CRSS

Conceptual Design Report

Arizona Canal Diversion Channel

Deep Tunnel Study

CRSS, Inc.
in association with
Brierley & Lyman, Inc.
SEA Consulting Engineers, Inc.

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ARIZONA CANAL DIVERSION CHANNEL

CONCEPTUAL DESIGN REPORT

DEEP TUNNEL STUDY

Prepared by

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ACDC DEEP TUNNEL STUDY

CONCEPTUAL DESIGN REPORT

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1.0 INTRODUCTION

1.01 Purpose and Scope

The purpose of this report is to provide written documentation of conceptual design criteria and cost estimates associated with two deep tunnel/siphon alternatives in lieu of open cut construction for the Arizona Canal Diversion Channel (ACDC). These two alternatives will be referred to as the short and the long tunnels, respectively. The short tunnel alternative extends from Sta. 895 to Sta. 945 and the long tunnel extends from Sta. 865 to Sta. 945.

The scope of work performed for preparation of this report included the following:

- o Perform a geotechnical exploration program along the ACDC alignment from Sta. 865 +/- to Sta. 945 +/-.
- o Make preliminary analyses relating to the subsurface conditions and the implications of those conditions to tunnel design and construction.
- o Meet with the Flood Control District (FCD) immediately after completion of the field and laboratory work and the preliminary analysis to present findings and discuss tunnel construction feasibility and alternatives.
- o Provide a cost estimate for the relocation of utilities along the proposed ACDC from Sta. 865 to Sta. 895 +/-.
- o Provide a cost estimate for construction of the ACDC from Sta. 865 +/- to Sta. 895 +/- on the basis of existing Corps of Engineers data.
- o Provide a hydraulic study for the tunnel/siphon concept.
- o Provide a conceptual design and cost estimate for a tunnel that will transmit the flow of the ACDC underground from Sta. 895 +/- to Sta. 945 +/- including inlet and outlet structures (short tunnel).
- o Provide a conceptual design and cost estimate for a tunnel that will transmit the flow of the ACDC underground from Sta. 865 +/- to Sta. 945 +/- including inlet and outlet structures (long tunnel).
- o Meet with the FCD to discuss the conceptual design and cost estimates.
- o Prepare and submit four (4) copies of a Draft Report for the above Scope of Services.

- o Meet with FCD to review the Draft Report.
- o Prepare and submit ten (10) copies of a Final Report for the above Scope of Services.

1.02 Background Information

A flood hazard exists in the Phoenix metropolitan area along and to the south of the Arizona Canal between Cudia City Wash and Skunk Creek. This problem was recognized in the early 1960's which prompted Congress to authorize the New River and Phoenix City Streams Flood Control Project under the Rivers and Harbors Public Works Act of 1965.

The Corps of Engineers recommended construction of the Arizona Canal Diversion Channel (ACDC) just north of the existing Arizona Canal as the best solution to the flooding problem. The ACDC is designed to carry the 100-year flood. The channel will be entrenched for its entire length to allow side inflows to enter over the channel walls. Confluence structures will be required at major tributary locations, pipe inlets and overflow spillways will be used where local ponding occurs, and drop inlet structures will be used along the covered channel. The ACDC was divided into four reaches for design and construction purposes.

Reach Four, the upstream part of the ACDC, will begin at Cudia City Wