	
Channel Slope	0.001000	ft/ft
Left Side Slope	1.500000	H : V
Right Side Slope	1.500000	H : V
Bottom Width	60.00	ft
Discharge	1,000.00	cfs

Results		
Depth	4.09	ft
Flow Area	270.73	ft^2
Wetted Perimeter	74.76	ft
Top Width	72.28	ft
Critical Depth	2.02	ft
Critical Slope	0.010805	ft/ft
Velocity	3.69	ft/s
Velocity Head	0.21	ft
Specific Energy	4.31	ft
Froude Number	0.34	
Flow is subcritical.		

Basin Flow at 1500 CFS
Worksheet for Trapezoidal Channel

Project Description	
Project File	p:\28900051\flow master\outer loop basin\flowdept.fm2
Worksheet	channel flow depth
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Channel Depth

Input Data		
Mannings Coefficient	0.030	
Channel Slope	0.001000	ft/ft
Left Side Slope	1.500000	H : V
Right Side Slope	1.500000	H : V
Bottom Width	60.00	ft
Discharge	1,500.00	cfs

Results		
Depth	5.20	ft
Flow Area	352.56	ft^2
Wetted Perimeter	78.75	ft
Top Width	75.60	ft
Critical Depth	2.63	ft
Critical Slope	0.009999	ft/ft
Velocity	4.25	ft/s
Velocity Head	0.28	ft
Specific Energy	5.48	ft
Froude Number	0.35	
Flow is subcritical.		

Basin Flow at 2000 CFS
Worksheet for Trapezoidal Channel

Project Description	
Project File	p:\28900051\flow master\outer loop basin\flowdept.fm2
Worksheet	channel flow depth
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Channel Depth

Input Data		
Mannings Coefficient	0.030	
Channel Slope	0.001000	ft/ft
Left Side Slope	1.500000	H : V
Right Side Slope	1.500000	H : V
Bottom Width	60.00	ft
Discharge	2,000.00	cfs

Results		
Depth	6.16	ft
Flow Area	426.22	ft²
Wetted Perimeter	82.20	ft
Top Width	78.47	ft
Critical Depth	3.17	ft
Critical Slope	0.009480	ft/ft
Velocity	4.69	ft/s
Velocity Head	0.34	ft
Specific Energy	6.50	ft
Froude Number	0.35	
Flow is subcritical.		

Basin Flow at 2500 CFS
Worksheet for Trapezoidal Channel

Project Description	
Project File	p:\28900051\flow master\outer loop basin\flowdept.fm2
Worksheet	channel flow depth
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Channel Depth

Input Data		
Mannings Coefficient	0.030	
Channel Slope	0.001000	ft/ft
Left Side Slope	1.500000	H : V
Right Side Slope	1.500000	H : V
Bottom Width	60.00	ft
Discharge	2,500.00	cfs

Results		
Depth	7.01	ft
Flow Area	494.52	ft^2
Wetted Perimeter	85.28	ft
Top Width	81.04	ft
Critical Depth	3.66	ft
Critical Slope	0.009106	ft/ft
Velocity	5.06	ft/s
Velocity Head	0.40	ft
Specific Energy	7.41	ft
Froude Number	0.36	
Flow is subcritical.		

Basin Flow at 3000 CFS
Worksheet for Trapezoidal Channel

Project Description	
Project File	p:\28900051\flow master\outer loop basin\flowdept.fm2
Worksheet	channel flow depth
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Channel Depth

Input Data		
Mannings Coefficient	0.030	
Channel Slope	0.001000	ft/ft
Left Side Slope	1.500000	H : V
Right Side Slope	1.500000	H : V
Bottom Width	60.00	ft
Discharge	3,000.00	cfs

Results		
Depth	7.80	ft
Flow Area	558.93	ft²
Wetted Perimeter	88.11	ft
Top Width	83.39	ft
Critical Depth	4.12	ft
Critical Slope	0.008819	ft/ft
Velocity	5.37	ft/s
Velocity Head	0.45	ft
Specific Energy	8.24	ft
Froude Number	0.37	
Flow is subcritical.		

Basin Flow at 4000 CFS
Worksheet for Trapezoidal Channel

Project Description	
Project File	p:\28900051\flow master\outer loop basin\flowdept.fm2
Worksheet	channel flow depth
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Channel Depth

Input Data		
Mannings Coefficient	0.030	
Channel Slope	0.001000	ft/ft
Left Side Slope	1.500000	H : V
Right Side Slope	1.500000	H : V
Bottom Width	60.00	ft
Discharge	4,000.00	cfs

Results		
Depth	9.20	ft
Flow Area	679.28	ft²
Wetted Perimeter	93.18	ft
Top Width	87.61	ft
Critical Depth	4.95	ft
Critical Slope	0.008395	ft/ft
Velocity	5.89	ft/s
Velocity Head	0.54	ft
Specific Energy	9.74	ft
Froude Number	0.37	
Flow is subcritical.		

Basin Flow at 4500 CFS
Worksheet for Trapezoidal Channel

Project Description	
Project File	p:\28900051\flow master\outer loop basin\flowdept.fm2
Worksheet	channel flow depth
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Channel Depth

Input Data		
Mannings Coefficient	0.030	
Channel Slope	0.001000	ft/ft
Left Side Slope	1.500000	H : V
Right Side Slope	1.500000	H : V
Bottom Width	60.00	ft
Discharge	4,500.00	cfs

Results		
Depth	9.85	ft
Flow Area	736.22	ft²
Wetted Perimeter	95.50	ft
Top Width	89.54	ft
Critical Depth	5.34	ft
Critical Slope	0.008232	ft/ft
Velocity	6.11	ft/s
Velocity Head	0.58	ft
Specific Energy	10.43	ft
Froude Number	0.38	
Flow is subcritical.		

Basin Flow at 5000 CFS
Worksheet for Trapezoidal Channel

Project Description	
Project File	p:\28900051\flow master\outer loop basin\flowdept.fm2
Worksheet	channel flow depth
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Channel Depth

Input Data		
Mannings Coefficient	0.030	
Channel Slope	0.001000	ft/ft
Left Side Slope	1.500000	H : V
Right Side Slope	1.500000	H : V
Bottom Width	60.00	ft
Discharge	5,000.00	cfs

Results		
Depth	10.46	ft
Flow Area	791.45	ft^2
Wetted Perimeter	97.70	ft
Top Width	91.37	ft
Critical Depth	5.71	ft
Critical Slope	0.008091	ft/ft
Velocity	6.32	ft/s
Velocity Head	0.62	ft
Specific Energy	11.08	ft
Froude Number	0.38	
Flow is subcritical.		

Basin Flow at 5500 CFS
Worksheet for Trapezoidal Channel

Project Description	
Project File	p:\28900051\flow master\outer loop basin\flowdept.fm2
Worksheet	channel flow depth
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Channel Depth

Input Data		
Mannings Coefficient	0.030	
Channel Slope	0.001000	ft/ft
Left Side Slope	1.500000	H : V
Right Side Slope	1.500000	H : V
Bottom Width	60.00	ft
Discharge	5,500.00	cfs

Results		
Depth	11.04	ft
Flow Area	845.19	ft^2
Wetted Perimeter	99.80	ft
Top Width	93.12	ft
Critical Depth	6.06	ft
Critical Slope	0.007968	ft/ft
Velocity	6.51	ft/s
Velocity Head	0.66	ft
Specific Energy	11.70	ft
Froude Number	0.38	
Flow is subcritical.		

Appendix B

AGRA Earth & Environmental, Inc.

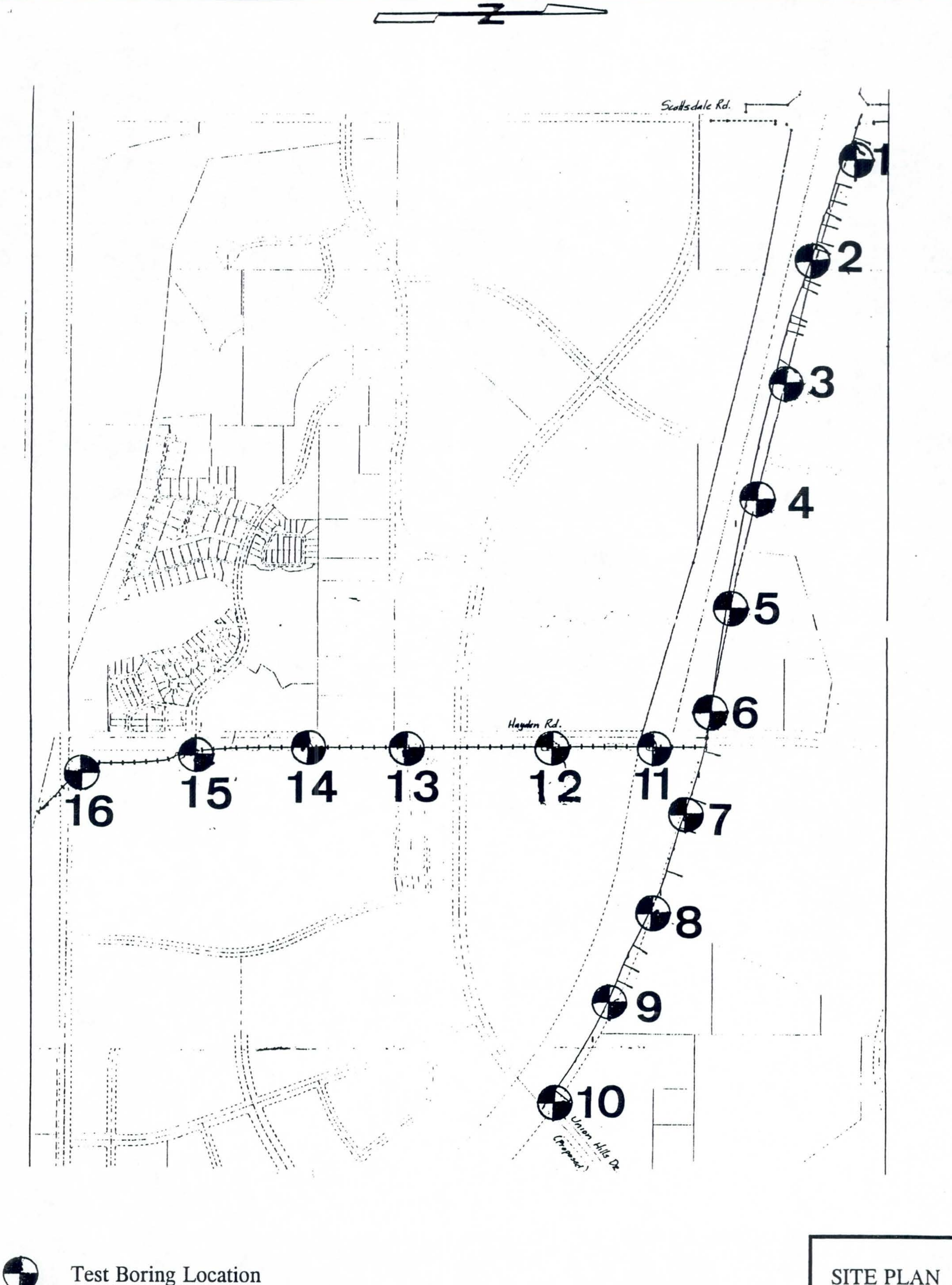
PROJECT:	DESERT GREENBELT - PHASE I	JOB NO:	E95-86
LOCATION:	PIMA ROAD BETWEEN BELL & PINNACLE PEAK	WORK ORDER NO:	6
		DATE SAMPLED:	07-21-95

MECHANICAL SIEVE ANALYSIS
GROUP SYMBOL, USCS (ASTM D-2487)

SIEVE SIZES

				Silt or	SAND								GRAVEL								
				Clay	Fine			Medium			Coarse		Fine				Coarse				
Location & Depth	USCS	LL	PI	#200	#100	#50	#40	#30	#16	#10	#8	#4	1/4"	3/8"	1/2"	3/4"	1"	1 1/2"	2"	3"	Lab #
				PERCENT PASSING BY WEIGHT																	
DB-1 @ 5 - 6'	CL	30	9	78	87	91	93	94	96	97	97	99	99	99	100	100	100	100	100	100	215
DB-1 @ 20 - 21.5'	SM	40	14	24	29	34	37	41	54	67	71	92	97	100	100	100	100	100	100	100	218
RP-1 @ 0 - 1.5'	SM	NV	NP	24	28	35	39	44	55	66	71	87	93	97	100	100	100	100	100	100	195
RP-3 @ 10 - 11.5'	SC	30	10	17	22	30	35	40	54	66	71	86	92	98	99	100	100	100	100	100	202
RP-4 @ 5 - 6.5'	SM	NV	NP	19	25	34	41	47	62	74	78	92	95	97	100	100	100	100	100	100	205
RP-5 @ 0 - 1.5'	SC	28	9	26	32	42	48	55	71	82	85	94	97	99	100	100	100	100	100	100	208
RP-6 @ 5 - 6.5'	SC	30	10	21	24	30	33	38	50	64	70	89	95	100	100	100	100	100	100	100	211
RP-6 @ 13 - 15'	GP-GC	33	16	8.2	9	11	13	15	21	29	33	47	54	64	71	85	98	100	100	100	213

AGRA Earth & Environmental



N
Scottsdale Rd.
Hayden Rd.
Union Hills Dr.
(Proposed)
1
2
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9
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11
12
13
14
15
16
Test Boring Location

SITE PLAN

LABORATORY TEST RESULTS

Date: 13-May-98

SAMPLE SOURCE: As noted below

TESTING PERFORMED: Percent Passing No. 200 Sieve, Atterberg Limits (ASTM D1140, D4318)

SAMPLED BY: RAM/Miller

RESULTS:

Sample Source	Percent Retained No. 4 Sieve	Percent Passing No. 200 Sieve	Liquid Limit	Plasticity Index
1 @ 5'-10'	18	32	27	7
1 @ 15'-20'	22	15	29	11
2 @ 0'-5'	14	44	28	10
2 @ 10'-15'	3	63	39	21
3 @ 5'-10'	18	27	26	8
3 @ 15'-20'	15	20	42	26
4 @ 0'-5'	12	30	24	5
4 @ 10'-15'	9	34	29	9
5 @ 5'-10'	15	26	26	8
5 @ 15'-20'	14	31	38	20
6 @ 0'-5'	8	24	20	4
6 @ 10'-15'	13	23	29	14
7 @ 5'-10'	13	31	22	5
7 @ 15'-20'	13	29	27	7
8 @ 0'-5'	18	21	20	3
8 @ 10'-15'	16	23	26	9

LABORATORY TEST RESULTS

Date: 13-May-98

SAMPLE SOURCE: As noted below

TESTING PERFORMED: Percent Passing No. 200 Sieve, Atterberg Limits (ASTM D1140, D4318)

SAMPLED BY: RAM/Miller

RESULTS:

Sample Source	Percent Retained No. 4 Sieve	Percent Passing No. 200 Sieve	Liquid Limit	Plasticity Index
9 @ 5'-10'	18	22	21	4
9 @ 15'-20'	14	30	28	10
10 @ 0'-5'	25	13	N/A	Non-Plastic
10 @ 10'-15'	16	22	26	6
11 @ 0'-5'	11	37	25	6
11 @ 5'-10'	11	30	23	3
12 @ 0'-5'	10	44	27	8
12 @ 10'-15'	25	31	36	13
13 @ 0'-5'	29	19	33	17
13@ 15'-20'	17	22	38	21
14 @ 0'-5'	10	42	26	9
14 @ 5'-10'	11	29	24	7
15 @ 0'-5'	11	41	31	13
15 @ 10'-15'	17	29	39	22
16 @ 0'-5'	15	35	27	11
16 @ 15'-20'	7	33	29	12

RICKER • ATKINSON • McBEE & ASSOCIATES, INC.

Geotechnical Engineering • Construction Materials Testing

R·A·M

Stantech Consulting
7776 Pointe Parkway West, Suite 290
Phoenix, Arizona 85044

June 18, 1998

Attention: Chuck Gopperton, P.E.

Subject: Geotechnical Engineering Report
Freeway Basin and Outlet Conduit
Pima 3 Basins Project
Loop 101 - Scottsdale Road to Union Hills Drive
Hayden Road - Loop 101 to Bell Road
Scottsdale, Arizona

R.A.M. Project No. G02281
Supplement No. 1

At your request, this firm has reviewed the geotechnical report for the subject project with respect to:

1. Soil gradation at the east end of basin for use in Stantech's scour analysis.

2. Review and comments on the flat edge drain strip and weep holes behind the liner in the basin sides.

3. The use of cast-in-place concrete pipe in the outlet conduit along Hayden Road.

Additional tests have been completed on soils samples from Test Borings 9 and 10 at the east end of the basin and the results are:

Percent Passing (Sieve Size)

Location	No. 200	No. 100	No.40	No. 16	No. 8	No. 4	No. 3/4"
9 @ 18' to 22'	43	49	57	69	81	93	100
10 @ 18' to 22'	33	38	47	62	77	93	100

Sands are angular to subangular.

The following drawings were reviewed with respect to the drainage system behind the liner and in the planter areas. The following comments are presented for your use.

2105 South Hardy Drive, Suite 13, Tempe, AZ 85282 • Telephone (602) 921-8100 • Facsimile (602) 921-4081

Sheet No.	Comments
D1; Section C	1. Since rip-rap spillway will be subjected to flows over the surface, will a geotextile filter fabric be required at the soil rip-rap interface to prevent piping of the soil into the rip-rap. 2. The rip-rap will fill with water. A way to drain this zone should be provided, such as using weep pipes which extend through the lining-soil-turndown or under the turndown.
D3; Section A	1. Planter should have a PVC or gunite bottom. 2. The planter should have weep pipes which drain the bottom of planter through turndown-soil-lining.
D5; Section A	1. The soil end of the outlet coupling (weep pipes) should be either covered with filter fabric or preferably terminated on and surrounded by the 12" flat edge drain. 2. Modify section or add a new section so that the drainage system extends down behind the lining below the planter.
D5; Plan	1. Limits of polyethylene 8 mil moisture barrier as shown would extend around the bottom and sides of the turndown along the top of the basin or planter and the turndown on the uphill side of the planter. This layer should be terminated at the turndown.

The use of cast-in-place pipe generally requires excavations be accomplished with a special rounded bucket. Due to zones of heavy cementation, excavation with this kind of bucket may be slow and difficult to accomplish and could require excavating with a conventional bucket or rock bucket below proposed grade, backfilling the lower half of the pipe zone and re-excavating with the special rounded bucket. In addition, some relatively clean sand lenses may be encountered which will not maintain the round bottom configuration before or during slip form placement of the concrete.

If you have any questions, please do not hesitate to call. This supplement should be attached to and made a part of the original report.

Respectfully submitted,

RICKER, ATKINSON, MCBEE & ASSOCIATES, INC.

Registered Professional Engineer (Civil)
CERTIFICATE NO.
10971
KENNETH L.
RICKER
Date Signed 6/11/98
ARIZONA U.S.A.

By: Kenneth L. Ricker, P.E.

/nk
Copies to: Addressee (5)

Appendix C

PROJECT NAME: Outer Loop Basin
PROJECT NO: 28900082
ANALYSIS BY: Stantech Consulting - cvg
DATE: 6-6-98

LOCAL SCOUR ANALYSIS
Types A and B - Natural Channel for Restriction and Bends and Bankline Structures

APPLICATIONS INCLUDE:

Type A	Type B
(a) Siphon Crossing	(e) Abutments to Bridge/Siphon Crossings
(b) Buried Pipeline	(f) Bank Slope Protection (Riprap)
(c) Nat'l Bank Stability	(g) Spur Dikes, Groins, etc.
(d) One-Span Bridge Waterway	(h) Pumping Plants
	(i) Canal Headworks

HYDRAULIC DATA

Discharge (cfs):	1000
Mean Depth (ft):	4.09
Mean Velocity (fps):	3.69
Unit Discharge (cfs/ft):	15.12
Threshold Velocity (fps):	2.46

SEDIMENT DATA

Material Grain Size, D50 (mm):	1.002

REACH INFORMATION

Straight Reach

RESULTS OF ANALYSIS

Methods	Scour Depth	
(a) USBR I Equation	NOT APPLICABLE	
(b) Lacey Equation	0.973 ft	0.297 m
(c) Blench Equation	3.010 ft	0.918 m
(d) USBR II Equation	1.023 ft	0.312 m
(e) Neill Equation	2.045 ft	0.623 m

COMMENTS:

REFERENCES:
(1) Pemberton, E.L. and J. M. Lara, Computing Degradation and Local Scour, Technical Guideline for Bureau of Reclamation, January 1984.

PROJECT NAME: Outer Loop Basin
PROJECT NO: 28900082
ANALYSIS BY: Stantech Consulting - cvg
DATE: 6-6-98

TOTAL SCOUR ANALYSIS

HYDRAULIC DATA

Discharge (cfs):	1000	Mean Velocity (ft/ft):	3.69
Bottom Width (ft):	60.0	Kinematic Visc.(sq.ft/s):	0.0000105
Mean Depth (ft):	4.09	Energy Slope (ft/ft):	0.00081
Side Slope (H:V):	1.50	Manning's n-Value:	0.035

SEDIMENT DATA

Material Grain Size, D16 (mm):	
Material Grain Size, D50 (mm):	1.0020
Material Grain Size, D84 (mm):	
Material Grain Size, D90 (mm):	
Unit Weight (pcf):	165.0000
Gradation Coefficient:	

RESULTS OF ANALYSIS

SCOUR COMPONENTS	DEPTH (ft)
(a) Local Scour	1.7628
(b) General Scour	0.0000
(c) Long-Term Scour	
(d) Low-Flow Incisement	1.0000
(e) Anti-Dune Scour	0.1860
(f) Bend Scour	
(g) Factor of Safety	
Total Scour Depth	2.9488

COMMENTS:

REFERENCES:

(1) Pemberton, E.L. and Lara, J.M. (1984), Computing Degradation and Local Scour Technical Guideline, Bureau of Reclamation, Engineering and Research Center Denver, Colorado, January 1984, pp. 48.

(2) Resource Consultants & Engineers, Inc. (1994), Sediment and Erosion Design Guide, prepared for Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA), RCE Ref. No. 90-560, November 1994.

(3) Simons, Li & Associates, Inc.,(1989), Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona, prepared for City of Tucson December 1989.

PROJECT NAME: Outer Loop Basin
PROJECT NO: 28900082
ANALYSIS BY: Stantech Consulting - cvg
DATE: 6-6-98

LOCAL SCOUR ANALYSIS

Types A and B - Natural Channel for Restriction and Bends and Bankline Structures

APPLICATIONS INCLUDE:

Type A	Type B
(a) Siphon Crossing	(e) Abutments to Bridge/Siphon Crossings
(b) Buried Pipeline	(f) Bank Slope Protection (Riprap)
(c) Nat'l Bank Stability	(g) Spur Dikes, Groins, etc.
(d) One-Span Bridge Waterway	(h) Pumping Plants
	(i) Canal Headworks

HYDRAULIC DATA

Discharge (cfs):	2000
Mean Depth (ft):	6.16
Mean Velocity (fps):	4.69
Unit Discharge (cfs/ft):	28.89
Threshold Velocity (fps):	2.81

SEDIMENT DATA

Material Grain Size, D50 (mm): 1.002

REACH INFORMATION

Straight Reach

RESULTS OF ANALYSIS

Methods	Scour Depth	
(a) USBR I Equation	NOT APPLICABLE	
(b) Lacey Equation	1.225 ft	0.373 m
(c) Blench Equation	4.635 ft	1.413 m
(d) USBR II Equation	1.540 ft	0.470 m
(e) Neill Equation	4.121 ft	1.256 m

COMMENTS:

REFERENCES:

(1) Pemberton, E.L. and J. M. Lara, Computing Degradation and Local Scour, Technical Guideline for Bureau of Reclamation, January 1984.

PROJECT NAME: Outer Loop Basin
PROJECT NO: 28900082
ANALYSIS BY: Stantech Consulting - cvg
DATE: 6-6-98

TOTAL SCOUR ANALYSIS

HYDRAULIC DATA

Discharge (cfs):	2000	Mean Velocity (ft/ft):	4.69
Bottom Width (ft):	60.0	Kinematic Visc.(sq.ft/s):	0.0000105
Mean Depth (ft):	6.16	Energy Slope (ft/ft):	0.00081
Side Slope (H:V):	1.50	Manning's n-Value:	0.035

SEDIMENT DATA

Material Grain Size, D16 (mm):	
Material Grain Size, D50 (mm):	1.0020
Material Grain Size, D84 (mm):	
Material Grain Size, D90 (mm):	
Unit Weight (pcf):	165.0000
Gradation Coefficient:	

RESULTS OF ANALYSIS

SCOUR COMPONENTS	DEPTH (ft)
(a) Local Scour	2.8802
(b) General Scour	0.0000
(c) Long-Term Scour	
(d) Low-Flow Incisement	1.0000
(e) Anti-Dune Scour	0.3004
(f) Bend Scour	
(g) Factor of Safety	
Total Scour Depth	4.1806

COMMENTS:

REFERENCES:

(1) Pemberton, E.L. and Lara, J.M. (1984), Computing Degradation and Local Scour Technical Guideline, Bureau of Reclamation, Engineering and Research Center Denver, Colorado, January 1984, pp. 48.
(2) Resource Consultants & Engineers, Inc. (1994), Sediment and Erosion Design Guide, prepared for Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA), RCE Ref. No. 90-560, November 1994.
(3) Simons, Li & Associates, Inc.,(1989), Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona, prepared for City of Tucson December 1989.

PROJECT NAME: Outer Loop Basin
PROJECT NO: 28900082
ANALYSIS BY: Stantech Consulting - cvg
DATE: 6-6-98

LOCAL SCOUR ANALYSIS
Types A and B - Natural Channel for Restriction and Bends and Bankline Structures

APPLICATIONS INCLUDE:

Type A	Type B
(a) Siphon Crossing	(e) Abutments to Bridge/Siphon Crossings
(b) Buried Pipeline	(f) Bank Slope Protection (Riprap)
(c) Nat'l Bank Stability	(g) Spur Dikes, Groins, etc.
(d) One-Span Bridge Waterway	(h) Pumping Plants
	(i) Canal Headworks

HYDRAULIC DATA

Discharge (cfs):	3000
Mean Depth (ft):	7.80
Mean Velocity (fps):	5.37
Unit Discharge (cfs/ft):	41.80
Threshold Velocity (fps):	3.05

SEDIMENT DATA

Material Grain Size, D50 (mm):	1.002

REACH INFORMATION

Straight Reach

RESULTS OF ANALYSIS

Methods	Scour Depth	
(a) USBR I Equation	NOT APPLICABLE	
(b) Lacey Equation	1.403 ft	0.428 m
(c) Blench Equation	5.929 ft	1.808 m
(d) USBR II Equation	1.950 ft	0.595 m
(e) Neill Equation	5.933 ft	1.809 m

COMMENTS:

REFERENCES:

(1) Pemberton, E.L. and J. M. Lara, Computing Degradation and Local Scour, Technical Guideline for Bureau of Reclamation, January 1984.

PROJECT NAME: Outer Loop Basin
PROJECT NO: 28900082
ANALYSIS BY: Stantech Consulting - cvg
DATE: 6-6-98

TOTAL SCOUR ANALYSIS

HYDRAULIC DATA

Discharge (cfs):	3000	Mean Velocity (ft/ft):	5.37
Bottom Width (ft):	60.0	Kinematic Visc.(sq.ft/s):	0.0000105
Mean Depth (ft):	7.80	Energy Slope (ft/ft):	0.00081
Side Slope (H:V):	1.50	Manning's n-Value:	0.035

SEDIMENT DATA

Material Grain Size, D16 (mm):	
Material Grain Size, D50 (mm):	1.0020
Material Grain Size, D84 (mm):	
Material Grain Size, D90 (mm):	
Unit Weight (pcf):	165.0000
Gradation Coefficient:	

RESULTS OF ANALYSIS

SCOUR COMPONENTS	DEPTH (ft)
(a) Local Scour	3.8038
(b) General Scour	0.0000
(c) Long-Term Scour	
(d) Low-Flow Incisement	1.0000
(e) Anti-Dune Scour	0.3939
(f) Bend Scour	
(g) Factor of Safety	
Total Scour Depth	5.1977

COMMENTS:

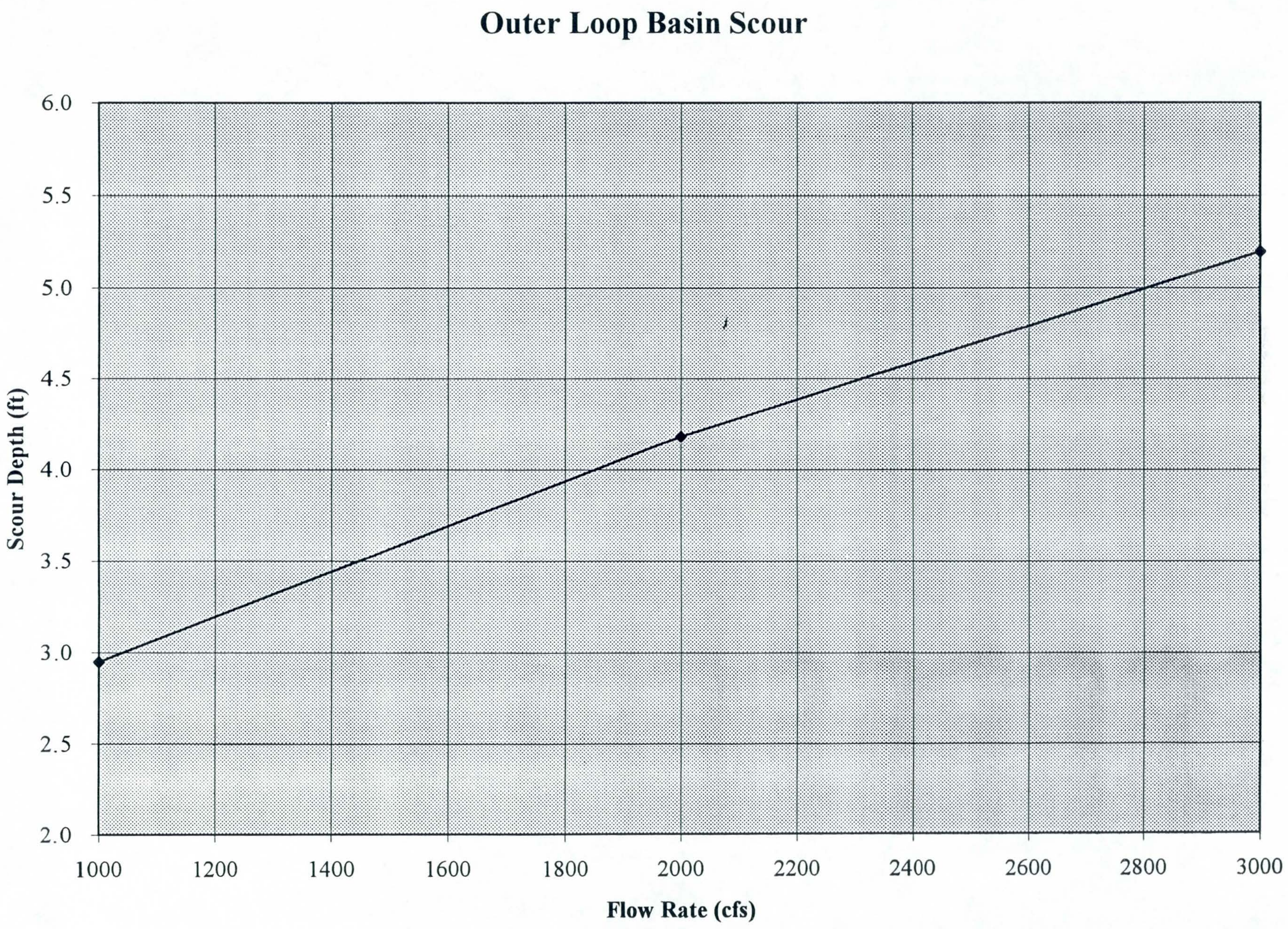
REFERENCES:

(1) Pemberton, E.L. and Lara, J.M. (1984), Computing Degradation and Local Scour Technical Guideline, Bureau of Reclamation, Engineering and Research Center Denver, Colorado, January 1984, pp. 48.
(2) Resource Consultants & Engineers, Inc. (1994), Sediment and Erosion Design Guide, prepared for Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA), RCE Ref. No. 90-560, November 1994.
(3) Simons, Li & Associates, Inc.,(1989), Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona, prepared for City of Tucson December 1989.

Outer Loop Basin Scour Calculations
Summary of input data and scour components

6-Jun-98

Input Data			
Flow Rate (cfs)	1000	2000	3000
Flow Depth (ft)	4.09	6.16	7.8
Flow Velocity (fps)	3.69	4.69	5.37
Unit Discharge (cfs/ft)	15.12	28.89	41.8
Threshold Velocity (fps)	2.46	2.81	3.05
D50 grain size mm (mm)	1.002	1.002	1.002
Unit Weight of soil (pcf)	165.00	165.00	165.00
Channel slope	0.001	0.001	0.001
Energy slope	0.00081	0.00081	0.00081
Mannings n	0.030	0.030	0.030
Side slope	1.5	1.5	1.5
Kinematic Viscosity	0.0000105	0.0000105	0.0000105
Local Scour			
USBR I Eq	N/A	N/A	N/A
Lacey Eq (z = 0.25)	0.9730	1.2250	1.4030
Blench Eq (z = 0.60)	3.0100	4.6350	5.9290
USBR II Eq (p 37, end of 3rd paragraph)	1.0230	1.5400	1.9500
Neill Eq	2.0450	4.1210	5.9330
Average local scour	1.7628	2.8803	3.8038
Anti-Dune depth			
Kennedy Eq	0.1860	0.3004	0.3939
Small watercourse low flow incisement	1.0000	1.0000	1.0000
General Scour	0.0000	0.0000	0.0000
Safety Factor	0.0000	0.0000	0.0000
Total Scour	2.9488	4.1807	5.1977



Outer Loop Basin Scour
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
Scour Depth (ft)
1000
1200
1400
1600
1800
2000
2200
2400
2600
2800
3000
Flow Rate (cfs)

APPENDIX A

BIBLIOGRAPHY

Appendix D

STANDARDS MANUAL FOR DRAINAGE DESIGN AND FLOODPLAIN MANAGEMENT IN TUCSON, ARIZONA

PREPARED FOR
CITY OF TUCSON
DEPARTMENT OF TRANSPORTATION,
ENGINEERING DIVISION

PREPARED BY
SIMONS, LI & ASSOCIATES, INC.

DECEMBER, 1989

$$Z_t = 1.3\ (Z_{gs} + 1/2Z_a + Z_{ls} + Z_{bs} + Z_{lft}) \tag{6.3}$$

Where:

Z_t	=	Design scour depth, *excluding* long-term aggradation/degradation, in feet;
Z_{gs}	=	General scour depth, in feet;
Z_a	=	Anti-dune trough depth, in feet;
Z_{ls}	=	Local scour depth, in feet;
Z_{bs}	=	Bend scour depth, in feet;
Z_{lft}	=	Low-flow thalweg depth, in feet; and,
1.3	=	Factor of safety to account for nonuniform flow distribution.

The various equations for depth of scour which are to follow were developed strictly for use in conjunction with sand-bed channels in which the bed material is erodible to the depth specified by the applicable equations. However, this situation does not always exist in channels located within the City of Tucson. In some areas of the city, the channel has degraded to a point where the exposed bed is no longer composed of strictly unconsolidated alluvial material, but rather of consolidated hard-pan or caliche. Channel beds composed of this type of material are not freely erodible, and thus the scour equations which follow may not strictly apply. Should such conditions be encountered, a geotechnical investigation should be submitted by an Arizona Registered Professional Civil Engineer to justify the use of a lesser scour depth than would be determined from the use of Equation 6.3.

6.6.1 General Scour

As previously discussed in Section 6.5 of this Manual, the depth of general scour is best estimated by performing a detailed sediment-transport analysis using the bed grain-size distribution, hydraulic conditions, sediment-transport capacity at different stages throughout the flow event, changes in bed levels throughout the event, and the sediment supply into the reach being studied. An analysis to this level of detail is beyond the scope of this Manual. However, there are several computer models commercially available to aid in making an estimate of general scour. Unfortunately, these models are very sensitive to input, and the results are best interpreted by someone with extensive experience in the field of sediment transport. A detailed discussion of sediment-transport analysis for computing general scour can be found in "Engineering Analysis of Fluvial Systems" (Simons, Li & Associates, 1982), and "Arizona Department of Water Resources Design Manual for Engineering Analysis of Fluvial Systems" (Simons, Li & Associates, 1985).

General scour on regional watercourses should be estimated by undertaking a detailed sediment-transport study, as described above, when and where it is feasible to do so. However, such a study is not usually practical on smaller watercourses. Therefore, as an alternative to the above, on watercourses other than regional watercourses, the following equation (Zeller, 1981) should be used to predict general scour:

$$Z_{gs} = Y_{max} \left[\frac{0.0685 V_m^{0.8}}{Y_h^{0.4} S_e^{0.3}} - 1 \right] \tag{6.4}$$

Where:

Z_{gs} = General scour depth, in feet;
V_m = Average velocity of flow, in feet per second;
Y_{max} = Maximum depth of flow, in feet;
Y_h = Hydraulic depth of flow, in feet; and,
S_e = Energy slope (or bed slope for uniform-flow conditions), in feet per foot.

NOTE: Should Z_{gs} become negative, assume that the general-scour component is equal to zero (i.e., Z_{gs} = 0).

6.6.2 Anti-Dune Trough Depth

Anti-dunes are bed forms, in the shape of dunes, which move in an upstream rather than a downstream direction within the channel; hence the term "anti-dunes." They form as trains of waves that build up from a plane bed and a plane water surface. Anti-dunes can form either during transitional flow, between subcritical and supercritical flow, or during supercritical flow. The wave length is proportional to the velocity of flow. The corresponding surface waves, which are in phase with the anti-dunes, tend to break like surf when the waves reach a height approximately equal to 0.14 times the wave length. A relationship between average channel velocity, V_m, and anti-dune trough depth, Z_a, can therefore be developed (Simons, Li & Associates, 1982). This relationship is:

$$Z_a = \frac{1}{2} (0.14) \frac{2\pi V_m^2}{g} = 0.0137 V_m^2 \tag{6.5}$$

A restriction on the above equation is that the anti-dune trough depth can never exceed one-half the depth of flow. Therefore, if the computed depth of Z_a obtained by using Equation 6.5 exceeds one-half of the depth of flow, the anti-dune trough depth should then be taken as equal to one-half the depth of flow. Figure 6.2 shows a definition sketch for anti-dune trough depth.

6.6.3 Low-Flow Thalweg

A low-flow thalweg is a small channel which forms within the bed of the main channel, and in which low discharges are carried. Low-flow thalwegs form when the width/depth ratio of the main channel is large. Rather than flow in a very wide, shallow state, low flows will develop a low-flow channel thalweg below the average channel bed elevation in order to provide more efficient conveyance of these discharges.

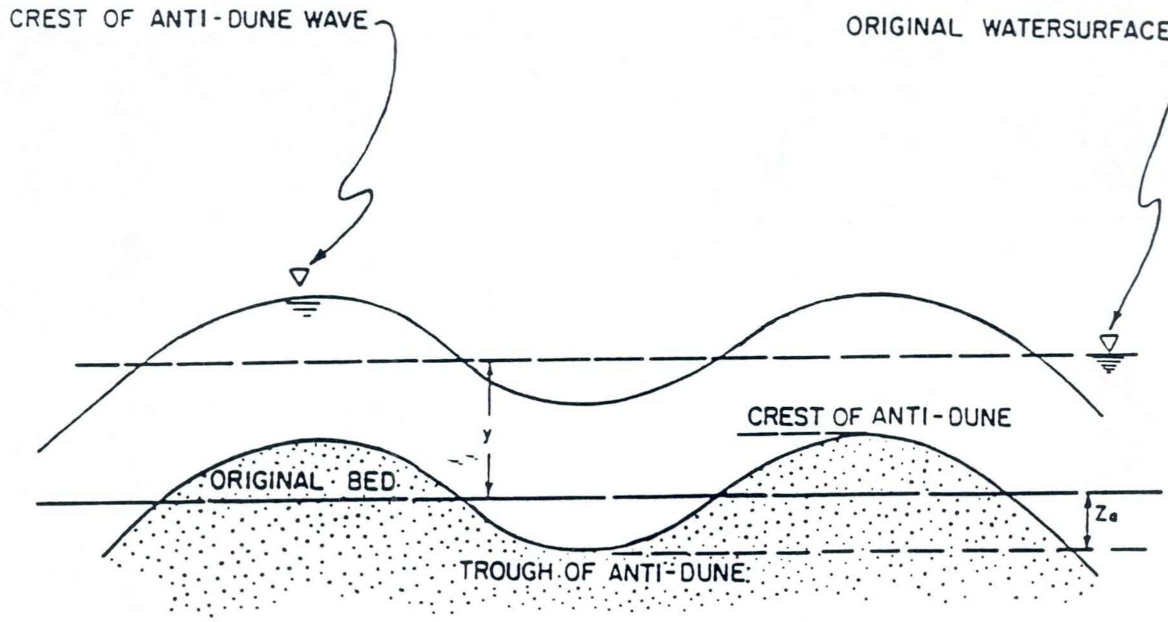


FIGURE 6.2
DEFINITION SKETCH FOR ANTI-DUNE TROUGH DEPTH

When the ratio of the flow width to the flow depth of a channel is greater than 1.15 times the average velocity of flow for the 100-year discharge, a low-flow thalweg must be included in all scour calculations. When the flow width or flow depth exceeds the top width and bank heights of the channel, use the top width and flow depth at bank-full conditions, instead of the actual flow width and flow depth. Presently, there is no known methodology for predicting low-flow thalweg depth. However, observation of channels in the Tucson area has revealed that low-flow thalwegs are normally one to two feet deep. Therefore, if a low-flow thalweg is predicted to be present, it should be assumed to be at least two feet deep within regional watercourses, and at least one foot deep within all other watercourses, unless field observations dictate otherwise.

6.6.4 Bend Scour

Bend scour normally occurs along the outside of bends, and is caused by spiral, transverse currents which form within the flow as the water moves around the bend. Presently, there is no single procedure which will consistently and accurately predict bend scour over a wide range of hydraulic conditions. However, the following relationship has been developed by Zeller (1981) for estimating bend scour in sand-bed channels based upon the assumption of the maintenance of constant stream power within the channel bend:

$$Z_{bs} = \frac{0.0685 Y_{max} V_m^{0.8}}{Y_h^{0.4} S_e^{0.3}} \left(2.1 \left[\frac{\sin^2(\alpha/2)}{\cos \alpha} \right]^{0.2} - 1 \right) \qquad (6.6)$$

Where:

Z_{bs} = Bend-scour component of total scour depth, in feet;
= 0 when $r_c/T \geq 10.0$, or $\alpha \leq 17.8°$
= computed value when $0.5 < r_c/T < 10.0$, or $17.8° < \alpha < 60°$
= computed value at $\alpha = 60°$ when $r_c/T \leq 0.5$, or $\alpha \geq 60°$

V_m = Average velocity of flow immediately upstream of bend, in feet per second;

Y_{max} = *Maximum* depth of flow immediately upstream of bend, in feet;

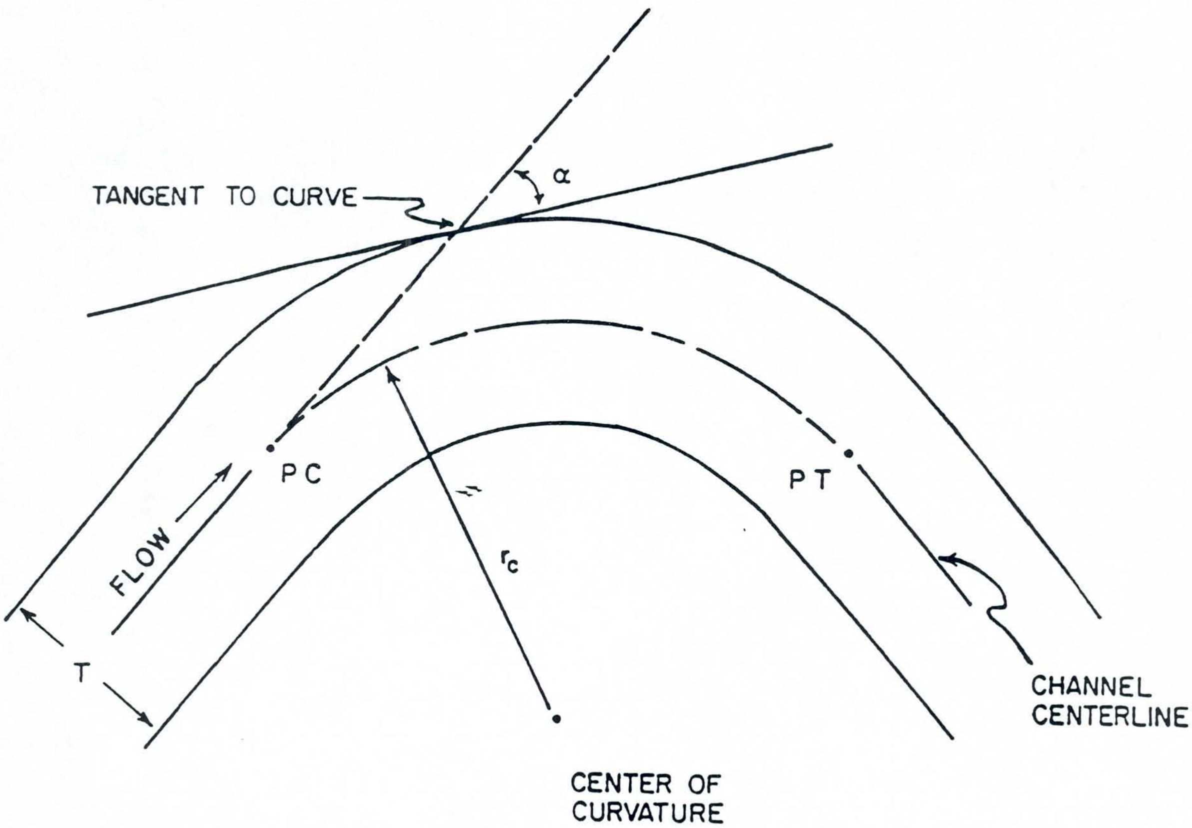
Y_h = Hydraulic depth of flow immediately upstream of bend, in feet;

S_e = Energy slope immediately upstream of bend (or bed slope for uniform-flow conditions), in feet per foot; and,

α = Angle formed by the projection of the channel centerline from the point of curvature to a point which meets a line tangent to the outer bank of the channel, in degrees (see Figure 6.3).

NOTE: Mathematically, it can be shown that, for a simple circular curve, the following relationship exists between α and the ratio of the centerline radius of curvature, r_c, to channel top width, T.

$$\frac{r_c}{T} = \frac{\cos \alpha}{4 \sin^2(\alpha/2)} \qquad (6.7)$$



PT = Downstream point of tangency to the centerline radius of curvature.
PC = Upstream point of curvature at the centerline radius of curvature.

FIGURE 6.3

ILLUSTRATION OF TERMINOLOGY FOR BEND-SCOUR CALCULATIONS

Where:

r_c = Radius of curvature along centerline of channel, in feet; and,
T = Channel top width, in feet.

If the bend deviates significantly from a simple circular curve, the curve should be divided into a series of circular curves, and the bend scour computed for each segment should be based upon the angle α applicable to that segment.

Equation 6.6 can be applied to obtain an approximation of the scour depth that can be expected in a bend during a specific water discharge. The impact that other simultaneously occurring phenomena such as sand waves, local scour, long-term degradation, etc., might have upon bend scour is not known for certain, given the present state of the art. Therefore, in order that the maximum scour in a bend not be underestimated, it is recommended that bend scour be considered as an independent channel adjustment that should be added to those adjustments computed for long-term degradation, general scour, and sand-wave troughs.

The longitudinal extent of the bend-scour component is as difficult to quantify as the vertical extent. Rozovskii (1961) developed an expression for predicting the distance from the end of a bend at which the secondary currents will have decayed to a negligible magnitude. This relationship, in a simplified form, can be expressed as:

$$x = \frac{0.6}{n} Y^{1.17} \tag{6.8}$$

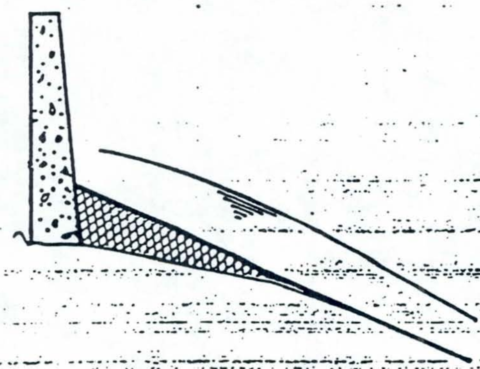
Where:

x = Distance from the end of channel curvature (point of tangency, PT) to the downstream point at which secondary currents have dissipated, in feet;
n = Manning's roughness coefficient;
g = Acceleration due to gravity, 32.2 ft/sec^2; and,
Y = Depth of flow (to be conservative, use maximum depth of flow, exclusive of scour, within the bend), in feet.

Equation 6.8 should be used for determining the distance downstream of a curve that secondary currents will continue to be effective in producing bend scour. As a conservative estimate of the longitudinal extent of bend scour, both through and downstream of the curve, it would be advisable to consider bend scour as commencing at the upstream point of curvature (PC), and extending a distance x (computed with Equation 6.8) beyond the downstream point of tangency (PT).

6.6.5 *Local Scour*

Local scour occurs whenever there is an abrupt change in the direction of flow. Abrupt changes in flow direction can be caused by obstructions to flow, such as bridge piers or abrupt contractions at bridge abutments.



COMPUTING DEGRADATION AND LOCAL SCOUR

TECHNICAL GUIDELINE FOR BUREAU OF RECLAMATION

U.S. Department of the Interior
Bureau of Reclamation

Inch-pound units	Metric units
$L_g = \frac{37.05}{0.00112}$	$L_g = \frac{1.625\ (6.94)}{0.00112}$
$L_g = 33\,100$ ft	$L_g = 10\,100$ m

and for the subreaches:

Inch-pound units	Metric units
$L_1 = \frac{22.8}{2\ (0.00112)} = 10\ 200$ ft	$L_1 = \frac{6.94}{2\ (0.00112)} = 3\ 100$ m
$L_2 = \frac{3\ (22.8)}{8\ (0.00112)} = 7\ 600$ ft	$L_2 = \frac{3\ (6.94)}{8\ (0.00112)} = 2\ 300$ m
$L_3 = \frac{3\ (22.8)}{4\ (0.00112)} = 15\ 300$ ft	$L_3 = \frac{3(6.94)}{4\ (0.00112)} = 4\ 700$ m

CHANNEL SCOUR DURING PEAK FLOODFLOWS

The design of any structure located either along the riverbank and flood plain or across a channel requires a river study to determine the response of the riverbed and banks to large floods. A knowledge of fluvial morphology combined with field experience is important in both the collection of adequate field data and selection of appropriate studies for predicting the erosion potential. In most studies, two processes must be considered, (1) natural channel scour, and (2) scour induced by structures placed by man either in or adjacent to the main river channel.

Natural scour occurs in any moveable bed river but is more severe when associated with restrictions in river widths, caused by morphological channel changes, and influenced by erosive flow patterns resulting from channel alinement such as a bend in a meandering river. Rock outcrops along the bed or banks of a stream can restrict the normal river movement and thus effect any of the above influencing factors. Manmade structures can have varying degrees of influence, usually dependent upon either the restriction placed upon the normal river movement or by turbulence in flow pattern directly related to the structure. Examples of structures that influence river movement would be (1) levees placed to control flood plain flows, thus increasing main channel discharges; (2) spur dikes, groins, riprapped banks, or bridge abutments used to control main channel movement; or (3) pumping plants or headworks to canals placed on a riverbank. Scour of the bed or banks caused by these structures is that created by higher local velocities or excessive turbulence at the strucutre. Structures placed directly in the river consist of (1) piers and piling for either highways or railroad bridges; (2) dams across the river for diversion or storage, (3) grade control structures such as rock cascades, gabion controls or concrete baffled apron drop structures; or (4) occasionally a powerline or tower structure placed in the flood plain but exposed to channel erosion with extreme shifting or movement of a river. All of the above may be subject to higher local velocities, but usually are subject to the more critical local scour caused by turbulence and helicoidal flow patterns.

The prediction of river channel scour due to floods is necessary for the design of many Reclamation structures. These Reclamation guidelines on scour represent a summary of some of the more applicable techniques which are described in greater detail in the reference publications by T. Blench (1969), National Cooperative Highway Research Program Synthesis 5 (1970), C. R. Neill (1973), D. B. Simons and F. Senturk (1977), and S. C. Jain (1981). The paper by S. C. Jain (1981) summarized many of the empirical equations developed for predicting scour of a streambed around a bridge pier. It should be recognized that the many equations are empirically developed from experimental studies. Some are regime-type based on practical conditions and considerable experience and judgment. Because of the complexity of scouring action as related to velocity, turbulence, and bed materials, it is difficult to prescribe a direct procedure. Reclamation practice is to compute scour by several methods and utilize judgment in averaging the results or selection of the most applicable procedures.

The equations for predicting local channel scour usually can be grouped into those applicable to the two previously described processes of either a natural channel scour or scour caused by a manmade structure. A further breakdown of these processes is shown in table 6 where Type A equations are those used for natural river erosion and Types B, C, and D cover various manmade structures.

The importance of experience and judgment in conducting a scour study cannot be overemphasized. It should be recognized that the techniques described in these guidelines merely provide a set of practical tools in guiding the investigator to estimate the amount of scour for use in design. The collection of adequate field data to define channel hydraulics and bed or bank materials to be scoured govern the accuracy of any study. They should be given as much emphasis as the methodology used in the analytical study. Field data are needed to compute water surface profiles for a reach of river in the determination of channel hydraulics for use in a scour study. With no restrictions in channel width, scour is computed from the average channel hydraulics for a reach. If a structure restricts the river width, scour is computed from the channel hydraulics at the restriction. In all cases, scour estimates should be based upon the portion of discharge in and hydraulic characteristics of the main channel only.

Table 6. - Classification of scour equation for various structure designs

Equation type	Scour	Design
A	Natural channel for restrictions and bends	Siphon crossing or any buried pipeline. Stability study of a natural bank. Waterway for one-span bridge.
B	Bankline structures	Abutments to bridge or siphon crossing. Bank slope protection such as riprap, etc. Spur dikes, groins, etc. Pumping plants. Canal headworks.
C	Midchannel structures	Piling for bridge. Piers for flume over river. Powerline footings. Riverbed water intake structures.
D	Hydraulic structures across channel	Dams and diversion dams. Erosion controls. Rock cascade drops, gabion controls, and concrete drops.

Although each scour problem must be analyzed individually, there are some general flow and sediment transport characteristics to be considered in making the judgmental decision on methodology. The general conclusion reached by Lane and Borland (1954) was that floods do not cause a general lowering of streambed, and rivers such as the Rio Grande may scour at the narrow sections but fill up at the wider downstream sections during a major flood. Another general sediment transport characteristic is the influence of a large sediment load on scour which includes the variation of sediment transport associated with a high peak, short duration flood hydrograph. The large sediment concentrations usually of clay and silt size material will occur on the rising stage of the hydrograph up and through the peak of the flood while the falling stage of the flood with deposition of coarser sediments in the bed of the channel may be accompanied by greater scour of the wetted channel banks. Channel scour also occurs when the capacity of streamflow with extreme high velocities in portions of the channel cross section will transport the bed material at a greater rate than replacement materials are supplied. Thus, maximum depth of channel scour during the flood is a function of the channel geometry, obstruction created by a structure (if any), the velocity of flow, turbulence, and size of bed material.

Design Flood

The first step in local scour study for design of a structure is selection of design flood frequency. Reclamation criteria for design of most structures shown in table 6 varies from a design flood estimated on a frequency basis from 50 to 100 years. This pertains to an adequate waterway for passage of the floodflow peak. The scour calculations for these same structures are always made for a 100-year flood peak. The use of the 100-year flood peak for scour is based on variability of channel hydraulics, bed material, and general complexity of the erosive process. The exception in the use of the 100-year flood peak for estimating scour would be the scour hole immediately below a large dam or a major structure where loss of structure could involve lives or represent a catastrophic event. In this case, the scour for use in design should be determined for a flow equal to 50 percent of the structure design flood.

Equation Types A and B (See Table 6)

Natural river channel scour estimates are required in design of a buried pipe, buried canal siphon, or a bankline structure. For most siphon crossings of a river, the cost of burying a siphon will dictate either the selection of a natural narrow reach of river or a restriction in width created by constructing canal bankline levees across a portion of the flood plain. A summary of available methods for computing scour at constrictions is given by Neill (1973). The four methods for estimating general scour at constricted waterways described by Neill (1973) are considered the proper approach for estimating scour for use in either design of a siphon crossing or where general scour is needed of the riverbed for a bankline structure. The four methods supplemented with Reclamation's procedure for application are given below:

Field measurments of scour method. - This method consists of observing or measuring the actual scoured depths either at the river under investigation or a similar type river. The measurements are taken during as high a flow as possible to minimize the influence of extrapolation.

A Reclamation unpublished study by Abbott (1963) analyzed U.S. Geological Survey discharge measurement notes from several streams in the southwestern United States, including the Galisteo Creek at Domingo, New Mexico, and developed an empirical curve enveloping observed scour at the gaging station. This envelope curve for use in siphon design was further supported by observed scour from crest-stage and scour gages on Gallegos, Kutz, Largo, Chaco, and Gobernador Canyons in northwest New Mexico collected during the period from 1963 to 1969. The scour gages consisted of a series of deeply anchored buried flexible tapes across the channel section that were resurveyed after a flood to determine the depth of scour at a specific location. The results of these measurements are shown on figure 8 along with the envelope curve for Galisteo Creek that support scour estimates for wide sandbed (D_{50} varying from 0.5 to 0.7 mm) ephemeral streams in the southwestern United States by the equation.

$$d_s = K\,(q)^{0.24} \qquad (24)$$

USBR I

where:

d_s = Depth of scour below streambed, ft (m)
K = 2.45 inch-pound units (1.32 metric units)
q = Unit water discharge, ft^3/s per ft of width (m^3/s per m of width)

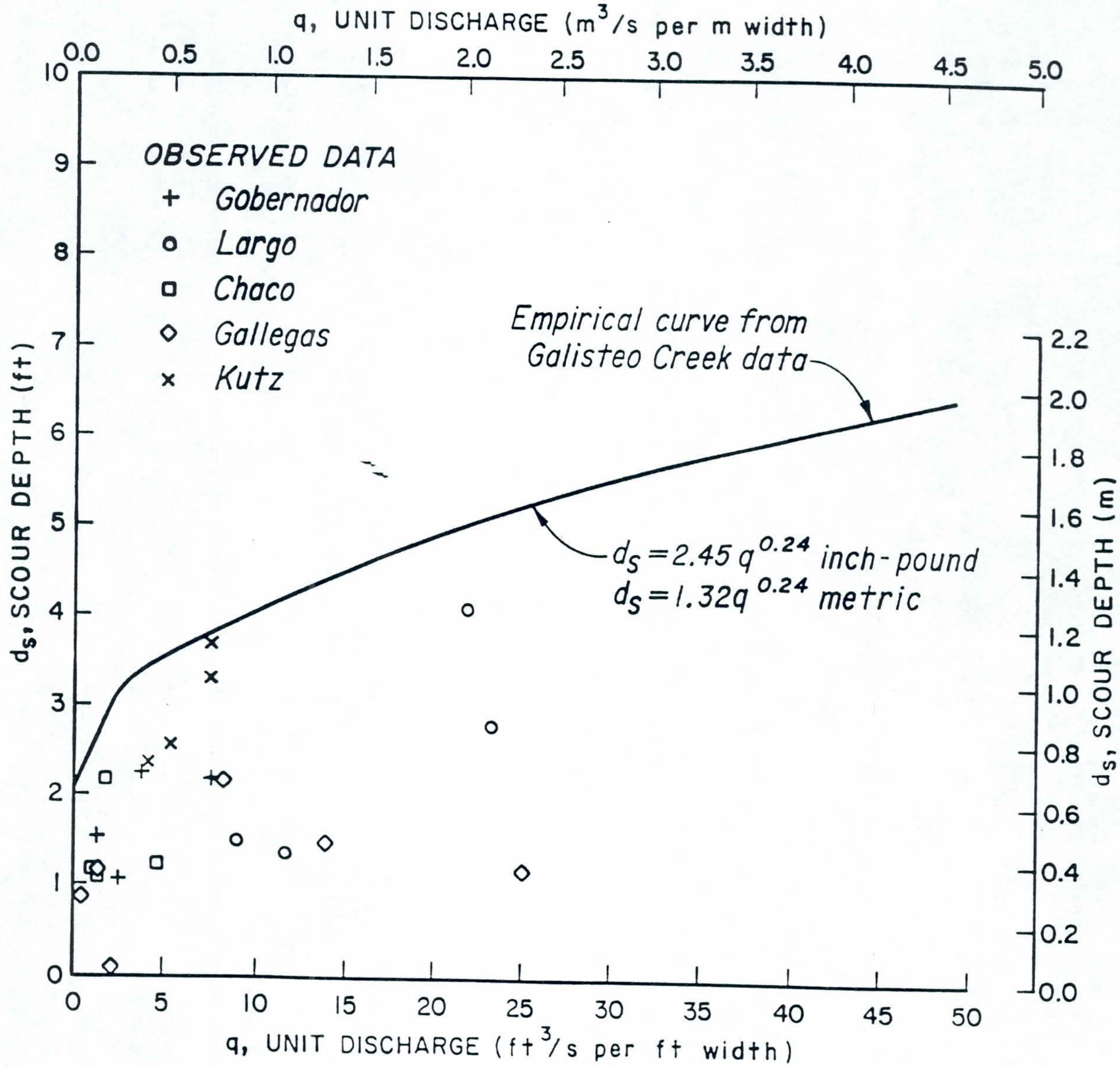


Figure 8. - Navajo Indian Irrigation Project - scour versus unit discharge.

The use of equation 24 except as a check on other methods would be limited to channels similar to those observed on relatively steep slopes ranging from 0.004 to 0.008 ft/ft (m/m). Because of shallow depths of flow and medium to coarse sand size bed material the bedload transport should also be very high.

Regime equations supported by field measurements method. - This approach as suggested by Neill (1973) on recommendations by Blench (1969) involves obtaining field measurements in an incised reach of river from which the bankfull discharge and hydraluics can be determined. From the bankfull hydraulics in the incised reach of river, the flood depths can be computed by:

$$d_f = d_i \left(\frac{q_f}{q_i}\right)^m \tag{25}$$

where:

d_f = Scoured depth below design floodwater level
d_i = Average depth at bankfull discharge in incised reach
q_f = Design flood discharge per unit width
q_i = Bankfull discharge in incised reach per unit width
m = Exponent varying from 0.67 for sand to 0.85 for coarse gravel

This method has been expanded for Reclamation use to include the empirical regime equation by Lacey (1930) and the method of zero bed-sediment transport by Blench (1969) in the form of the Lacey equation:

$$d_m = 0.47 \left(\frac{Q}{f}\right)^{1/3} \tag{26}$$

Lacey

where:

d_m = Mean depth at design discharge, ft (m)
Q = Design discharge, ft^3/s (m^3/s)
f = Lacey's silt factor equals 1.76 $(D_m)^{1/2}$ where D_m equal mean grain size of bed material in millimeters

and the Blench equation for "zero bed factor":

Blench

$$d_{fo} = \frac{q_f^{\ 2/3}}{F_{bo}^{\ 1/3}} \tag{27}$$

where:

d_{fo} = Depth for zero bed sediment transport, ft (m)
q_f = Design flood discharge per unit width, ft^3/s per ft (m^3/s per m)
F_{bo} = Blench's "zero bed factor" in ft/s^2 (m/s^2) from figure 9

The maximum natural channel scour depth for design of any structure placed below the streambed (i.e., siphon) or along the bank of a channel must

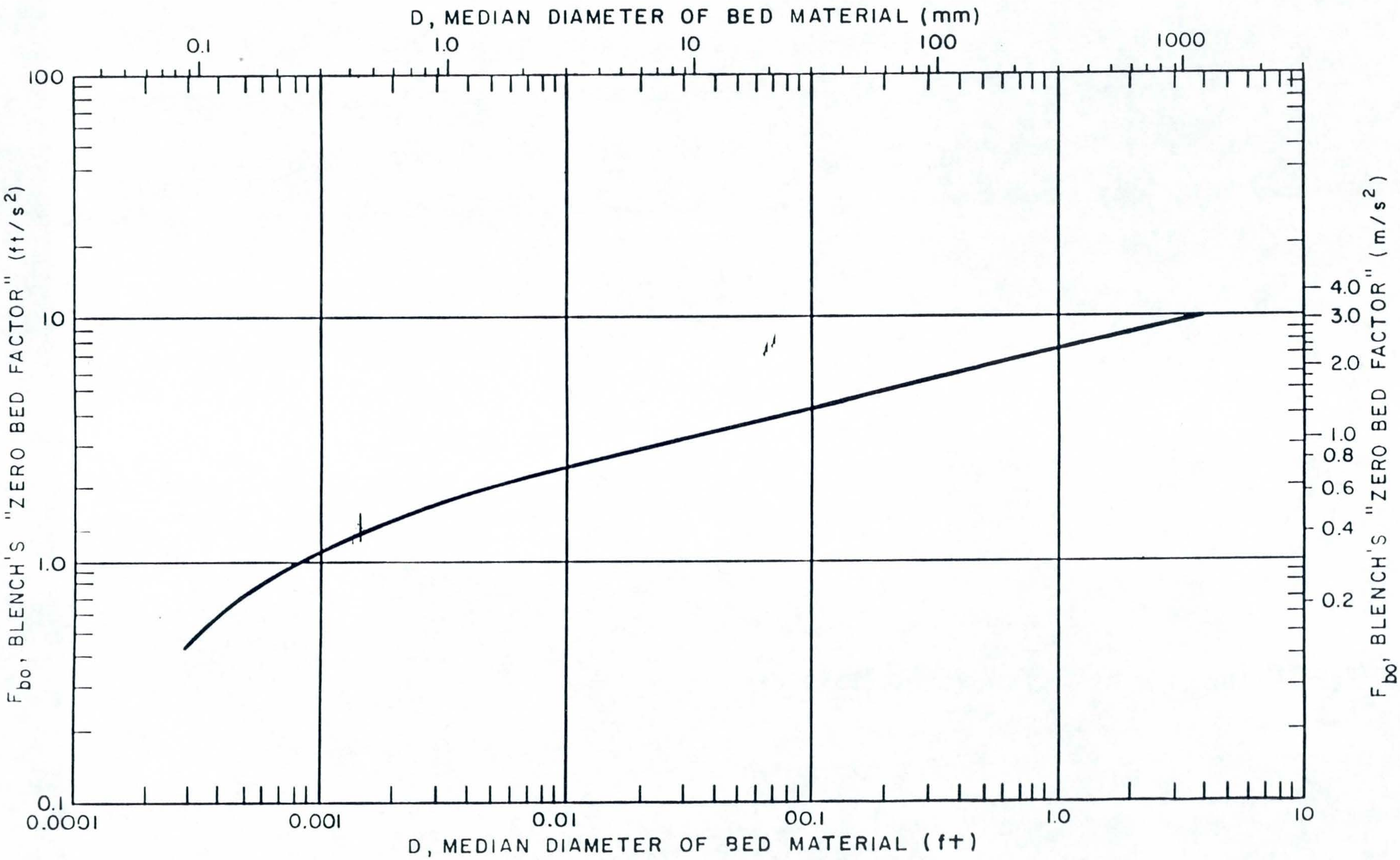


CHART FOR ESTIMATING F_{bo} (AFTER BLENCH)

Figure 9. - Chart for estimating F_{bo} (after Blench, 1969).

consider the probable concentration of floodflows in some portion of the natural channel. Equations 25, 26, or 27 for predicting this maximum depth are to be adjusted by the empirical multiplying factors, Z, shown for formula Types A and B (table 6), in table 7. An illustration of maximum scour depth associated with a flood discharge is shown in a sketch of a natural channel, figure 10. As shown in table 7 and on figure 10, the d_s equals depth of scour below streambed.

$$d_s = Z\,d_f \tag{28}$$

NB. ~~USBR II~~

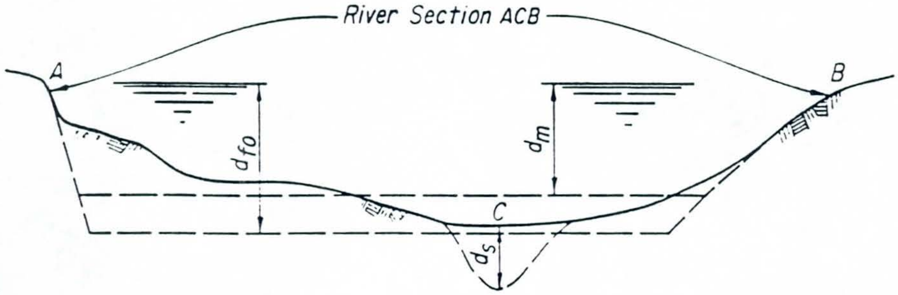
$$d_s = Z\,d_m \tag{29}$$

$$d_s = Z\,d_{fo} \tag{30}$$

Table 7. - Multiplying factors, Z, for use in scour depths by regime equations

Condition	Value of Z Neill $d_s = Z\,d_f$	 Lacey $d_s = Z\,d_m$	 Blench $d_s = Z\,d_{fo}$
<u>Equation Types A and B</u>			
Straight reach	0.5	0.25	} 1/ 0.6
Moderate bend	0.6	0.5	
Severe bend	0.7	0.75	
Right angle bends		1.0	1.25
Vertical rock bank or wall		1.25	
<u>Equation Types C and D</u>			
Nose of piers	1.0		0.5 to 1.0
Nose of guide banks	0.4 to 0.7	1.50 to 1.75	1.0 to 1.75
Small dam or control across river		1.5	0.75 to 1.25

1/ Z value selected by USBR for use on bends in river.



NOTE: $d_{fo} > d_f > d_m$. Point C is low point of natural section.

Figure 10. - Sketch of natural channel scour by regime method.

Although not shown on figure 10, the d_f from Neill's equation 25 is usually less than the d_{fo} from Blench's equation 27 but greater than the d_m from Lacey's equation 26.

The design of a structure under a river channel such as a siphon is based on applying the scoured depth, d_s, as obtained from table 7 to the low point in a surveyed section, as shown by point C on figure 10. This criteria is considered by Reclamation as an adequate safety factor for use in design. In an alluvial streambed, designs should also be based on scour occuring at any location in order to provide for channel shifting with time.

Mean velocity from field measurements method. - This approach represents an adjustment in surveyed channel geometry based on an extrapolated design flow velocity. In Reclamation's application of this method, a series of at least four cross sections are surveyed and backwater computations made for the design discharge by use of Reclamation's Water Surface Profile Computer Program. In addition to the surveyed cross sections observed, water surface elevations at a known or measured discharge are needed to provide a check on Manning's "n" channel roughness coefficient. This procedure allows for any proposed waterway restrictions to be analyzed for channel hydraulic characteristics including mean velocity at the design discharge. The usual Reclamation application of this method is to determine the mean channel depth, d_m, from the computer output data and apply the Z values defined by Lacey in table 7 to compute a scour depth, d_s, by equation 29 where $d_s = Z\ d_m$. USBR II

Examples of more unique solutions to scour problems were Reclamation studies on the Colorado River near Parker, Arizona, and Salt River near Granite Reef Diversion Dam, Arizona, where an adjustment in "n" based on particle size along with a Z value from table 7 provided a method of computing bed scour. The selection of a particle size "n" associated with scour in the above two examples was computed from the Strickler (1923) equation for roughness of a channel based on diameter of particles where:

$$K = \frac{C}{D_{90}^{1/6}} \qquad (31)$$

$C \approx 26$ from Nikuradse (1933) and "n" = 1/K. The appropriate "n" values for the two rivers based on particle size and engineering judgment were selected as follows:

River	D (mm)	Particle size "n"	Selected "n"
Colorado	0.2	0.01	0.014
Salt	18	0.02	0.02

In the Colorado River study, the existing channel "n" value of 0.022 was adjusted down to 0.014 due to bed material particle size to give a computed water surface at design discharge representative of a scoured channel. With a Z value of 0.5, the scoured section in the form of a trianglular section combined with the accepted "n" of 0.022 provided a close check on the water surface computed without scour. An illustration of this technique is shown in sketch on figure 11a. Another example is shown on figure 11b for a Salt River scour study where the particle size "n" of 0.02 gave a reduced mean depth. Scour was assumed to be in the shape of a triangle where the average depth of scour would be equal the depth at an "n" equal to 0.02 subtracted from depth at an "n" equal to 0.03. (See example problem in subsequent paragraph.)

Competent or limiting velocity control to scour method. - This method assumes that scour will occur in the channel cross section until the mean velocity is reduced to that where little or no movement of bed material is taking place. It gives the maximum limit to scour existing in only the deep scour hole portion of the channel cross section and is similar to the Blench equation 27 for a "zero bed factor."

The empirical curves, figure 12, derived by Neill (1973) for competent velocity with sand or coarser bed material (>0.30 mm) represent a combining of regime criteria, Shields (1936) criterion for material >1.0 mm, and a mean velocity formula relating mean velocity V_m to the shear velocity. The competent velocities for erosion of cohesive materials recommended by Neill (1973) are given in table 8. The scour depth or increase in area of scoured channel section with corresponding increase in depth for competent velocity, V_c, is determined by relationship of mean velocity, V_m, to V_c in the equation:

$$d_s = d_m \left(\frac{V_m}{V_c} - 1 \right) \qquad (32)$$ Neill

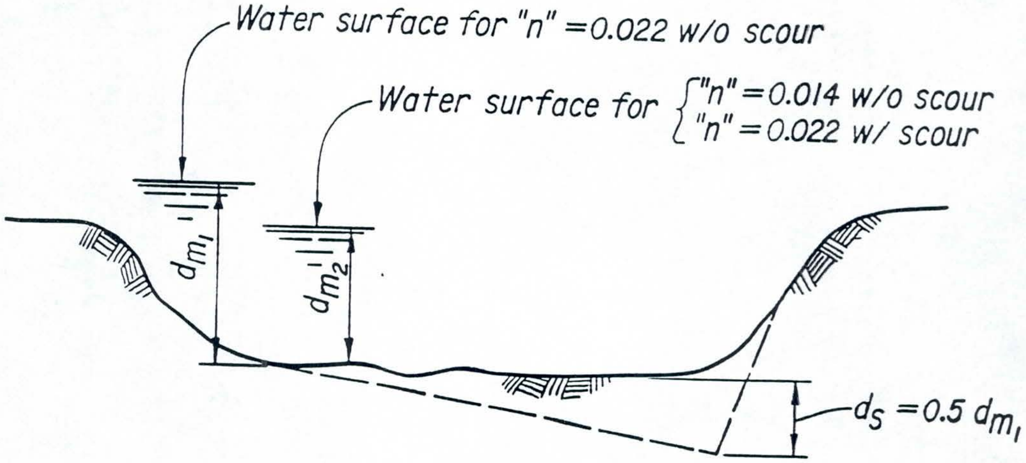
where:

d_s = Scour depth below streambed, ft (m)
d_m = Mean depth, ft (m)

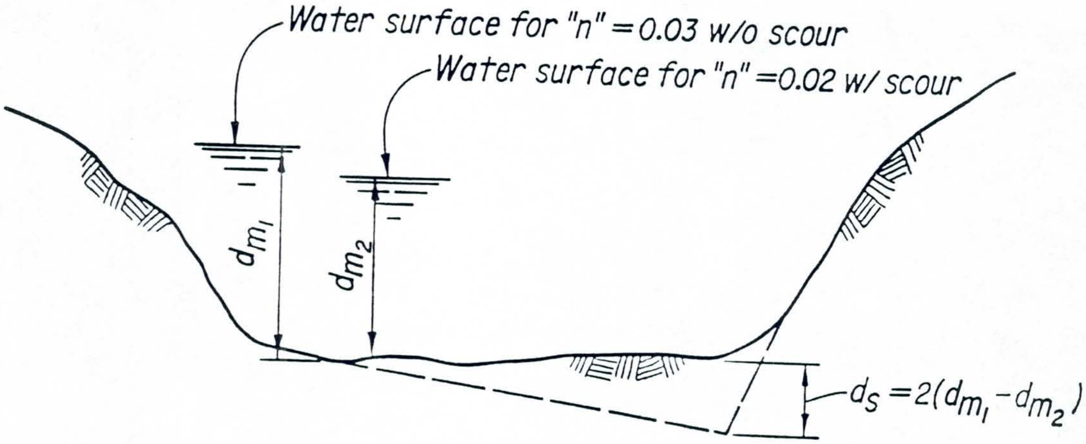
Table 8. - Tentative guide to competent velocities for erosion of cohesive materials* (after Neill, 1973)

Depth of flow		Competent mean velocity					
		Low values - easily erodible material		Average values		High values - resistant material	
ft	m	ft/s	m/s	ft/s	m/s	ft/s	m/s
5	1.5	1.9	0.6	3.4	1.0	5.9	1.8
10	3	2.1	0.65	3.9	1.2	6.6	2.0
20	6	2.3	0.7	4.3	1.3	7.4	2.3
50	15	2.7	0.8	5.0	1.5	8.6	2.6

* Notes: (1) This table is to be regarded as a rough guide only, in the absence of data based on local experience. Account must be taken of the expected condition of the material after exposure to weathering and saturation. (2) It is not considered advisable to relate the suggested low, average, and high values to soil shear strength or other conventional indices, because of the predominating effects of weathering and saturation on the erodibility of many cohesive soils.



a. Colorado River Study



b. Salt River Study

Figure 11. - Sketch of scour from water surface profile computations and reduced "n" for scour.

The use of figure 12 and table 8 recommended by Neill (1973) has had limited application in Reclamation, but appears to be a potential useful technique for many Reclamation studies on scour and armoring of the channel.

Equation Type C (See Table 6)

The principal references for design of midchannel structures for scour such as at bridge piers are National Cooperative Highway Research Program Synthesis 5 (1970), C. R. Neill (1973), Federal Highway Administration, Training and Design Manual (1975), Federal Highway Administration (1980), and S. C. Jain (1981). The numerous empirical relationships for computing scour at bridge piers include one or more of the following hydraulic parameters: pier width and skewness, flow depth, velocity, and size of sediment. The many relations available were further broken down by Jain (1981) to two different approaches: (1) regime, and (2) rational.

The Federal Highway Administration has funded numerous research projects to assist in improving their designs of bridge piers. This research has not resulted in any one recommended procedure. Reclamation's need for scour estimates at midchannel structures is limited. The procedures adopted are to try at least two techniques and apply engineering judgment in selecting an average or most reliable method. The regime approach is to use either equations 26, 27, 28, or 30 and a Z value from table 7. An appropriate Z value to use for piers is 1.0 as found for the railway bridge piers applied to the Lacey equation 29 reported by Central Board of Irrigation and Power (1971).

The rational equation selected for scour at piers is described by Jain (1981) in the form:

$$\frac{d_s}{b} = 1.84 \left(\frac{d}{b}\right)^{0.3} \left(F_c\right)^{0.25} \qquad (33)$$

where:

d_s = Depth of scour below streambed, ft (m)
b = Pier size, ft (m)
d = Flow depth, ft (m)
$F_c = V_c/\sqrt{gd}$ = Threshold Froude number
V_c = Threshold velocity, ft/s (m/s) from figure 12
g = Acceleration due to gravity, 32.2 ft/s^2 (9.81 m/s^2)

Equation Type D (See Table 6)

Immediately downstream from any hydraulic structure the riverbed is subject to the erosive action created by the structure. Some type of stilling basin or energy dissipator as described by Reclamation (1977) is provided in the design of such structures to dissipate the energy thereby reducing the erosion potential. There still remains at most structures, below the point where the structure ends and the natural riverbed material begins, a potential for scour. The magnitude of this scour hole will depend on a combination of flow velocity, turbulence, and vortices generated by the structure. Simons and Senturk (1977) describe many of the available equations.

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Memo

To: File

From: George V. Sabol

Date: 19 August 1998

Reference: **PR3B – TRASHRACK HEAD LOSSES**
FILE: 28900082

For the PR3B Project, trashracks will be required at basin outlet structures and at lateral inflows to conduits and storm drains. The trashracks will serve to keep large debris from entering the basin outlet works and from being introduced to conduits and storm drains. Such trashracks are necessary to provide a reasonable level of assurance that those facilities are not clogged or their hydraulic performance impaired by trash accumulation. Trashracks also serve a safety function by inhibiting unauthorized intentional or accidental entrance into those hydraulic structures by persons.

A review of trashrack design criteria was conducted, particularly in regard to estimating head losses through those structures. Appendix A provides copies of procedures that were considered for head loss estimation.

Two of those procedures were evaluated; 1) the procedure in the Maricopa County Drainage Design Manual, Volume II, Hydraulics, and 2) the procedure in the USBR Design of Small Dams. It was determined that the procedure in Design of Small Dams is most appropriate for the trashracks to be installed on the basin outlet works. The procedure in the Maricopa County manual is essentially the same as that contained in the FHWA Hydraulic Design of Highway Culverts. That procedure accounts for approach angle which may be more appropriate for trashracks on inlets to the conduits or at entrances to culverts.

The following are generally design considerations from the references that are provided in Appendix A.

1. Approach velocities less than 3 fps do not require accounting for trashrack head losses. Velocities greater than 3 fps require computation of head losses (FCDMC).
2. Velocity through the trashrack ordinarily should not exceed 2 fps if the rack is inaccessible for cleaning. Velocity up to 5 fps may be tolerated for racks that are accessible for cleaning (USBR).
3. Open area between the bars should be 1.5 to 3.0 times the area of the outlet entrance, depending on the anticipated volume and size of debris (FCDMC).
4. Where maximum head loss values are desired, assume that 50 percent of the rack area is clogged (USBR).
5. For minimum head loss, assume no clogging or neglect the loss entirely (USBR).
6. Head loss equations are empirical and should be used with caution (FHWA).

In certain design conditions, both the maximum head loss and the minimum head loss conditions must be considered. For example, trashracks on outlets from basins that discharge to downstream conduits or storm sewers must consider maximum head loss since that loss will affect the stage in the basin, and also minimum head loss must be considered since that condition will maximize the discharge into the conduit. Since the degree of clogging cannot be anticipated, the entire system must function either under the no clogging condition (minimum basin stage and maximum outlet discharge), and also under the maximum clogging condition (maximum basin stage and minimum outlet discharge).

An example of trashrack head loss calculations for the Pima Freeway Basin outlet to the Hayden Road Conduits is provided. A sketch of such a trashrack installed on that outlet is shown in Appendix B. For a discharge of 2,000 cfs, the head loss for 50 percent rack clogging is 1.8 feet by the Maricopa County manual procedure and 1.6 feet by the Design of Small Dams procedure (see Appendix B for calculations). Use of the Design of Small Dams procedure to incorporate trashrack head losses into the outlet rating curve for that basin is provided in Appendix B. Two rating curves are developed, one for an unclogged rack and the other for a 50 percent clogged rack. Results are presented in Appendix B. For the 50 percent clogging case, trashrack head losses are not added to the conduit inlet head losses until the rack is submerged. This is reasonable and practical since it assumes that the trashrack is maintained and not clogged at the onset of use, and that clogging is not meaningful prior to full submergence of the rack. In the example, clogged trashrack head losses are estimated for discharge in excess of 1,260 cfs. For the 0 percent clogging case, the trashrack head loss is assumed to be insignificant and the only loss is that at the conduit inlet. The resulting two Pima Freeway Basin outlet rating curves for the double-barrel 108 inch outlet conduits are provided in Appendix B.

Using those two rating curves with an HEC-1 reservoir routing model of the Pima Freeway Basin (100-year, 24-hour storm), produces the following results:

Maximum head loss (50 percent clogging):

Maximum water surface elevation = 1,607.7 feet

Maximum discharge into conduits = 2,038 cfs

Minimum head loss (no clogging):

Maximum water surface elevation = 1,607.2 feet

Maximum discharge into conduits = 2,150 cfs

This procedure will be used in the analysis of basin outlets for the PR3B project. For inlets to the conduits from channels (such as the Sierra Pinta channel), appropriate head losses at the trashrack will need to be incorporated into the analysis to verify that hydraulic performance is acceptable under an appropriate trashrack clogging condition.

George V. Sabol, PhD, PE
Senior Associate
Water Resources Division

Attachment

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Appendix A

DRAINAGE DESIGN MANUAL

for Maricopa County, Arizona

Volume II

HYDRAULICS

January 28, 1996

Flood Control District
of Maricopa County
2801 West Durango Street
Phoenix, Arizona 85009
(602) 506-1501

The formula for y' is based on momentum considerations and is as follows:

$$y' = \frac{Q_2 V_2 - Q_1 V_1 - Q_3 V_3 \cos\theta_j}{0.5(A_1 + A_2)g} \tag{5.4}$$

The subscripts 1, 2, and 3 refer to the outlet pipe, the upstream pipe, and the lateral pipe respectively.

5.2.2.11: Trash Racks: For trash racks with approach velocities less than 3 fps, it is not necessary to include a loss for the trash rack; however, for velocities greater than 3 fps, such computations are required.

Trash racks can promote debris buildup and the subsequent reduction of hydraulic performance. Thorough anaylsis of this potential should be undertaken prior to their use. Depending on the anticipated volume and size of the debris an open area between the bars of 1.5 to 3.0 times the area of the culvert entrance should be provided.

$$H_g = 1.5 \frac{[V_g^2 - V_a^2]}{2g} \tag{5.5}$$

Trash rack losses are a function of velocity, bar thickness, bar spacing, and orientation of the flow entering the rack, the latter condition being an important factor. Trash racks with bars oriented horizontally are not permitted, and horizontal bars used to support vertically oriented bars should be as small as practical and kept to the minimum required to meet structural requirements.

The expected loss from a trash rack is greatly affected by the approach angle. The loss computed by Equation 5.5 should be multiplied by the appropriate value from Table 5.1, when the approach channel and culvert are at an angle to each other.

Table 5.1
Loss Factors for Approach Angle Skewed to Entrance

Approach Angle, degrees	Loss Factor
0	1.0
20	1.7
40	3.0
60	6.0

UNITED STATES DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

DESIGN OF SMALL DAMS

A Water Resources Technical Publication

Second Edition
1973
Revised Reprint
1974

iary equipment such as ventilating fans, heaters, flow measuring and recording meters, air pumps, small power-generator sets, and equipment needed for maintenance.

229. *Intake Structures.*—In addition to forming the entrance into the outlet works, an intake structure may accommodate control devices. It also supports necessary auxiliary appurtenances (such as trashracks, fish screens, and bypass devices), and it may include temporary diversion openings and provisions for installation of bulkhead or stoplog closure devices.

An intake structure may take on many forms, depending on the functions it must serve as noted above, on the range in reservoir head under which it must operate, on the discharge it must handle, on the frequency of reservoir drawdown, on the trash conditions in the reservoir which will determine the need for or the frequency of cleaning of the trashracks, on reservoir ice conditions or wave action which could affect the stability, and on other such considerations. An intake structure may either be submerged or extended as a tower to some height above the maximum reservoir water surface, depending on its function. A tower must be provided if the controls are placed at the intake, or if an operating platform is needed for trash raking, maintaining and cleaning of fish screens, or installing stoplogs. Where the structure serves only as an entrance to the outlet conduit and where trash cleaning ordinarily will not be required, a submerged structure can be adopted.

The conduit entrance can be placed vertically, inclined, or horizontally, depending on intake requirements. Where a sill level higher than the conduit level is desired, the intake can be a drop inlet similar to the entrance of a drop inlet spillway. A vertical entrance is usually provided for inlets at the conduit level. In certain instances at small installations where the gate is placed and operated on the upstream slope of a low dam, an inclined entrance can be adopted. Such an arrangement is typified by the Ortega Reservoir outlet shown on figure 306. In most cases conduit entrances should be rounded or bellmouthed to reduce hydraulic entrance losses.

The necessity for trashracks on an outlet works depends on the size of the sluice or conduit, the type of control device used, the nature of the trash burden in the reservoir, the utilization of the water, the need for excluding small trash from the outflow, and other factors. These factors will determine the type of trashracks and the size of the openings. Where an outlet consists of a small conduit with valve controls, closely spaced trashbars will be needed to exclude small trash. Where an outlet involves a large conduit with large slide gate controls, the racks can be more widely spaced. If there is no danger of clogging or damage from small trash, a trashrack may consist simply of struts and beams placed to exclude only the larger trees and such floating debris. The rack arrangement will also depend on accessibility for removing accumulated trash. Thus, a submerged rack which seldom will be unwatered must be more substantial than one which is at or near the surface. Similarly, an outlet with controls at the entrance where the gates can be jammed by trash protruding through the rack bars must have a more substantial rack arrangement than if the controls are not at the entrance.

Trash bars usually consist of thin, flat steel bars which are placed on edge from 3 to 6 inches apart and assembled in rack sections. The required area of the trashrack is fixed by a limiting velocity through the rack, which in turn depends on the nature of the trash which must be excluded. Where the trashracks are inaccessible for cleaning, the velocity through the racks ordinarily should not exceed 2 feet per second. A velocity of up to approximately 5 feet per second may be tolerated for racks which are accessible for cleaning.

Trashrack structures also may take on varied shapes, depending on how they are mounted or arranged on the intake structure. Trashracks for a drop inlet intake are generally formed as a cage surmounting the entrance. They may be arranged as an open box placed in front of a vertical entrance or they may be positioned along the front side of a tower structure. Figures 300 through 306 illustrate various arrangements of trashracks at entrances to outlet works.

(c) *Trashrack Losses.*—Trashrack structures which consist of widely spaced structural members without rack bars will cause very little head loss, and trashrack losses in such a case might be neglected in computing conduit losses. When the trash structure consists of racks of bars, the loss will depend on the bar thickness, depth, and spacing. An average approximation can be obtained [2] from the equation:

$$\text{Loss}=K_t\frac{v_n^2}{2g}$$

where:

$$K_t=1.45-0.45\frac{a_n}{a_g}-\left(\frac{a_n}{a_g}\right)^2 \qquad (11)$$

In the above:

K_t=the trashrack loss coefficient (empirical),
a_n=the net area through the rack bars,
a_g=the gross area of the racks and supports, and,
v_n=the velocity through the net trashrack area.

Where maximum loss values are desired, assume that 50 percent of the rack area is clogged. This will result in twice the velocity through the trashrack. For minimum trashrack losses, assume no clogging of the openings when computing the loss coefficient, or neglect the loss entirely.

(d) *Entrance Losses.*—The loss of head at the entrance of a conduit is comparable to the loss in a short tube or in a sluice. If H is the head producing the discharge, C is the coefficient of discharge, and a is the area, the discharge

Q is equal to $Ca\sqrt{2gH}$

and the velocity

v is equal to $C\sqrt{2gH}$.

Or,

$$H=\frac{1}{C^2}\frac{v^2}{2g} \qquad (12)$$

Since H is the sum of the velocity head h_v and the head lost at the entrance h_e, equation (12) may be written:

$$\frac{v^2}{2g}+h_e=\frac{1}{C^2}\frac{v^2}{2g} \text{ or } h_e=\left(\frac{1}{C^2}-1\right)\frac{v^2}{2g}$$

Then:

$$K_e=\left(\frac{1}{C^2}-1\right) \qquad (13)$$

Coefficients of discharge for square sluice entrances are shown on figure 309. Coefficients of discharge and loss coefficients for typical entrances for conduits, as given in various texts and technical papers, are listed in table 33.

TABLE 33.—*Coefficients of discharge and loss coefficients for conduit entrances*

	Coefficient C			Loss coefficient K_e		
	Maximum	Minimum	Average	Maximum	Minimum	Average
(a) Gate in thin wall—unsuppressed contraction	0.70	0.60	0.63	1.80	1.00	1.50
(b) Gate in thin wall—bottom and sides suppressed.	.81	.68	.70	1.20	0.50	1.00
(c) Gate in thin wall—corners rounded	.95	.71	.82	1.00	.10	0.50
(d) Square-cornered entrances	.85	.77	.82	.70	.40	.50
(e) Slightly rounded entrances	.92	.79	.90	.60	.18	.23
(f) Fully rounded entrances $\frac{r}{D}\geq 0.15$	.96	.88	.95	.27	.08	.10
(g) Circular bellmouth entrances	.98	.95	.98	.10	.04	.05
(h) Square bellmouth entrances	.97	.91	.93	.20	.07	.16
(i) Inward projecting entrances	.80	.72	.75	.93	.56	.80

(e) *Bend Losses.*—Bend losses in closed conduits in excess of those due to friction loss through the length of the bend are a function of the bend radius, pipe diameter, and the angle through which the bend turns. Although experimental data on bend losses in large pipes are meager, the loss can be related to those determined for smaller pipe. Figure 311(A) shows the coefficients found by various investigators for 90° bends for various ratios of radius of bend to diameter of pipe, and an adjusted curve assumed to be suitable for large pipes.

HYDRAULIC DESIGN OF HIGHWAY CULVERTS

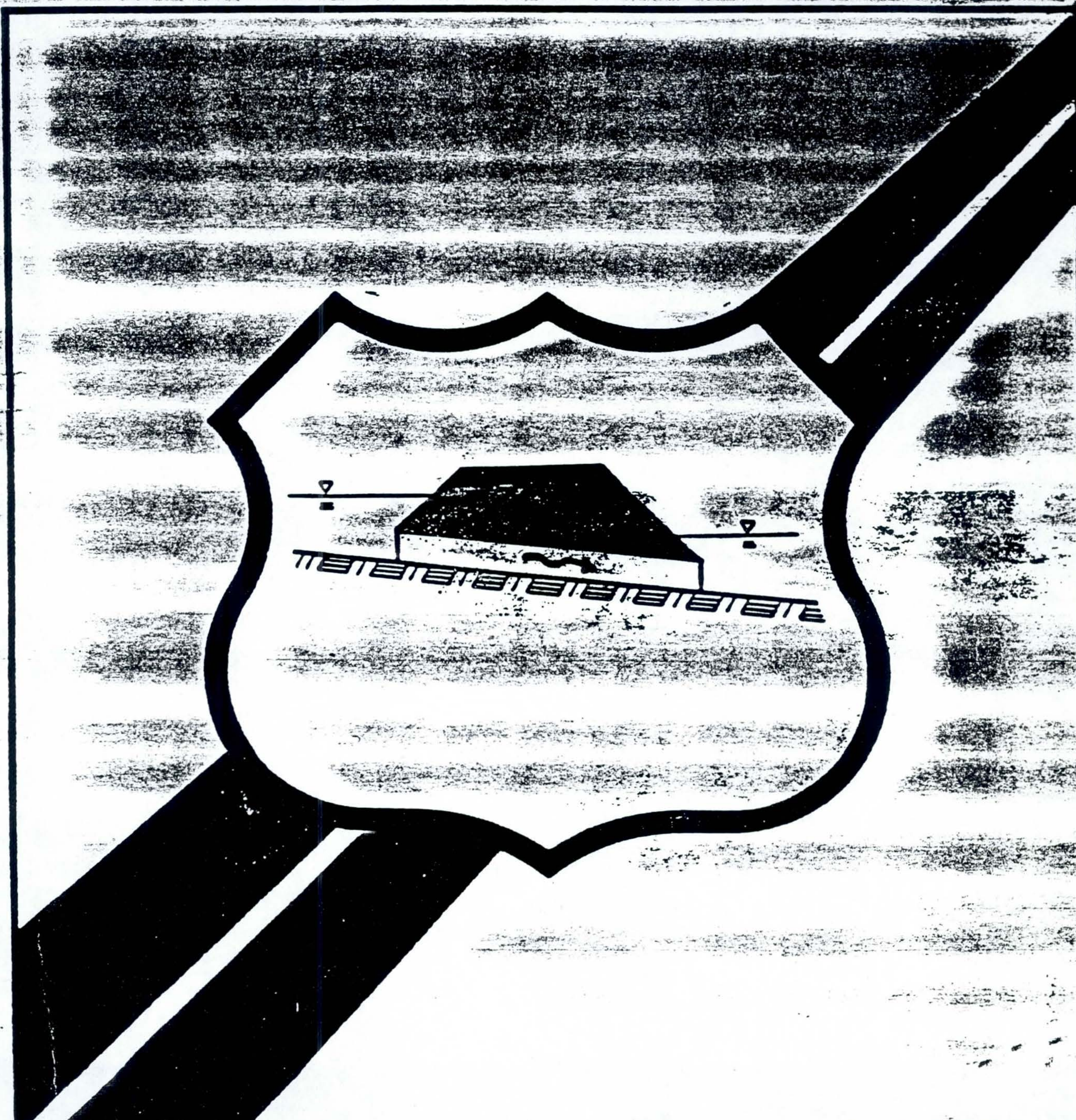
Research, Development, and Technology
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, Virginia 22101

U.S. Department of Transportation
Federal Highway Administration

Hydraulic Design Series No. 5

Report No. FHWA-IP-85-15

September 1985



For culvert ends with headwalls, fill should be warped behind them to limit their exposure. (Markers should be placed on concealed culvert ends to protect roadside maintenance personnel.)

2. Safety Barriers and Grates. Additional traffic safety can be achieved by the installation of safety barriers and grates. Safety barriers should be considered in the form of guardrails along the roadside near a culvert when adequate recovery distance cannot be achieved, or for abnormally steep fill slopes. (figure VI-28) Traversible grates placed over culvert openings will reduce vehicle impact forces and the likelihood of overturning. (figure VI-29)

Safety grates promote debris buildup and the subsequent reduction of hydraulic performance. Thorough analysis of this potential should be undertaken prior to the selection of this safety alternative. Good design practice provides an open area between bars of 1.5 to 3.0 times the area of the culvert entrance depending on the anticipated volume and size of debris. Bar grates placed against the entrance of the culvert are unacceptable. (figure VI - 30). Reference (47) indicates that the head loss due to a bar grate can be estimated as follows.

$$H_g = 1.5 \left(\frac{V_g^2 - V_u^2}{2g} \right) \qquad (24)$$

H_g is the head loss due to the bar grate, ft

V_g is the velocity between the bars, ft/s

V_u is the approach velocity, ft/s

Another formula for the head loss in bar racks with vertical bars is found in reference (48).

$$H_g = K_g \left(\frac{w}{x} \right) \left(\frac{V_u^2}{2g} \right) \sin \theta_g \qquad (49)$$

K_g is a dimensionless bar shape factor, equal to:

2.42 - sharp-edged rectangular bars

1.83 - rectangular bars with semi-circular upstream face

1.79 - circular bars

1.67 - rectangular bars with semi-circular upstream and downstream faces

w is the maximum cross-sectional width of the bars facing the flow, ft

x is the minimum clear spacing between bars, ft

θ_g is the angle of the grate with respect to the horizontal, degrees

Both of the above equations are empirical and should be used with caution. Research on loss coefficients in safety grates is documented in reference (47). In all cases, the head losses are for clean grates and they must be increased to account for debris buildup.

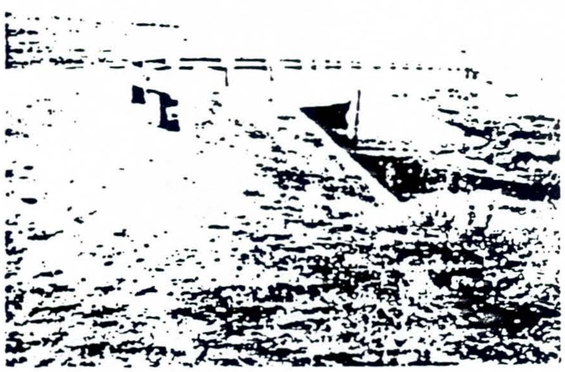


Figure VI-28--Guardrail adjacent to culvert headwall.

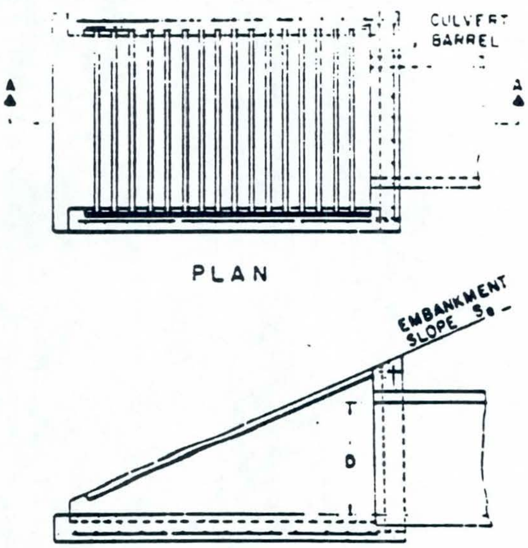


Figure VI-29--Endwall for safety grate.

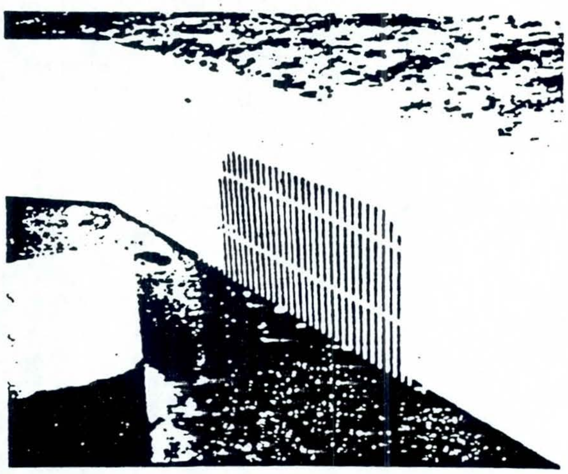


Figure VI-30--Safety grate flush with culvert entrance.

Culverts have always attracted the attention and curiosity of children. In high population areas where hazards could exist, access to culverts should be prevented. Safety grates can serve this function. If clogging by debris is a problem, fencing around the culvert ends is an acceptable alternative to grates.

G. Structural Considerations.

Proper structural design is critical to the performance and service life of a culvert. The structural design of a highway culvert begins with the analysis of moments, thrusts, and shears caused by embankment and traffic loads, and by hydrostatic and hydrodynamic forces. The culvert barrel, acting in harmony with the bedding and fill, must be able to resist these sizeable forces. Anchorage devices, endwalls, and wingwalls are often required to maintain the structural integrity of a culvert barrel by resisting flotation and inlet or outlet movement and distortion.

1. General Structural Analysis. Loads affecting culvert barrel design include the culvert weight, fluid loads, earth and pavement loads, and the weight and impact of surface vehicles. Culvert weights per unit length are available from culvert manufacturers. The weight of fluid per unit length can be obtained from the culvert barrel geometry and the unit weight of water.

The magnitude of the earth and pavement load (dead load) is dependent upon the weight of the prism above the barrel and the soil-structure interaction factor. The soil-structure interaction factor is the ratio of the earth prism load on the culvert to the earth prism weight. Conditions which affect this factor include soil type, backfill compaction, culvert material (rigid or flexible), and the type of culvert installation.

Two common types of culvert installations are depicted in figure VI-31. In the positive projecting embankment installation, the culvert barrel is supported on the original streambed or compacted fill and covered by the embankment material. A negative projecting embankment is similar except that additional load support is gained from the existing banks of a deep stream bed. Each of these installations requires the establishment of an appropriate soil structure interaction factor or the determination of the load by appropriate tests, finite element analysis, or previous experience.

39. "Design Considerations and Calculations for Fishways Through Box Culverts," Fred F.M. Chang and J. M. Normann, Unpublished Text, Hydraulics Branch, Bridge Division, FHWA, Office of Engineering, Washington, D.C. 20590, September 1976.

40. "Hydraulic Design of Energy Dissipators for Culverts and Channels," M. L. Corry, P.L. Thompson, F.J. Watts, J.S. Jones and D.L. Richards, HEC No. 14, Hydraulics Branch, Bridge Division, Office of Engineering, FHWA, Washington, D.C. 20590, September 1983.

41. "Evaluating Scour at Culvert Outlets," J.F. Ruff and S.R. Abt, Transportation Research Board Record N785, Transportation Research Board, Publications Office, 2101 Constitution Avenue, NW Washington, D.C. 20418, 1980.

42. "Field and Laboratory Evaluation of Energy Dissipators for Culverts and Storm Drain Outlets. Volume I - Modular Energy Dissipators, Internal Energy Dissipators and Rock Channel Protection," S. Sarikelle and A.L. Simon, FHWA-OH-79-03, Akron University Department of Civil Engineering, Akron, Ohio 44325, December 1980. Available from the National Technical Information Service, Springfield, Virginia 22161.

43. "Field and Laboratory Evaluation of Energy Dissipators for Culvert and Storm Outlets. Volume II--Field Performance of Corrugated Metal Culverts," S. Sarikelle and A.L. Simon, FHWA-OH-79-04, Akron University, 302 East Buchtel Avenue, Akron, Ohio 44325, December 1980. Available from the National Technical Information Service, Springfield, Virginia 22161.

44. "Highways in the River Environment - Hydraulic and Environmental Design Considerations," Civil Engineering Department, Colorado State University, Fort Collins, Colorado, May 1975.

45. "Debris Control Structures," G. Reihsen and L. J. Harrison, HEC No. 9, Hydraulics Branch, Bridge Division, Office of Engineering, FHWA, Washington, D.C. 20590, 1971.

46. "The Design of Encroachments on Flood Plains Using Risk Analysis," M.L. Corry, J.S. Jones and P.L. Thompson, HEC No. 17, Hydraulics Branch, Bridge Division, Office of Engineering, FHWA, Washington, D.C. 20590, October 1980.

47. "Handbook of Applied Hydraulics," Calvin V. Davis, McGraw-Hill Book Co., Inc., New York, N.Y., 1952.

48. "Wastewater Engineering," Metcalf & Eddy, Inc., McGraw-Hill Book Co., New York, N.Y., 1972

49. "Hydraulic Performance of Culverts with Safety Grates," L.W. Mays, M.E. Walker, M.S. Bennet, and R.P. Arbuckle, FHWA/TX-82/55T301-1F; PB83-219626, Texas University, Austin Center for Transportation Research, Austin, Texas 78712, March 1983. Available from the National Technical Information Service, Springfield, Virginia 22161.

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OPEN-CHANNEL HYDRAULICS

VEN TE CHOW, Ph.D.

Professor of Hydraulic Engineering
University of Illinois

McGRAW-HILL BOOK COMPANY

New York Toronto London

1959

equal to 4. For ratios of 7 and 13, the effect of increasing length on backwater is shown in Fig. 17-36, which is plotted with Yarnell's data. It seems that the backwater caused by the long piers is greater when the pier ends are semicircular than when they are square. It is probable that an abrupt entrance in the case of square pier ends tends to decrease friction losses for a short distance downstream because of its effect on the velocity distribution, since the velocity along the walls is reduced. The effect of bridge piers present in a constriction has been considered in Art. 17-6 and in Fig. 17-23*d*.

17-11. Flow through Pile Trestles. Yarnell's investigation [41] indicates that the Nagler formula may be suitably applied to subcritical flow passing a pile trestle and the d'Aubuisson formula to supercritical flow passing a pile trestle. The following coefficients are recommended for use in these formulas:

Type of trestle	K_N	K_A
Single-track 5-pile trestle bent		
Parallel to current	0.90	0.96
At 10° angle with current	0.90	
At 20° angle with current	0.89	
At 30° angle with current	0.87	
Double-track 10-pile trestle bent	0.82	0.88
Two single-track 5-pile trestle bents	0.79	0.86

The amount of channel contraction is to be taken as the average diameter of the piles plus the thickness of the sway bracing, disregarding the angle at which the bent is set with the current.

The effect of trestle piles present in a constriction has been considered in Art. 17-6 and in Fig. 17-23*c*.

17-12. Flow through Trash Racks. For flow through trash racks, the designer is primarily concerned with the amount of head loss due to the resistance of the rack. This may be expressed in terms of the velocity head of the approach flow, or

$$h_f = c\,\frac{V^2}{2g} \tag{17-32}$$

where V is the velocity of approach ahead of the rack and where c is a coefficient depending on the cross-sectional form, thickness s, length L of the rack bar, clear distance b between bars, angle δ of inclination of the bar from the horizontal, and angle α between the direction of flow and the length of the bar.

On the basis of the experimental data for rack bars of various forms and with $\alpha = 0$, Kirschmer [52,53] has set up the following formula for c:

$$c = \beta \left(\frac{s}{b}\right)^{4/3} \sin\delta \tag{17-33}$$

where β is a coefficient having the values listed below:

Form of rack bar	*Value of β*
Square nose and tail, $L/s = 5$	2.42
Square nose and semicircular tail, $L/s = 5$	1.83
Semicircular nose and tail, $L/s = 5$	1.67
Round	1.79
Airfoil	0.76

Spangler [55] has extended the experiment and determined the value of β for $\alpha = 30°$, $45°$, and $60°$.

According to Fellenius [54], an empirical formula for c can be given as follows:

$$c = \mu \left(\frac{s}{s + b} \right)^2 \sin^x \delta \tag{17-34}$$

where the coefficient μ and exponent x have the following values:

Form of rack bar	*Value of μ*	*Value of x*
Square nose and tail		
With sharp corners, $L/s = 10$	7.1	1.0
With sharp corners, $L/s = 12$	6.2	1.0
With slightly rounded corners, $L/s = 8$ to 11	6.1	1.0
Semicircular nose and tail, $L/s = 7$	5.6	1.5

In general, $x = 1.0$ for bars having sharp or slightly rounded corners and $x = 1.5$ for bars having rounded corners. For cross-connected and clipped rack bars, the value of μ should be increased by about 22.5%.

Scimemi [56] and Koženy [50] have provided values of c, β, and μ, and other data for racks installed in several hydropower plants.

17-13. Underflow Gates. Certain control gates in canals may be called *underflow gates*[1] from the fact that water passes underneath the structure. Common examples are the sluice gate, Tainter gate (or radial gate), and rolling gate (Fig. 17-37). In designing such gates the hydraulic engineer is most interested in two major features: the head-discharge relationship and the pressure distribution over the gate surfaces for various positions of the gate and forms of the gate lip. The form of the lip will not only affect the velocity and pressure distributions and the energy loss in flow through the gate opening, but may also develop very disturbing vibrations that should be avoided during gate operation. As

[1] In contrast to the underflow gate is the *overflow gate* through which the water flows over the structure. The drum gate (Art. 14-9) is an example of an overflow gate. Hydraulically speaking, the overflow gate acts like a weir as much as the underflow gate acts like an orifice. There are also designs for which the water flows above and below the structure at the same time (Fig. 17-37).

G. BRADY

UNITED STATES
DEPARTMENT OF THE INTERIOR
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TRANSMITTAL OF DESIGN STANDARDS — **Release No.** DS-3-5

Number and Title:

Design Standards No. 3 - CANALS AND RELATED STRUCTURES

Insert Sheets:	Remove Sheets:
Design Standards No. 3 (182 sheets)	Design Standards No. 3

Other revisions:

Summary of changes:

This Design Standards has been completely revised and updated to present current Bureau practice in the design of canals and related structures. Among the more significant changes or additions are the following:

1. A statement of the Bureau's new waterway policy, which places emphasis on lining waterways or placing them in pipes.

2. A description of several types of automatic control features, and a discussion of the importance of considering automation of a canal system in the planning and design stage.

3. An increase in the maximum concrete compressive stress from 3,000 to 3,750 pounds per square inch.

NOTE: This is a complete replacement for Design Standards No. 3.

Approved:

Wm H Wolf

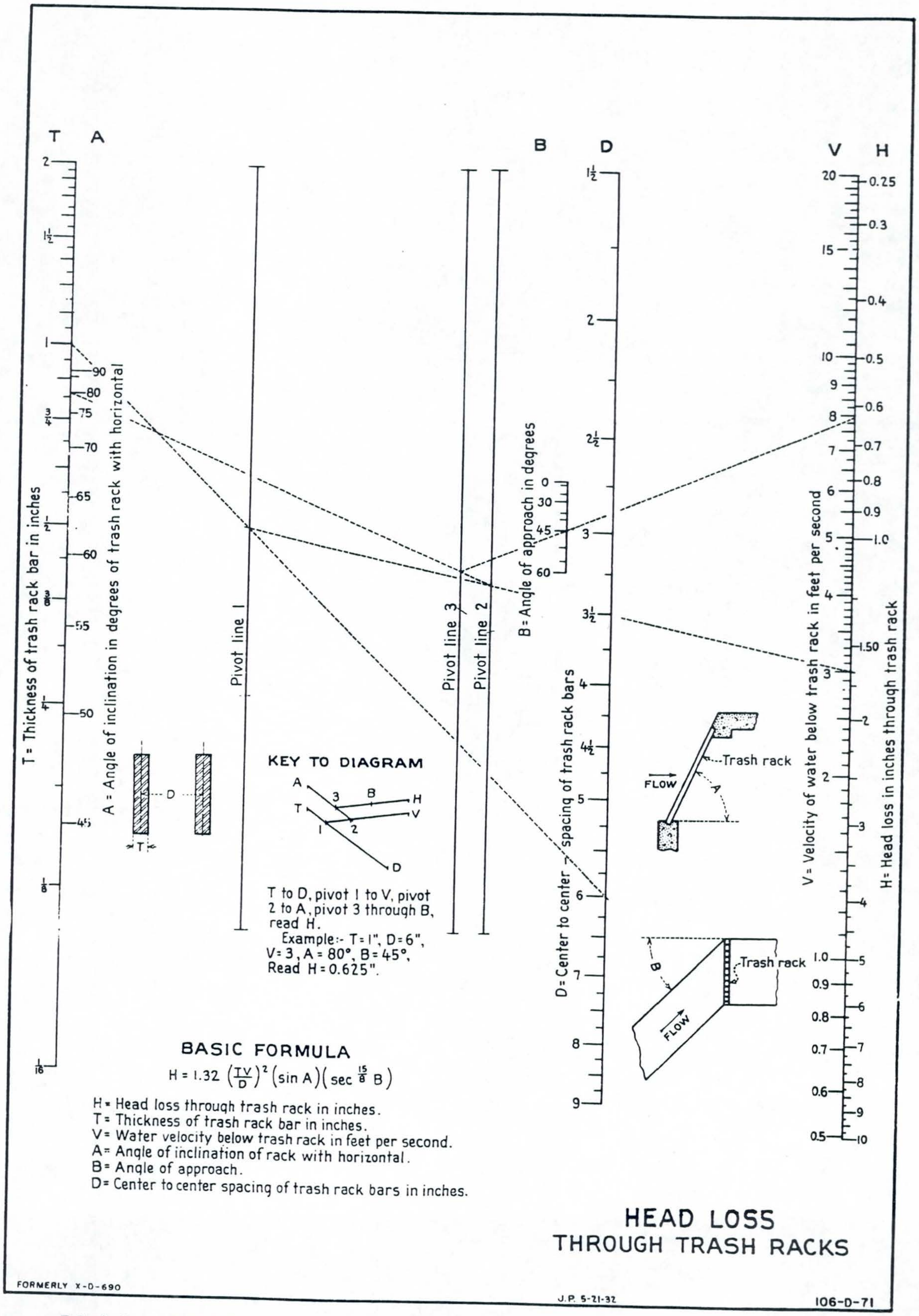
Acting Chief Engineer — December 8, 1967 — **Date**

(To be filled in by employee who files this release in appropriate folder.

The above change has been made in the Design Standards.

Signature — **Date**

Fig. 15 Par. 2.25D HEAD LOSS THROUGH TRASHRACKS



2.31

RIPRAP--Continued

SIPHONS AND TUNNELS .31 The following protection is considered minimum for siphons and tunnels [in] earth-surfaced canals:

d = water depth adjacent to structure (feet)	Inlet	Outlet
0 to 2.00	None	None
2.01 to 3.50	None	Type 1
3.51 to 7.00	Type 1	Type 2
7.01 to 10.00	Type 2	Type 3

Water depths over 10 feet require special consideration.

Protection called for on inlets may be omitted if the velocity is less than 2.5 feet per second.

Where protection is required on inlets, length = d (3.0 feet minimum).

Where protection is required on outlets, length = 2.5d (5.0 feet minimum).

PARSHALL FLUMES, CHECKS, CHECK DROPS, INCLINED DROPS, CHUTES, AND CLOSED-CONDUIT DROPS .32 The following protection is considered minimum for Parshall flumes, checks, check drops, inclined drops, chutes, and closed-conduit drops with control section on concrete, that is, where critical depth does not occur off the concrete. Where critical depth may occur off the concrete, the next higher type of protection should be used at the inlet.

d = water depth adjacent to structure (feet)	Inlet	Outlet
0 to 2.00	None	Type 2
2.01 to 3.50	None	Type 2
3.51 to 7.00	Type 1	Type 3
7.01 to 10.00	Type 2	Type 4

Water depths over 10 feet require special consideration.

Protection called for on inlets may be omitted if velocity is less than 2.5 feet per second.

Where protection is required on inlets, length = d (3.0 feet minimum).

Length of protection on outlets = 2.5d (5.0 feet minimum).

Where turbulent water may occur at the outlet, the length of protection should be increased to 4d.

Gates or stoplogs near the outlet increase turbulence.

TURNOUTS .33 Protection is not required on the inlets to most small turnouts. If the turnout capacity is 50 percent or more of the capacity of the canal, the protection recommended for siphon inlets should be used. Protection at the outlets of turnouts should be the same as for siphons, based on the water depth in the lateral adjacent to the outlet transition.

Appendix B

SOURCES
(1) FCDMC : DRAINAGE DESIGN MANUAL, VOL. II
(2) USBR : DESIGN OF SMALL DAMS
(3) USBR : DESIGN STANDARDS NO. 3, CANALS AND RELATED STRUCTURE
(4) FHWA : HYDRAULIC DESIGN OF HIGHWAY CULVERTS
(5) KIRSCHMER : OPEN CHANNEL HYDRAULICS (CHOW)

EQUATIONS

(1) $H = 1.5\left(\frac{V_g^2 - V_a^2}{2g}\right)$ WHERE: V_g = VELOCITY BETWEEN BARS
V_a = APPROACH VELOCITY

* MULTIPLY BY APPROACH ANGLE FACTORS
* BAR SPACING SHOULD BE 1.5 - 3.0 TIME CULVERT OPENING

(2) $H = \left[1.45 - 0.45\frac{a_n}{a_g} - \left(\frac{a_n}{a_g}\right)^2\right]\frac{V_n^2}{2g}$ WHERE: a_n = NET AREA THROUGH BA
a_g = GROSS AREA OF TRASH
V_n = VELOCITY BETWEEN B

* FOR MAX. LOSSES ASSUME 50% CLOGGING BY USING $2V_n$

(3) $H = 1.32\left(\frac{TV}{D}\right)^2(\sin A)(\sec B)^{\frac{15}{8}}$ WHERE: T = BAR THICKNESS
V = APPROACH VELOCITY
D = BAR SPACING
A = ANGLE OF TRASHRACK INCLINATION
B = ANGLE OF APPROACH

(4) $H = 1.5\left(\frac{V_g^2 - V_a^2}{2g}\right)$

* SAME AS (1) EXCEPT NO CONSIDERATION TO APPROACH ANGLE

(5) $H = \beta\left(\frac{S}{B}\right)^{4/3}\sin\delta$ FOR APPROACH ANGLE = 0°

WHERE: β = BAR SHAPE FACTOR
S = BAR THICKNESS
B = CLEAR DISTANCE BETWEEN BARS
δ = ANGLE OF INCLINATION OF TRASHRA

* SIMILAR TO OLDER FCDMC EQUATION

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Stantech Consulting

Project: PIMA ROAD 3 BASINS Project Number:

Notes: Scale:

Page 2 of Page(s)

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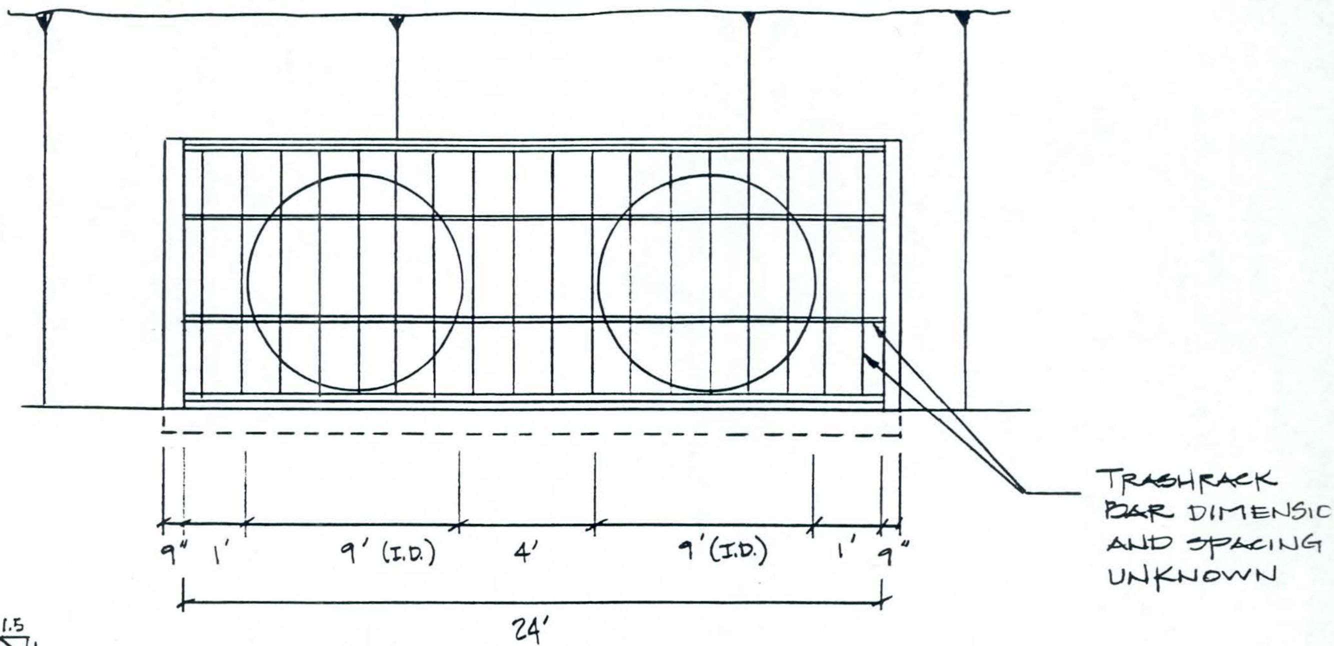

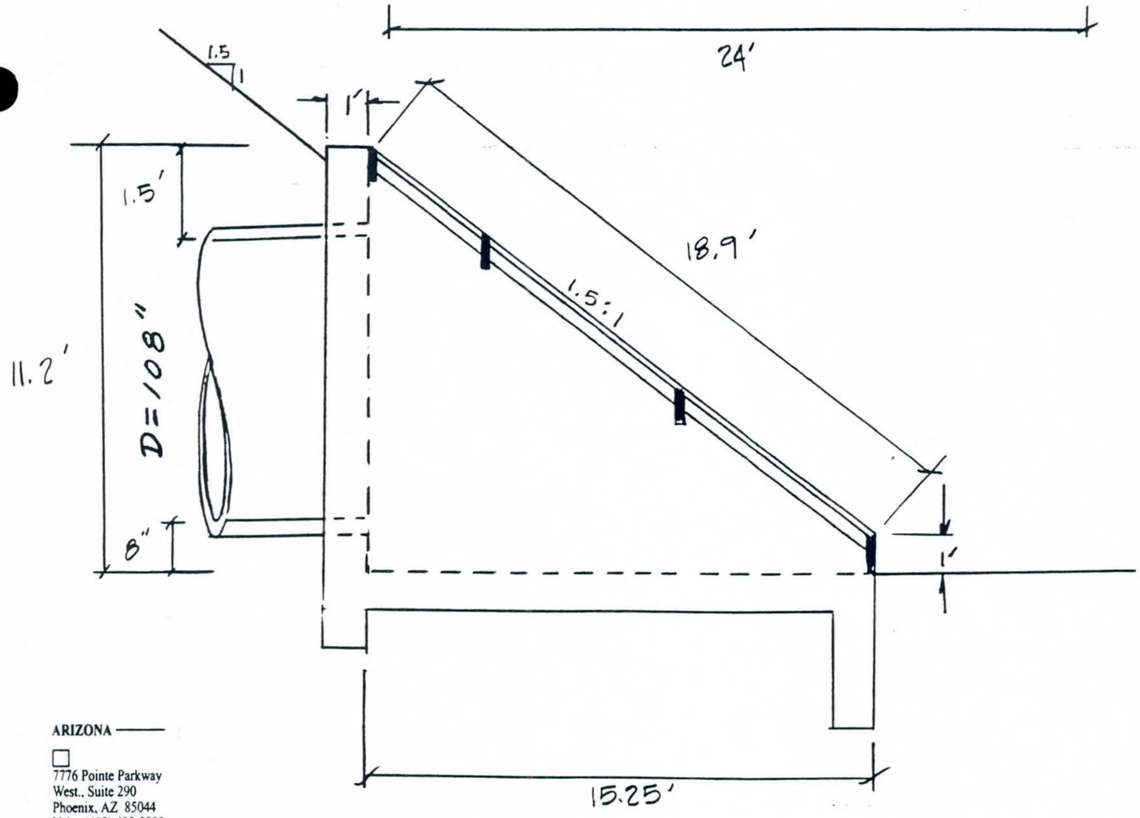

AREA OF TRASHRACK (GROSS), $A_g = (18.9)(24) = 453.6$ SF

* ASSUME 10% OF GROSS AREA FOR BARS

AREA OF TRASHRACK OPENINGS (NET), $A_n = (0.9)(453.6) = 408.2$ SF

AREA OF CONDUIT, $A_c = 2\left(\pi \frac{9^2}{4}\right) = 127.2$ SF

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* FLDMC TRASHRACK EQN

$$H = 1.5\left(\frac{V_g^2 - V_a^2}{2g}\right)^{*}$$

* TO ACCOUNT FOR APPROACH ANGLE REFER TO MANUAL

ASSUME THE APPROACH VELOCITY CAN BE CALCULATED FROM CONTINUITY EQN AT $Q = 2{,}000^{CFS}$ AND THE GROSS TRASHRACK AREA

$$V_a = \frac{Q}{A_g} = \frac{2{,}000}{453.6} = 4.4^{FPS}$$

ASSUME THE VELOCITY THROUGH THE BARS CAN BE CALCULATED FROM CONTINUITY EQN AT $Q = 2{,}000^{CFS}$ AND 50% OF THE NET TRASHRACK AREA

$$V_g = \frac{Q}{0.5A_n} = \frac{2{,}000}{(0.5)(408.2)} = 9.8^{FPS}$$

$$H = 1.5\left(\frac{9.8^2 - 4.4^2}{2g}\right) = 1.79'$$

* USBR (DESIGN OF SMALL DAMS) TRASHRACK EQN

CASE 1: ASSUME THE VELOCITY THROUGH THE BARS IS BASED ON 50% OF THE NET AREA

$$H = K_t \frac{V_n^2}{2g} \; ; \; V_n = 9.8^{FPS}$$

$$K_t = 1.45 - 0.45\frac{A_n}{A_g} - \left(\frac{A_n}{A_g}\right)^2$$ WHERE A_n = 50% OF A_n

$$= 1.45 - 0.45\left(\frac{204.1}{453.6}\right) - \left(\frac{204.1}{453.6}\right)^2$$

$$K_t = 1.04$$

$$H = 1.04\,\frac{9.8^2}{2g} = 1.55'$$

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HEADLOSS CALCULATIONS USING USBR EQN.

CASE (1) 50% CLOGGING

DISCHARGE CFS	TRASHRACK AREA* GROSS (A_g) SF	NET (A_n) SF	HEADLOSS COEFFICIENT K_t	VELOCITY THROUGH BARS (V_n) FPS	HEADLOSS FT
1,260	453.6	204.1	1.04	6.17	0.61
1,470	453.6	204.1	1.04	7.20	0.84
1,680	453.6	204.1	1.04	8.23	1.09
1,890	453.6	204.1	1.04	9.26	1.38
2,100	453.6	204.1	1.04	10.29	1.71

* ONLY CONSIDERS FULL SUBMERENCE OF TRASH RACK. ASSUME FULL SUMBERGIENCE BEGINS AT Q = 1,260 CFS.

CASE (4) 0% CLOGGING, HEADLOSS THROUGH TRASHRACK IS INSIGNIFICANT

OUTLET STRUCTURE RATING CURVES

DISCHARGE CFS	STAGE CASE 1* FT	CASE 4 FT
0	1590.00	1590.00
210	1593.39	1593.11
420	1595.07	1594.89
630	1596.37	1596.30
840	1597.58	1597.53
1050	1598.89	1598.70
1260	1600.55	1599.94
1470	1602.16	1601.32
1680	1603.98	1602.89
1890	1606.07	1604.69
2100	1608.44	1606.73
2265	---	1608.50

CASE 1

STAGE = 1,607.71
OUTFLOW = 2,038 CFS

CASE 4

STAGE = 1,607.24
OUTFLOW = 2,150 CFS

* THE STAGES FOR Q = 210 CFS TO Q = 1,050 CFS ARE ASSUMED TO BE THE STAGE FOR A MITERED INLET TO ACCOUNT FOR TRASHRACK HEADLOSS FOR PARTIALLY SUBMERGED CONDITIONS.

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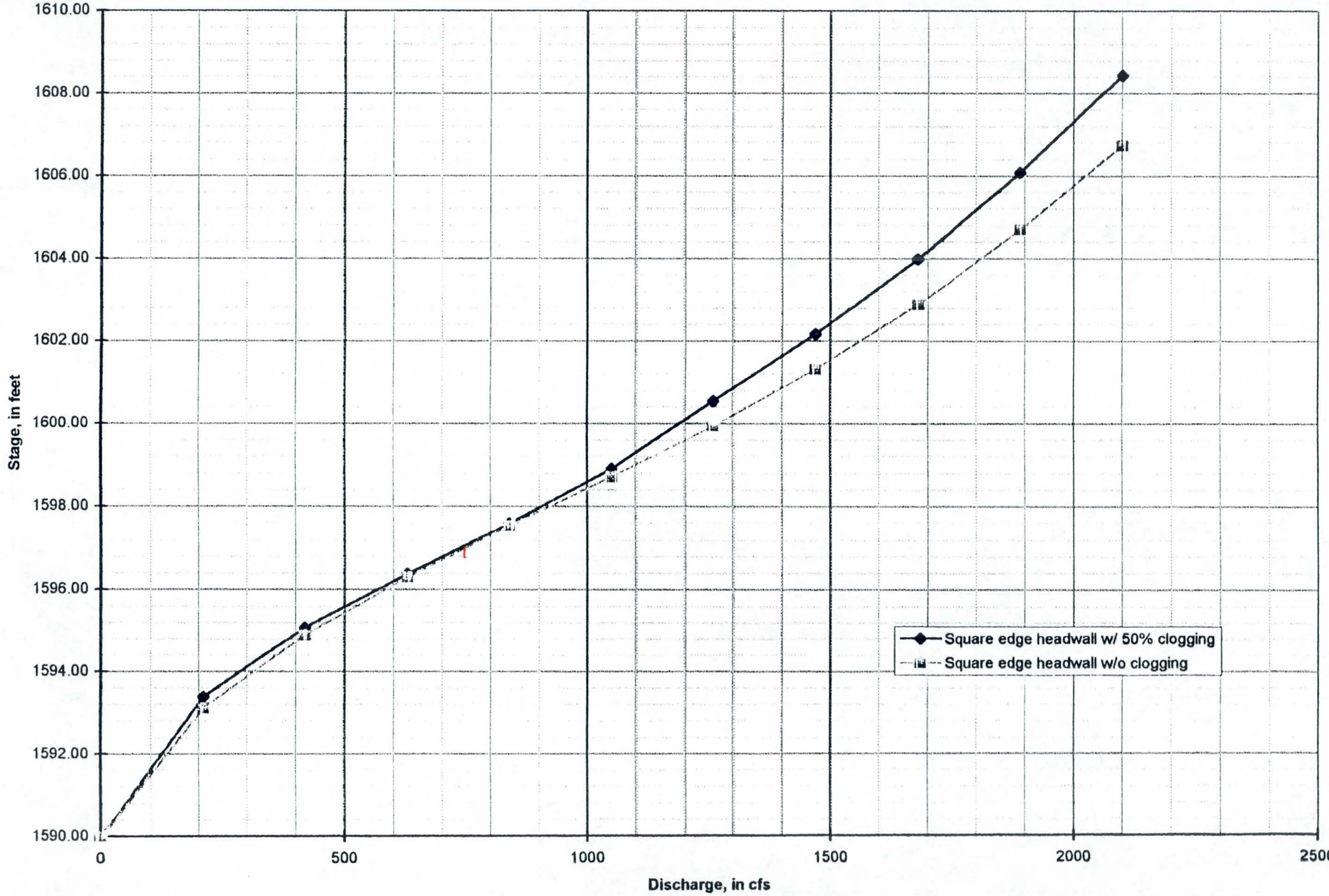
Stage - Discharge curve for the
Pima Freeway outlet conduit

	Stage, in feet	
Discharge cfs	**50% Clogging**	**No Clogging**
0	1590.00	1590.00
210	1593.39	1593.11
420	1595.07	1594.89
630	1596.37	1596.30
840	1597.58	1597.53
1050	1598.89	1598.70
1260	1600.55	1599.94
1470	1602.16	1601.32
1680	1603.98	1602.89
1890	1606.07	1604.69
2100	1608.44	1606.73



Pima Freeway outlet rating curves
2 - 108" conduits

Stantech
Consulting

Memo

To: File

From: Chuck Gopperton, PE

Date: 2 September 1998

Reference: **OUTER LOOP BASIN RE-SIZING STUDY**
FILE: 28900082

Due to changes in the design criteria for the Outer Loop Basin (OLB) and a delay in the construction of the freeway, it was determined that additional analysis of the basin size, location, operation and inflows was necessary. The study is a phased approach with the analysis of two scenarios and based on the results of this initial analysis, further analysis or additional scenarios will be authorized. Scenario 1 analyzes the revised hydrology and routes it through the single basin as presently designed. Scenario 2 investigates a double basin concept with the same grading plan but separate outlet works for the east and west basins.

Note: This memo was discussed at the 9 September bi-weekly meeting. As a result of that discussion, clarifications were made to the draft memo and decisions made to proceed with Scenario 2. The East Basin design will proceed as described herein. The feasibility of diverting flow in order to reduce the size of the West Basin will be investigated as Scenario 3.

Hydrology

The OLB is to be designed for the 100-year, 6-hour storm with 1 foot freeboard. That design must function satisfactorily for the 50-year, 24-hour storm. In addition, the 100-year, 24-hour runoff is routed through the basin to verify that water is contained within the public right of way and that flooding of private property doesn't occur. The revised hydrology was prepared and a summary was presented at the August 26th bi-weekly meeting. Table 1 presents a summary of the inflow hydrology peak flow rates and total volumes.

Table 1
Summary of inflow hydrographs at the Outer Loop Freeway

	100-year, 6-hour		100-year, 24-hour		50-year, 6-hour	
Runoff Condition	**Peak Inflow cfs**	**Total Volume acre-ft**	**Peak Inflow cfs**	**Total Volume acre-ft**	**Peak Inflow cfs**	**Total Volume acre-ft**
Interim:						
East side only	4161	411	5159	592	4084	507
West side only	1381	248	1802	361	1417	308
Combined	5376	660	6612	954	5356	815
Ultimate:						
East side only	4397	898	5459	1317	4410	1126
West side only	911	72	1068	102	859	87
Combined	5151	963	6517	1419	5269	1213

The inflow hydrographs are included in Appendix A.

Scenario 1

The basin stage/storage/discharge relationship for both the east and west side basins is assumed to remain as presently designed. Maximum water surface elevation is to remain at 1608 +/- with a maximum outflow of 2,150 cfs assuming no clogging from the trashrack. Table 2 is a summary of the ultimate condition routing.

Table 2
Scenario 1 - Ultimate condition, Summary of peak outflow and maximum stage

Hydrologic Condition	Peak Outflow (cfs)	Maximum WSEL (ft)
100-year, 6-hour	1,913	1,604.92
50-year, 24-hour	1,946	1,605.24
100-year, 24-hour	2,152	1,607.29

The results of the analysis show that by changing the basin inflow hydrology to the 50-year, 24-hour or the 100-year, 6-hour hydrology, the maximum water surface elevation is reduced. The overall basin size could be reduced to raise the 100-year, 6-hour WSEL up to the maximum water surface elevation of 1608.

The basin stage/storage curves, stage/storage/discharge curves and the basin outflow hydrographs for Scenario 1 are included in Appendix B.

Scenario 2

The basin stage/storage relationship for both the east and west side basins is assumed to remain as presently designed, however the two basins are not connected at Hayden Road. The maximum water surface elevation will increase to 1612 on the east side and remains at 1608 on the west. The west basin will have an outlet pipe which will connect with the east side outlet pipes a sufficient distance south of the freeway such that no backwater from the east side will affect the west basin outlet works. Different sizes of outlet works for both basins under both ultimate and interim hydrology conditions are analyzed. The hydrologically combined outflows from the east and west basin are limited to 2,150 cfs assuming no clogging of the trashracks. Table 3 summarizes the analysis for the ultimate condition hydrology for the east basin.

Table 3
Scenario 2, East basin - Ultimate condition, Summary of peak outflow and maximum stage

Basin Outlet Configuration	Hydrologic Condition	Peak Outflow (cfs)	Maximum WSEL (ft)
2 - 108" outlet	100-year, 6-hour	2,287	1,608.75
	50-year, 24-hour	2,315	1,609.07
	100-year, 24-hour	2,591	1,612.43
2 - 96" outlet	100-year, 6-hour	1,982	1610.74
	50-year, 24-hour	2,006	1,611.11
	100-year, 24-hour	2,233	1,614.77
2 - 84" outlet	100-year, 6-hour	1,660	1,613[1]
	50-year, 24-hour		[2]
	100-year, 24-hour		[2]

1. east basin overflows
2. rating curve exceeded, east basin overflows

The analysis shows that for the existing grading plan, the east basin would require an outlet works slightly larger than a double 96" pipe in order to eliminate the overflow during the 100-year, 24-hour event. With a slight increase in storage capacity of the basin or an increase in the maximum allowable stage, a double 96" outlet would work. With an additional increase in storage capacity of the basin, a double 84" outlet would work.

Table 4 summarizes the analysis for the ultimate condition hydrology for the west basin.

Table 4
Scenario 2, West basin - Ultimate condition, Summary of peak outflow and maximum stage

Hydrologic Condition	Peak Outflow (cfs)	Maximum WSEL (ft)
1 - 96" outlet		
100-year, 6-hour	347	1,597.15
50-year, 24-hour	341	1,597.07
100-year, 24-hour	393	1,597.77

Table 4
Scenario 2, West basin - Ultimate condition, Summary of peak outflow and maximum stage

Hydrologic Condition	Peak Outflow (cfs)	Maximum WSEL (ft)
1 - 48" outlet		
100-year, 6-hour	146	1,598.70
50-year, 24-hour	145	1,598.57
100-year, 24-hour	157	1,599.49
1 - 24" outlet		
100-year, 6-hour	48	1,600.67
50-year, 24-hour	47	1,600.37
100-year, 24-hour	50	1,601.48

This analysis shows that the west basin is oversized for the ultimate condition hydrology. The maximum water surface elevation for any size outlet is only 1,601.48. In fact, the west basin could retain the entire inflow volume for the ultimate 100-year, 24-hour storm of 102 acre-feet with a stage of only 1,605.69.

Combined flows for various combinations of pipe outlets are shown in the Table 5.

Table 5
Scenario 2 - Ultimate condition, Summary of combined peak outflow

Basin Outlet Configuration	Hydrologic Condition	Peak Outflow (cfs)
East Basin 2 - 96"	100-year, 6-hour	2,125
West Basin 1-48"	50-year, 24-hour	2,148
	100-year, 24-hour	2,377
East Basin 2 - 96"	100-year, 6-hour	2,026
West Basin 1-24"	50-year, 24-hour	2,050
	100-year, 24-hour	2,269
East Basin 2 - 108"	100-year, 6-hour	2,330
West Basin 1-24"	50-year, 24-hour	2,358
	100-year, 24-hour	2,637

Under ultimate conditions, with no changes to the basin grading plan, the preferred option would be the second one which uses a double 96" outlet on the east side and a single 24" outlet on the west. However, under interim conditions, the runoff to the west basin is increased and it requires a 48" outlet.

Table 6 shows peak flows for the east and west basin under interim conditions.

Table 6
Scenario 2, Interim condition, Summary of peak outflow and maximum stage

Hydrologic Condition	Peak Outflow (cfs)	Maximum WSEL (ft)
East Basin 2 - 96" outlet		
100-year, 6-hour	1,849	1,608.71
50-year, 24-hour	1847	1,608.70
100-year, 24-hour	2044	1,611.73
West Basin 1 - 48" outlet		
100-year, 6-hour	237	1,607.53
50-year, 24-hour	238	1,607.69
100-year, 24-hour	256	1,609.98[1]
West Basin 1 - 24" outlet		
100-year, 6-hour	65	1610.97[1]
50-year, 24-hour		[1]
100-year, 24-hour		[1]

1. rating curve exceeded, west basin overflows

Combined flows for various combinations of pipe outlets under interim conditions are shown in the Table 7.

Table 7
Scenario 2 - Interim condition, Summary of combined peak outflow

Basin Outlet Configuration	Hydrologic Condition	Peak Outflow (cfs)
East Basin 2 - 96"	100-year, 6-hour	2,041
West Basin 1-48"	50-year, 24-hour	2,046
	100-year, 24-hour	2,254[1]
East Basin 2 - 96"	100-year, 6-hour	1,901[1]
West Basin 1-24"	50-year, 24-hour	[1]
	100-year, 24-hour	[1]

1. rating curve exceeded, west basin overflows

The basin stage/storage curves, stage/storage/discharge curves and the basin outflow hydrographs for Scenario 2 are included in Appendix C.

Summary

The analysis shows that for Scenario 1, some savings in cost for excavation could be realized by raising the bottom of the basin or making the basin narrower. This would optimize the basin by raising the maximum water surface elevation closer to 1608.

For Scenario 2, an increase in basin size or maximum stage on the east side and a decrease in storage on the west side would help to optimize the cost. While interim hydrology has little effect on the size of the Scenario 1 basin, under Scenario 2, it requires that the west basin be designed with substantially more capacity than required for the ultimate condition. Construction of the powerline channel, a portion of the Phase 2 - Pima Road Channel north to Pinnacle Peak Road or some other method of diverting the interim flows into the east basin would greatly reduce the required storage volume of the west basin.

Chuck Gopperton, PE
Project Engineer

Attachment

cc: Doug Cullinane, City of Scottsdale
Collis Lovely, City of Scottsdale
Mark Landsiedel, City of Scottsdale
John Rodriguez, Flood Control District of Maricopa County
Scott Ogden, Flood Control District of Maricopa County
Marty Bressor, Pentacore

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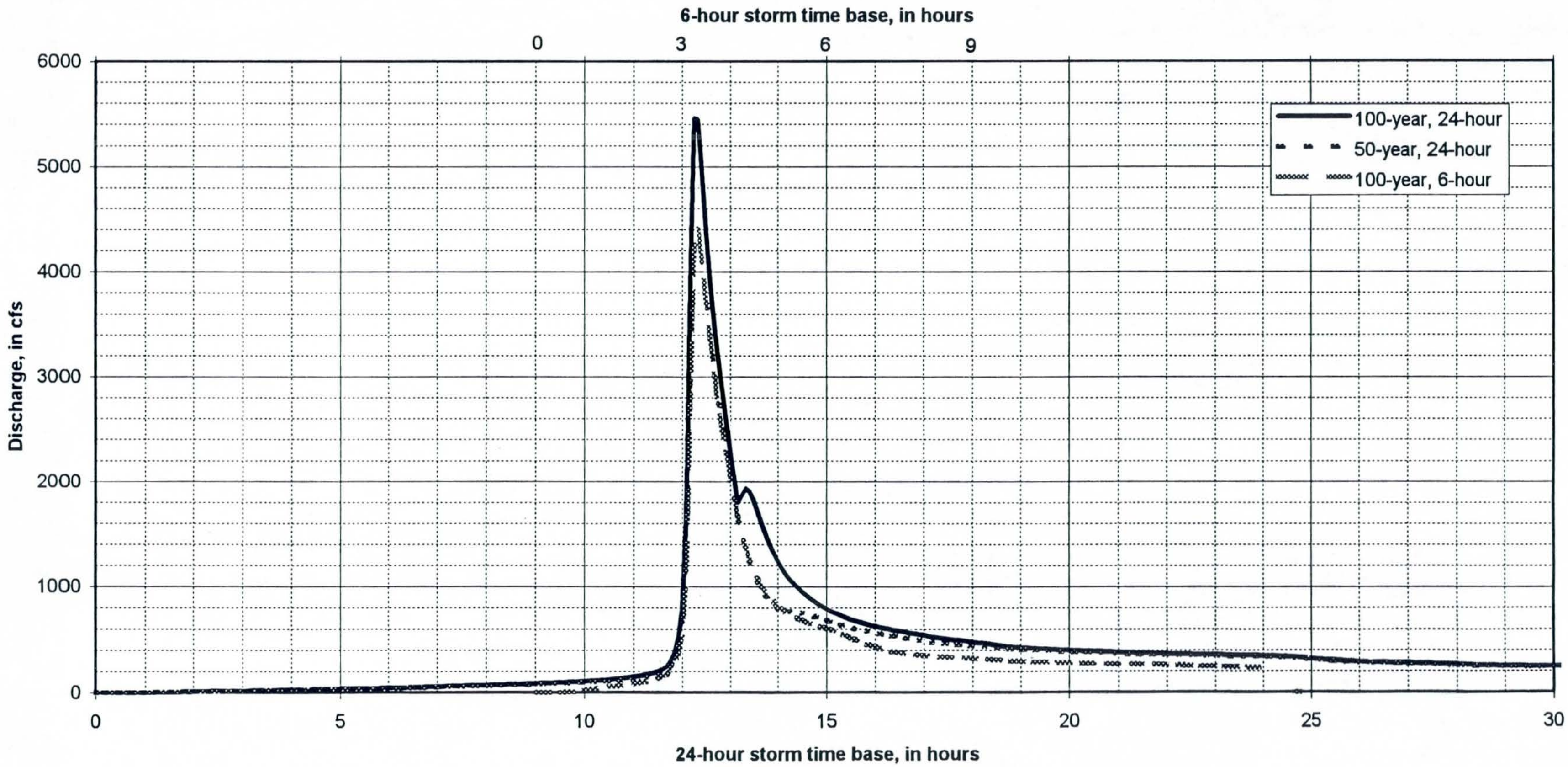
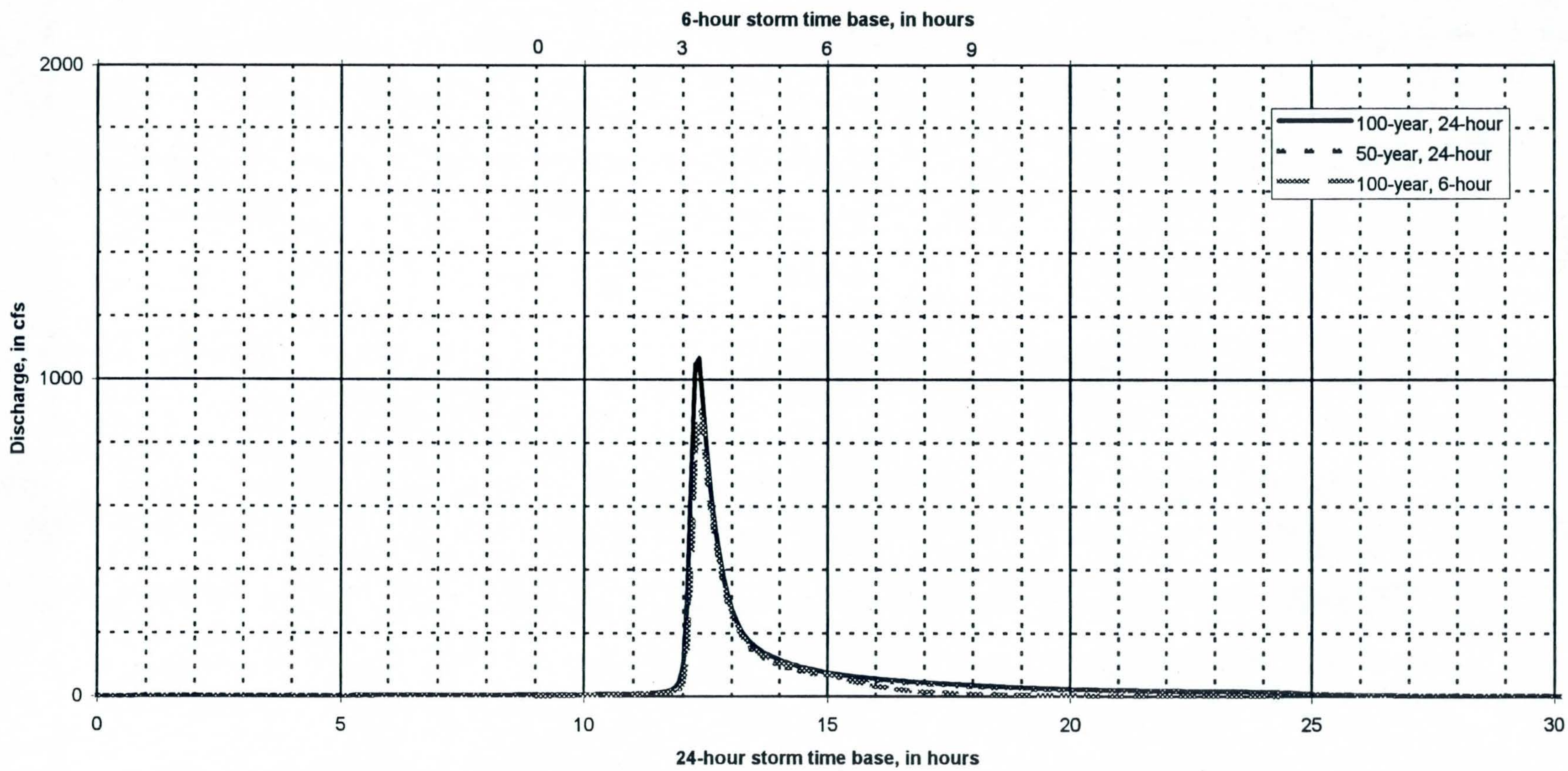
APPENDIX A

Inflow Hydrographs at Outer Loop Freeway

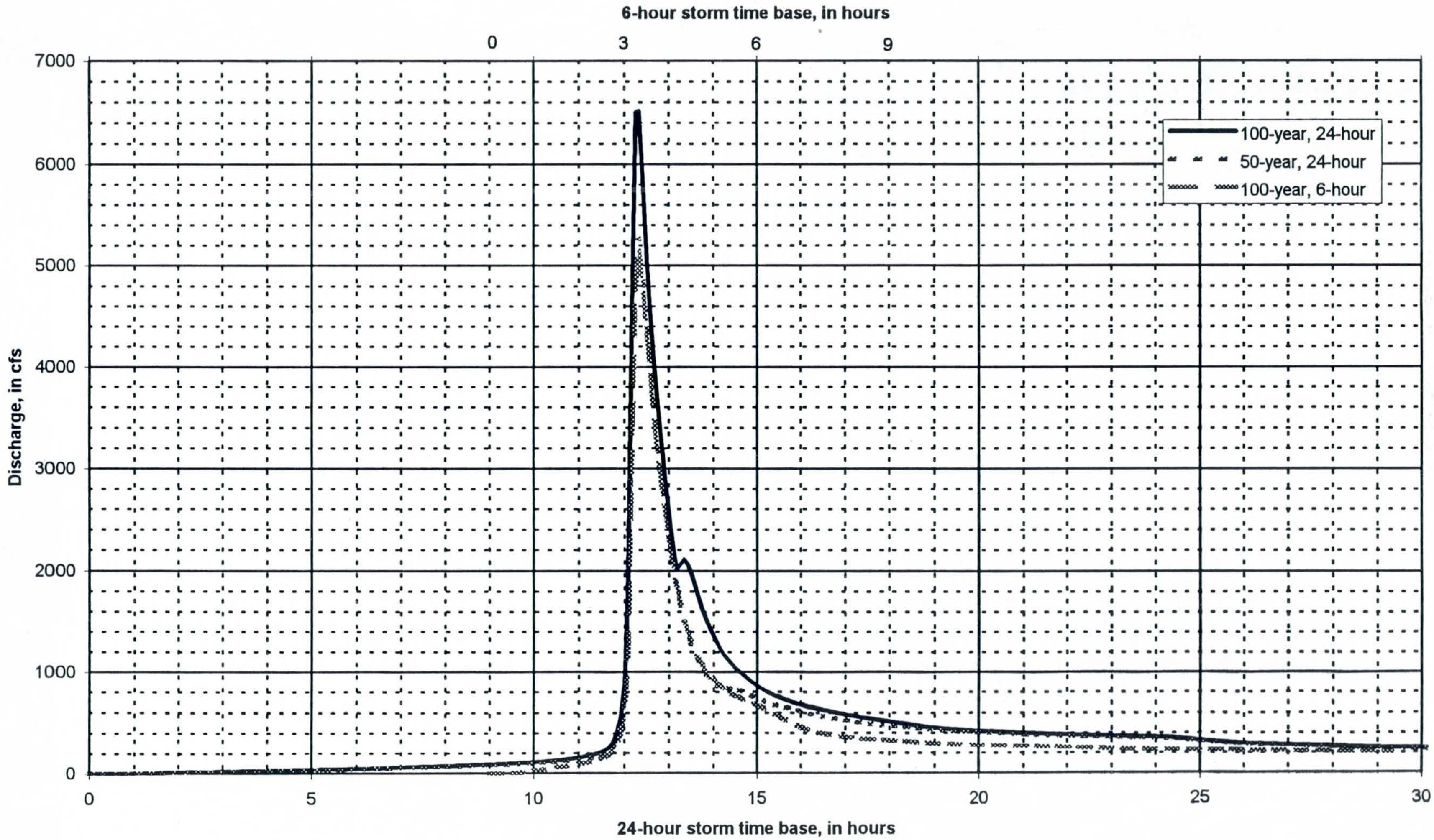
East basin inflow hydrograph, ultimate condition

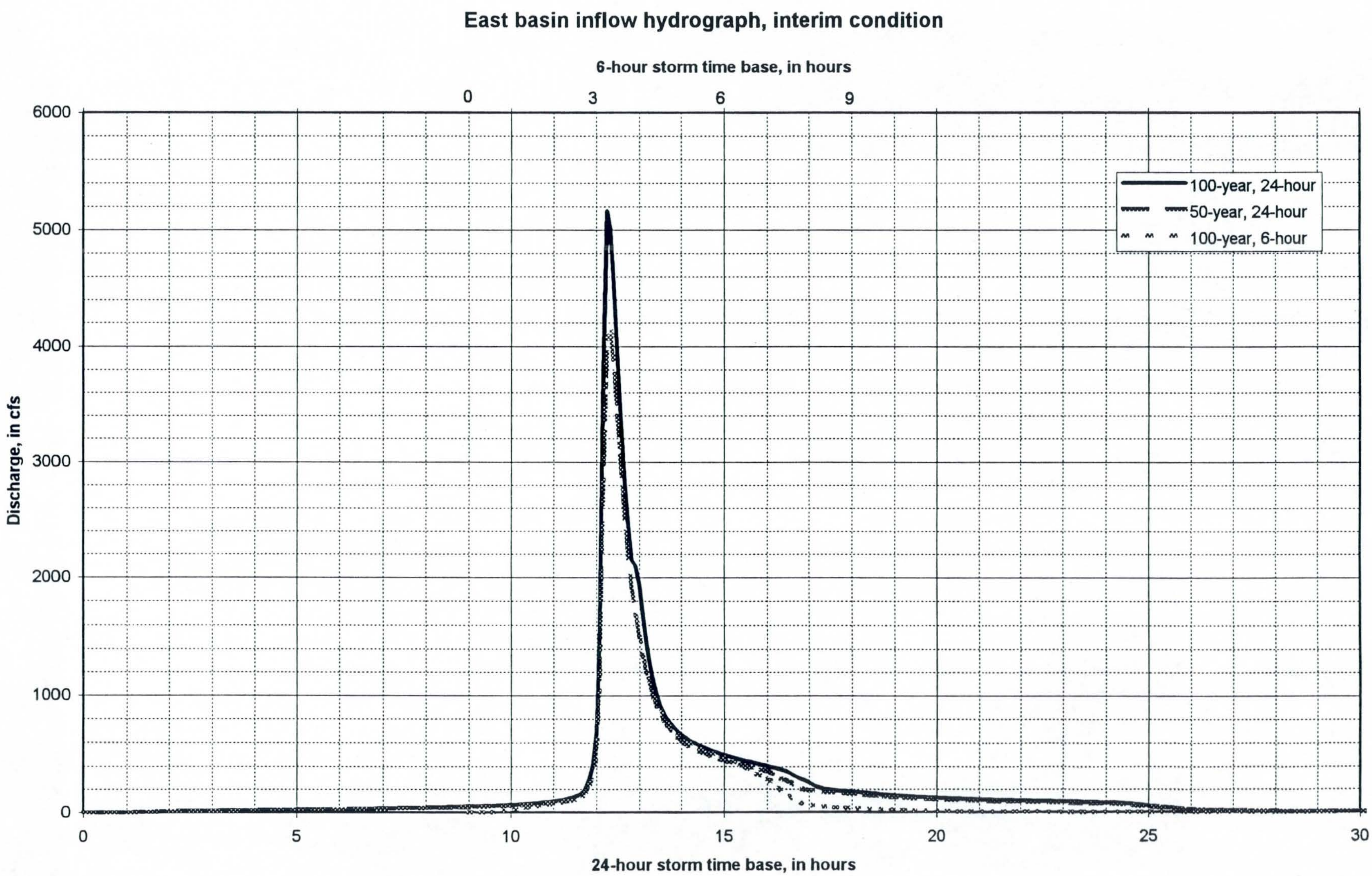


West basin inflow hydrograph, ultimate condition

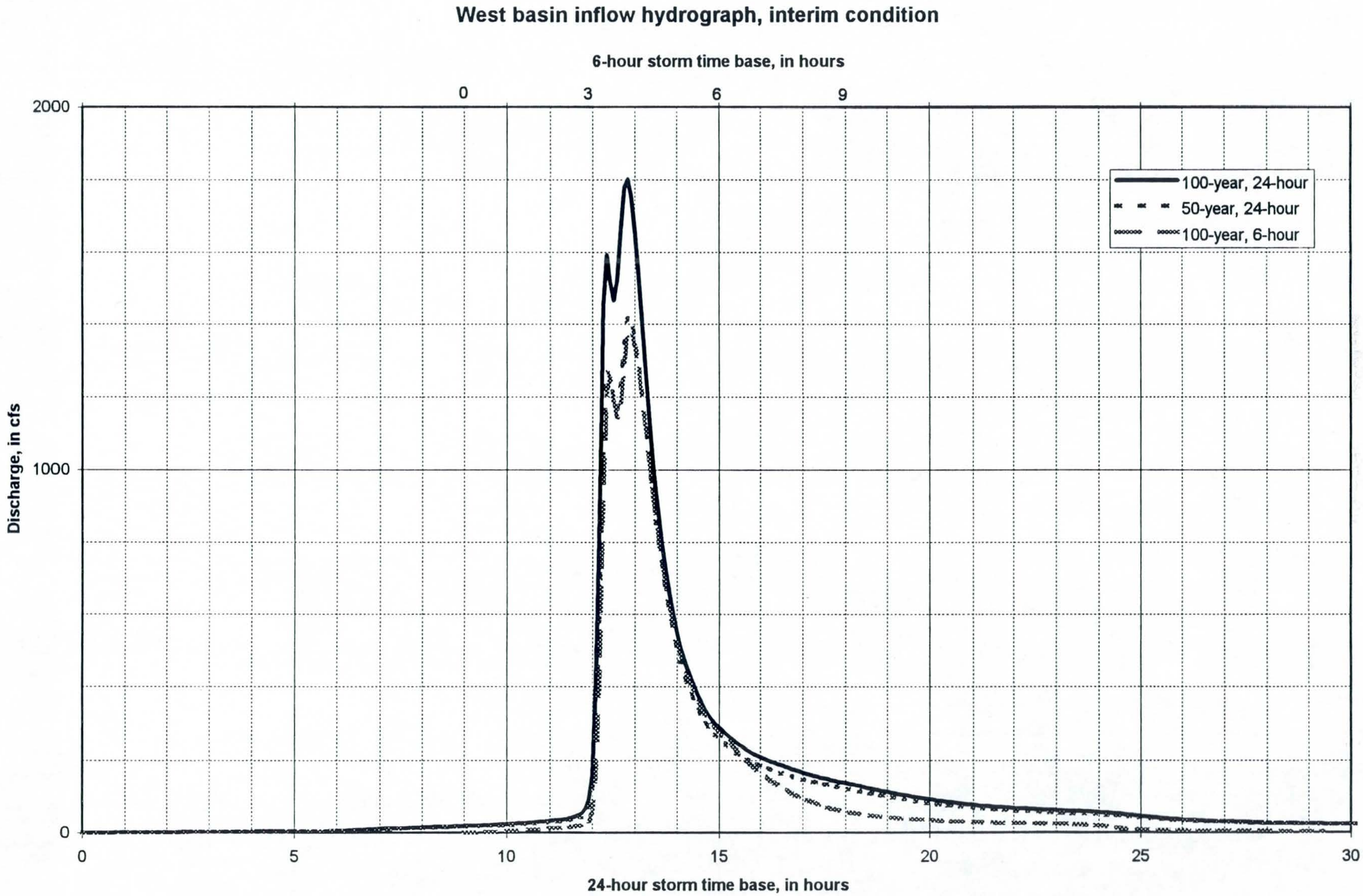


Combined basin inflow hydrograph, ultimate condition



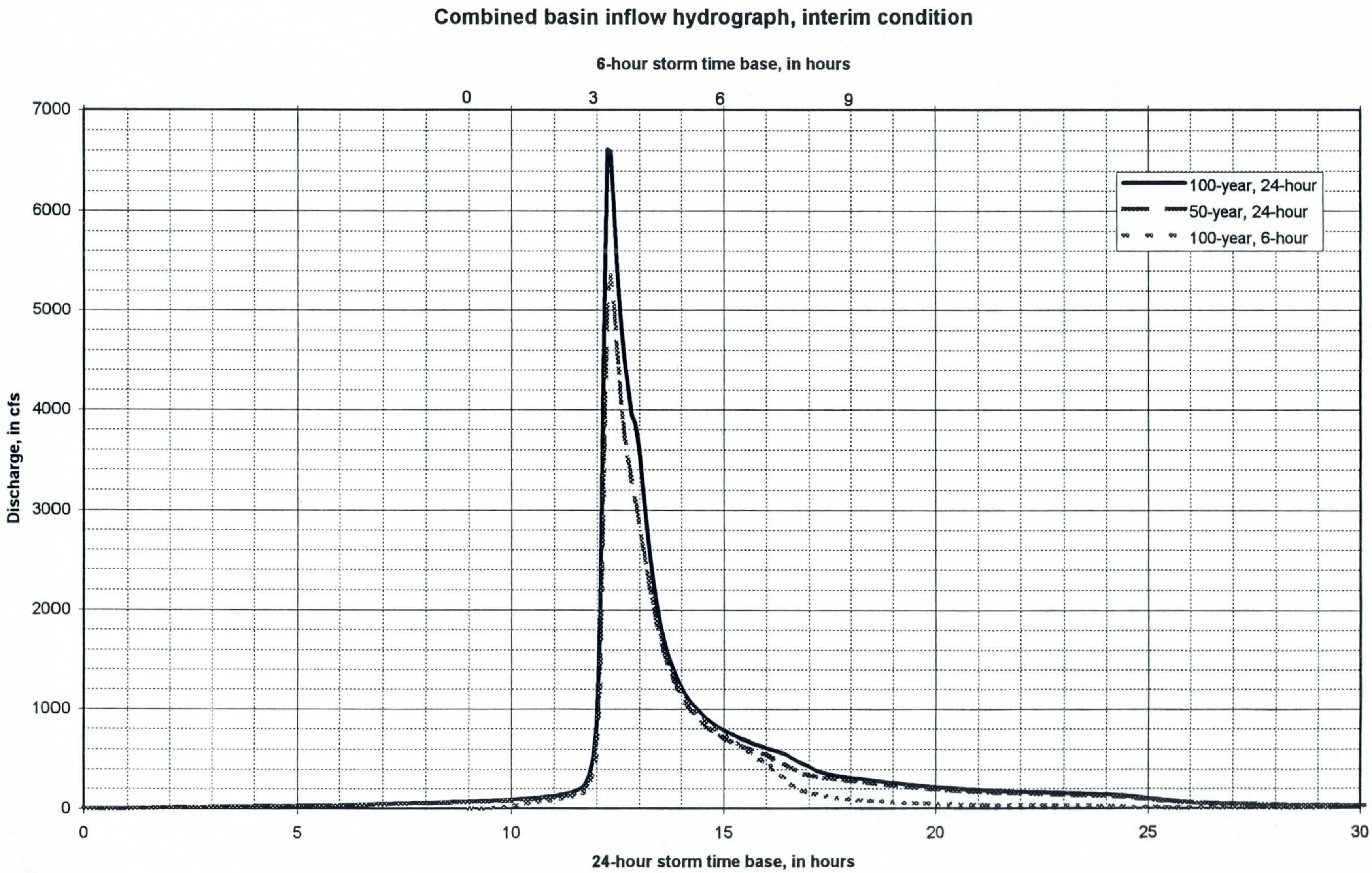


East basin inflow hydrograph, interim condition
6-hour storm time base, in hours
0
3
6
9
6000
5000
4000
3000
2000
1000
0
Discharge, in cfs
100-year, 24-hour
50-year, 24-hour
100-year, 6-hour
0
5
10
15
20
25
30
24-hour storm time base, in hours



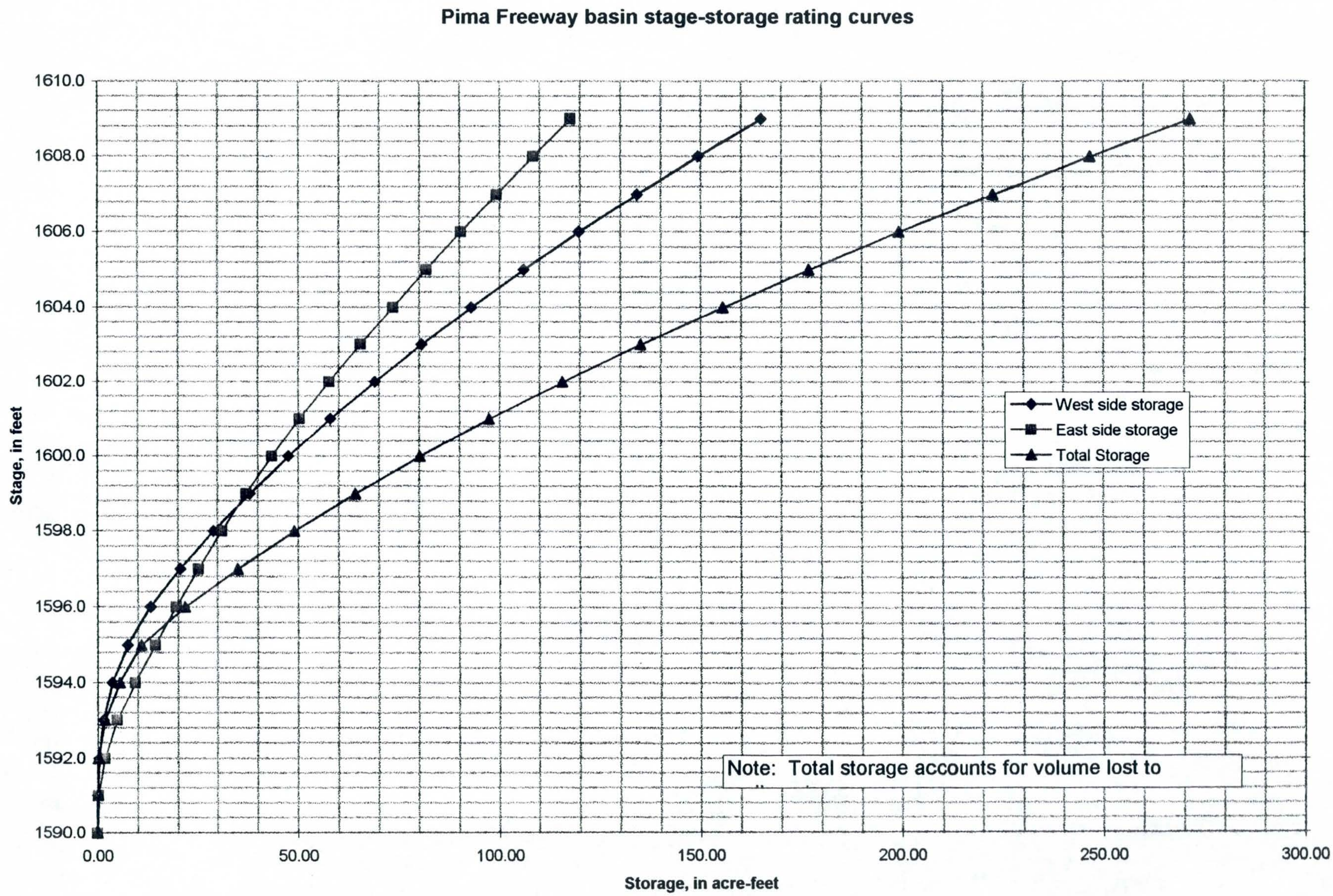
Miller - Graph Chart 1

9/8/98 5:31 PM

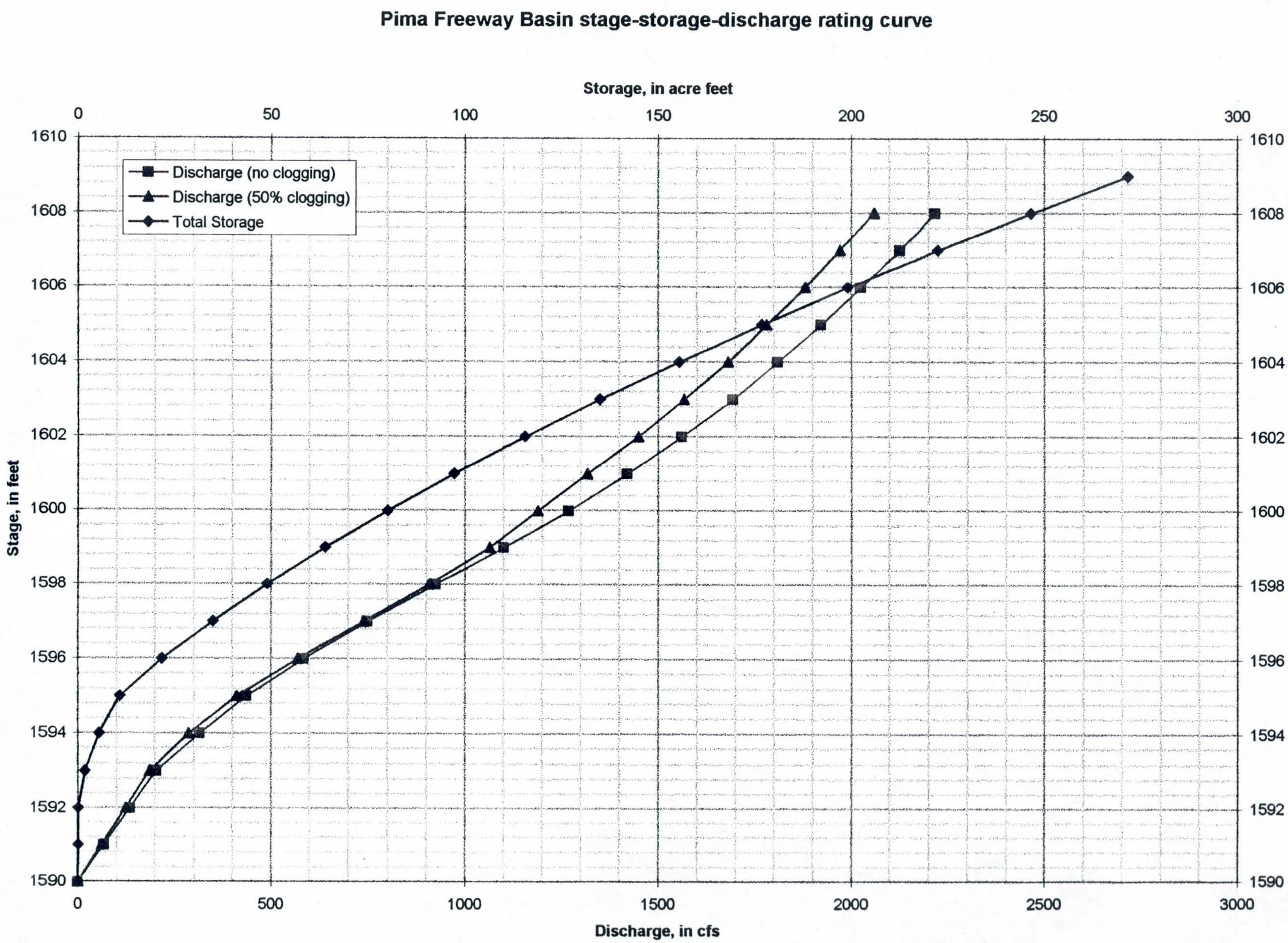

Combined basin inflow hydrograph, interim condition
6-hour storm time base, in hours
0
3
6
9
7000
6000
5000
4000
3000
2000
1000
0
Discharge, in cfs
100-year, 24-hour
50-year, 24-hour
100-year, 6-hour
0
5
10
15
20
25
30
24-hour storm time base, in hours

APPENDIX B

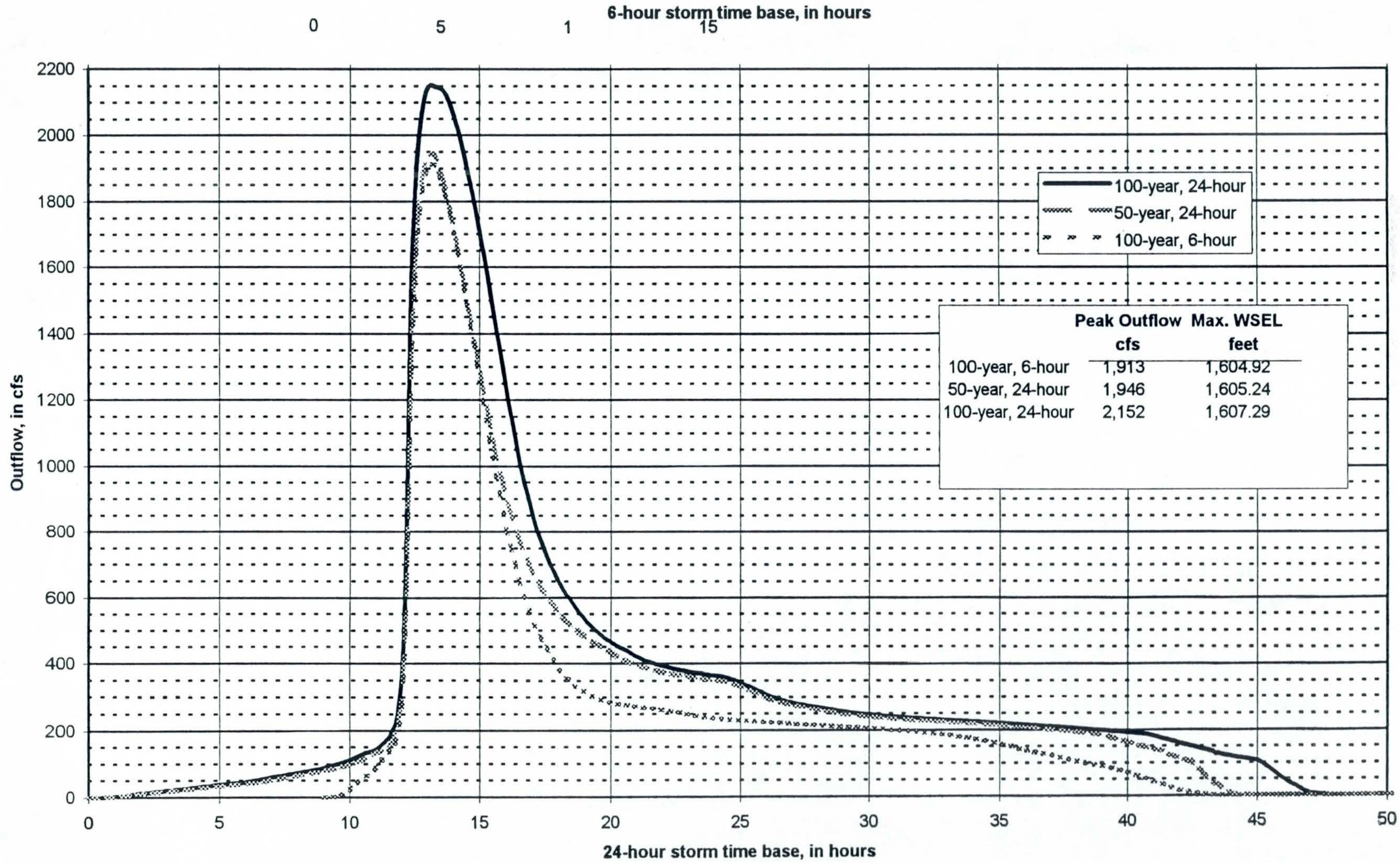
Outer Loop Basin Scenario 1 - Stage/Storage/Discharge and Hydrologic Routing



Pima Freeway basin stage-storage rating curves
1610.0
1608.0
1606.0
1604.0
1602.0
1600.0
1598.0
1596.0
1594.0
1592.0
1590.0
Stage, in feet
0.00
50.00
100.00
150.00
200.00
250.00
300.00
Storage, in acre-feet
West side storage
East side storage
Total Storage
Note: Total storage accounts for volume lost to


Pima Freeway Basin stage-storage-discharge rating curve
Storage, in acre feet
0
50
100
150
200
250
300
Stage, in feet
1590
1592
1594
1596
1598
1600
1602
1604
1606
1608
1610
Discharge (no clogging)
Discharge (50% clogging)
Total Storage
0
500
1000
1500
2000
2500
3000
Discharge, in cfs

Combined basin outflow hydrographs, ultimate condition



APPENDIX C

Outer Loop Basin Scenario 2 - Hydrologic Routing

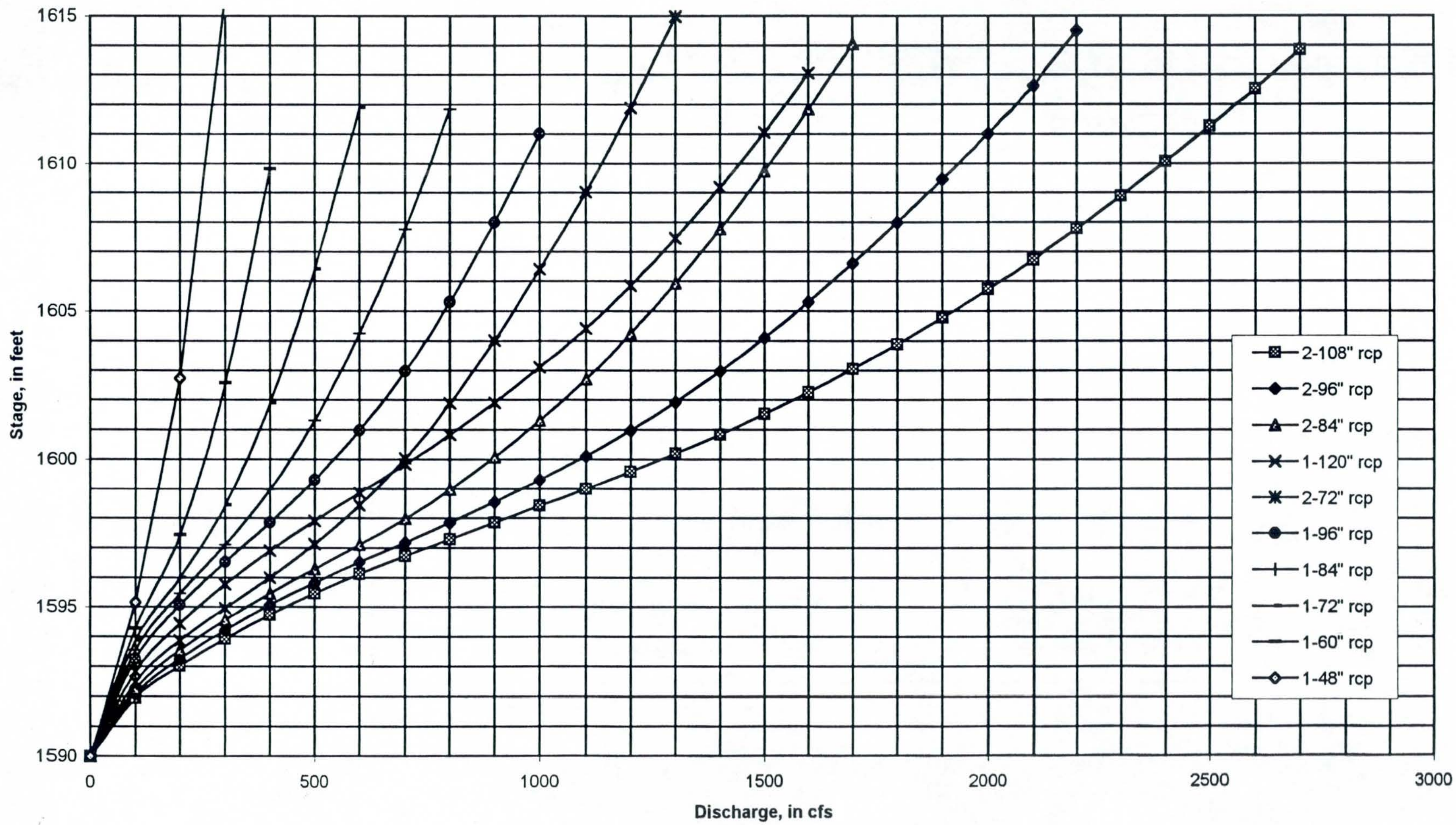
Table 2
Stage-storage-discharge rating curves for the East Pima Freeway Basin
storage is for a 60' bottom width basin

		Discharge for various outlet conduit sizes				
Stage feet	**Storage acre-feet**	**2-108" cfs**	**2-96" cfs**	**2-84" cfs**	**1-120" cfs**	**2-72" cfs**
1590	0.00	0	0	0	0	0
1591	0.48	51	48	44	35	40
1592	1.62	104	96	89	71	81
1593	3.29	199	179	158	111	137
1594	6.00	309	277	245	173	212
1595	9.06	436	391	348	242	303
1596	12.35	582	526	466	320	400
1597	17.86	748	672	588	410	488
1598	23.60	925	820	701	509	566
1599	29.56	1102	959	803	614	635
1600	35.75	1269	1088	894	717	698
1601	42.35	1423	1202	975	815	752
1602	49.53	1564	1307	1049	907	805
1603	57.28	1694	1403	1119	991	852
1604	65.46	1812	1491	1184	1068	899
1605	73.92	1923	1574	1244	1140	941
1606	82.60	2027	1653	1303	1209	983
1607	91.51	2126	1728	1358	1272	1023
1608	100.66	2220	1800	1412	1332	1061
1609	110.05	2309	1868	1462	1390	1099
1610	119.66	2395	1934	1512	1444	1134
1611	129.51	2478	1999	1560	1497	1169
1612	139.59	2557	2060	1608	1547	1204
1613	149.91	2635	2119	1653	1597	1236

Table 2
Stage-storage-discharge rating curves for the West Pima Freeway Basin
storage is for a 60' bottom width basin

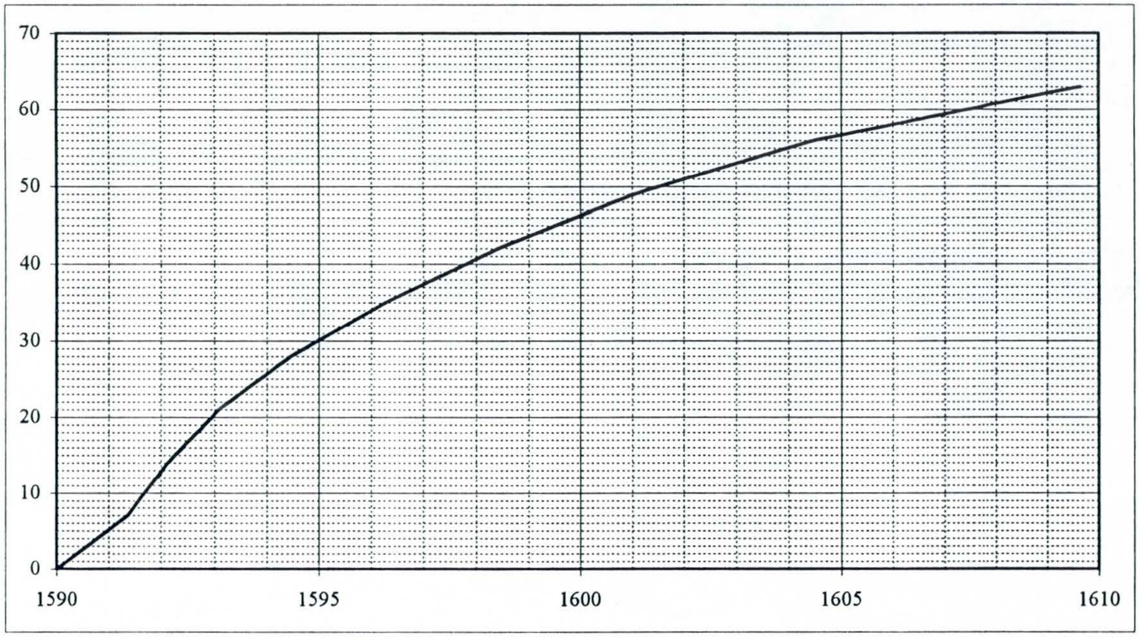
		Discharge for various outlet conduit sizes					
Stage feet	**Storage acre-feet**	**1-96" cfs**	**1-84" cfs**	**1-72" cfs**	**1-60" cfs**	**1-48" cfs**	**1-24" cfs**
1590	0.00	0	0	0	0	0	0
1591	0.02	31	28	26	23	19	5
1592	0.35	62	56	52	47	39	13
1593	1.43	92	85	78	70	58	20
1594	3.63	141	124	106	94	77	25
1595	7.44	196	176	153	123	97	30
1596	13.16	264	233	200	155	111	34
1597	20.71	336	294	241	186	124	37
1598	29.03	410	348	282	211	137	41
1599	38.00	479	401	316	230	151	43
1600	47.63	542	444	345	250	164	46
1601	57.94	601	487	374	269	177	49
1602	68.92	651	523	402	289	190	51
1603	80.59	701	557	425	306	202	53
1604	92.94	744	591	447	320	210	55
1605	106.02	787	621	469	333	217	57
1606	119.79	826	650	491	347	225	58
1607	134.22	863	678	511	361	233	59
1608	149.29	900	706	529	375	240	61
1609	164.96	933	730	547	389	248	62

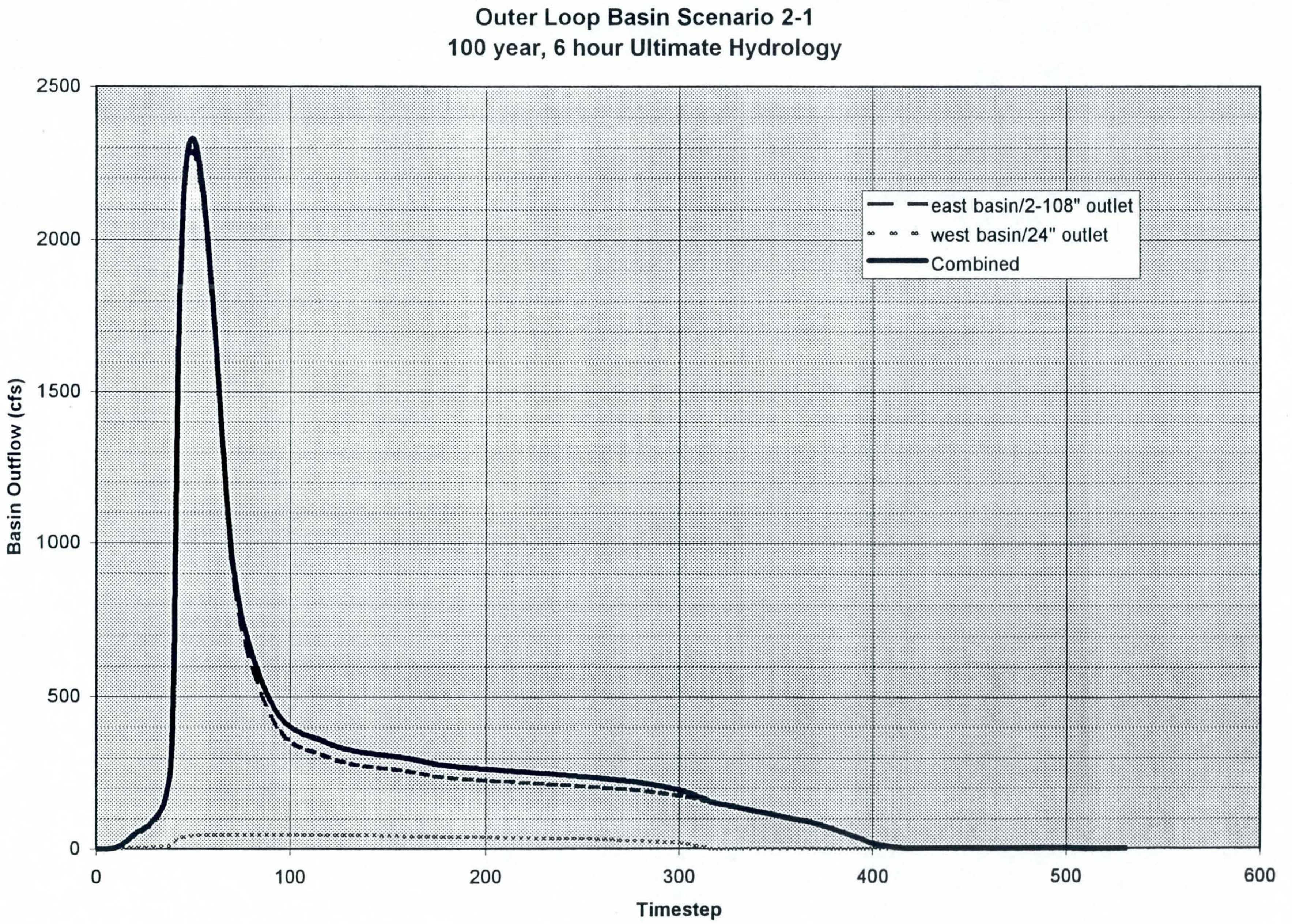
Stage-discharge rating curves



1-24" rcp

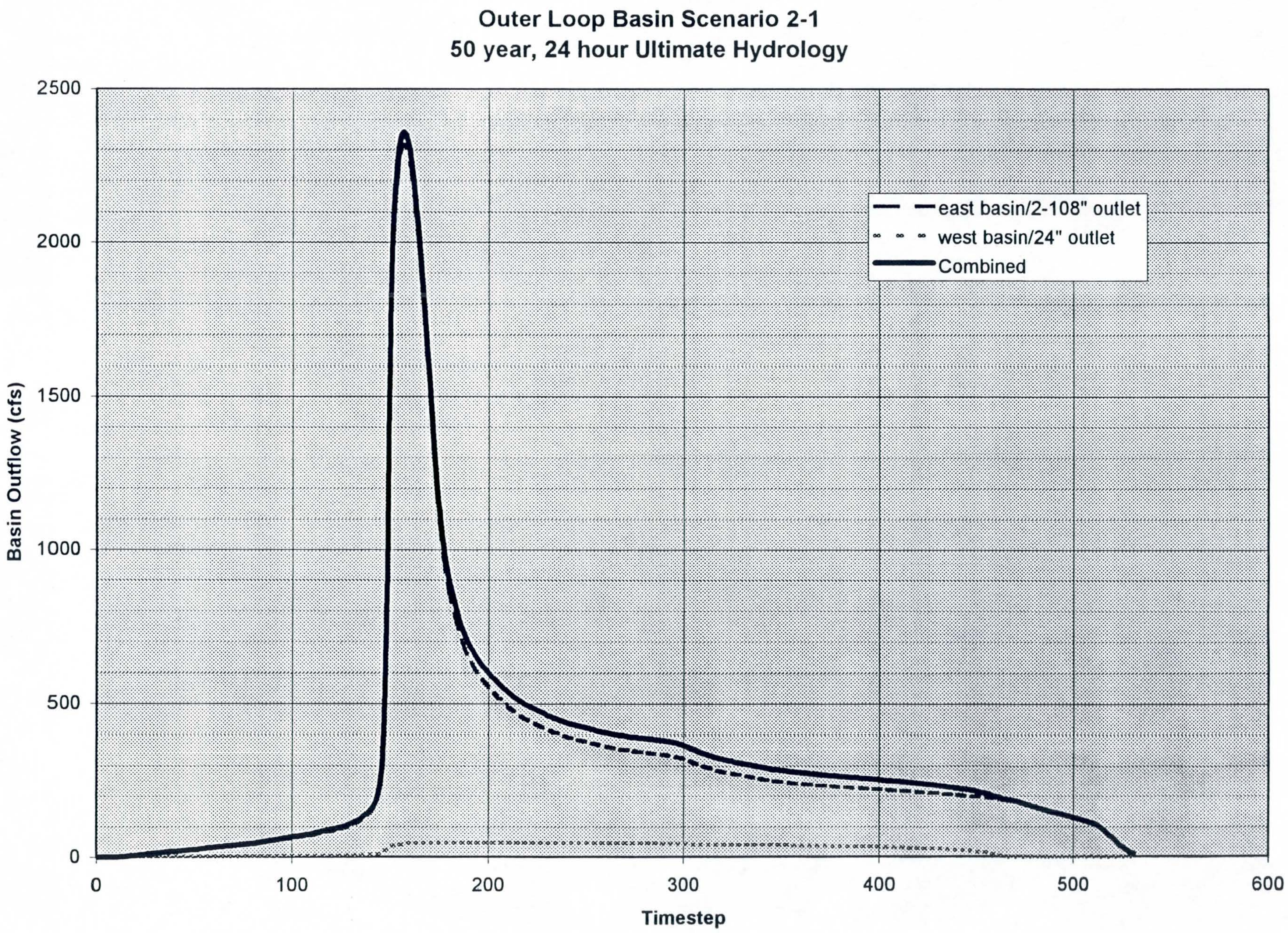
stage	discharge
1590	0
1591.36	7
1592.13	14
1593.09	21
1594.47	28
1596.26	35
1598.43	42
1601.04	49
1604.49	56
1609.64	63





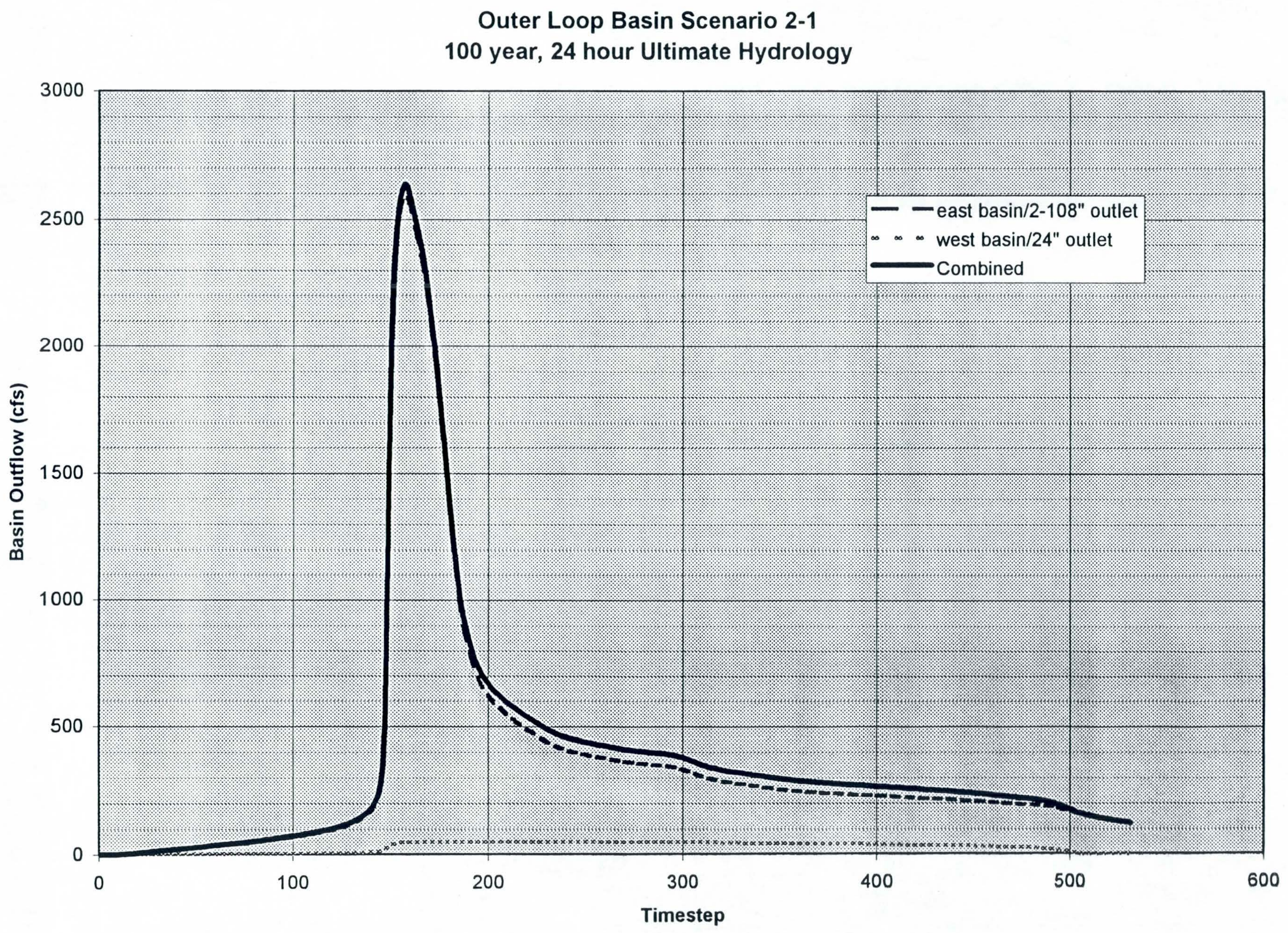
Ultimate outlet hydrographs.xls Chart1 (2)

9/4/98 1:02 PM



Ultimate outlet hydrographs.xls Chart1

9/4/98 12:53 PM



Ultimate outlet hydrographs.xls Chart1 (3)

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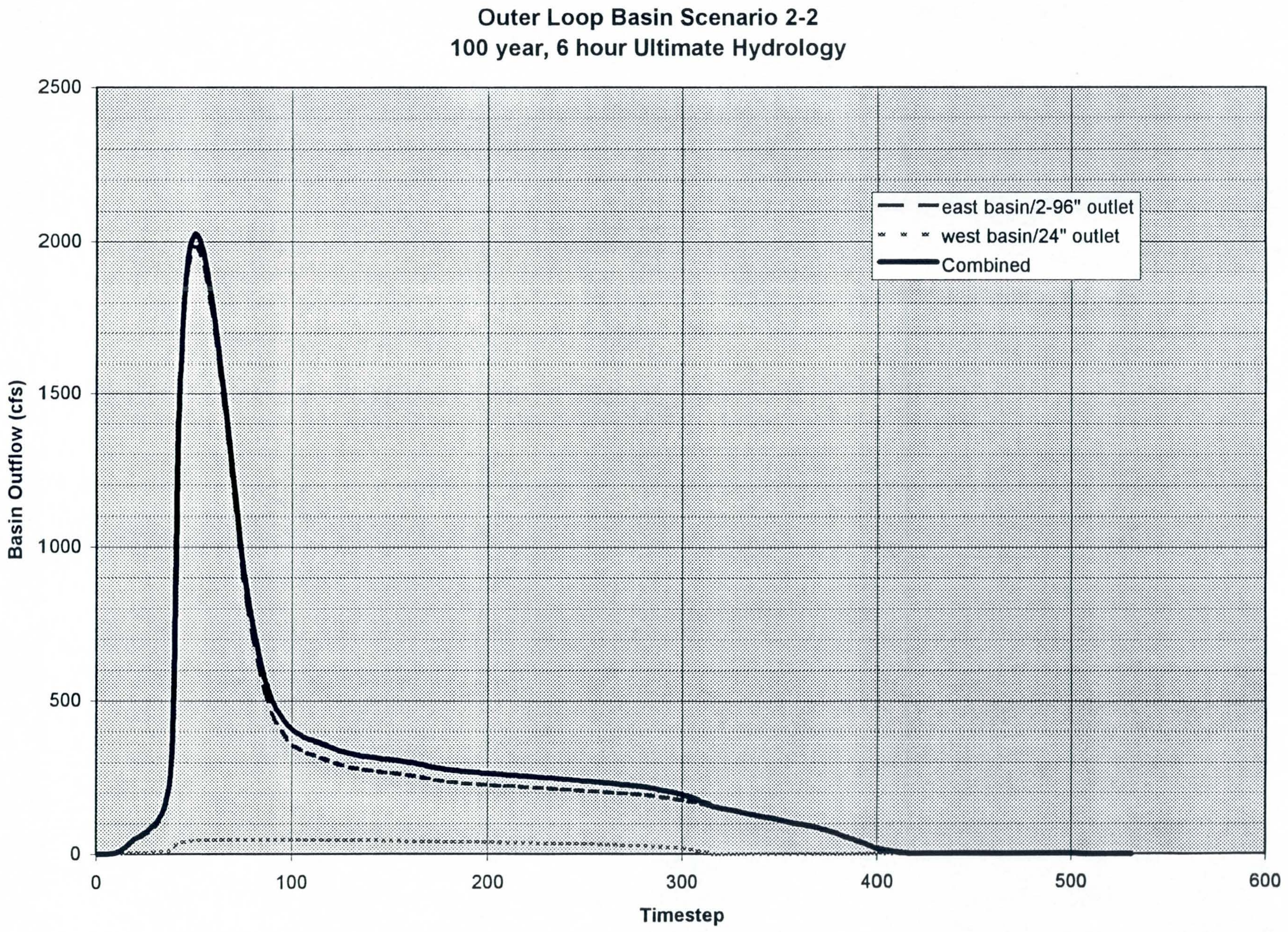
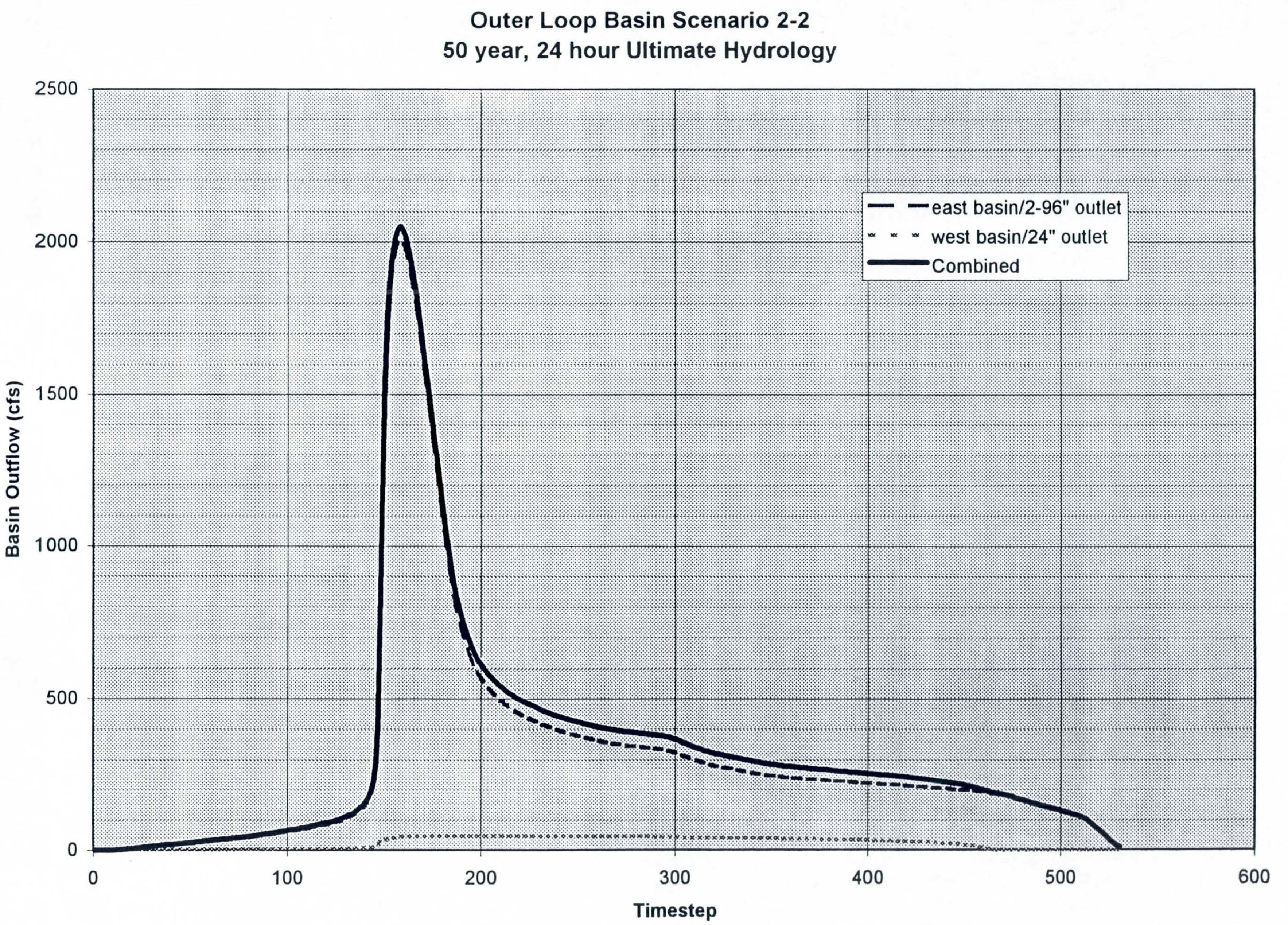
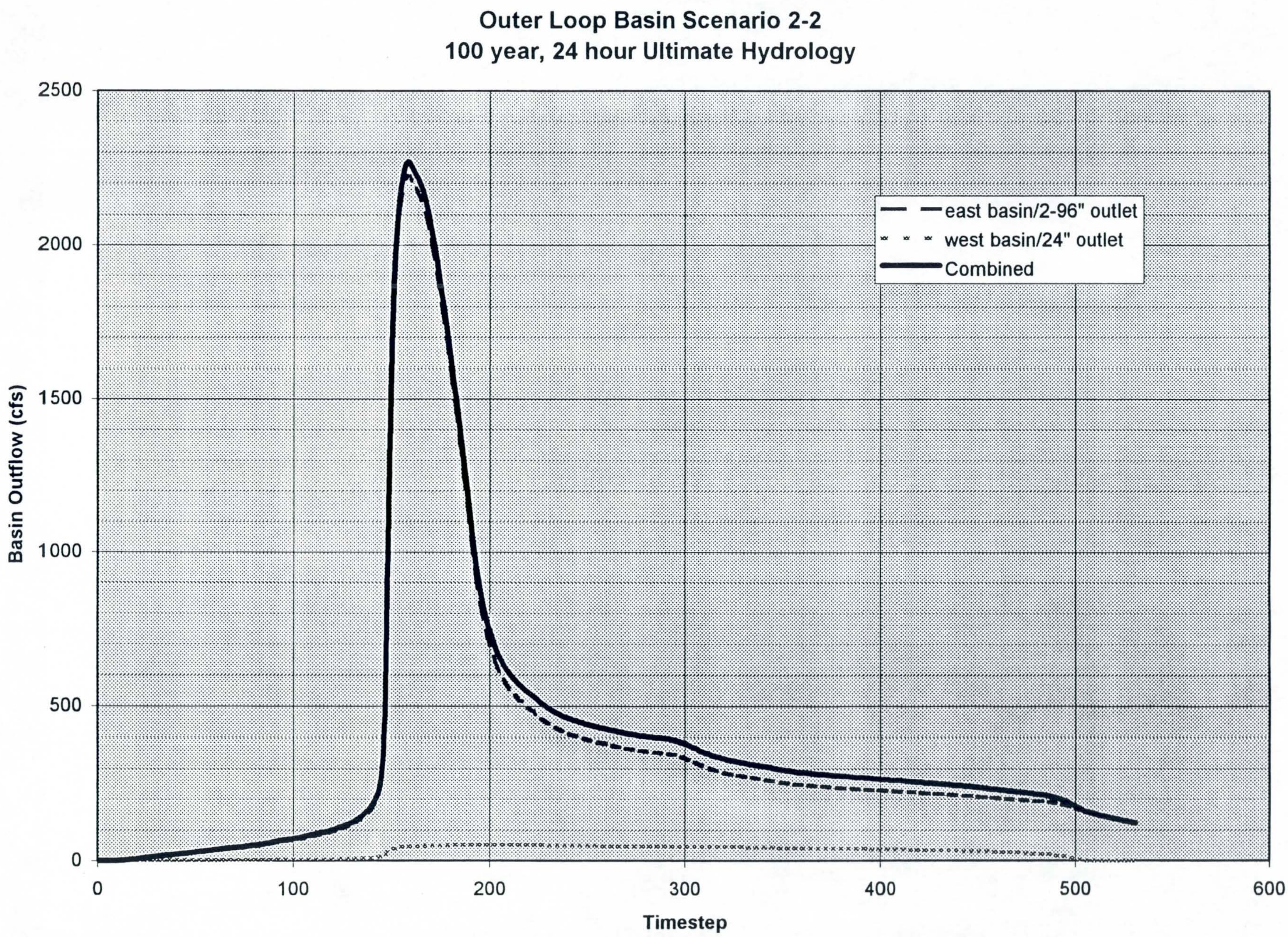


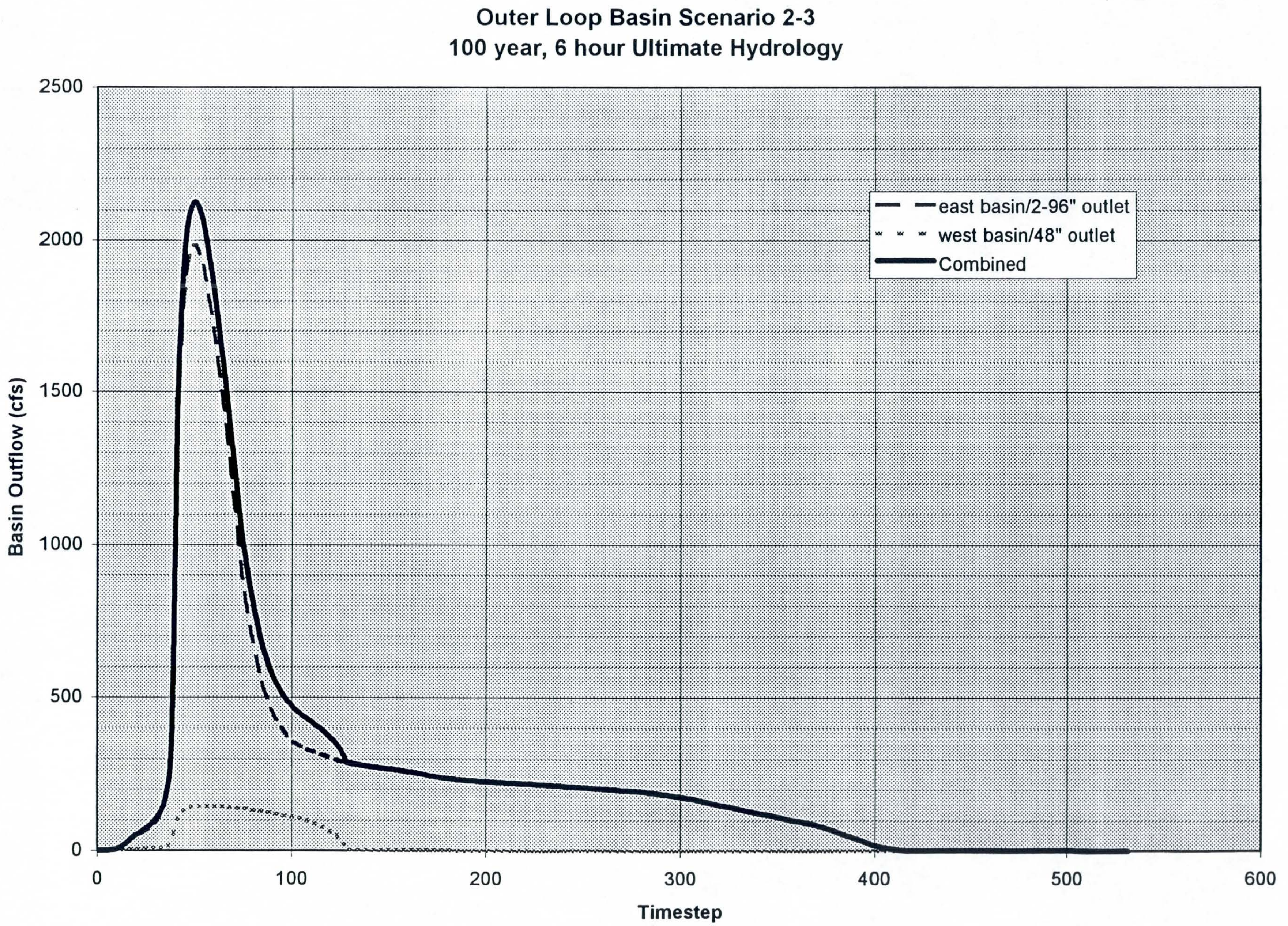
Ultimate outlet hydrographs.xls Chart2 (1)

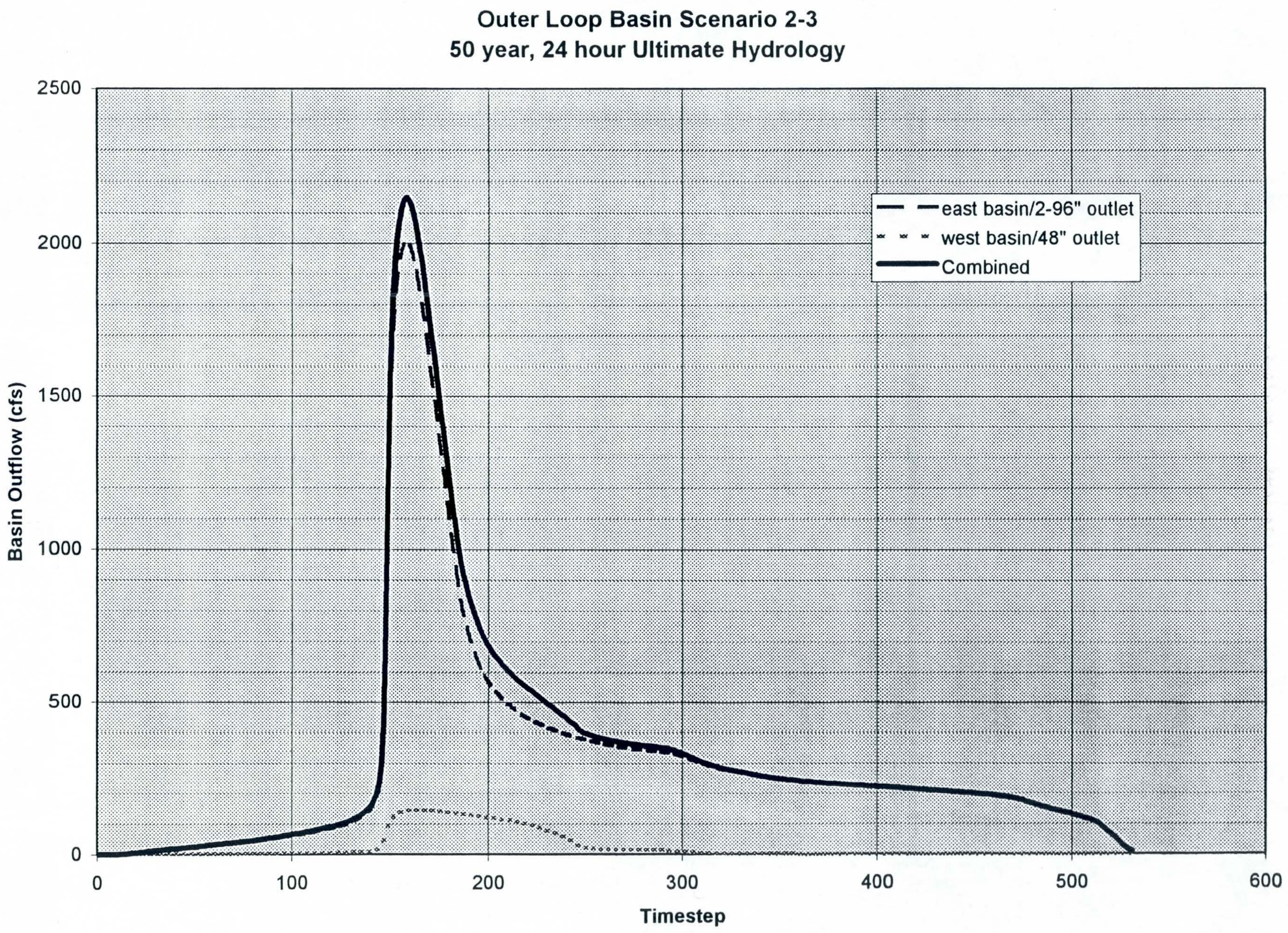
9/4/98 1:20 PM


Outer Loop Basin Scenario 2-2
50 year, 24 hour Ultimate Hydrology
Basin Outflow (cfs)
Timestep
0
500
1000
1500
2000
2500
0
100
200
300
400
500
600
east basin/2-96" outlet
west basin/24" outlet
Combined



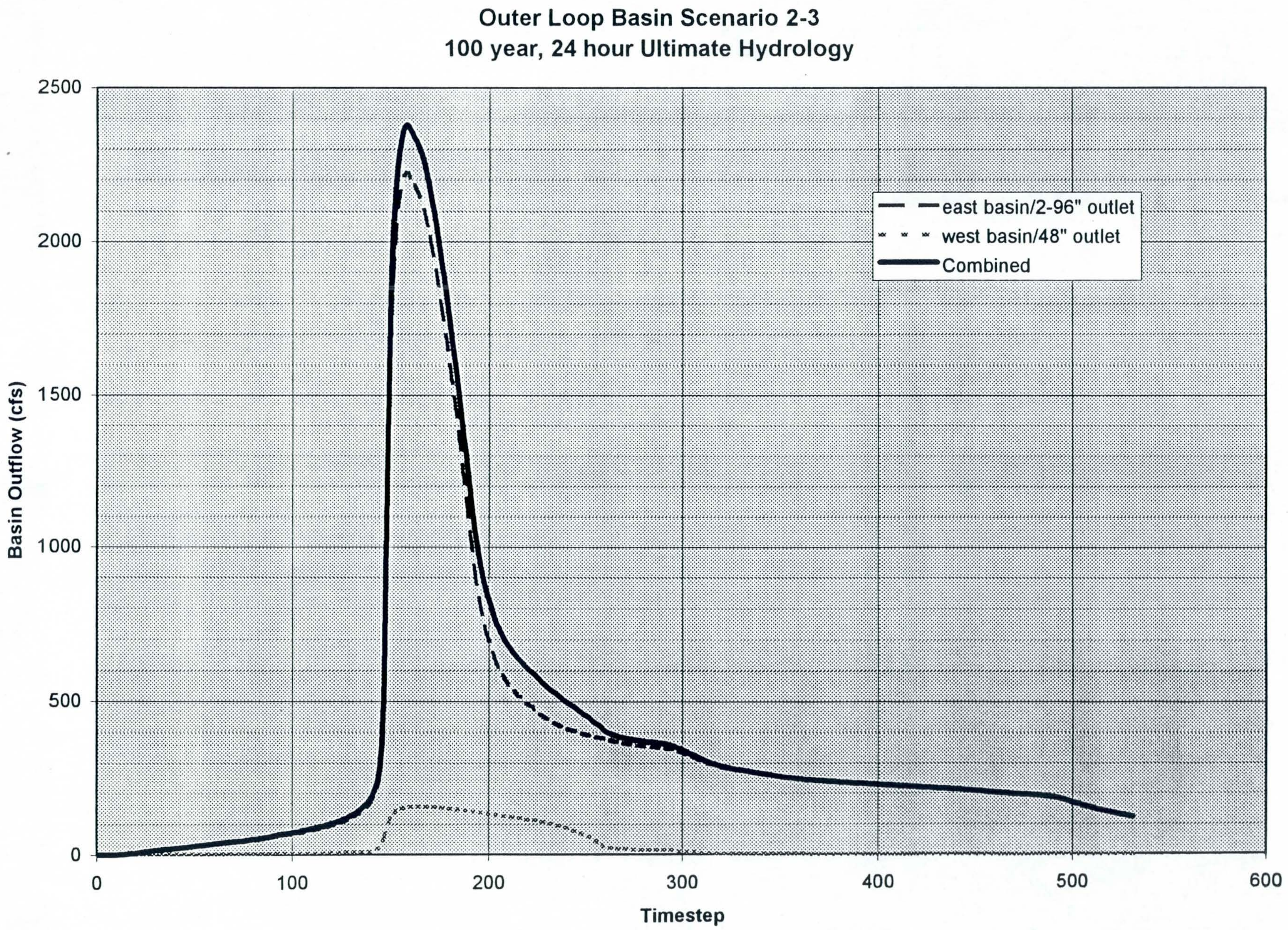
Outer Loop Basin Scenario 2-2
100 year, 24 hour Ultimate Hydrology
Basin Outflow (cfs)
2500
2000
1500
1000
500
0
0
100
200
300
400
500
600
Timestep
east basin/2-96" outlet
west basin/24" outlet
Combined


Outer Loop Basin Scenario 2-3
100 year, 6 hour Ultimate Hydrology
east basin/2-96" outlet
west basin/48" outlet
Combined
Basin Outflow (cfs)
2500
2000
1500
1000
500
0
0
100
200
300
400
500
600
Timestep



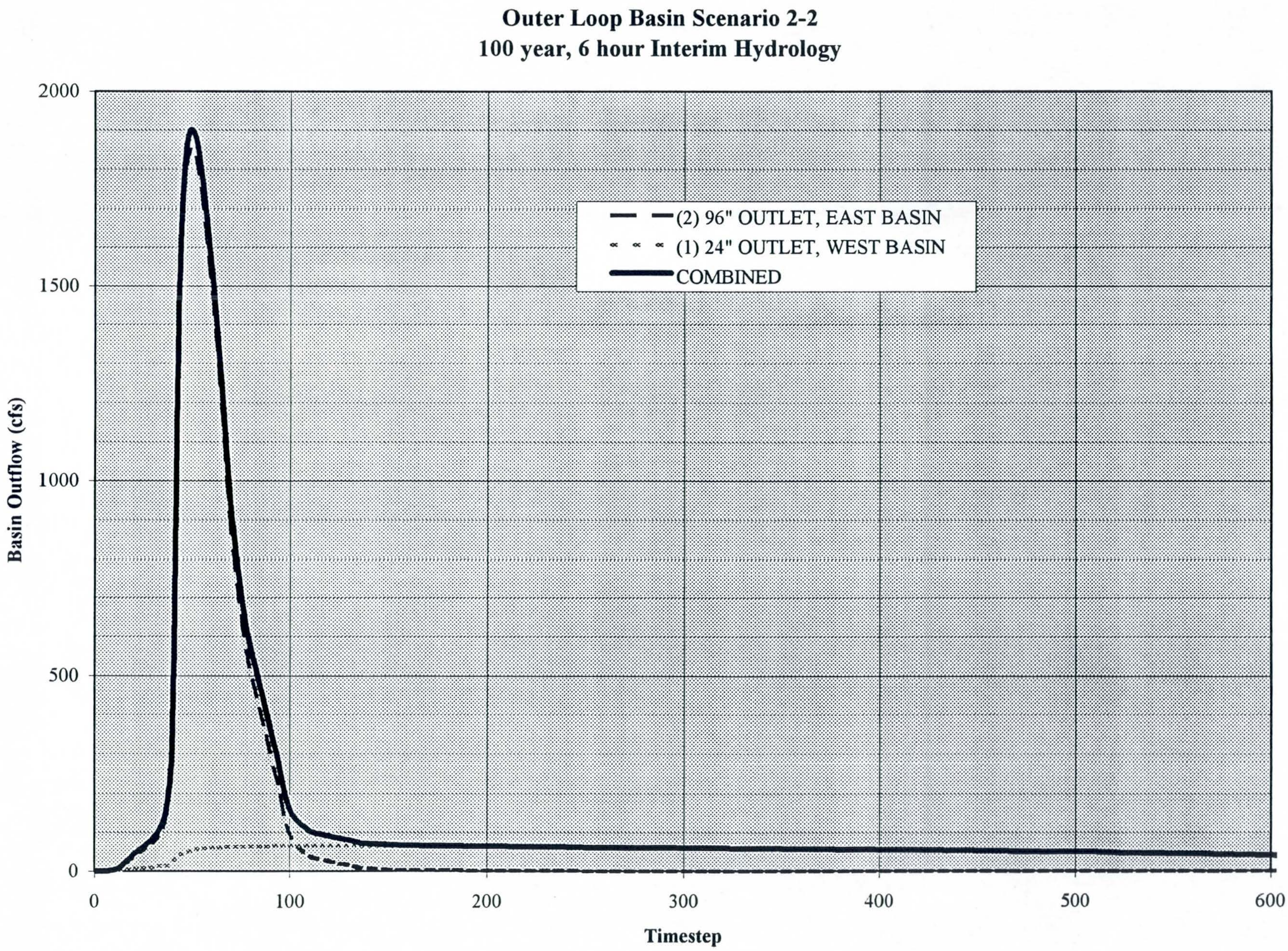
Ultimate outlet hydrographs.xls Chart3 (2)

9/4/98 1:41 PM

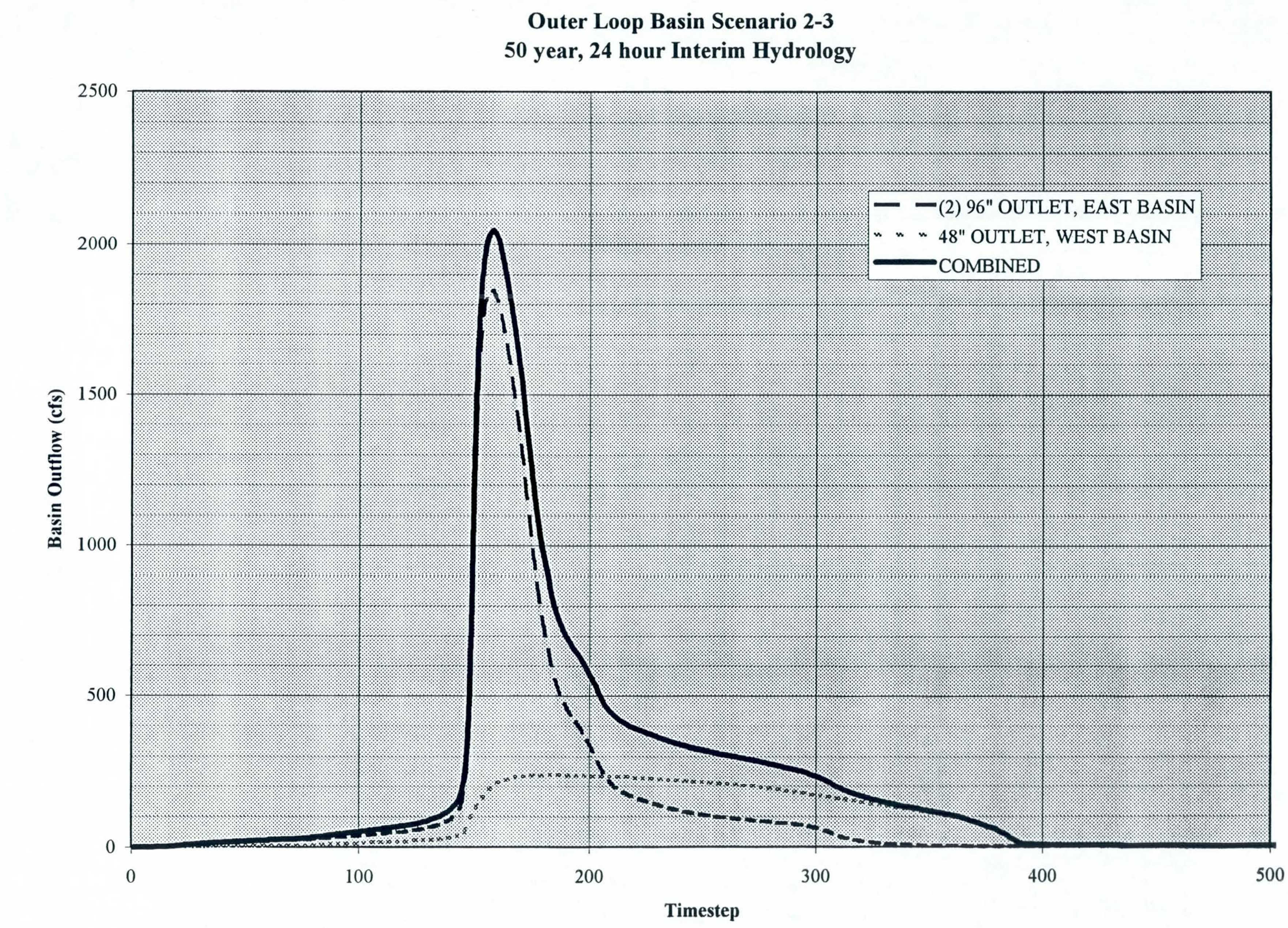


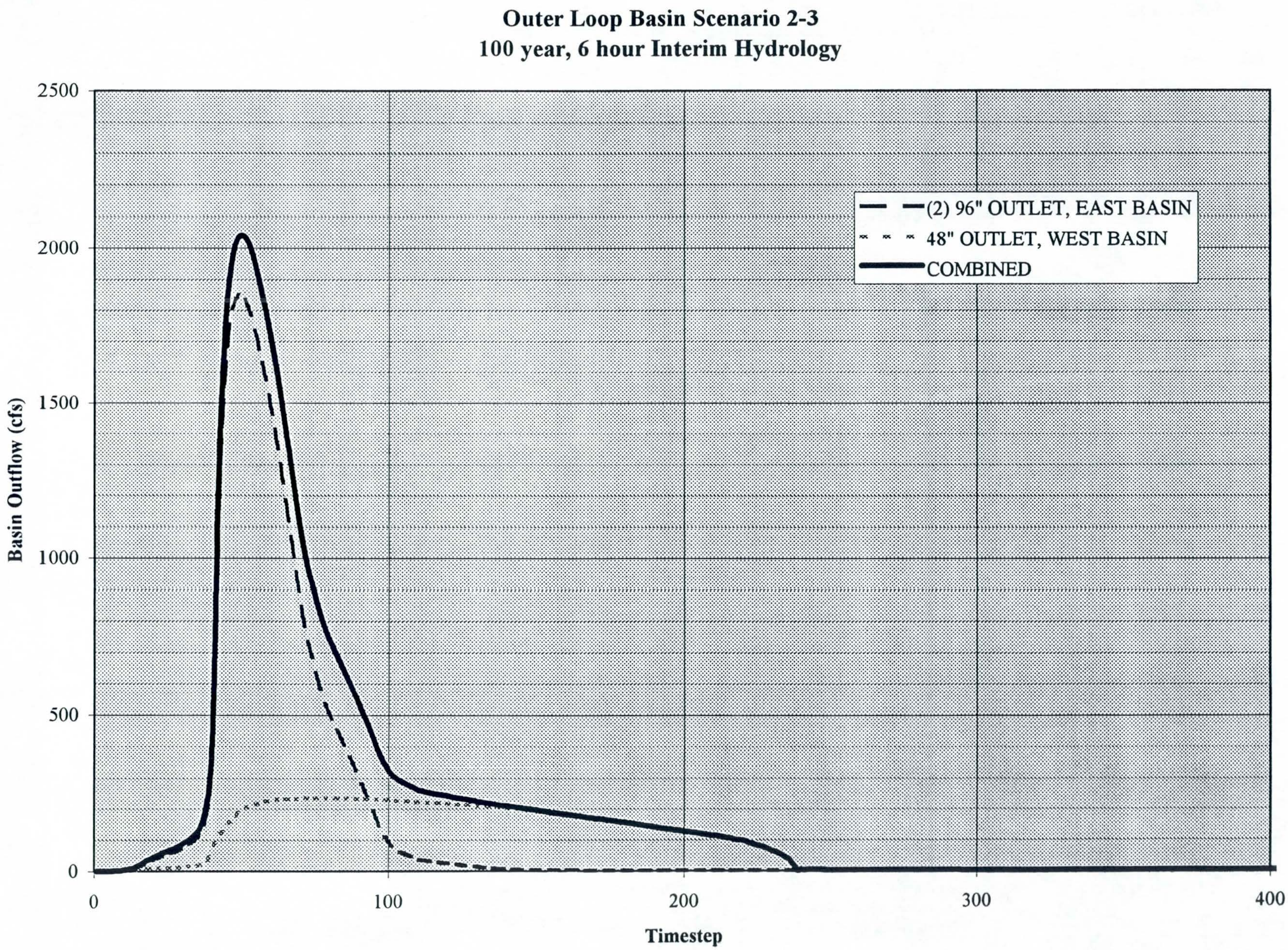
Ultimate outlet hydrographs.xls Chart3 (3)

9/4/98 1:42 PM



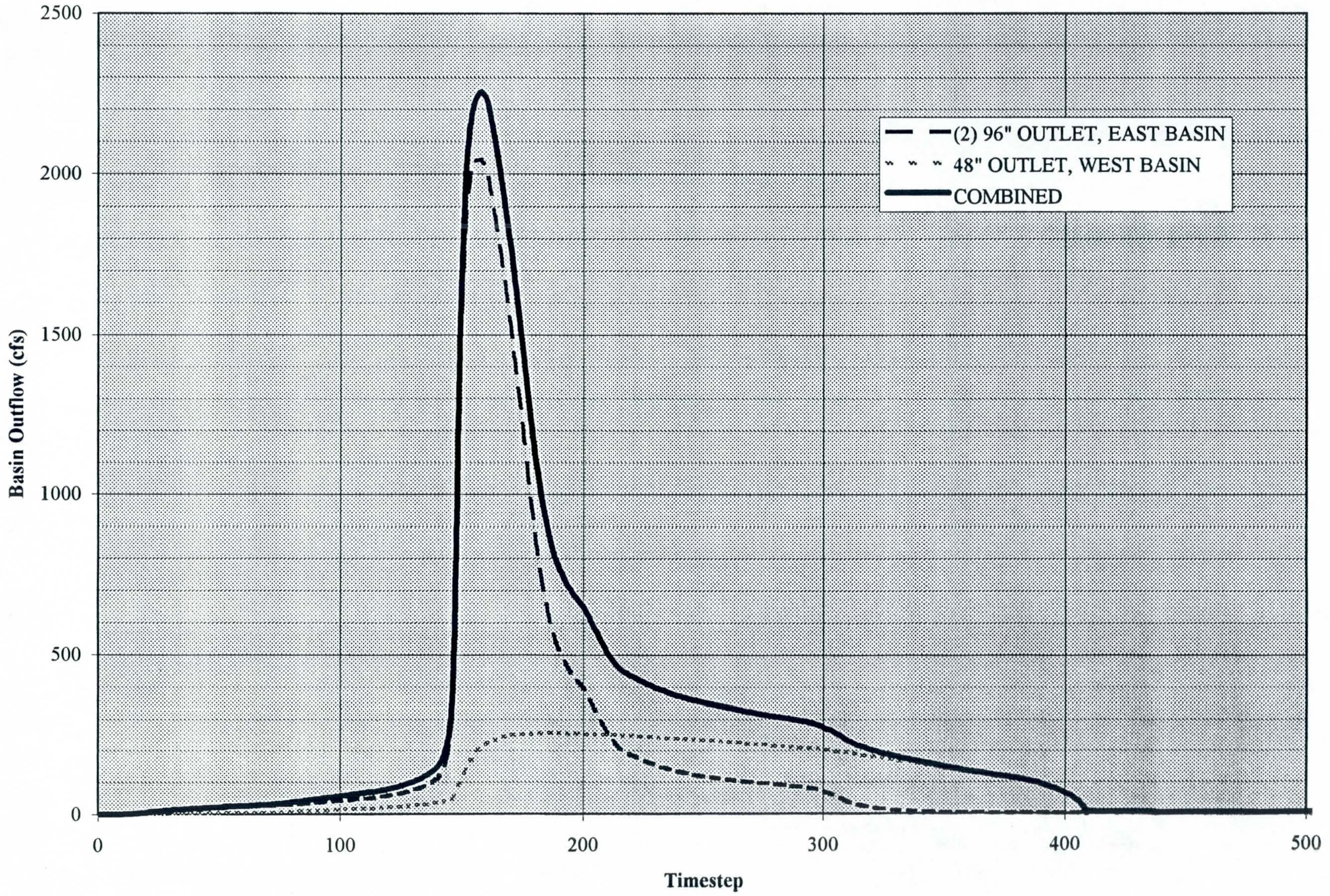
Outer Loop Basin Scenario 2-2
100 year, 6 hour Interim Hydrology
Basin Outflow (cfs)
2000
1500
1000
500
0
0
100
200
300
400
500
600
Timestep
(2) 96" OUTLET, EAST BASIN
(1) 24" OUTLET, WEST BASIN
COMBINED


Outer Loop Basin Scenario 2-3
50 year, 24 hour Interim Hydrology
(2) 96" OUTLET, EAST BASIN
48" OUTLET, WEST BASIN
COMBINED
Basin Outflow (cfs)
2500
2000
1500
1000
500
0
100
200
300
400
500
Timestep



interim outlet hydrographs.xls Chart2-3,100-6 9/8/98 1:44 PM

Outer Loop Basin Scenario 2-3
100 year, 24 hour Interim Hydrology



Memo

Stantec

To: File

File: 28900082

From: Chuck Gopperton

Date: 13 November 1998

Re: PR3B – Pima Freeway Basin Revised Scour Analysis and Toe-Down

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Basin Configuration and Operation

The Pima Freeway East Basin is a linear detention basin that extends from Hayden Road on the west to the Union Hills Drive interchange with the Pima Freeway on the east. The basin is on the north side of the Pima Freeway and essentially parallel to it throughout that approximately 3,500 ft length. Benches with tree planters are located on the north bank. The side-slope of the basin is 1V:1.5H on the south bank. On the north side, the slope is 1V:1.5H above and below each tree planter bench. The slope is about 1V:2.7H at rundown spillways that are between tree planters. The shotcrete lined side-slope extends 1 foot above the maximum water surface elevation. The basin has a bottom width of 60 ft with invert slopes of 0.1% and a basin invert low elevation of approximately 1,590 ft at the outlet works near Hayden Road. A 1 ft deep low flow channel is graded along the centerline of the basin. The bed of the basin will be unlined and landscaped with non-irrigated, native grasses.

The outlet from the basin is a double-barrel, 96 inch diameter concrete conduit that connects to the Hayden Road Conduits. Outflow passes through that outlet and is discharged to the USBR basin at the TPC Desert Golf Course on the east side of Hayden Road.

The basin is sized to route the larger of the 100-yr, 6-hr or 50-yr, 24-hr design storm and all lesser magnitude runoff events while maintaining a MWSE of approximately 1,611.5 ft. The major source of inflow to the basin is the Pima Road Conduits at the eastern end of the basin. Runoff from north of the basin, including the Grayhawk development, enters the basin through rundown spillways located at existing flow paths along the north side of the basin. The design peak discharges to the basin are summarized as follows:

Location of Basin Inflow	Peak Inflow, cfs (50-yr, 24-hour storm)	Peak Inflow, cfs (100-yr, 6-hour storm)
Pima Road Conduits	2,848	2,849
Total spillway inflow	1,865	2,084

As flow enters the basin and prior to the onset of detention ponding within the basin, the basin will function as a channel conveying moderate discharges throughout

**Reference: PR3B – Pima Freeway Basin Revised
Scour Analysis and Toe-Down**

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major lengths of that basin. Once ponding occurs, water continues to flow through the basin with depths and velocities high enough to cause scouring. In addition to longitudinal flow through the basin, overland flows also enter the basin through the spillways. Scour may occur due to several processes: 1. The low flow channel will tend to meander away from the centerline, 2. Flows entering through the spillways will create a scour hole at the bottom of each spillway, 3. High velocity flows entering from the Pima Road Conduits will tend to scour until the basin begins to pond, 4. At higher basin stages, longitudinal flow through the basin increases causing additional scour. Scour potential is the greatest at the toe of each spillway due to spillway flows and longitudinally along the edges of the basin adjacent to the shotcrete lining due to movement of water through the basin towards the outlet works. Due to the high potential for scour, scour protection along that lining is recommended to protect the bank lining from undercutting and potential failure of that lining.

The most critical scour condition would be sustained inflow to the basin from the Pima Road Conduits wherein the water would tend to move from the east end of the basin toward the outlet structure. Actual inflow conditions that would affect tailwater conditions, flow velocity and flow depth are too complex and varied to fully anticipate. In addition, it is difficult to predict the velocity profile at different basin stages. For this reason, scour potential is estimated for a range of basin flow from 0 to 2,000 cfs which corresponds to the maximum stage/discharge of the basin. In addition, scour potential is also estimated for the spillway aprons. Based on those results, reasonably prudent scour depths are estimated for which basin lining toe-down depth and scour protection are provided in the basin design.

Basin Hydraulics

Hydraulics (discharge-depth-velocity) relations for channel flow conditions are provided in Appendix A. The relations were analyzed for both normal depth channel flow and for stage-discharge continuity relations for the basin. It was determined that except for very low flows of 1 foot deep or less, the depth and velocity of flow within the basin is controlled by ponding in the basin and the discharge capacity of the outlet works. The hydraulic relations used for the scour analysis are therefore primarily based on hydrologic routing and flow continuity through the basin and fully consider the backwater due to impoundment. Flow velocities range from 4.48 fps to 1.91 fps with basin stage ranging from 1594 to 1611 and discharges from 128 cfs to 2001 cfs. Calculated velocities are mean values assuming a uniform velocity profile from the flowline to the water surface. Actual velocities at the flowline and along the boundary layers can not readily be estimated.

Spillway Hydraulics

Hydraulics (discharge-depth-velocity) relations and scour analysis for the spillway are provided in Appendix C. Unit discharges vary from 2.5 cfs/ft design discharge to 5.0 cfs/ft maximum capacity. Incoming flow velocities are estimated to be 5 fps and

**Reference: PR3B – Pima Freeway Basin Revised
Scour Analysis and Toe-Down**

velocities at the toe of the spillway are estimated to be 10 fps or less. No basin tailwater is assumed. This is the worst case scenario for conditions at the toe of the spillways where no flow is coming from the Pima Road Conduits, and only local inflow from the spillways.

Basin Bed-Material Size Gradation

Two sources of material size gradation data are available for the Pima Freeway Basin:

1. Geotechnical Investigation Report, Desert Greenbelt Phase 1 Channels, Pima Road and CAP Canal, Scottsdale, Arizona: AGRA Earth & Environmental, 25 August 1995.
2. Geotechnical Engineering Report, Freeway Basin and Outlet Conduit, Pima Three Basins Project : Ricker, Atkinson, McBee & Associates, Inc., 27 May 1998, and Supplement No. 1, 18 June 1998.

One size gradation sample is available for use from (1). It is identified as DB-1 @ 20-21.5 ft (see Appendix B for data). Two sets of size gradation data are available from (2), (see Appendix B for data). Percent retained on the #4 sieve and percent passing the #200 sieve are provided for 10 sample locations at various depths. That data are shown in Table 1 for the sample depths most closely representing the basin bed material. Two additional size gradation data are provided for sample locations 9 and 10, both at depths of 18 to 22 feet. Those gradation data are provided in Table 2.

TABLE 1

Basin bed-material size gradation data from RAM

Sample No.	Sample Depth, in feet	% Retained #4 Sieve (6.35 mm)	% Passing #200 sieve (0.127 mm)
(1)	(2)	(3)	(4)
1	10-15	22	15
2	10-15	3	63
3	15-20	15	20
4	10-15	9	34
5	15-20	14	31
6	10-15	13	23
7	15-20	13	29
8	10-15	16	23

**Reference: PR3B – Pima Freeway Basin Revised
Scour Analysis and Toe-Down**

TABLE 1

Basin bed-material size gradation data from RAM

Sample No. (1)	Sample Depth, in feet (2)	% Retained #4 Sieve (6.35 mm) (3)	% Passing #200 sieve (0.127 mm) (4)
9	15-20	14	30
10	10-15	16	22

The ten RAM (Table 1) samples show fairly consistent size gradation except sample No. 2, which shows exceptionally fine material. Disregarding Sample No. 2 as nonrepresentative, the averages of column (3) and (4) are 15% and 25%, respectively. The size gradation data are presented in Table 2. Notice that the average for the nine RAM samples agrees favorably with the one AGRA sample. Notice that the two samples from RAM for sample locations No. 9 and No. 10 (Table 2) are for soil that is finer than is represented by either the one AGRA sample or the average of the other nine RAM samples. The AGRA size gradation data is used in the scour analysis as generally representing the basin bed material. Also notice that 67% of the material is sand or finer with about 24% in the silt and clay size fraction. Virtually all of the material is smaller than fine gravel and is classified as alluvial silts and sand deposits. Clearly, this material is susceptible to erosion and scour even with low velocities. It is also noted that the RAM sample No. 2 indicates that zones of extremely fine sand, silt and clay may be exposed in the basin excavation.

TABLE 2

Basin bed-material size gradation

		% Finer			
			RAM		
Sieve (1)	Sieve Size, in mm (2)	AGRA[1] (3)	No. 9	No. 10	Avg[2]
200	0.075	24	43	33	25
100	0.150	29	49	38	---
50	0.300	34	---	---	---
40	0.425	37	57	47	---
30	0.600	41	---	---	---
16	1.18	54	69	62	---
10	2.00	67	---	---	---

**Reference: PR3B – Pima Freeway Basin Revised
Scour Analysis and Toe-Down**

TABLE 2

Basin bed-material size gradation

Sieve (1)	Sieve Size, in mm (2)	% Finer AGRA[1] (3)	% Finer RAM No. 9	% Finer RAM No. 10	% Finer Avg[2]
8	2.36	71	81	77	---
4	4.75	92	93	93	---
0.25"	6.35	97	---	---	85
0.375"	9.5	100	100	100	---

Notes: 1 – From Reference (1)
2 – Average of nine samples from Reference (2)

Scour Analysis

Scour analyses are performed for basin outlet works discharges ranging from 128 to 2,001 cfs. Scour analyses are also performed for spillway discharges of 2.5 cfs/ft and 5 cfs/ft. A summary of the results are presented in Table 3 and 4. Detailed scour calculations are provided in Appendix C. Reference material for the scour equations and procedures are provided in Appendix D.

TABLE 3

Basin scour depth estimation

Scour Components (1)	128 (2)	403 (3)	540 (4)	833 (5)	1094 (6)	1497 (7)	1802 (8)	2001 (9)
	Discharge, in cfs[4] / Depth of Scour, in feet							
Local Scour[1]	0.78	1.43	1.44	1.52	2.11	2.73	3.27	3.64
Low-Flow Incisement[2]	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Anti-Dune Scour[3]	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Calculated Scour	1.78	2.43	2.44	2.52	3.11	3.73	4.27	4.64
Safety Factor (0.3)	0.53	0.73	0.73	0.76	0.93	1.12	1.28	1.39
Total Scour	2.32	3.16	3.17	3.27	4.04	4.85	5.55	6.03

Notes:
1 - Local scour is average of scour depth estimated by four methods

**Reference: PR3B – Pima Freeway Basin Revised
Scour Analysis and Toe-Down**

(see Appendix C)
2 - Assumed depth
3 - Not applicable for sub-critical flow
4 - Scour depths for 128 cfs flow are based on normal depth channel hydraulics, all other flow rates are based on stage-discharge continuity

TABLE 4

Spillway Scour depth estimation

	Discharge, in cfs/ft	
	2.5	5.0
Scour Components	Depth of Scour, in feet	
(1)	(2)	(3)
Local Scour[1]	2.36	3.93
Safety Factor (0.3)	0.71	1.18
Total Scour	3.07	5.11

Notes:
1 – Local scour is the average of scour depth estimated by three methods (see Appendix C)

Basin scour depth is estimated for local scour and low-flow incisement. Anti-dune depths are not considered since the flow in the basin is subcritical. Long term degradation is also not considered for several reasons including: 1. The basin will receive frequent maintenance, 2. The east and west ends of the basin have grade control, 3. The maximum elevation difference between the two ends of the basin is only three feet which is less than the potential local scour depth. A safety factor of 30% increase in scour depth is recommended to be added to all calculated scour depths. Reasonable estimates of total scour depth (including safety factor) adjacent to the shotcrete bank lining (on both sides) are estimated at 2 - 6 feet for longitudinal flow in the basin and 3 - 5 feet at the base of the spillways (north side only).

Bank Lining Toe-Down and Scour Protection

Potential scour depths range from 2.32 feet to 6.03 feet along the entire basin due to longitudinal flow. Potential scour depths at the spillways range from 3.07 feet to 5.11 feet. These figures include a 30% safety factor of increased depth. Without the safety factor, scour from longitudinal flow ranges from 1.78 feet to 4.64 feet and scour from spillway flows ranges from 2.36 feet to 3.93 feet. It is recommended that the toe-down on the shotcrete bank lining be extended to the maximum depth of potential scour which is 4.64 feet. Additionally, it is recommended that riprap be

**Reference: PR3B – Pima Freeway Basin Revised
Scour Analysis and Toe-Down**

---

constructed as shown in Figure 1 along the entire length of the basin. The riprap, provides a safety factor for scour protection if local scour along the riprap should exceed the toe-down depth. In that case, the loose riprap will tumble into the scour hole thus armoring that bank against scour migration toward the bank lining. Additional depth of riprap should be provided at spillways where the maximum flow of 5 cfs/ft is expected. That riprap also (D_{50} = 8 inch) provides an apron and additional scour protection for the spillways and any uncontrolled runoff that could pass over the north slope basin lining. At the spillways, the riprap will extend to the surface and the one foot of native material backfill will not be used.

This scour analysis is based on limited size gradation analyses and assumptions. Actual size gradation of that basin bed material, and its variability over that approximately 3,500 feet of basin length, will not be known until that basin is excavated. Based on that basin excavation, and appropriate size gradation analyses, it may be necessary to refine these scour analyses and scour protection facilities. Additional scour analysis and protection is recommended immediately downstream of the Pima Road Conduits energy dissipater. That analysis and recommended design for the energy dissipater and associated scour protection will be presented in a separate memorandum.

Basin Maintenance

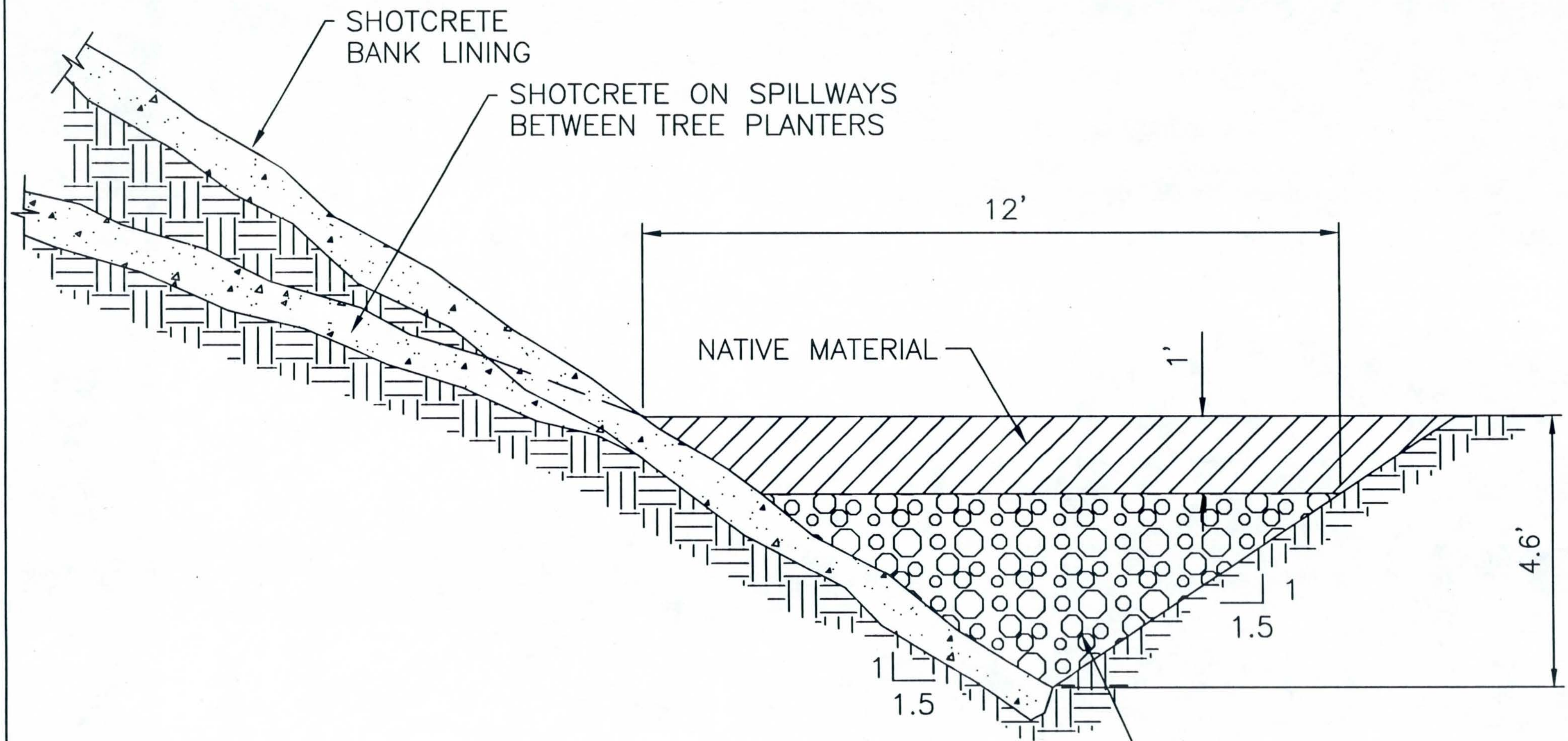
The basin is to be inspected annually and after each significant runoff event. Scour holes must be backfilled with riprap or other competent material. Low-flow incisement must be monitored and corrective measures taken to avoid flow concentration along the bank lining.

STANTEC CONSULTING INC.

Chuck Gopperton

Chuck Gopperton, PE
Water Resources Engineer
cgopperton@stantec.com

p:\28900082\correspondence\memos\memo to file, revised scour analysis & toe down 11-6.doc



Stantec

FIGURE 1

P:\28900051\ACADR13\DETAILS\CHNLDET.dwg

Appendix A

Stantech Consulting

Project: Pima Freeway East Basin Project Number: 28900082

Notes: Basin Scour Analysis Scale:

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Previous scour analysis used normal depth channel flow to calculate the depth, velocity and unit discharge at the basin for scour calculations. However this does not fit well with the calculated stage-discharge relationship which shows significant ponding even at low flows.

A better method is to use the stage-discharge relationship and calculate the velocity and unit discharge. By plotting the hydraulic data it is shown that the velocity decreases and the depth of water increases as the discharge through the basin outlet increases.

Even though inflow to the basin is over 4,000 cfs, outflow never exceeds 2,000 cfs. The scour analysis should consider the full range of flows through the basin which is very nearly equal to the outflow from the basin. Therefore - consider flows ranging from 0 - 2000 cfs.

Spillway flows should be analyzed for the design condition of 2.5 cfs and the maximum flow condition of 5.0 cfs per foot of length. Due to the distributive nature of the alluvial fan upstream of the basin, it is very likely that from time to time the flows discharging to any one spillway could reach the 5.0 cfs maximum capacity. Therefore this should be the controlling criteria.

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(702) 361-9050
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950 Industrial Way
Sparks, NV 89431
(702) 358-6931
Fax (702) 358-6954

Stantech Consulting

Project: PIMA FREEWAY EAST BASIN
Project Number: 28900082
Notes: SCOUR ANALYSIS
Scale:
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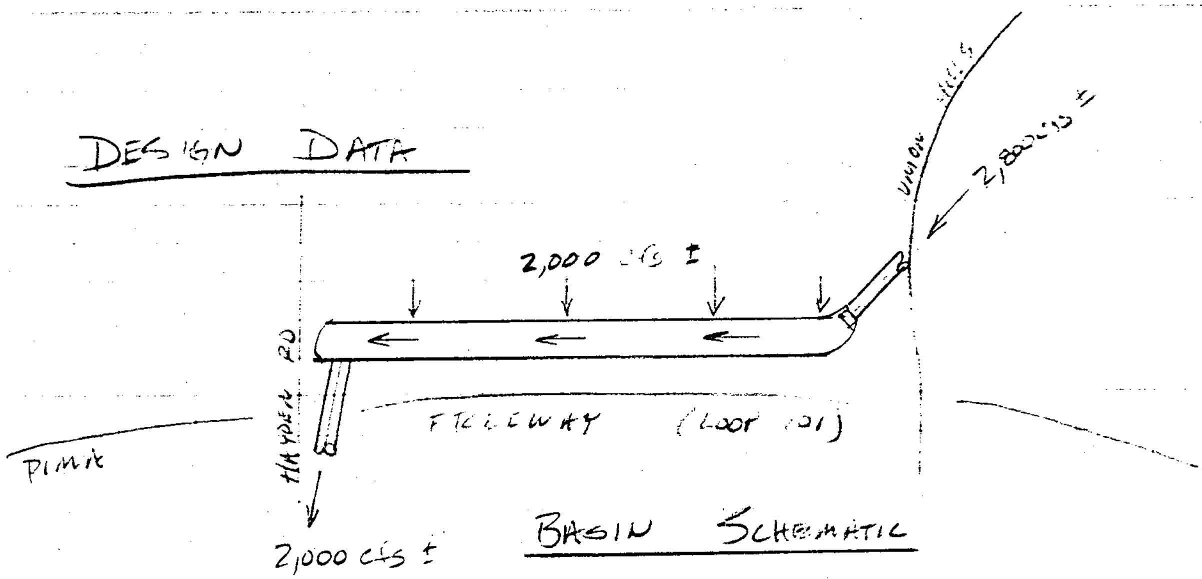
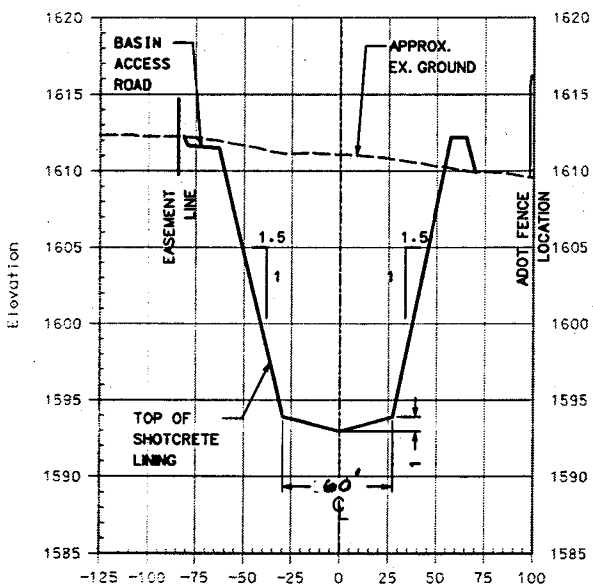
Date: 12/30/98
Checked By:
Date:

DESIGN DATA



BASIN SCHEMATIC



TYPICAL SECTION

ARIZONA

7776 Pointe Parkway
West., Suite 290
Phoenix, AZ 85044
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NEVADA

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950 Industrial Way
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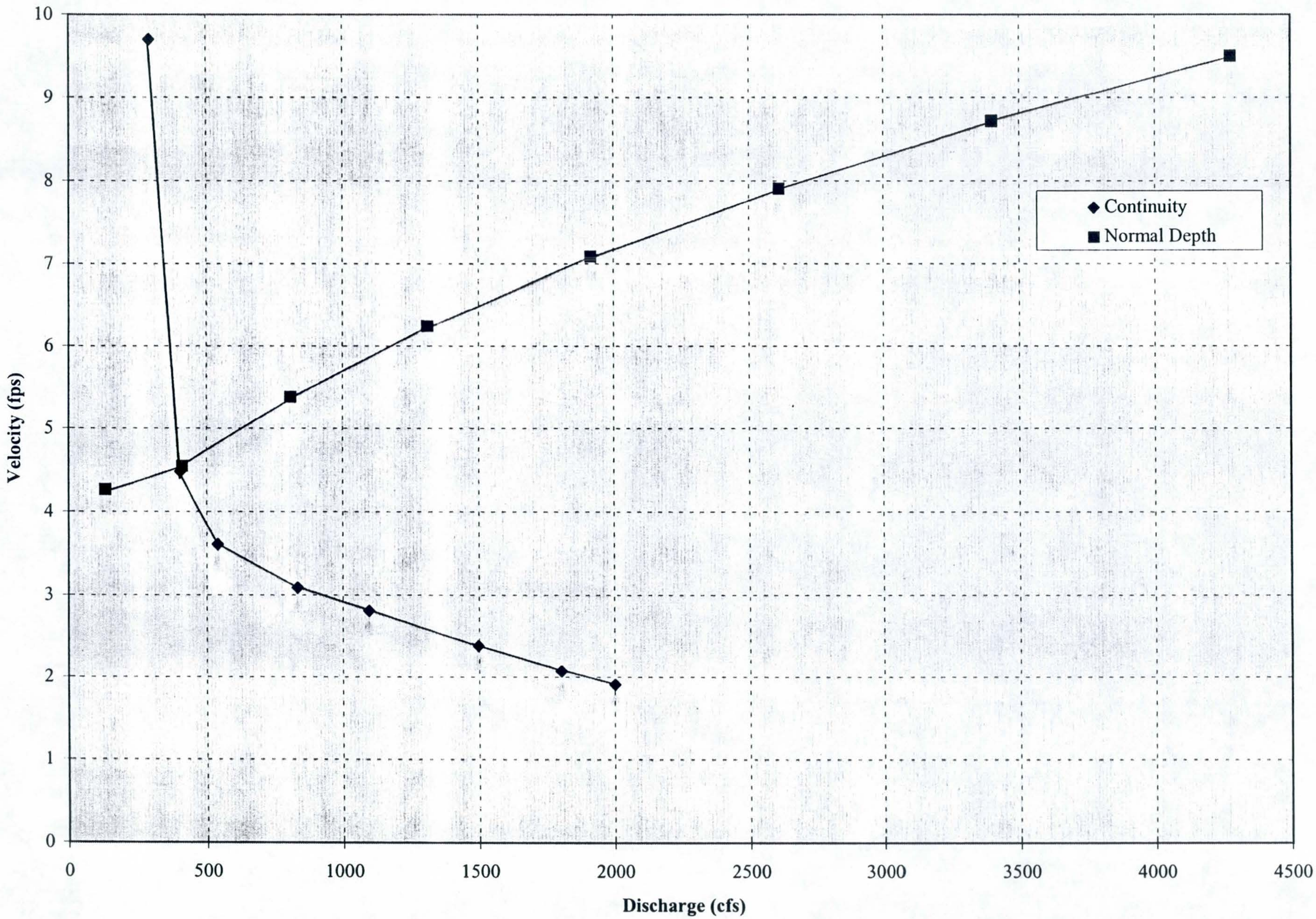
(702) 358-6931
Fax (702) 358-6954

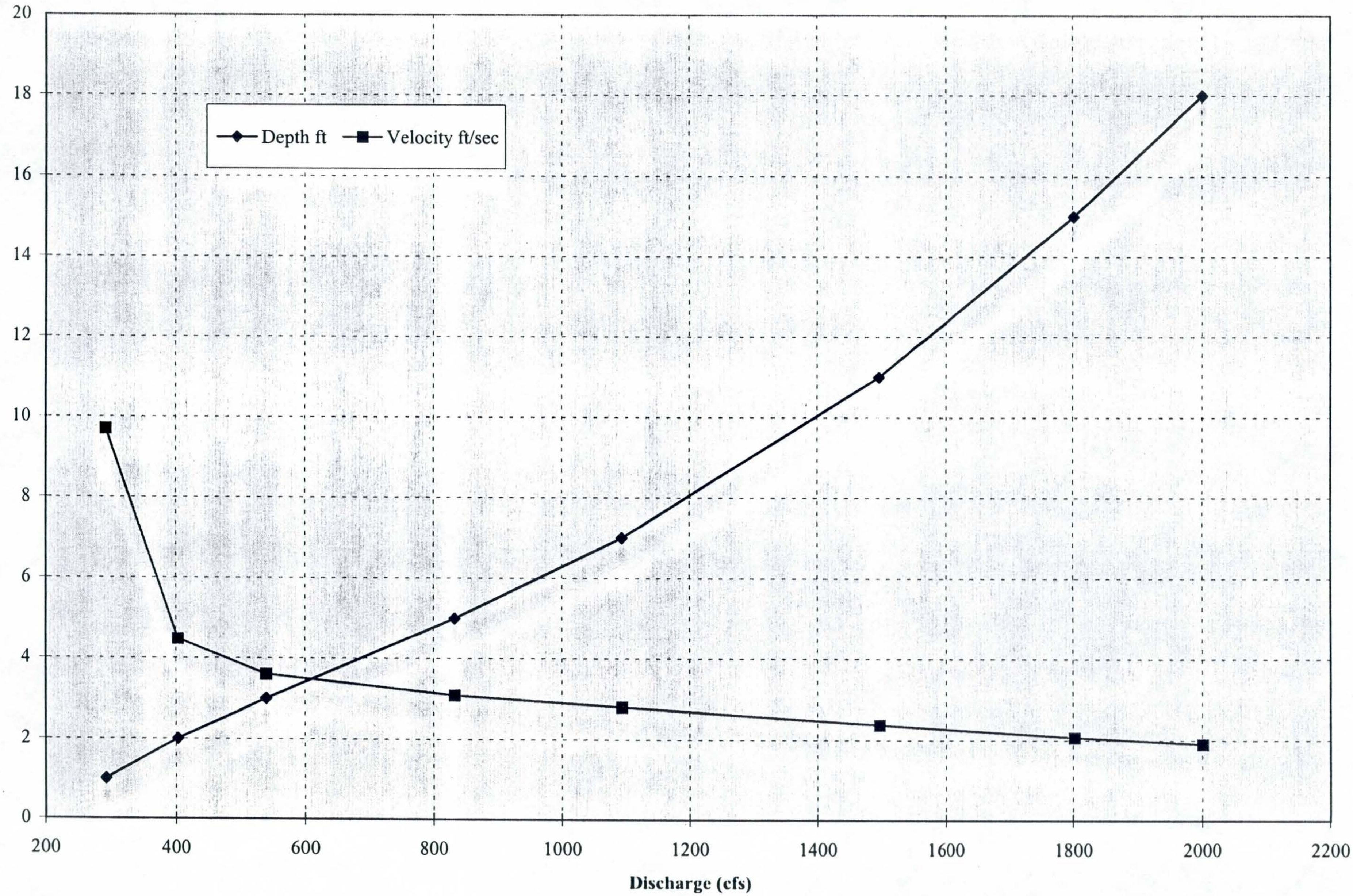
Pima Freeway East Basin						12-Nov-98
Flow parameters for scour analysis						
velocity calculated using continuity equation						
					Equiv	Unit
Discharge	Stage	Depth	Area	Velocity	Width	Discharge
cfs	ft	ft	sq ft	ft/sec	ft	cfs/ft
291	1594	1	30.00	9.70	30.00	9.70
403	1595	2	90.00	4.48	45.00	8.96
540	1596	3	150.00	3.60	50.00	10.80
833	1598	5	270.00	3.09	54.00	15.43
1094	1600	7	390.00	2.81	55.71	19.64
1497	1604	11	630.00	2.38	57.27	26.14
1802	1608	15	870.00	2.07	58.00	31.07
2001	1611	18	1050.00	1.91	58.33	34.30

Pima Freeway East Basin							12-Nov-98
Flow parameters for scour analysis							
velocity calculated using mannings normal depth equation							
					Equiv	Unit	
Discharge	Stage	Depth	Area	Velocity	Width	Discharge	
cfs	ft	ft	sq ft	ft/sec	ft	cfs/ft	
128	1594	1	30.00	4.27	30.00	4.27	
409	1595	2	90.00	4.54	45.00	9.09	
808	1596	3	150.00	5.39	50.00	16.16	
1312	1597	4	210.00	6.25	52.50	24.99	
1914	1598	5	270.00	7.09	54.00	35.44	
2610	1599	6	330.00	7.91	55.00	47.45	
3399	1600	7	390.00	8.72	55.71	61.01	
4278	1601	8	450.00	9.51	56.25	76.05	

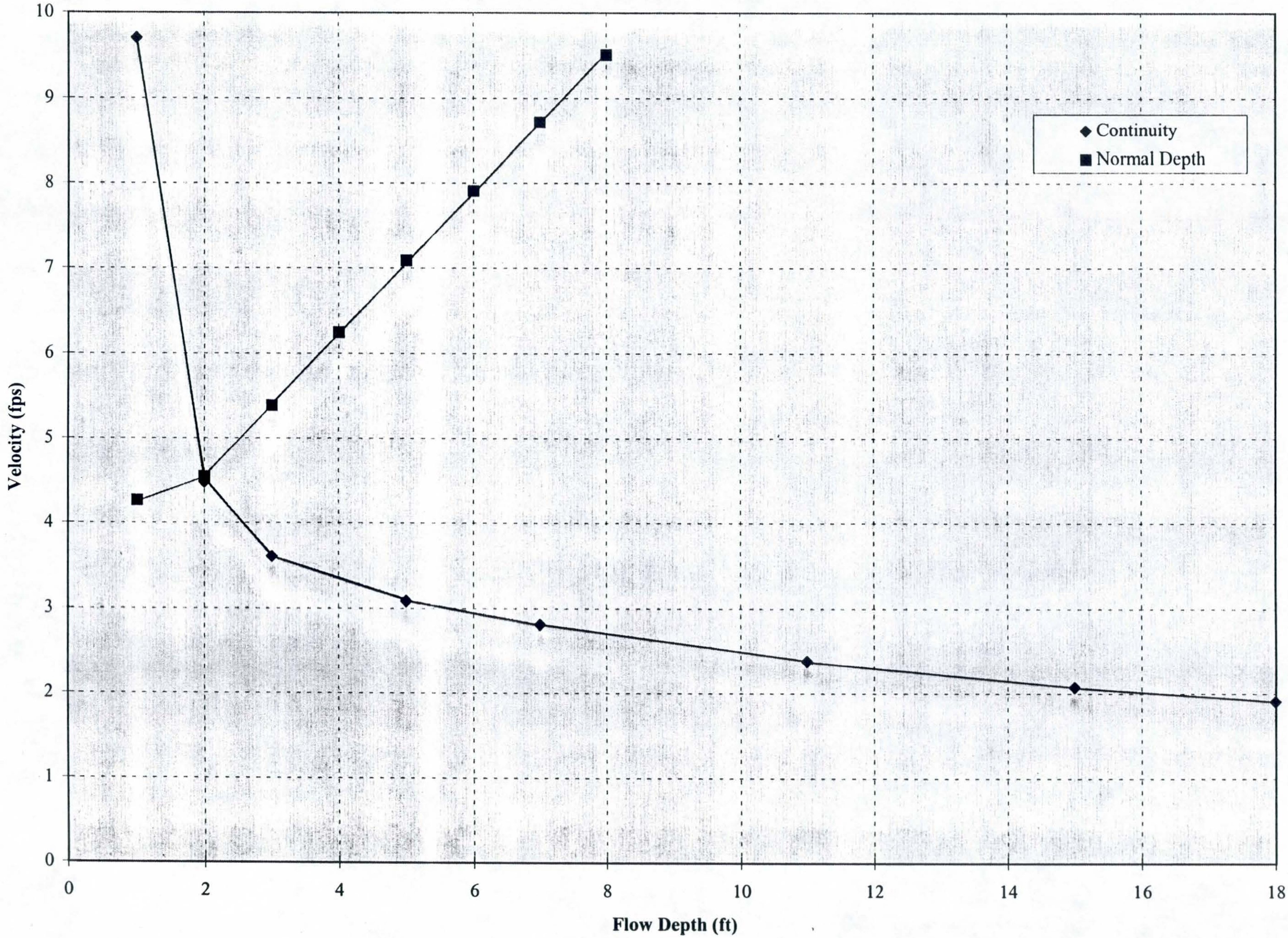
Pima Freeway East Basin Velocity-Discharge Relationship



Pima Freeway East Basin Depth-Discharge-Velocity
(Continuity Equation)



Pima Freeway East Basin, Depth vs Velocity



Scour.xls - Chart6

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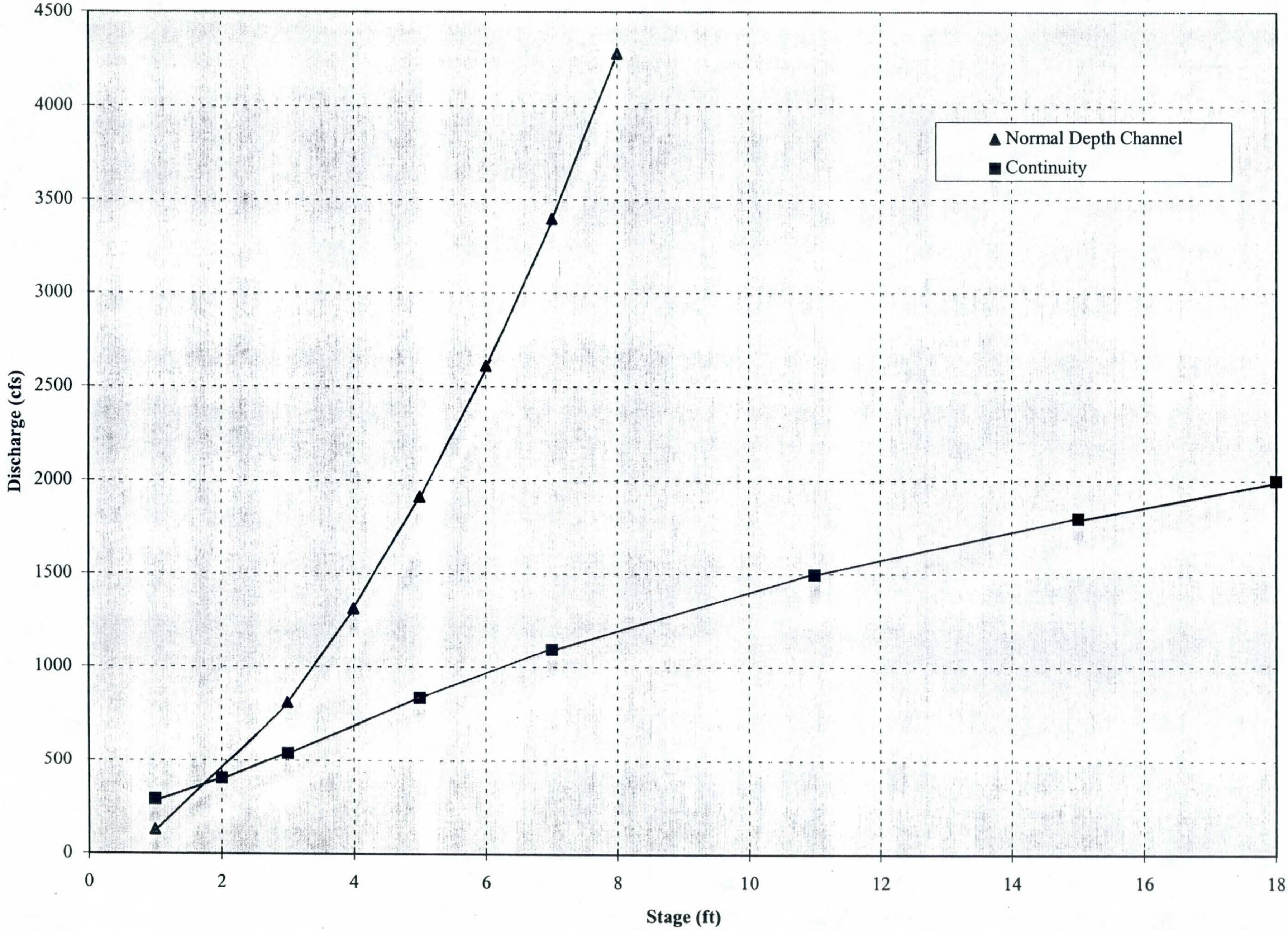
Pima Freeway East Basin, Stage - Discharge Relationship



Scour.xls - Chart7

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Appendix B

AGRA Earth & Environmental, Inc.

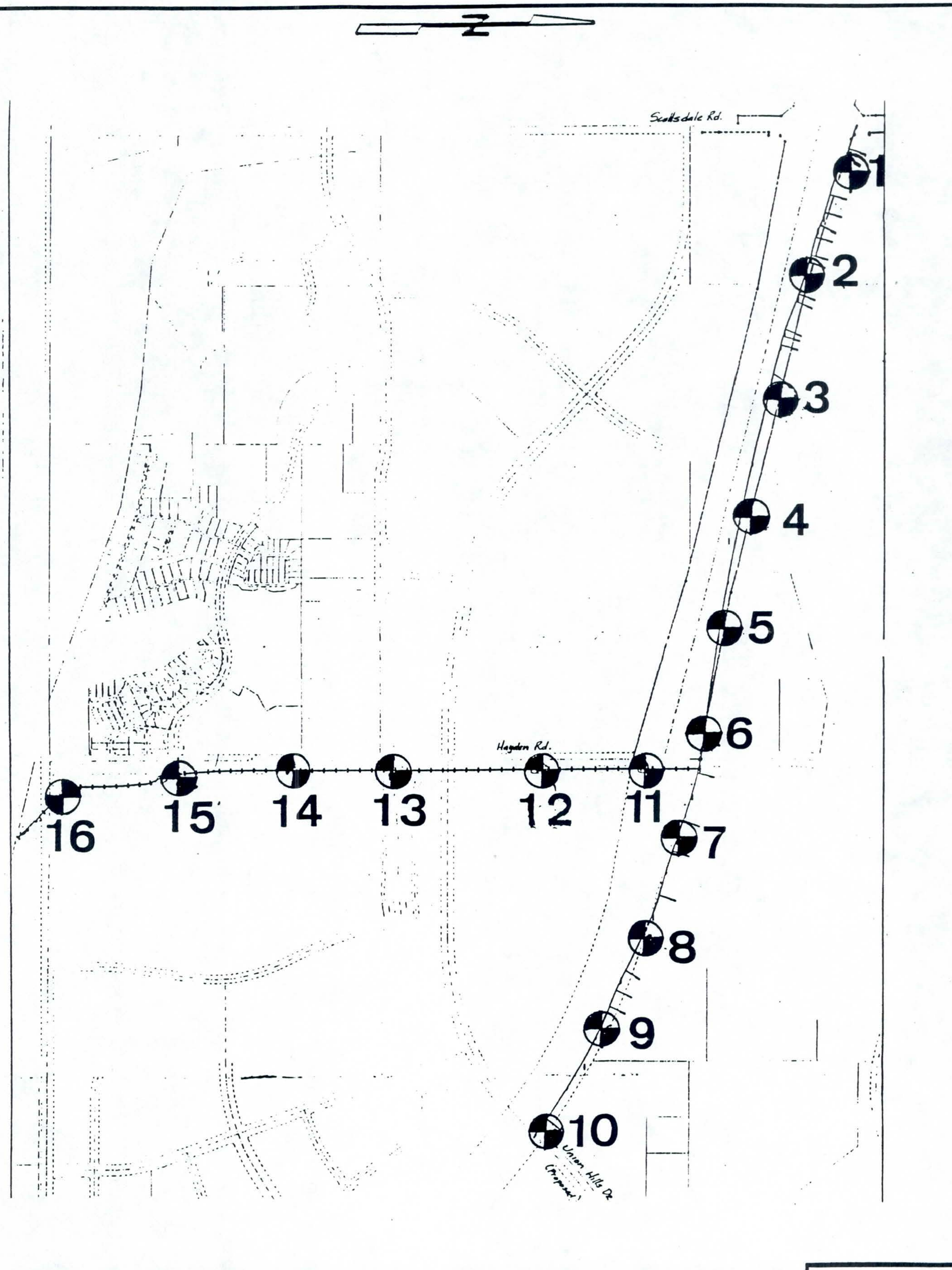
PROJECT:	DESERT GREENBELT - PHASE I	**JOB NO:**	E95-86
LOCATION:	PIMA ROAD BETWEEN BELL & PINNACLE PEAK	**WORK ORDER NO:**	6
		DATE SAMPLED:	07-21-95

MECHANICAL SIEVE ANALYSIS
GROUP SYMBOL, USCS (ASTM D-2487)

SIEVE SIZES

				Silt or Clay	SAND								GRAVEL								
					Fine			Medium			Coarse		Fine				Coarse				
Location & Depth	**USCS**	**LL**	**PI**	**#200**	**#100**	**#50**	**#40**	**#30**	**#16**	**#10**	**#8**	**#4**	**1/4"**	**3/8"**	**1/2"**	**3/4"**	**1"**	**1 1/2"**	**2"**	**3"**	**Lab #**
				PERCENT PASSING BY WEIGHT																	
DB-1 @ 5 - 6'	CL	30	9	78	87	91	93	94	96	97	97	99	99	99	100	100	100	100	100	100	215
DB-1 @ 20 - 21.5'	SM	40	14	24	29	34	37	41	54	67	71	92	97	100	100	100	100	100	100	100	218
RP-1 @ 0 - 1.5'	SM	NV	NP	24	28	35	39	44	55	66	71	87	93	97	100	100	100	100	100	100	195
RP-3 @ 10 - 11.5'	SC	30	10	17	22	30	35	40	54	66	71	86	92	98	99	100	100	100	100	100	202
RP-4 @ 5 - 6.5'	SM	NV	NP	19	25	34	41	47	62	74	78	92	95	97	100	100	100	100	100	100	205
RP-5 @ 0 - 1.5'	SC	28	9	26	32	42	48	55	71	82	85	94	97	99	100	100	100	100	100	100	208
RP-6 @ 5 - 6.5'	SC	30	10	21	24	30	33	38	50	64	70	89	95	100	100	100	100	100	100	100	211
RP-6 @ 13 - 15'	GP-GC	33	16	8.2	9	11	13	15	21	29	33	47	54	64	71	85	98	100	100	100	213

AGRA
Earth & Environmental



Scottsdale Rd.
1
2
3
4
5
6
Hayden Rd.
16
15
14
13
12
11
7
8
9
10
Union Hills Dr
(Proposed)
Test Boring Location

SITE PLAN

LABORATORY TEST RESULTS

Date: 13-May-98

SAMPLE SOURCE: As noted below

TESTING PERFORMED: Percent Passing No. 200 Sieve, Atterberg Limits (ASTM D1140, D4318)

SAMPLED BY: RAM/Miller

RESULTS:

Sample Source	Percent Retained No. 4 Sieve	Percent Passing No. 200 Sieve	Liquid Limit	Plasticity Index
1 @ 5'-10'	18	32	27	7
1 @ 15'-20'	22	15	29	11
2 @ 0'-5'	14	44	28	10
2 @ 10'-15'	3	63	39	21
3 @ 5'-10'	18	27	26	8
3 @ 15'-20'	15	20	42	26
4 @ 0'-5'	12	30	24	5
4 @ 10'-15'	9	34	29	9
5 @ 5'-10'	15	26	26	8
5 @ 15'-20'	14	31	38	20
6 @ 0'-5'	8	24	20	4
6 @ 10'-15'	13	23	29	14
7 @ 5'-10'	13	31	22	5
7 @ 15'-20'	13	29	27	7
8 @ 0'-5'	18	21	20	3
8 @ 10'-15'	16	23	26	9

LABORATORY TEST RESULTS

Date: 13-May-98

SAMPLE SOURCE: As noted below

TESTING PERFORMED: Percent Passing No. 200 Sieve, Atterberg Limits (ASTM D1140, D4318)

SAMPLED BY: RAM/Miller

RESULTS:

Sample Source	Percent Retained No. 4 Sieve	Percent Passing No. 200 Sieve	Liquid Limit	Plasticity Index
9 @ 5'-10'	18	22	21	4
9 @ 15'-20'	14	30	28	10
10 @ 0'-5'	25	13	N/A	Non-Plastic
10 @ 10'-15'	16	22	26	6
11 @ 0'-5'	11	37	25	6
11 @ 5'-10'	11	30	23	3
12 @ 0'-5'	10	44	27	8
12 @ 10'-15'	25	31	36	13
13 @ 0'-5'	29	19	33	17
13@ 15'-20'	17	22	38	21
14 @ 0'-5'	10	42	26	9
14 @ 5'-10'	11	29	24	7
15 @ 0'-5'	11	41	31	13
15 @ 10'-15'	17	29	39	22
16 @ 0'-5'	15	35	27	11
16 @ 15'-20'	7	33	29	12

RICKER • ATKINSON • McBEE & ASSOCIATES, INC.

Geotechnical Engineering • Construction Materials Testing

R·A·M

Stantech Consulting
7776 Pointe Parkway West, Suite 290
Phoenix, Arizona 85044

June 18, 1998

Attention: Chuck Gopperton, P.E.

Subject: Geotechnical Engineering Report
Freeway Basin and Outlet Conduit
Pima 3 Basins Project
Loop 101 - Scottsdale Road to Union Hills Drive
Hayden Road - Loop 101 to Bell Road
Scottsdale, Arizona

R.A.M. Project No. G02281
Supplement No. 1

At your request, this firm has reviewed the geotechnical report for the subject project with respect to:

1. Soil gradation at the east end of basin for use in Stantech's scour analysis.

2. Review and comments on the flat edge drain strip and weep holes behind the liner in the basin sides.

3. The use of cast-in-place concrete pipe in the outlet conduit along Hayden Road.

Additional tests have been completed on soils samples from Test Borings 9 and 10 at the east end of the basin and the results are:

Percent Passing (Sieve Size)

Location	No. 200	No. 100	No.40	No. 16	No. 8	No. 4	No. 3/4"
9 @ 18' to 22'	43	49	57	69	81	93	100
10 @ 18' to 22'	33	38	47	62	77	93	100

Sands are angular to subangular.

The following drawings were reviewed with respect to the drainage system behind the liner and in the planter areas. The following comments are presented for your use.

Sheet No.	Comments
D1; Section C	1. Since rip-rap spillway will be subjected to flows over the surface, will a geotextile filter fabric be required at the soil rip-rap interface to prevent piping of the soil into the rip-rap. 2. The rip-rap will fill with water. A way to drain this zone should be provided, such as using weep pipes which extend through the lining-soil-turndown or under the turndown.
D3; Section A	1. Planter should have a PVC or gunite bottom. 2. The planter should have weep pipes which drain the bottom of planter through turndown-soil-lining.
D5; Section A	1. The soil end of the outlet coupling (weep pipes) should be either covered with filter fabric or preferably terminated on and surrounded by the 12" flat edge drain. 2. Modify section or add a new section so that the drainage system extends down behind the lining below the planter.
D5; Plan	1. Limits of polyethylene 8 mil moisture barrier as shown would extend around the bottom and sides of the turndown along the top of the basin or planter and the turndown on the uphill side of the planter. This layer should be terminated at the turndown.

The use of cast-in-place pipe generally requires excavations be accomplished with a special rounded bucket. Due to zones of heavy cementation, excavation with this kind of bucket may be slow and difficult to accomplish and could require excavating with a conventional bucket or rock bucket below proposed grade, backfilling the lower half of the pipe zone and re-excavating with the special rounded bucket. In addition, some relatively clean sand lenses may be encountered which will not maintain the round bottom configuration before or during slip form placement of the concrete.

If you have any questions, please do not hesitate to call. This supplement should be attached to and made a part of the original report.

Respectfully submitted,

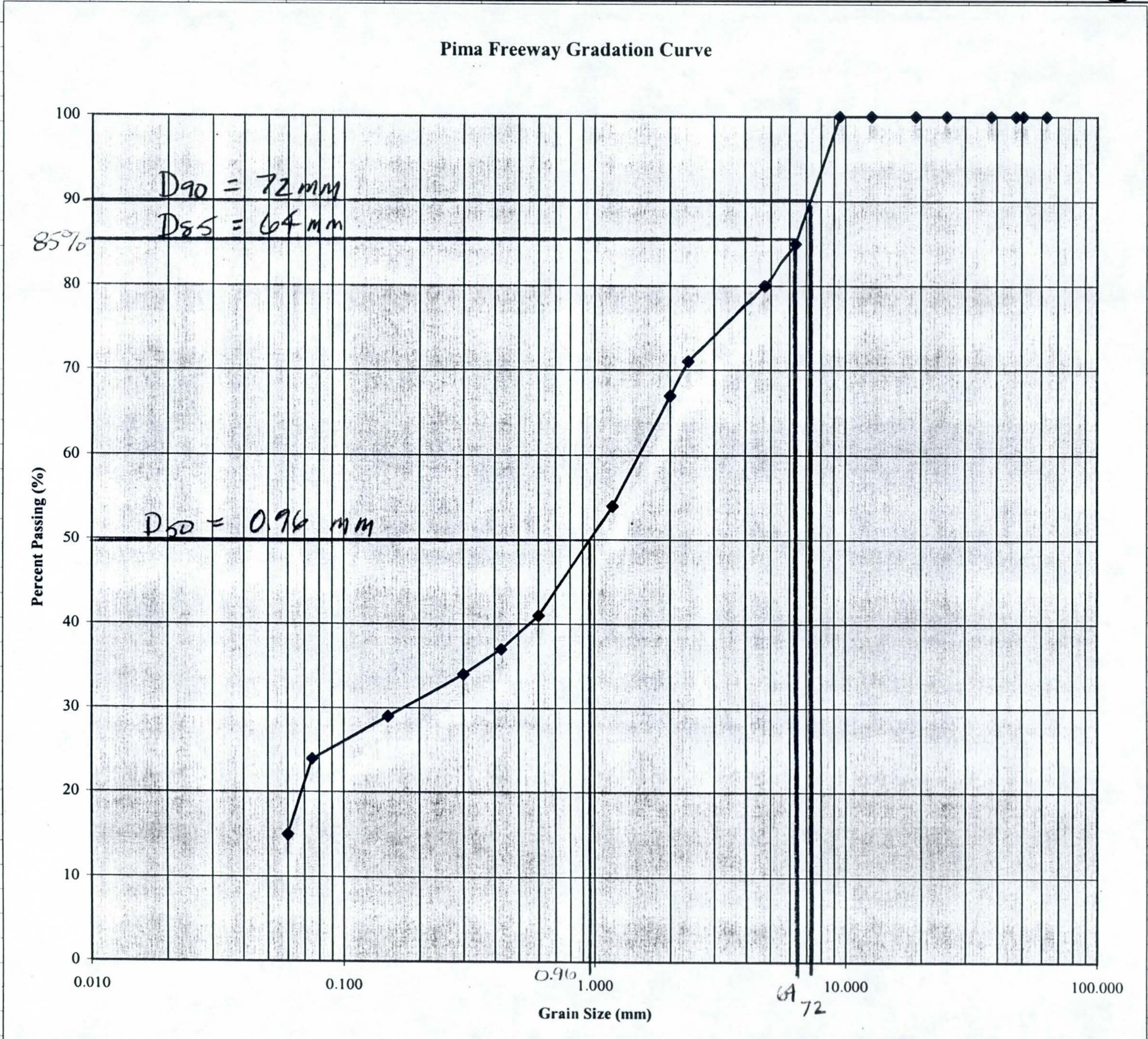
RICKER, ATKINSON, MCBEE & ASSOCIATES, INC.

Registered Professional Engineer (Civil)
CERTIFICATE NO.
10971
KENNETH L.
RICKER
Date Signed
ARIZONA U.S.A.

By: Kenneth L. Ricker, P.E.

/nk
Copies to: Addressee (5)

Size (mm)	% Finer
0.060	15
0.075	24
0.150	29
0.300	34
0.425	37
0.600	41
1.180	54
2.000	67
2.360	71
4.750	80
6.350	85
9.500	100
12.700	100
19.050	100
25.400	100
38.100	100
47.600	100
50.800	100
63.000	100



Appendix C

Flood Control District of Maricopa County
2801 West Durango Street
Phoenix, Arizona 85009-6399
(602) 506-1501
FAX: (602) 506-4601
TT: (602) 506-5897

15 October, 1998

MEMO TO: John Rodriguez

FROM: W. Scott ~~Ogden~~ WSO

CC: Pedro Calza

SUBJECT: PR3B – Pima Freeway Basin Scour Report

I have reviewed the 23 June 1998 design memo and find the scour analyses and proposed toe down depth acceptable. Usually the District requires a factor of safety of 1.3 to be applied to all scour calculations as long as estimates of all scour components (i.e. long term, general, local, low flow incisement, and anti-dune) are calculated. Ed Raleigh attended the biweekly meeting at which the results of this memo were discussed and it was agreed that the rock rip-rap placed at the bottom of the toe down (as indicated in the memo), will adequately serve as the factor of safety for this design.

If you have any questions or require further discussion, please let me know.

PROJECT NAME: Pima Freeway East Basin
PROJECT NO: 28900082
ANALYSIS BY: Stantech Consulting
DATE: 11-12-98

LOCAL SCOUR ANALYSIS
Types A and B - Natural Channel for Restriction and Bends and Bankline Structures

APPLICATIONS INCLUDE:

Type A	Type B
(a) Siphon Crossing	(e) Abutments to Bridge/Siphon Crossings
(b) Buried Pipeline	(f) Bank Slope Protection (Riprap)
(c) Nat'l Bank Stability	(g) Spur Dikes, Groins, etc.
(d) One-Span Bridge Waterway	(h) Pumping Plants
	(i) Canal Headworks

HYDRAULIC DATA

Discharge (cfs):	128
Mean Depth (ft):	1.00
Mean Velocity (fps):	4.27
Unit Discharge (cfs/ft):	4.27
Threshold Velocity (fps):	2.04

SEDIMENT DATA

Material Grain Size, D50 (mm): 1.002

REACH INFORMATION

Straight Reach

RESULTS OF ANALYSIS

Methods	Scour Depth	
(a) USBR I Equation	NOT APPLICABLE	
(b) Lacey Equation	0.490 ft	0.149 m
(c) Blench Equation	1.296 ft	0.395 m
(d) USBR II Equation	0.250 ft	0.076 m
(e) Neill Equation	1.093 ft	0.333 m
AVERAGE SCOUR DEPTH	0.782 ft	0.239 m

COMMENTS:

REFERENCES:
(1) Pemberton, E.L. and J. M. Lara, Computing Degradation and Local Scour, Technical Guideline for Bureau of Reclamation, January 1984.

PROJECT NAME: Pima Freeway East Basin
PROJECT NO: 28900082
ANALYSIS BY: Stantech Consulting
DATE: 11-12-98

LOCAL SCOUR ANALYSIS
Types A and B - Natural Channel for Restriction and Bends and Bankline Structures

APPLICATIONS INCLUDE:

Type A	Type B
(a) Siphon Crossing	(e) Abutments to Bridge/Siphon Crossings
(b) Buried Pipeline	(f) Bank Slope Protection (Riprap)
(c) Nat'l Bank Stability	(g) Spur Dikes, Groins, etc.
(d) One-Span Bridge Waterway	(h) Pumping Plants
	(i) Canal Headworks

HYDRAULIC DATA

Discharge (cfs):	403
Mean Depth (ft):	2.00
Mean Velocity (fps):	4.48
Unit Discharge (cfs/ft):	8.96
Threshold Velocity (fps):	2.04

SEDIMENT DATA

Material Grain Size, D50 (mm):	1.002

REACH INFORMATION

Straight Reach

RESULTS OF ANALYSIS

Methods	Scour Depth	
(a) USBR I Equation	NOT APPLICABLE	
(b) Lacey Equation	0.718 ft	0.219 m
(c) Blench Equation	2.124 ft	0.648 m
(d) USBR II Equation	0.500 ft	0.152 m
(e) Neill Equation	2.392 ft	0.729 m
AVERAGE SCOUR DEPTH	1.434 ft	0.437 m

COMMENTS:

REFERENCES:
(1) Pemberton, E.L. and J. M. Lara, Computing Degradation and Local Scour, Technical Guideline for Bureau of Reclamation, January 1984.

PROJECT NAME: Pima Freeway East Basin
PROJECT NO: 28900082
ANALYSIS BY: Stantech Consulting
DATE: 11-12-98

LOCAL SCOUR ANALYSIS

Types A and B - Natural Channel for Restriction and Bends and Bankline Structures

APPLICATIONS INCLUDE:

Type A	Type B
(a) Siphon Crossing	(e) Abutments to Bridge/Siphon Crossings
(b) Buried Pipeline	(f) Bank Slope Protection (Riprap)
(c) Nat'l Bank Stability	(g) Spur Dikes, Groins, etc.
(d) One-Span Bridge Waterway	(h) Pumping Plants
	(i) Canal Headworks

HYDRAULIC DATA

Discharge (cfs):	540
Mean Depth (ft):	3.00
Mean Velocity (fps):	3.60
Unit Discharge (cfs/ft):	10.80
Threshold Velocity (fps):	2.24

SEDIMENT DATA

Material Grain Size, D50 (mm): 1.002

REACH INFORMATION

Straight Reach

RESULTS OF ANALYSIS

Methods	Scour Depth	
(a) USBR I Equation	NOT APPLICABLE	
(b) Lacey Equation	0.792 ft	0.241 m
(c) Blench Equation	2.405 ft	0.733 m
(d) USBR II Equation	0.750 ft	0.229 m
(e) Neill Equation	1.821 ft	0.555 m
AVERAGE SCOUR DEPTH	1.442 ft	0.440 m

COMMENTS:

REFERENCES:

(1) Pemberton, E.L. and J. M. Lara, Computing Degradation and Local Scour, Technical Guideline for Bureau of Reclamation, January 1984.

PROJECT NAME: Pima Freeway East Basin
PROJECT NO: 28900082
ANALYSIS BY: Stantech Consulting
DATE: 11-12-98

LOCAL SCOUR ANALYSIS

Types A and B - Natural Channel for Restriction and Bends and Bankline Structures

APPLICATIONS INCLUDE:

Type A	Type B
(a) Siphon Crossing	(e) Abutments to Bridge/Siphon Crossings
(b) Buried Pipeline	(f) Bank Slope Protection (Riprap)
(c) Nat'l Bank Stability	(g) Spur Dikes, Groins, etc.
(d) One-Span Bridge Waterway	(h) Pumping Plants
	(i) Canal Headworks

HYDRAULIC DATA

Discharge (cfs):	833
Mean Depth (ft):	5.00
Mean Velocity (fps):	3.09
Unit Discharge (cfs/ft):	15.43
Threshold Velocity (fps):	2.64

SEDIMENT DATA

Material Grain Size, D50 (mm):	1.002

REACH INFORMATION

Straight Reach

RESULTS OF ANALYSIS

Methods	Scour Depth	
(a) USBR I Equation	NOT APPLICABLE	
(b) Lacey Equation	0.915 ft	0.279 m
(c) Blench Equation	3.051 ft	0.930 m
(d) USBR II Equation	1.250 ft	0.381 m
(e) Neill Equation	0.852 ft	0.260 m
AVERAGE SCOUR DEPTH	1.517 ft	0.463 m

COMMENTS:

REFERENCES:

(1) Pemberton, E.L. and J. M. Lara, Computing Degradation and Local Scour, Technical Guideline for Bureau of Reclamation, January 1984.

PROJECT NAME: Pima Freeway East Basin
PROJECT NO: 28900082
ANALYSIS BY: Stantech Consulting
DATE: 11-12-98

LOCAL SCOUR ANALYSIS

Types A and B - Natural Channel for Restriction and Bends and Bankline Structures

APPLICATIONS INCLUDE:

Type A	Type B
(a) Siphon Crossing	(e) Abutments to Bridge/Siphon Crossings
(b) Buried Pipeline	(f) Bank Slope Protection (Riprap)
(c) Nat'l Bank Stability	(g) Spur Dikes, Groins, etc.
(d) One-Span Bridge Waterway	(h) Pumping Plants
	(i) Canal Headworks

HYDRAULIC DATA

Discharge (cfs):	1094
Mean Depth (ft):	7.00
Mean Velocity (fps):	2.81
Unit Discharge (cfs/ft):	19.64
Threshold Velocity (fps):	2.93

SEDIMENT DATA

Material Grain Size, D50 (mm):	1.002

REACH INFORMATION

Straight Reach

RESULTS OF ANALYSIS

Methods	Scour Depth	
(a) USBR I Equation	NOT APPLICABLE	
(b) Lacey Equation	1.002 ft	0.305 m
(c) Blench Equation	3.583 ft	1.092 m
(d) USBR II Equation	1.750 ft	0.534 m
(e) Neill Equation	~~0.287 ft~~	~~0.088 m~~ N/A
AVERAGE SCOUR DEPTH	1.512 ft	0.461 m

COMMENTS:
AVERAGE = 2.11

REFERENCES:

(1) Pemberton, E.L. and J. M. Lara, Computing Degradation and Local Scour, Technical Guideline for Bureau of Reclamation, January 1984.

PROJECT NAME: Pima Freeway East Basin
PROJECT NO: 28900082
ANALYSIS BY: Stantech Consulting
DATE: 11-12-98

LOCAL SCOUR ANALYSIS
Types A and B - Natural Channel for Restriction and Bends and Bankline Structures

APPLICATIONS INCLUDE:

Type A	Type B
(a) Siphon Crossing	(e) Abutments to Bridge/Siphon Crossings
(b) Buried Pipeline	(f) Bank Slope Protection (Riprap)
(c) Nat'l Bank Stability	(g) Spur Dikes, Groins, etc.
(d) One-Span Bridge Waterway	(h) Pumping Plants
	(i) Canal Headworks

HYDRAULIC DATA

Discharge (cfs):	1497
Mean Depth (ft):	11.00
Mean Velocity (fps):	2.38
Unit Discharge (cfs/ft):	26.14
Threshold Velocity (fps):	3.51

SEDIMENT DATA

Material Grain Size, D50 (mm): 1.002

REACH INFORMATION

Straight Reach

RESULTS OF ANALYSIS

Methods	Scour Depth		
(a) USBR I Equation	NOT APPLICABLE		
(b) Lacey Equation	1.113 ft	0.339 m	
(c) Blench Equation	4.336 ft	1.322 m	
(d) USBR II Equation	2.750 ft	0.838 m	
(e) Neill Equation	~~3.541 ft~~	~~1.080 m~~	N/A
AVERAGE SCOUR DEPTH	1.165 ft	0.355 m	

COMMENTS: AVERAGE = 2.73

REFERENCES:

(1) Pemberton, E.L. and J. M. Lara, Computing Degradation and Local Scour, Technical Guideline for Bureau of Reclamation, January 1984.

PROJECT NAME: Pima Freeway East Basin
PROJECT NO: 28900082
ANALYSIS BY: Stantech Consulting
DATE: 11-12-98

LOCAL SCOUR ANALYSIS
Types A and B - Natural Channel for Restriction and Bends and Bankline Structures

APPLICATIONS INCLUDE:

Type A	Type B
(a) Siphon Crossing	(e) Abutments to Bridge/Siphon Crossings
(b) Buried Pipeline	(f) Bank Slope Protection (Riprap)
(c) Nat'l Bank Stability	(g) Spur Dikes, Groins, etc.
(d) One-Span Bridge Waterway	(h) Pumping Plants
	(i) Canal Headworks

HYDRAULIC DATA

Discharge (cfs):	1802
Mean Depth (ft):	15.00
Mean Velocity (fps):	2.07
Unit Discharge (cfs/ft):	31.07
Threshold Velocity (fps):	4.06

SEDIMENT DATA

Material Grain Size, D50 (mm): 1.002

REACH INFORMATION

Straight Reach

RESULTS OF ANALYSIS

Methods	Scour Depth	
(a) USBR I Equation	NOT APPLICABLE	
(b) Lacey Equation	1.184 ft	0.361 m
(c) Blench Equation	4.865 ft	1.483 m
(d) USBR II Equation	3.750 ft	1.143 m
(e) Neill Equation	~~7.352 ft~~	~~2.241 m~~ N/A
AVERAGE SCOUR DEPTH	0.612 ft	0.186 m

COMMENTS: AVERAGE = 3.27

REFERENCES:

(1) Pemberton, E.L. and J. M. Lara, Computing Degradation and Local Scour, Technical Guideline for Bureau of Reclamation, January 1984.

PROJECT NAME: Pima Freeway East Basin
PROJECT NO: 28900082
ANALYSIS BY: Stantech Consulting
DATE: 11-12-98

LOCAL SCOUR ANALYSIS
Types A and B - Natural Channel for Restriction and Bends and Bankline Structures

APPLICATIONS INCLUDE:

Type A	Type B
(a) Siphon Crossing	(e) Abutments to Bridge/Siphon Crossings
(b) Buried Pipeline	(f) Bank Slope Protection (Riprap)
(c) Nat'l Bank Stability	(g) Spur Dikes, Groins, etc.
(d) One-Span Bridge Waterway	(h) Pumping Plants
	(i) Canal Headworks

HYDRAULIC DATA

Discharge (cfs):	2001
Mean Depth (ft):	18.00
Mean Velocity (fps):	1.91
Unit Discharge (cfs/ft):	34.30
Threshold Velocity (fps):	4.48

SEDIMENT DATA

Material Grain Size, D50 (mm): 1.002

REACH INFORMATION

Straight Reach

RESULTS OF ANALYSIS

Methods	Scour Depth		
(a) USBR I Equation	NOT APPLICABLE		
(b) Lacey Equation	1.226 ft	0.374 m	
(c) Blench Equation	5.197 ft	1.584 m	
(d) USBR II Equation	4.500 ft	1.372 m	
(e) Neill Equation	~~-10.326 ft~~	~~-3.148 m~~	N/A
AVERAGE SCOUR DEPTH	0.149 ft	0.046 m	

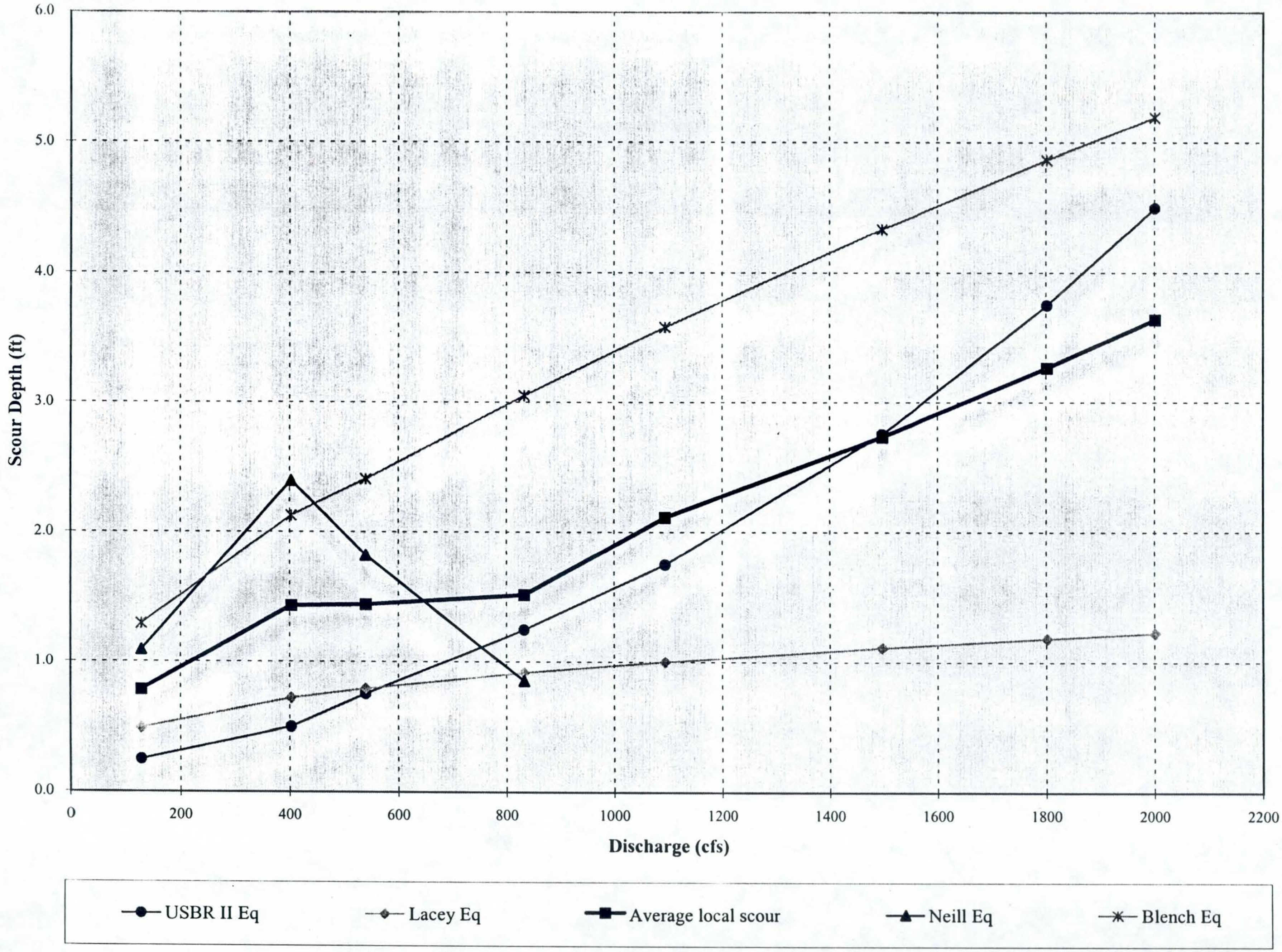
COMMENTS:
AVERAGE = 3.64

REFERENCES:
(1) Pemberton, E.L. and J. M. Lara, Computing Degradation and Local Scour, Technical Guideline for Bureau of Reclamation, January 1984.

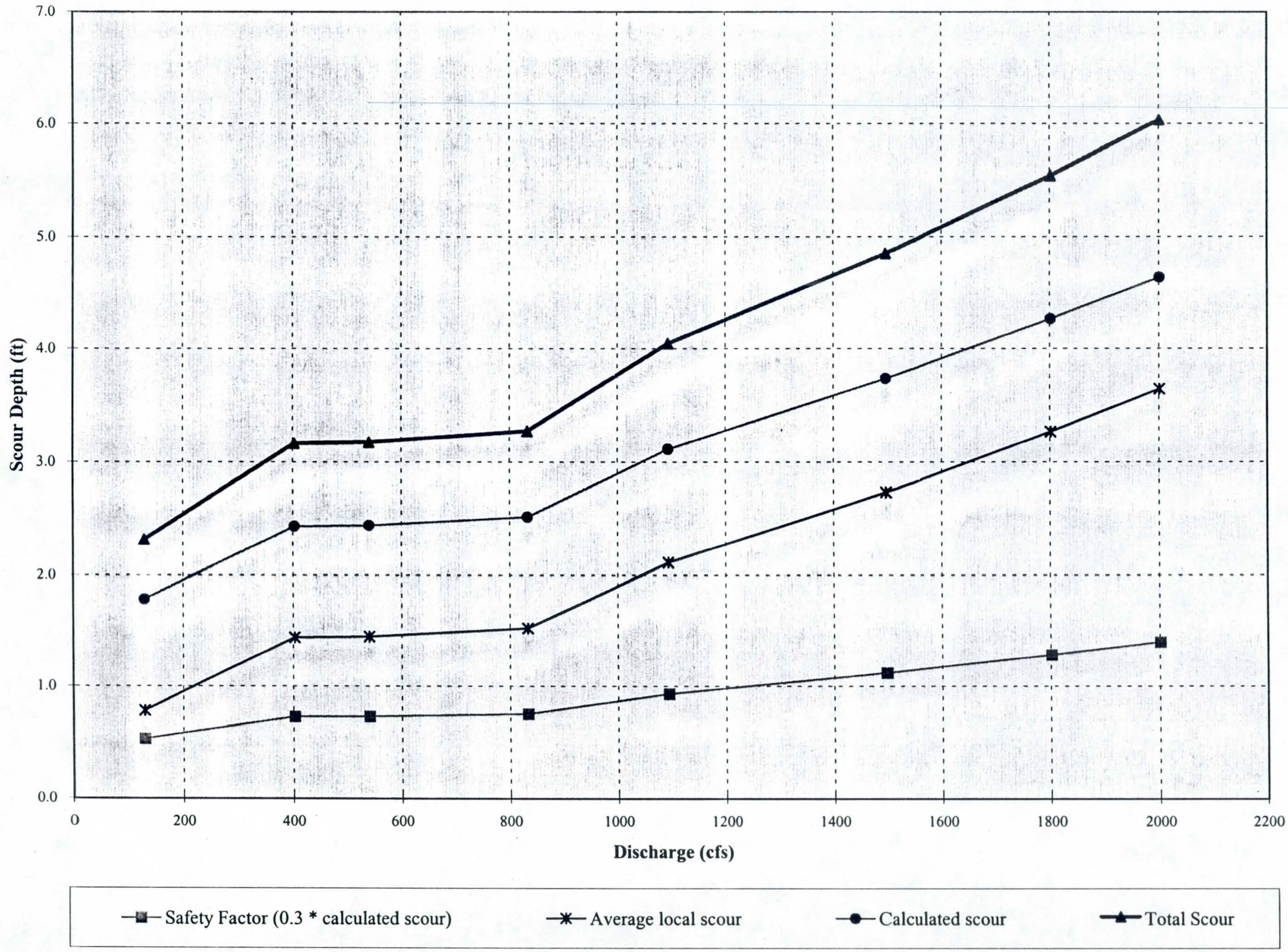
Outer Loop Basin Scour Calculations				
Summary of input data and scour components				
11/12/98				
Input Data				
Flow Rate (cfs)	128	403	540	833
Flow Depth (ft)	1	2	3	5
Flow Velocity (fps)	4.27	4.48	3.6	3.09
Unit Discharge (cfs/ft)	4.27	8.96	10.8	15.43
Threshold Velocity (fps)	2.04	2.04	2.24	2.64
D50 grain size mm (mm)	1.002	1.002	1.002	1.002
Unit Weight of soil (pcf)	165.00	165.00	165.00	165.00
Channel slope	0.001	0.001	0.001	0.001
Mannings n	0.022	0.022	0.022	0.022
Side slope	1.5	1.5	1.5	1.5
Kinematic Viscosity	0.0000105	0.0000105	0.0000105	0.0000105
Local Scour				
USBR I Eq	N/A	N/A	N/A	N/A
Lacey Eq	0.4900	0.7180	0.7920	0.9150
Blench Eq	1.2960	2.1240	2.4050	3.0510
USBR II Eq	0.2500	0.5000	0.7500	1.2500
Neill Eq	1.0930	2.3920	1.8210	0.8520
Average local scour	0.7823	1.4335	1.4420	1.5170
Anti-Dune depth				
Kennedy Eq	0.0000	0.0000	0.0000	0.0000
Small watercourse low flow incisement	1.0000	1.0000	1.0000	1.0000
Calculated scour	1.7823	2.4335	2.4420	2.5170
Safety Factor (0.3 * calculated scour)	0.5347	0.7301	0.7326	0.7551
Total Scour	2.3169	3.1636	3.1746	3.2721

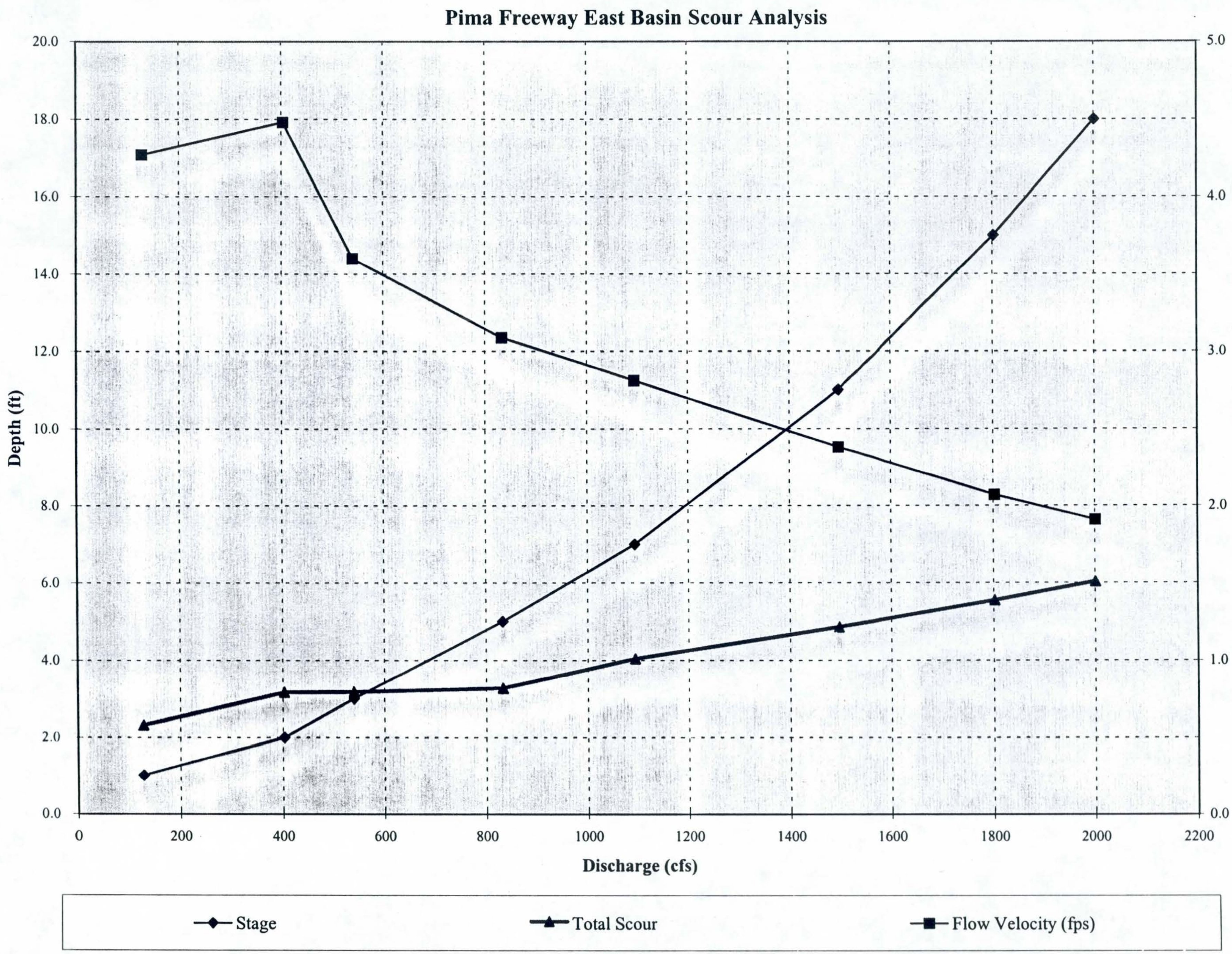
Outer Loop Basin Scour Calculations				
Summary of input data and scour components				
11/12/98				
Input Data				
Flow Rate (cfs)	1094	1497	1802	2001
Flow Depth (ft)	7	11	15	18
Flow Velocity (fps)	2.81	2.38	2.07	1.91
Unit Discharge (cfs/ft)	19.64	26.14	31.07	34.3
Threshold Velocity (fps)	2.93	3.51	4.06	4.48
D50 grain size mm (mm)	1.002	1.002	1.002	1.002
Unit Weight of soil (pcf)	165.00	165.00	165.00	165.00
Channel slope	0.001	0.001	0.001	0.001
Mannings n	0.022	0.022	0.022	0.022
Side slope	1.5	1.5	1.5	1.5
Kinematic Viscosity	0.0000105	0.0000105	0.0000105	0.0000105
Local Scour				
USBR I Eq	N/A	N/A	N/A	N/A
Lacey Eq	1.0020	1.1130	1.1840	1.2260
Blench Eq	3.5830	4.3360	4.8650	5.1970
USBR II Eq	1.7500	2.7500	3.7500	4.5000
Neill Eq	N/A	N/A	N/A	N/A
Average local scour	2.1117	2.7330	3.2663	3.6410
Anti-Dune depth				
Kennedy Eq	0.0000	0.0000	0.0000	0.0000
Small watercourse low flow incisement	1.0000	1.0000	1.0000	1.0000
Calculated scour	3.1117	3.7330	4.2663	4.6410
Safety Factor (0.3 * calculated scour)	0.9335	1.1199	1.2799	1.3923
Total Scour	4.0452	4.8529	5.5462	6.0333

Pima Freeway East Basin Local Scour Analysis



Pima Freeway East Basin Total Scour Analysis




Pima Freeway East Basin Scour Analysis
Depth (ft)
Velocity (fps)
Discharge (cfs)
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
1.0
2.0
3.0
4.0
5.0
0
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
Stage
Total Scour
Flow Velocity (fps)

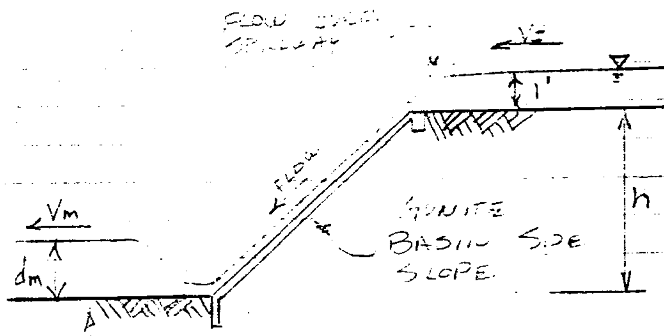
Stantech Consulting

Project: PIMA FREEWAY EAST BASIN   Project Number: 28900082

Notes: SCOUR ANALYSIS OF SPILLWAYS   Scale:

Page 1 of 3 Page(s)

Computed By: [illegible]   Date: [illegible] 98   Checked By: Date:



ASSUMPTIONS

ANALYZE THE SPILLWAY FOR 1 FOOT WIDE (UNIT WIDTH)

SPILLWAY DESIGN MAX FLOW IS 2.5 CFS/FT HOWEVER, SPILLWAY MAX CAPACITY IS 5.0 CFS/FT. SCOUR SHOULD BE ANALYZED FOR BOTH 2.5 & 5 CFS/FT

NEAR [illegible] STA

TYPICAL SECTION THROUGH SPILLWAY

THE VARIOUS SCOUR METHODS REQUIRE THE FOLLOWING INPUT PARAMETERS:

HEAD DIFFERENCE - ASSUME 30 FT

ENERGY DIFFERENCE - ASSUME UPSTREAM VELOCITY (V1) IS 5 FPS $V^2/2g$ = 0.388 FT SO THE ENERGY DIFFERENCE IS 30.39 FT

MEAN DEPTH (dm) IS ASSUMED AT [illegible] FT

MEAN VELOCITY (Vm) IS ASSUMED AT 10 FPS

UNIT DISCHARGE IS THEREFORE 2.5 OR 5.0 CFS

A SAFETY FACTOR OF 1.3 SHOULD BE USED

ARIZONA
7776 Pointe Parkway West, Suite 290
Phoenix, AZ 85044
Voice (602) 438-2200
Fax (602) 431-9562

NEVADA
1100 Grier Drive
Las Vegas, NV 89119
(702) 361-9050
Fax (702) 361-0659

950 Industrial Way
Sparks, NV 89431
(702) 358-6931
Fax (702) 358-6954

PROJECT NAME: Pima Freeway East Basin
PROJECT NO: 28900082
ANALYSIS BY: Stantec Consulting
DATE: 11-5-98

LOCAL SCOUR ANALYSIS
Type D - Hydraulic Structures Across Channel

APPLICATIONS INCLUDE:

(a) Dams and Diversion Dams
(b) Erosion Controls
(c) Rock Cascade Drops
(d) Gabion Controls
(e) Concrete Drops

HYDRAULIC DATA

Discharge (cfs):	2.5
Mean Depth (ft):	1
Mean Velocity (fps):	10
Unit Discharge (cfs/ft):	2.5
Head Difference (ft):	20
Energy Difference (ft):	20.39

SEDIMENT DATA

Material Grain Size, D50 (mm):	.96
D85 (mm):	64
D90 (mm):	72

STRUCTURAL DATA

Dams/Diversion Dams

RESULTS OF ANALYSIS

METHOD	SCOUR DEPTH	
(a) Schoklitsch Equation	1.46 ft	.445 m
(b) Veronese Equation	3.267 ft	.996 m
(c) Zimmerman\Maniak Equation	~~.1 ft = .03 m~~ N/A	

COMMENTS:

AVERAGE IS 2.36' X 1.3 = 3.07'
MAXIMUM IS 3.267 X 1.3 = 4.25'

REFERENCES:

(1) Pemberton, E.L. and J. M. Lara, Computing Degradation and Local Scour, Technical Guideline for Bureau of Reclamation, January 1984.

PROJECT NAME: Pima Freeway East Basin
PROJECT NO: 28900082
ANALYSIS BY: Stantec Consulting
DATE: 11-5-98

LOCAL SCOUR ANALYSIS
Type D - Hydraulic Structures Across Channel

APPLICATIONS INCLUDE:

(a) Dams and Diversion Dams
(b) Erosion Controls
(c) Rock Cascade Drops
(d) Gabion Controls
(e) Concrete Drops

HYDRAULIC DATA

Discharge (cfs):	5
Mean Depth (ft):	1
Mean Velocity (fps):	10
Unit Discharge (cfs/ft):	5
Head Difference (ft):	20
Energy Difference (ft):	20.39

SEDIMENT DATA

Material Grain Size, D50 (mm):	.96
D85 (mm):	64
D90 (mm):	72

STRUCTURAL DATA

Dams/Diversion Dams

RESULTS OF ANALYSIS

METHOD	SCOUR DEPTH	
(a) Schoklitsch Equation	2.652 ft	.809 m
(b) Veronese Equation	5.203 ft	1.586 m
(c) Zimmerman\Maniak Equation	~~.034 ft~~	~~.01 m~~ N/A

COMMENTS:

AVERAGE IS 3.928' x 1.3 = 5.11'
MAXIMUM IS 5.203' x 1.3 = 6.76'

REFERENCES:

(1) Pemberton, E.L. and J. M. Lara, Computing Degradation and Local Scour, Technical Guideline for Bureau of Reclamation, January 1984.

Appendix D

STANDARDS MANUAL FOR DRAINAGE DESIGN AND FLOODPLAIN MANAGEMENT IN TUCSON, ARIZONA

PREPARED FOR
CITY OF TUCSON
DEPARTMENT OF TRANSPORTATION,
ENGINEERING DIVISION

PREPARED BY
SIMONS, LI & ASSOCIATES, INC.

DECEMBER, 1989

degradation changes occurring throughout the river, and to establish the new channel configuration for the next time step.

This methodology has been successfully applied to a number of practical engineering problems. It provides a feasible and relatively cost-effective approach to design problems in alluvial rivers.

6.5.3 Dynamic Mathematical Modeling

Dynamic mathematical modeling of water and sediment routing is the next level of sophistication and complexity in determining alluvial-channel changes. It involves unsteady, non-uniform flow routing for determining the hydraulic conditions to be used to calculate sediment transport, aggradation, and degradation.

Unsteady, non-uniform flow routing solves equations governing the motion of water in open channels. These equations are mathematical descriptions of the physical phenomena. The two basic principles for water routing are continuity and momentum. Continuity states that water coming into a reach is either stored in the reach or passes downstream without gaining or losing water.

The momentum principle balances the forces and accelerations acting on flowing water. Generally, the continuity and momentum equations, along with a resistance to flow equation involving Manning's n or Chezy's C, are solved numerically in finite-difference form. The results are the hydraulic variables of velocity, depth, and width for unsteady, non-uniform flow. These are then used to route sediment. Sediment movement is controlled by the shear forces acting on the bed, transport capacity of the flow, and both availability and supply. Equations used in these calculations are described in most sedimentation textbooks. To compute aggradation and degradation, the sediment-continuity equation is used.

While dynamic mathematical modeling can give excellent results, it is very complex. Fortunately, it is not often required to solve many of the more straightforward, practical problems that designers will usually encounter within the Tucson area. In fact, most aggradation and degradation problems can be solved to an acceptable degree of accuracy by the several methods previously described within this chapter of the Manual.

6.6 Depth of Scour

Scour, or lowering of a channel bed (excluding long-term aggradation/ degradation), can be caused by discontinuity in the sediment-transport capacity of the flow during a runoff event (general scour); the formation of anti-dunes in the channel bed during a runoff event; transverse currents within the flow through a bend (bend scour) during a runoff event; local disturbances, such as abutments or bridge piers, during a runoff event; and the formation of a low-flow channel thalweg. The design depth of scour (*excluding* long-term aggradation/degradation, which must be added for toe-down design) is the sum of all these individual scour components, and can be expressed by:

$$Z_{gs} = Y_{max} \left[\frac{0.0685 V_m^{0.8}}{Y_h^{0.4} S_e^{0.3}} - 1 \right] \tag{6.4}$$

Where:

Z_{gs} = General scour depth, in feet;
V_m = Average velocity of flow, in feet per second;
Y_{max} = Maximum depth of flow, in feet;
Y_h = Hydraulic depth of flow, in feet; and,
S_e = Energy slope (or bed slope for uniform-flow conditions), in feet per foot.

NOTE: Should Z_{gs} become negative, assume that the general-scour component is equal to zero (i.e., $Z_{gs} = 0$).

6.6.2 Anti-Dune Trough Depth

Anti-dunes are bed forms, in the shape of dunes, which move in an upstream rather than a downstream direction within the channel; hence the term "anti-dunes." They form as trains of waves that build up from a plane bed and a plane water surface. Anti-dunes can form either during transitional flow, between subcritical and supercritical flow, or during supercritical flow. The wave length is proportional to the velocity of flow. The corresponding surface waves, which are in phase with the anti-dunes, tend to break like surf when the waves reach a height approximately equal to 0.14 times the wave length. A relationship between average channel velocity, V_m, and anti-dune trough depth, Z_a, can therefore be developed (Simons, Li & Associates, 1982). This relationship is:

$$Z_a = \frac{1}{2} (0.14) \frac{2\pi V_m^2}{g} = 0.0137 V_m^2 \tag{6.5}$$

A restriction on the above equation is that the anti-dune trough depth can never exceed one-half the depth of flow. Therefore, if the computed depth of Z_a obtained by using Equation 6.5 exceeds one-half of the depth of flow, the anti-dune trough depth should then be taken as equal to one-half the depth of flow. Figure 6.2 shows a definition sketch for anti-dune trough depth.

6.6.3 Low-Flow Thalweg

A low-flow thalweg is a small channel which forms within the bed of the main channel, and in which low discharges are carried. Low-flow thalwegs form when the width/depth ratio of the main channel is large. Rather than flow in a very wide, shallow state, low flows will develop a low-flow channel thalweg below the average channel bed elevation in order to provide more efficient conveyance of these discharges.

When the ratio of the flow width to the flow depth of a channel is greater than 1.15 times the average velocity of flow for the 100-year discharge, a low-flow thalweg must be included in all scour calculations. When the flow width or flow depth exceeds the top width and bank heights of the channel, use the top width and flow depth at bank-full conditions, instead of the actual flow width and flow depth. Presently, there is no known methodology for predicting low-flow thalweg depth. However, observation of channels in the Tucson area has revealed that low-flow thalwegs are normally one to two feet deep. Therefore, if a low-flow thalweg is predicted to be present, it should be assumed to be at least two feet deep within regional watercourses, and at least one foot deep within all other watercourses, unless field observations dictate otherwise.

6.6.4 Bend Scour

Bend scour normally occurs along the outside of bends, and is caused by spiral, transverse currents which form within the flow as the water moves around the bend. Presently, there is no single procedure which will consistently and accurately predict bend scour over a wide range of hydraulic conditions. However, the following relationship has been developed by Zeller (1981) for estimating bend scour in sand-bed channels based upon the assumption of the maintenance of constant stream power within the channel bend:

$$Z_{bs} = \frac{0.0685 Y_{max} V_m^{0.8}}{Y_h^{0.4} S_e^{0.3}} \left(2.1 \left[\frac{\sin^2(\alpha/2)}{\cos \alpha} \right]^{0.2} - 1 \right) \tag{6.6}$$

Where:

Z_{bs} = Bend-scour component of total scour depth, in feet;
= 0 when $r_c/T \geq 10.0$, or $\alpha \leq 17.8°$
= computed value when $0.5 < r_c/T < 10.0$, or $17.8° < \alpha < 60°$
= computed value at $\alpha = 60°$ when $r_c/T \leq 0.5$, or $\alpha \geq 60°$

V_m = Average velocity of flow immediately upstream of bend, in feet per second;

Y_{max} = *Maximum* depth of flow immediately upstream of bend, in feet;

Y_h = Hydraulic depth of flow immediately upstream of bend, in feet;

S_e = Energy slope immediately upstream of bend (or bed slope for uniform-flow conditions), in feet per foot; and,

α = Angle formed by the projection of the channel centerline from the point of curvature to a point which meets a line tangent to the outer bank of the channel, in degrees (see Figure 6.3).

NOTE: Mathematically, it can be shown that, for a simple circular curve, the following relationship exists between α and the ratio of the centerline radius of curvature, r_c, to channel top width, T.

$$\frac{r_c}{T} = \frac{\cos \alpha}{4 \sin^2(\alpha/2)} \tag{6.7}$$

Where:

r_c	=	Radius of curvature along centerline of channel, in feet; and,
T	=	Channel top width, in feet.

If the bend deviates significantly from a simple circular curve, the curve should be divided into a series of circular curves, and the bend scour computed for each segment should be based upon the angle α applicable to that segment.

Equation 6.6 can be applied to obtain an approximation of the scour depth that can be expected in a bend during a specific water discharge. The impact that other simultaneously occurring phenomena such as sand waves, local scour, long-term degradation, etc., might have upon bend scour is not known for certain, given the present state of the art. Therefore, in order that the maximum scour in a bend not be underestimated, it is recommended that bend scour be considered as an independent channel adjustment that should be added to those adjustments computed for long-term degradation, general scour, and sand-wave troughs.

The longitudinal extent of the bend-scour component is as difficult to quantify as the vertical extent. Rozovskii (1961) developed an expression for predicting the distance from the end of a bend at which the secondary currents will have decayed to a negligible magnitude. This relationship, in a simplified form, can be expressed as:

$$x = \frac{0.6}{n} Y^{1.17} \quad \textbf{(6.8)}$$

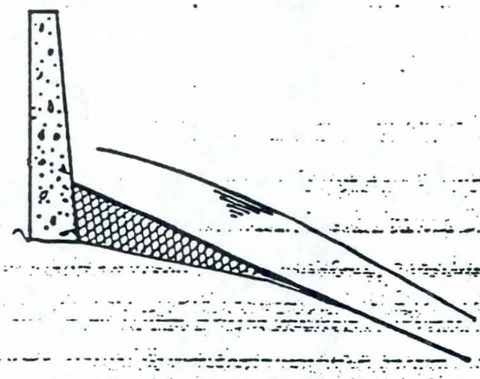
Where:

x	=	Distance from the end of channel curvature (point of tangency, PT) to the downstream point at which secondary currents have dissipated, in feet;
n	=	Manning's roughness coefficient;
g	=	Acceleration due to gravity, 32.2 ft/sec^2; and,
Y	=	Depth of flow (to be conservative, use maximum depth of flow, exclusive of scour, within the bend), in feet.

Equation 6.8 should be used for determining the distance downstream of a curve that secondary currents will continue to be effective in producing bend scour. As a conservative estimate of the longitudinal extent of bend scour, both through and downstream of the curve, it would be advisable to consider bend scour as commencing at the upstream point of curvature (PC), and extending a distance x (computed with Equation 6.8) beyond the downstream point of tangency (PT).

6.6.5 Local Scour

Local scour occurs whenever there is an abrupt change in the direction of flow. Abrupt changes in flow direction can be caused by obstructions to flow, such as bridge piers or abrupt contractions at bridge abutments.



COMPUTING DEGRADATION AND LOCAL SCOUR

TECHNICAL GUIDELINE FOR BUREAU OF RECLAMATION

U.S. Department of the Interior
Bureau of Reclamation

Inch-pound units

$$L_g = \frac{37.05}{0.00112}$$

$$L_g = 33\,100 \text{ ft}$$

Metric units

$$L_g = \frac{1.625\ (6.94)}{0.00112}$$

$$L_g = 10\,100 \text{ m}$$

and for the subreaches:

Inch-pound units

$$L_1 = \frac{22.8}{2\ (0.00112)} = 10\ 200 \text{ ft}$$

$$L_2 = \frac{3\ (22.8)}{8\ (0.00112)} = 7\ 600 \text{ ft}$$

$$L_3 = \frac{3\ (22.8)}{4\ (0.00112)} = 15\ 300 \text{ ft}$$

Metric units

$$L_1 = \frac{6.94}{2\ (0.00112)} = 3\ 100 \text{ m}$$

$$L_2 = \frac{3\ (6.94)}{8\ (0.00112)} = 2\ 300 \text{ m}$$

$$L_3 = \frac{3(6.94)}{4\ (0.00112)} = 4\ 700 \text{ m}$$

CHANNEL SCOUR DURING PEAK FLOODFLOWS

The design of any structure located either along the riverbank and flood plain or across a channel requires a river study to determine the response of the riverbed and banks to large floods. A knowledge of fluvial morphology combined with field experience is important in both the collection of adequate field data and selection of appropriate studies for predicting the erosion potential. In most studies, two processes must be considered, (1) natural channel scour, and (2) scour induced by structures placed by man either in or adjacent to the main river channel.

Natural scour occurs in any moveable bed river but is more severe when associated with restrictions in river widths, caused by morphological channel changes, and influenced by erosive flow patterns resulting from channel alinement such as a bend in a meandering river. Rock outcrops along the bed or banks of a stream can restrict the normal river movement and thus effect any of the above influencing factors. Manmade structures can have varying degrees of influence, usually dependent upon either the restriction placed upon the normal river movement or by turbulence in flow pattern directly related to the structure. Examples of structures that influence river movement would be (1) levees placed to control flood plain flows, thus increasing main channel discharges; (2) spur dikes, groins, riprapped banks, or bridge abutments used to control main channel movement; or (3) pumping plants or headworks to canals placed on a riverbank. Scour of the bed or banks caused by these structures is that created by higher local velocities or excessive turbulence at the strucutre. Structures placed directly in the river consist of (1) piers and piling for either highways or railroad bridges; (2) dams across the river for diversion or storage, (3) grade control structures such as rock cascades, gabion controls or concrete baffled apron drop structures; or (4) occasionally a powerline or tower structure placed in the flood plain but exposed to channel erosion with extreme shifting or movement of a river. All of the above may be subject to higher local velocities, but usually are subject to the more critical local scour caused by turbulence and helicoidal flow patterns.

The prediction of river channel scour due to floods is necessary for the design of many Reclamation structures. These Reclamation guidelines on scour represent a summary of some of the more applicable techniques which are described in greater detail in the reference publications by T. Blench (1969), National Cooperative Highway Research Program Synthesis 5 (1970), C. R. Neill (1973), D. B. Simons and F. Senturk (1977), and S. C. Jain (1981). The paper by S. C. Jain (1981) summarized many of the empirical equations developed for predicting scour of a streambed around a bridge pier. It should be recognized that the many equations are empirically developed from experimental studies. Some are regime-type based on practical conditions and considerable experience and judgment. Because of the complexity of scouring action as related to velocity, turbulence, and bed materials, it is difficult to prescribe a direct procedure. Reclamation practice is to compute scour by several methods and utilize judgment in averaging the results or selection of the most applicable procedures.

The equations for predicting local channel scour usually can be grouped into those applicable to the two previously described processes of either a natural channel scour or scour caused by a manmade structure. A further breakdown of these processes is shown in table 6 where Type A equations are those used for natural river erosion and Types B, C, and D cover various manmade structures.

The importance of experience and judgment in conducting a scour study cannot be overemphasized. It should be recognized that the techniques described in these guidelines merely provide a set of practical tools in guiding the investigator to estimate the amount of scour for use in design. The collection of adequate field data to define channel hydraulics and bed or bank materials to be scoured govern the accuracy of any study. They should be given as much emphasis as the methodology used in the analytical study. Field data are needed to compute water surface profiles for a reach of river in the determination of channel hydraulics for use in a scour study. With no restrictions in channel width, scour is computed from the average channel hydraulics for a reach. If a structure restricts the river width, scour is computed from the channel hydraulics at the restriction. In all cases, scour estimates should be based upon the portion of discharge in and hydraulic characteristics of the main channel only.

Table 6. - Classification of scour equation for various structure designs

Equation type	Scour	Design
A	Natural channel for restrictions and bends	Siphon crossing or any buried pipeline. Stability study of a natural bank. Waterway for one-span bridge.
B	Bankline structures	Abutments to bridge or siphon crossing. Bank slope protection such as riprap, etc. Spur dikes, groins, etc. Pumping plants. Canal headworks.
C	Midchannel structures	Piling for bridge. Piers for flume over river. Powerline footings. Riverbed water intake structures.
D	Hydraulic structures across channel	Dams and diversion dams. Erosion controls. Rock cascade drops, gabion controls, and concrete drops.

Although each scour problem must be analyzed individually, there are some general flow and sediment transport characteristics to be considered in making the judgmental decision on methodology. The general conclusion reached by Lane and Borland (1954) was that floods do not cause a general lowering of streambed, and rivers such as the Rio Grande may scour at the narrow sections but fill up at the wider downstream sections during a major flood. Another general sediment transport characteristic is the influence of a large sediment load on scour which includes the variation of sediment transport associated with a high peak, short duration flood hydrograph. The large sediment concentrations usually of clay and silt size material will occur on the rising stage of the hydrograph up and through the peak of the flood while the falling stage of the flood with deposition of coarser sediments in the bed of the channel may be accompanied by greater scour of the wetted channel banks. Channel scour also occurs when the capacity of streamflow with extreme high velocities in portions of the channel cross section will transport the bed material at a greater rate than replacement materials are supplied. Thus, maximum depth of channel scour during the flood is a function of the channel geometry, obstruction created by a structure (if any), the velocity of flow, turbulence, and size of bed material.

Design Flood

The first step in local scour study for design of a structure is selection of design flood frequency. Reclamation criteria for design of most structures shown in table 6 varies from a design flood estimated on a frequency basis from 50 to 100 years. This pertains to an adequate waterway for passage of the floodflow peak. The scour calculations for these same structures are always made for a 100-year flood peak. The use of the 100-year flood peak for scour is based on variability of channel hydraulics, bed material, and general complexity of the erosive process. The exception in the use of the 100-year flood peak for estimating scour would be the scour hole immediately below a large dam or a major structure where loss of structure could involve lives or represent a catastrophic event. In this case, the scour for use in design should be determined for a flow equal to 50 percent of the structure design flood.

Equation Types A and B (See Table 6)

Natural river channel scour estimates are required in design of a buried pipe, buried canal siphon, or a bankline structure. For most siphon crossings of a river, the cost of burying a siphon will dictate either the selection of a natural narrow reach of river or a restriction in width created by constructing canal bankline levees across a portion of the flood plain. A summary of available methods for computing scour at constrictions is given by Neill (1973). The four methods for estimating general scour at constricted waterways described by Neill (1973) are considered the proper approach for estimating scour for use in either design of a siphon crossing or where general scour is needed of the riverbed for a bankline structure. The four methods supplemented with Reclamation's procedure for application are given below:

Field measurments of scour method. - This method consists of observing or measuring the actual scoured depths either at the river under investigation or a similar type river. The measurements are taken during as high a flow as possible to minimize the influence of extrapolation.

A Reclamation unpublished study by Abbott (1963) analyzed U.S. Geological Survey discharge measurement notes from several streams in the southwestern United States, including the Galisteo Creek at Domingo, New Mexico, and developed an empirical curve enveloping observed scour at the gaging station. This envelope curve for use in siphon design was further supported by observed scour from crest-stage and scour gages on Gallegos, Kutz, Largo, Chaco, and Gobernador Canyons in northwest New Mexico collected during the period from 1963 to 1969. The scour gages consisted of a series of deeply anchored buried flexible tapes across the channel section that were resurveyed after a flood to determine the depth of scour at a specific location. The results of these measurements are shown on figure 8 along with the envelope curve for Galisteo Creek that support scour estimates for wide sandbed (D_{50} varying from 0.5 to 0.7 mm) ephemeral streams in the southwestern United States by the equation.

$$d_s = K\,(q)^{0.24} \qquad (24)$$ USBR I

where:

d_s = Depth of scour below streambed, ft (m)
K = 2.45 inch-pound units (1.32 metric units)
q = Unit water discharge, ft^3/s per ft of width (m^3/s per m of width)

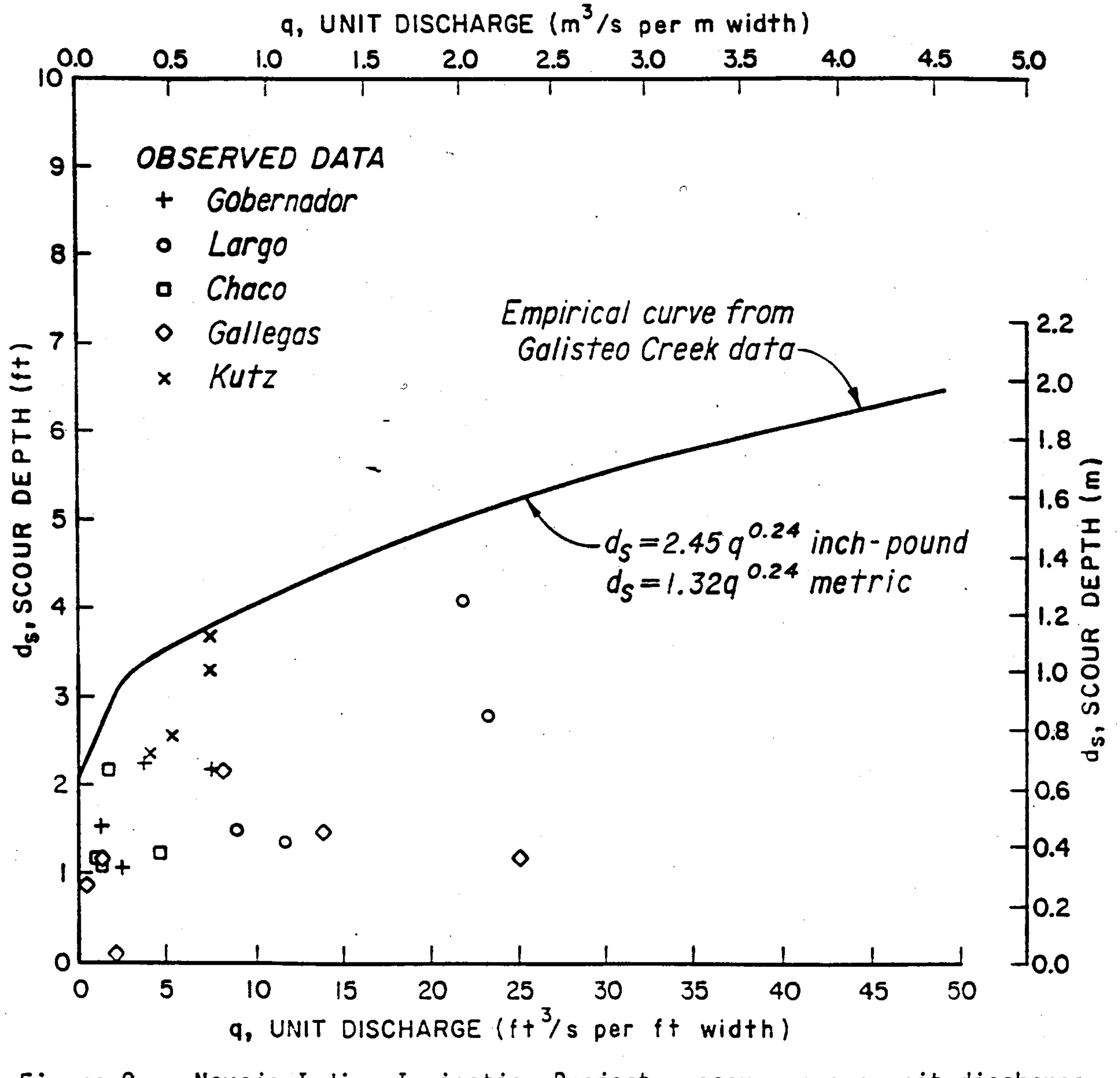


Figure 8. - Navajo Indian Irrigation Project - scour versus unit discharge.

The use of equation 24 except as a check on other methods would be limited to channels similar to those observed on relatively steep slopes ranging from 0.004 to 0.008 ft/ft (m/m). Because of shallow depths of flow and medium to coarse sand size bed material the bedload transport should also be very high.

Regime equations supported by field measurements method. - This approach as suggested by Neill (1973) on recommendations by Blench (1969) involves obtaining field measurements in an incised reach of river from which the bankfull discharge and hydraluics can be determined. From the bankfull hydraulics in the incised reach of river, the flood depths can be computed by:

$$d_f = d_i \left(\frac{q_f}{q_i}\right)^m \tag{25}$$

where:

d_f = Scoured depth below design floodwater level
d_i = Average depth at bankfull discharge in incised reach
q_f = Design flood discharge per unit width
q_i = Bankfull discharge in incised reach per unit width
m = Exponent varying from 0.67 for sand to 0.85 for coarse gravel

This method has been expanded for Reclamation use to include the empirical regime equation by Lacey (1930) and the method of zero bed-sediment transport by Blench (1969) in the form of the Lacey equation:

$$d_m = 0.47 \left(\frac{Q}{f}\right)^{1/3} \tag{26}$$

Lacey

where:

d_m = Mean depth at design discharge, ft (m)
Q = Design discharge, ft^3/s (m^3/s)
f = Lacey's silt factor equals 1.76 $(D_m)^{1/2}$ where D_m equal mean grain size of bed material in millimeters

and the Blench equation for "zero bed factor":

Blench

$$d_{fo} = \frac{q_f^{\ 2/3}}{F_{bo}^{\ 1/3}} \tag{27}$$

where:

d_{fo} = Depth for zero bed sediment transport, ft (m)
q_f = Design flood discharge per unit width, ft^3/s per ft (m^3/s per m)
F_{bo} = Blench's "zero bed factor" in ft/s^2 (m/s^2) from figure 9

The maximum natural channel scour depth for design of any structure placed below the streambed (i.e., siphon) or along the bank of a channel must

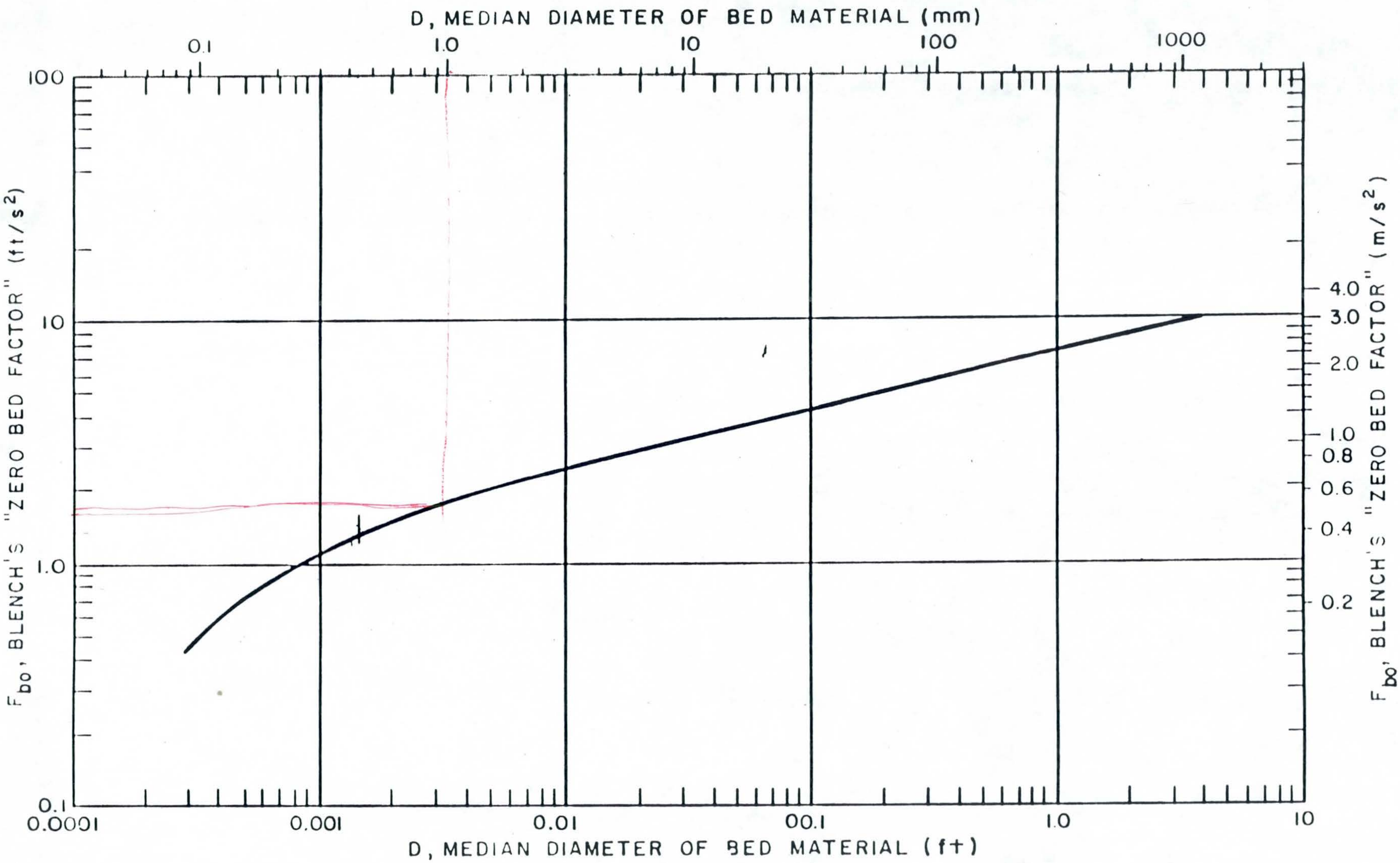


CHART FOR ESTIMATING F_{bo} (AFTER BLENCH)

Figure 9. - Chart for estimating F_{bo} (after Blench, 1969).

consider the probable concentration of floodflows in some portion of the natural channel. Equations 25, 26, or 27 for predicting this maximum depth are to be adjusted by the empirical multiplying factors, Z, shown for formula Types A and B (table 6), in table 7. An illustration of maximum scour depth associated with a flood discharge is shown in a sketch of a natural channel, figure 10. As shown in table 7 and on figure 10, the d_s equals depth of scour below streambed.

$$d_s = Z\, d_f \qquad (28)$$

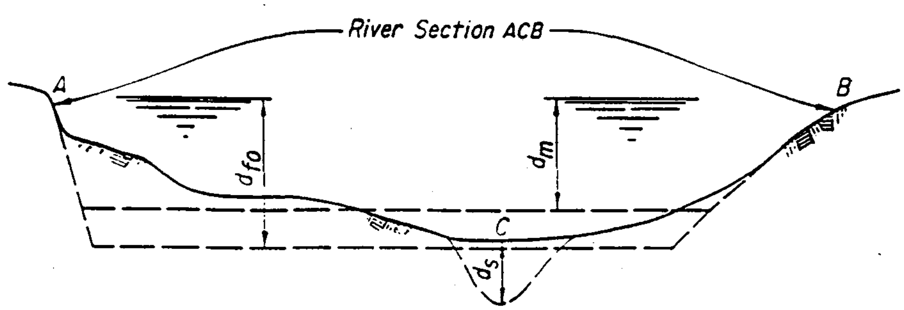
$$d_s = Z\, d_m \qquad (29)$$ USBR II

$$d_s = Z\, d_{fo} \qquad (30)$$

Table 7. - Multiplying factors, Z, for use in scour depths by regime equations

Condition	Value of Z		
	Neill $d_s = Z\, d_f$	Lacey $d_s = Z\, d_m$	Blench $d_s = Z\, d_{fo}$
<u>Equation Types A and B</u>			
Straight reach	0.5	0.25	1/ 0.6
Moderate bend	0.6	0.5	
Severe bend	0.7	0.75	
Right angle bends		1.0	1.25
Vertical rock bank or wall		1.25	
<u>Equation Types C and D</u>			
Nose of piers	1.0		0.5 to 1.0
Nose of guide banks	0.4 to 0.7	1.50 to 1.75	1.0 to 1.75
Small dam or control across river		1.5	0.75 to 1.25

1/ Z value selected by USBR for use on bends in river.



NOTE: $d_{fo} > d_f > d_m$. Point C is low point of natural section.

Figure 10. - Sketch of natural channel scour by regime method.

Although not shown on figure 10, the d_f from Neill's equation 25 is usually less than the d_{fo} from Blench's equation 27 but greater than the d_m from Lacey's equation 26.

The design of a structure under a river channel such as a siphon is based on applying the scoured depth, d_s, as obtained from table 7 to the low point in a surveyed section, as shown by point C on figure 10. This criteria is considered by Reclamation as an adequate safety factor for use in design. In an alluvial streambed, designs should also be based on scour occuring at any location in order to provide for channel shifting with time.

Mean velocity from field measurements method. - This approach represents an adjustment in surveyed channel geometry based on an extrapolated design flow velocity. In Reclamation's application of this method, a series of at least four cross sections are surveyed and backwater computations made for the design discharge by use of Reclamation's Water Surface Profile Computer Program. In addition to the surveyed cross sections observed, water surface elevations at a known or measured discharge are needed to provide a check on Manning's "n" channel roughness coefficient. This procedure allows for any proposed waterway restrictions to be analyzed for channel hydraulic characteristics including mean velocity at the design discharge. The usual Reclamation application of this method is to determine the mean channel depth, d_m, from the computer output data and apply the Z values defined by Lacey in table 7 to compute a scour depth, d_s, by equation 29 where $d_s = Z\ d_m$.

Examples of more unique solutions to scour problems were Reclamation studies on the Colorado River near Parker, Arizona, and Salt River near Granite Reef Diversion Dam, Arizona, where an adjustment in "n" based on particle size along with a Z value from table 7 provided a method of computing bed scour. The selection of a particle size "n" associated with scour in the above two examples was computed from the Strickler (1923) equation for roughness of a channel based on diameter of particles where:

$$K = \frac{C}{D_{90}^{1/6}} \tag{31}$$

$C \approx 26$ from Nikuradse (1933) and "n" = 1/K. The appropriate "n" values for the two rivers based on particle size and engineering judgment were selected as follows:

River	D (mm)	Particle size "n"	Selected "n"
Colorado	0.2	0.01	0.014
Salt	18	0.02	0.02

In the Colorado River study, the existing channel "n" value of 0.022 was adjusted down to 0.014 due to bed material particle size to give a computed water surface at design discharge representative of a scoured channel. With a Z value of 0.5, the scoured section in the form of a trianglular section combined with the accepted "n" of 0.022 provided a close check on the water surface computed without scour. An illustration of this technique is shown in sketch on figure 11a. Another example is shown on figure 11b for a Salt River scour study where the particle size "n" of 0.02 gave a reduced mean depth. Scour was assumed to be in the shape of a triangle where the average depth of scour would be equal the depth at an "n" equal to 0.02 subtracted from depth at an "n" equal to 0.03. (See example problem in subsequent paragraph.)

Competent or limiting velocity control to scour method. - This method assumes that scour will occur in the channel cross section until the mean velocity is reduced to that where little or no movement of bed material is taking place. It gives the maximum limit to scour existing in only the deep scour hole portion of the channel cross section and is similar to the Blench equation 27 for a "zero bed factor."

The empirical curves, figure 12, derived by Neill (1973) for competent velocity with sand or coarser bed material (>0.30 mm) represent a combining of regime criteria, Shields (1936) criterion for material >1.0 mm, and a mean velocity formula relating mean velocity V_m to the shear velocity. The competent velocities for erosion of cohesive materials recommended by Neill (1973) are given in table 8. The scour depth or increase in area of scoured channel section with corresponding increase in depth for competent velocity, V_c, is determined by relationship of mean velocity, V_m, to V_c in the equation:

$$d_s = d_m \left(\frac{V_m}{V_c} - 1\right) \qquad (32)$$ Neill

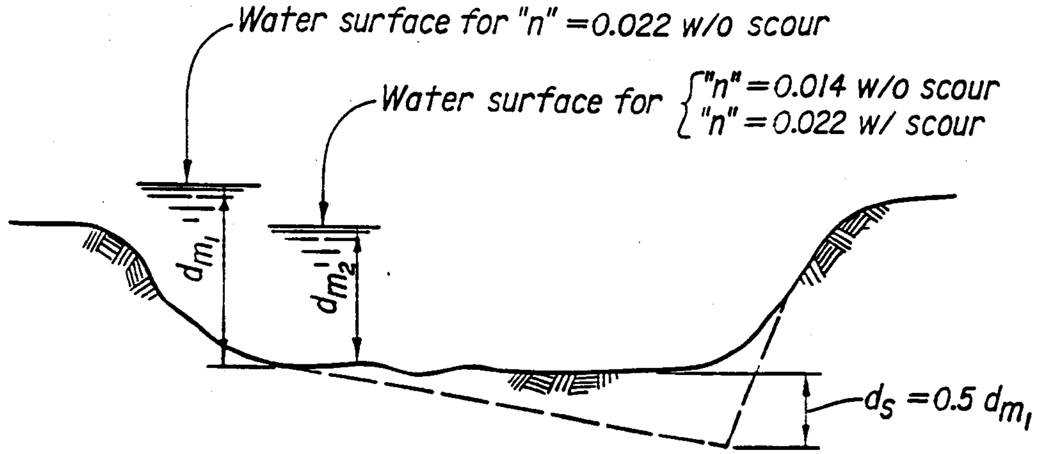
where:

d_s = Scour depth below streambed, ft (m)
d_m = Mean depth, ft (m)

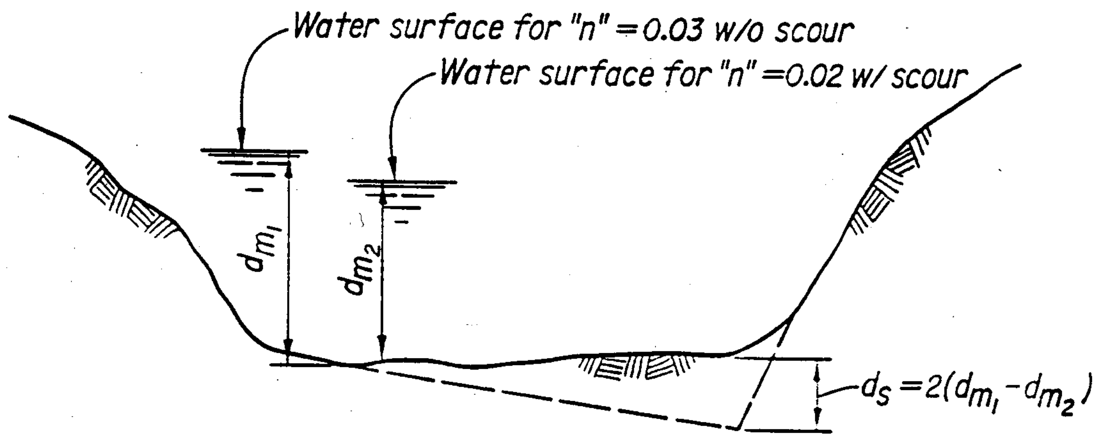
Table 8. - Tentative guide to competent velocities for erosion of cohesive materials* (after Neill, 1973)

Depth of flow		Competent mean velocity					
		Low values - easily erodible material		Average values		High values - resistant material	
ft	m	ft/s	m/s	ft/s	m/s	ft/s	m/s
5	1.5	1.9	0.6	3.4	1.0	5.9	1.8
10	3	2.1	0.65	3.9	1.2	6.6	2.0
20	6	2.3	0.7	4.3	1.3	7.4	2.3
50	15	2.7	0.8	5.0	1.5	8.6	2.6

* Notes: (1) This table is to be regarded as a rough guide only, in the absence of data based on local experience. Account must be taken of the expected condition of the material after exposure to weathering and saturation. (2) It is not considered advisable to relate the suggested low, average, and high values to soil shear strength or other conventional indices, because of the predominating effects of weathering and saturation on the erodibility of many cohesive soils.



a. Colorado River Study



b. Salt River Study

Figure 11. - Sketch of scour from water surface profile computations and reduced "n" for scour.

The use of figure 12 and table 8 recommended by Neill (1973) has had limited application in Reclamation, but appears to be a potential useful technique for many Reclamation studies on scour and armoring of the channel.

Equation Type C (See Table 6)

The principal references for design of midchannel structures for scour such as at bridge piers are National Cooperative Highway Research Program Synthesis 5 (1970), C. R. Neill (1973), Federal Highway Administration, Training and Design Manual (1975), Federal Highway Administration (1980), and S. C. Jain (1981). The numerous empirical relationships for computing scour at bridge piers include one or more of the following hydraulic parameters: pier width and skewness, flow depth, velocity, and size of sediment. The many relations available were further broken down by Jain (1981) to two different approaches: (1) regime, and (2) rational.

The Federal Highway Administration has funded numerous research projects to assist in improving their designs of bridge piers. This research has not resulted in any one recommended procedure. Reclamation's need for scour estimates at midchannel structures is limited. The procedures adopted are to try at least two techniques and apply engineering judgment in selecting an average or most reliable method. The regime approach is to use either equations 26, 27, 28, or 30 and a Z value from table 7. An appropriate Z value to use for piers is 1.0 as found for the railway bridge piers applied to the Lacey equation 29 reported by Central Board of Irrigation and Power (1971).

The rational equation selected for scour at piers is described by Jain (1981) in the form:

$$\frac{d_s}{b} = 1.84 \left(\frac{d}{b}\right)^{0.3} \left(F_c\right)^{0.25} \tag{33}$$

where:

d_s = Depth of scour below streambed, ft (m)
b = Pier size, ft (m)
d = Flow depth, ft (m)
$F_c = V_c/\sqrt{gd}$ = Threshold Froude number
V_c = Threshold velocity, ft/s (m/s) from figure 12
g = Acceleration due to gravity, 32.2 ft/s^2 (9.81 m/s^2)

Equation Type D (See Table 6)

Immediately downstream from any hydraulic structure the riverbed is subject to the erosive action created by the structure. Some type of stilling basin or energy dissipator as described by Reclamation (1977) is provided in the design of such structures to dissipate the energy thereby reducing the erosion potential. There still remains at most structures, below the point where the structure ends and the natural riverbed material begins, a potential for scour. The magnitude of this scour hole will depend on a combination of flow velocity, turbulence, and vortices generated by the structure. Simons and Senturk (1977) describe many of the available equations.

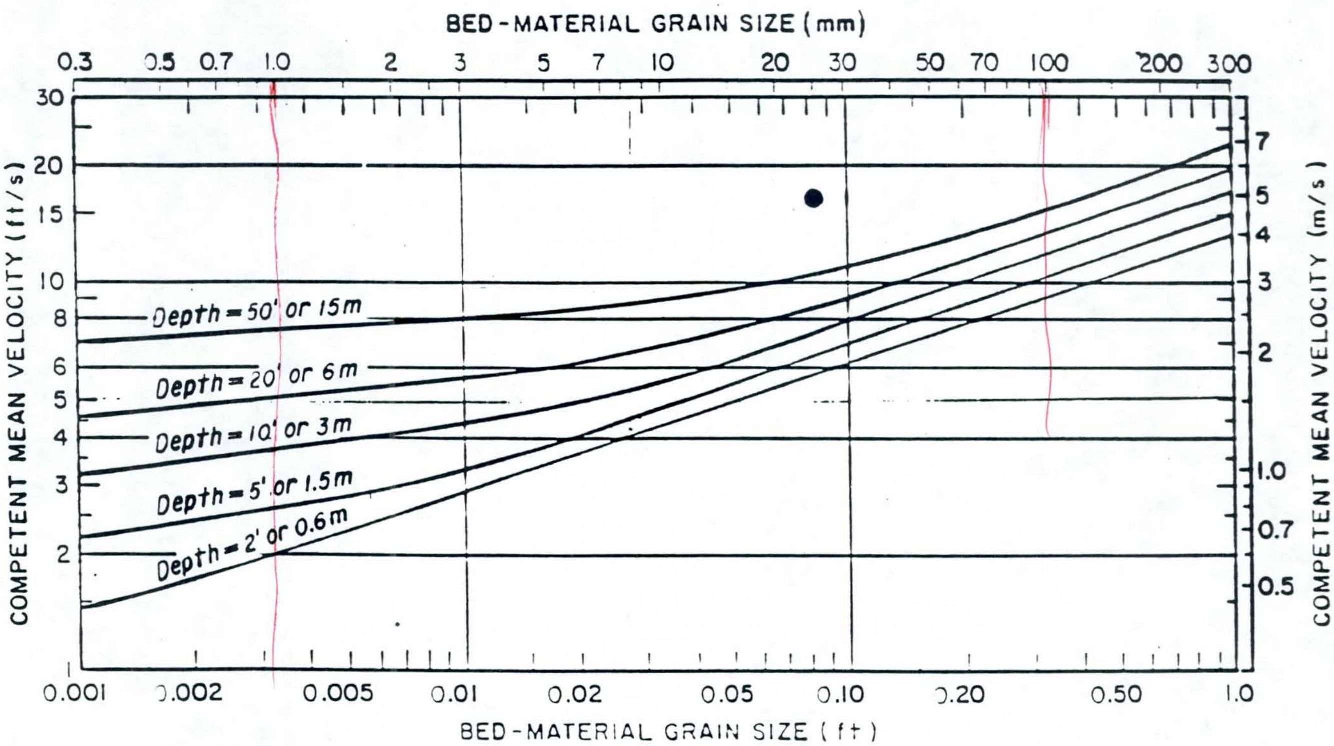


Figure 12. - Suggested competent mean velocities for significant bed movement of cohesionless materials, in terms of grain size and depth of flow (after Neill, 1973).

Methods adopted by Reclamation for computing local scour below a hydraulic structure across the river channel are based on either the regime or rational approach. Scour computations should be made by several methods and engineering judgment used to select the most appropriate. In the regime approach, the Lacey or Blench equations 26, 27, 29, and 30, respectively, with Z values from table 7 are applicable.

The most appropriate empirically developed rational methods for scour below a structure are those by Schoklitsch (1932), Veronese (1937), or Zimmerman and Maniak (1967). Scour computations by Schoklitsch are made by:

$$d_s = \frac{K\,(H)^{0.2}\,q^{0.57}}{D_{90}^{0.32}} - d_m \tag{34}$$

where:

d_s = Depth of scour below streambed, ft (m)
K = 3.15 inch-pound units (K = 4.70 metric units)
H = Vertical distance between the water level upstream and downstream of the structure, ft (m)
q = Design discharge per unit width, ft^3/s per ft (m^3/s per m)
D_{90} = Particle size for which 90 percent is finer than, mm
d_m = Downstream mean water depth, ft (m)

The Veronese (1937) equation for computing the scour hole depth below a low head stilling basin design is as follows:

$$d_s = K\,H_T^{0.225}\,q^{0.54} - d_m \tag{35}$$

where:

d_s = Maximum depth of scour below streambed, ft (m)
K = 1.32 inch-pound units (K = 1.90 metric units)
H_T = The head from upstream reservoir to tailwater level, ft (m)
q = Design discharge per unit width, ft^3/s per ft (m^3/s per m)
d_m = Downstream mean water depth, ft (m)

The Zimmerman and Maniak (1967) equation for local scour below a stilling basin can be calculated by:

$$d_s = K\left(\frac{q^{0.82}}{D_{85}^{0.23}}\right)\left(\frac{d_m}{q^{2/3}}\right)^{0.93} - d_m \tag{36}$$

where:

d_s = Depth of scour below streambed, ft (m)
K = 1.95 inch-pound units (K = 2.89 metric units)
q = Design discharge per unit width, ft^3/s per ft (m^3/s per m)
D_{85} = Particle size for which 85 percent is finer than, mm
d_m = Downstream mean water depth, ft (m)

REFERENCES

ASCE (1975), "Sedimentation Engineering," Manual No. 54.

Abbott, P. O. (1963), "Scour Study for Navajo Indian Irrigation Project," USBR memorandum.

Blench, T. (1969), "Mobile-Bed Fluviology", Edmonton: University of Alberta Press.

Bureau of Reclamation (1955), "Step Method for Computing Total Sediment Load by the Modified Einstein Procedure," Denver, Colorado.

Bureau of Reclamation (1957), "Guide for Computing Water Surface Profiles."

Bureau of Reclamation (1966), "Computation of 'Z's' for Use in the Modified Einstein Procedure, Denver, Colorado.

Bureau of Reclamation (1977), "Design of Small Dams."

Central Board of Irrigation and Power (1971), "River Behavior Control and Training," India.

Colby, B. R., and C. H. Hembree (1955), "Computations of Total Sediment Discharge, Niobrara River near Cody, Nebraska," U.S. Department of the Interior, Geological Survey Water Supply Paper No. 1357.

Corps of Engineers (1977), "Scour and Deposition on Rivers and Reservoirs," HEC-6 Users Manual for a Generalized Computer Program, Exhibit 3.

Federal Highway Administration, (1975) "Training and Design Manual."

Federal Highway Administration (1980), "Scour Around Bridge Piers."

Federal Inter-Agency Sedimentation Project (1963), A study of methods used in measurement and analysis of sediment loads in streams, report No. 14, determination of fluvial sediment discharge, St. Anthony Falls Hydraulic Laboratory, Minneapolis, Minnesota.

Fortier, S., and F. C. Scobey (1926), "Permissible Canal Velocities," Transactions, ASCE, vol. 89.

Glover, R., Q. Florey, and W. W. Lane (1951), "Stable Channel Profiles," Hydraulic Laboratory Report No. Hyd-325, U.S. Department of the Interior, Bureau of Reclamation.

Jain, S. C. (1981), "Maximum Clear-Water Scour Around Circular Piers," Journal of the Hydraulics Division, ASCE.

Kellerhals, R. (1967), "Stable Channels with Gravel-Based Beds," Journal of the Waterways and Harbors Division, ASCE, vol. 93.

Kellerhals, R., and D. I. Bray (1971), "Sampling Procedures for Coarse Fluvial Sediments," Journal Hydraulic Division, ASCE.

Lacey, G. (1930), "Stable Channels in Alluvium," in Proceedings of the Institution of Civil Engineers, vol. 229.

Lane, E. W. (1948), "An Estimate of the Magnitude of the Degradation Which will Result in the Middle Rio Grande Channel from the Construction of the Proposed Sediment Storage Basins and Contraction Works," U.S. Department of the Interior, Bureau of Reclamation.

Lane, E. W. (1952), "Progress Report on Results of Studies on Design of Stable Channels," HYD-352, Bureau of Reclamation.

Lane, E. W. (1955), "The Importance of Fluvial Morphology in Hydraulic Engineering," Proceedings, ASCE, vol. 81.

Lane, E. W. (1955), "Design of Stable Channels", Transactions, ASCE, vol. 120.

Lane, E. W., and W. M. Borland (1954), "River-Bed Scour During Floods," Paper No. 2712, Transactions, ASCE, vol. 119.

Lara, J. M., and E. L. Pemberton (1965), "Initial Unit Weight of Deposited Sediments," Proceedings of Federal Inter-Agency Sedimentation Conference, 1963, Miscellaneous Publication No. 970, USDA, ARS.

Leopold, L. B. (1970), "An Improved Method for Size Distribution of Stream Bed Gravel," Water Resources Research, vol. 6.

Mavis, F. T., and L. M. Laushey (1948), "A Reappraisal of the Beginning of Bed-Movement-Competent Velocity," International Association for Hydraulic Research, Second Meeting, Stockholm.

Mengis, R. C. (1981), "Modeling of a Transient Streambed in the Rio Grande, Cochiti Dam to Near Albuquerque, New Mexico," U.S. Geological Survey Open-File Report 82-106.

Meyer-Peter, E., and R. Muller (1948), "Formulas for Bed-Load Transport," International Association for Hydraulic Structure, Second Meeting, Stockholm.

National Cooperative Highway Research Program Synthesis 5 (1970), "Scour at Bridge Waterways," Highway Research Board.

Neill, C. R. (1973), "Guide to Bridge Hydraulics", Published for Roads and Transportation Association of Canada.

Nikuradse, V.D.J. (1933), Forschungsheft 361, Berlin, "Stromungsgesetze in rauhen Rohren."

Rubey, W. W. (1933), "Settling Velocities of Gravel, Sand, and Silt Particles," American Journal of Science, Fifth Series, vol. XXV, No. 148.

Schoklitsch (1932), "Prevention of Scour and Energy Dissipation," translated from German by E. F. Wilsey, USBR.

Sheppard, John R. (1960), "Investigation of Meyer-Peter, Muller Bedload Formulas," Bureau of Reclamation.

Shields, A.(1936), "Anwendung der Aenlichkeitsmechanik und der Turbulenzforschung anf die Geschiebebewegung, Mitteil, PVWES, Berlin, No. 26.

Shen, H. W. (1981), "Sediment Transport", Chapter in Niobrara River Whooping Crane Habitat Study prepared by ERT, Inc., Fort Collins, Colorado.

Shulits, S. (1935), "The Schoklitsch Bed-Load Formula," Engineering, London, England.

Simons, D. B., and F. Senturk (1977) "Sediment Transport Technology," Water Resources Publication.

Strand, R. I., and E. L. Pemberton (1982), "Reservoir Sedimentation," Technical Guideline for Bureau of Reclamation.

Strickler (1923), Mitteilung No. 16 des Eidg. Amtes fur Wasserwirthschaft, "Beitrage zur Frage der Geschwindigk eits formel."

Wolman, M. G. (1954), "A method of Sampling Coarse River Bed Material," Transactions, AGU, vol. 35.

Veronese, A. (1937), "Erosioni Di Fondo A Valle Di Uno Searico."

Zimmerman and Maniak (1967), "Scour Studies," Journal of Hydraulics Division, ASCE.

Yang, C. T. (1973), "Incipient Motion and Sediment Transport," Journal of the Hydraulics Division, ASCE, vol. 99, No. HY10, Proceeding Paper 10067.

GPO 843-068

Memo

Stantec

To: File

From: George V. Sabol

File: 28900082

Date: 04 December 1998

Re: PR3B – Pima Freeway Basin Spillway Capacity

The Pima Freeway East-Basin will receive overland runoff from a 0.41 square mile area east of Hayden Road (see Plate 1) and the Pima Freeway West-Basin will receive runoff from a 0.90 square mile area west of Hayden Road (see Plate 2). Runoff enters each basin via rundown spillways to be constructed in the north slope of each basin. The drainage area south of the Power Line Corridor producing that runoff is generally undeveloped, and the drainage network is distributary. Therefore, there is uncertainty as to the magnitude of design discharges for each of those spillways. Additionally, future land development may result in grading and drainage plans that would redefine the present drainage pattern, and there is a need to provide for future drainage planning in regard to spillway size and location.

Because of these factors, the rundown spillways in the Pima Freeway East- and West-Basins are located and sized using the following criteria:

- Spillways are generally located where existing washes are incised along the north bank alignment of the basins.
- The contributing drainage area(s) to each spillway are defined based on present land contours.
- The discharge from each minor subbasin is calculated by prorating, as shown in Table 2, the HEC-1 results (Table 1) for each model major subbasin.
- Spillways are sized to account for uncertainty in flow due to the present distributary drainage pattern, and to provide flexibility for future drainage planning.
- Spillways receiving uncontrolled surface runoff are generally sized for 2.5 cfs/ft of spillway length.
- Most of the spillways are sized to accept runoff from multiple subbasins. This provides some "redundancy" in spillway capacity and flexibility for future development to "redirect" some surface runoff to spillways other than those that drain "naturally" to a particular spillway.

The 100-year, 6-hour peak discharges from the project HEC-1 model are listed in Table 1. Those discharges are prorated based on spillway drainage area as show in Table 2. The calculation of spillway size is shown in Table 3. Note that using this approach results in oversizing the cumulative length of spillways. For example, the total peak discharge to the spillways for the East-Basin is 719 cfs (see Table 1). Using 2.5 cfs/ft, this would result in a total spillway length of 288 feet (719/2.5), whereas the total spillway length that is recommended is 385 feet (Table 3).

Reference: PR3B – PIMA FREEWAY BASIN SPILLWAY CAPACITY

Therefore, there is 97 feet more accumulated spillway length in those 11 spillways than is "needed" – assuming that each spillway were to receive its proportional amount of the peak discharge (which cannot be relied upon in the distributary drainage network). This design approach provides reasonable assurance that each spillway is adequately sized for present drainage network uncertainty and provides ample future drainage planning flexibility.

Spillway E9 is designed for 5 cfs/ft, which is the maximum hydraulic capacity for the spillways. The remainder of the spillways are sized for 2.5 cfs/ft. The runoff to E9 is not expected to vary appreciably from the 334 cfs design discharge. That is because much of the drainage area to E9 is already developed and there is little uncertainty about the magnitude of discharge to be directed to that spillway. Additionally, E9 is the location of the Power Line Corridor Channel outfall to the East-Basin and it is expected that when the Power Line Corridor Channel is completed and extended to the E9 location, the 70-foot rundown spillway will be replaced with an appropriately designed and sized spillway for the increased discharge. That 70-foot spillway should provide ample space for an appropriately designed energy dissipater spillway for the increased discharge that will ultimately enter the basin from the Power Line Corridor Channel.

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TABLE 1

Peak discharges (100-year, 6-hour) for the HEC-1 modeling subbasins that drain to the Pima Freeway East- and West-Basins

HEC-1 Model Subbasin (1)	Drainage Area sq. mi. (2)		100-year, 6-hour Peak Discharge Cfs (3)
	East-Basin		
SCN5C	0.1904		240
SCN5D	0.0499		64
SCN6C	0.1388		347
SCN6D	0.0369		68
		Total:	**719**
	West-Basin		
SCNA1	0.1168		142
SCNA2	0.2755		346
SCNA3	0.1230		126
SCNA4	0.1582		204
		Total:	**818**

TABLE 2

Peak discharges (100-year, 6-hour) for each spillway drainage subarea

Drainage Subarea	HEC-1 Model Subbasin	Drainage Area sq. mi.	Prorated Discharge cfs	Drainage Subarea Peak Discharge cfs
(1)	(2)	(3)	(4)	(5)
		East-Basin		
1	SCN5C	0.0603	240(.0603/.1904) =	76
2	SCN5C	0.0214	240(.0214/.1904) =	27
3	SCN5C	0.0456	240(.0456/.1904) =	58
4	SCN5C	0.0213	240(.0213/.1904) =	26
5	SCN5C	0.0086	240(.0086/.1904) =	11
6	SCN5C	0.0142	240(.0142/.1904) =	18
7	SCN5C	0.0050	240(.0050/.1904) =	6
8	SCN5C	0.0140	240(.0140/.1904) =	18
9	SCN5D	0.0072	64(.0072/.0499) =	6
10	SCN5D	0.0055	64(.0055/.0499) =	18
11	SCN5D	0.0372	64(.0372/.0499) =	48
12	SCN6C	0.0087	347(.0087/.1388) =	22
13	SCN6C	0.0047	347(.0047/.1388) =	12
14	SCN6C	0.0125	347(.0125/.1388) =	313
15	SCN6D	0.0050	68(.0500/.0369) =	9
16	SCN6D	0.0319	68(.0319/.0369) =	59
			Total:	**727**
		West-Basin		
17	SCNA1	0.0165	142(0.0165/0.1168) =	20
18	SCNA1	0.0258	142(0.0258/0.1168) =	31
19	SCNA1	0.0369	142(0.0369/0.1168) =	45
20	SCNA1	0.0246	142(0.0246/0.1168) =	30
21	SCNA2	0.0485	346(0.0485/0.2755) =	61
22	SCNA2	0.0044	346(0.0044/0.2755) =	5
23	SCNA2	0.2227	346(0.2227/0.2755) =	280
24	SCNA3	0.0205	126(0.0205/0.1230) =	21
25	SCNA3	0.0321	126(0.0321/0.1230) =	33
26	SCNA3	0.0703	126(0.0703/0.1230) =	72
27	SCNA4	0.0693	204(0.0693/0.1582) =	89
28	SCNA4	0.0856	204(0.0856/0.1582) =	110
29	SCNA4	0.0034	204(0.0034/0.1582) =	4
			Total:	**818**

TABLE 3

Allocation of drainage subareas to each spillway and spillway sizing

Spillway (1)	Potential Contributing Drainage Subareas (2)	Spillway Design Discharge (Q) cfs (3)		Spillway Length[1] ft. (4)
		East-Basin		
E1	1	76		30
E2	2 + 3	85		35
E3	3 + 4 + ½ (5 + 6)	101		40
E4	½ (5 + 6) + 7 + 8	39		20
E5	7 + 8 + 9 + 10	40		20
E6	9 + 10 + 11	64		25
E7	11 + 12 + 13	82		35
E8	12 + 13	34		15
E9	13 + 14 + 15	334		70[2]
E10	½ (14) + 15	166		70
E11	16	59		25
			Total:	**385**
		West-Basin		
W1	W1A	20		10
W2	W2A	31		15
W3	W3A	45		20
W4	W4A	30		15
W5	W5B	5		10
W6	W5A+1/2W6A (not including CP1L1)	201		80
W7	W6A (including CP1L1)	434		175
W8	W6B+1/2W6A (including CP1L1)	315		130
W9	W7A+1/2W6A (not including CP1L1)	173		70
W10	W8A+1/2W9A	117		50
W11	W9A+1/2W8A	125		55
W12	W10A	110		45
W13	W11A+1/2 (East-Basin 1)	42		20
			Total:	**695**

1 = Spillway Length = Q/2.5
2 = Spillway Length = Q/5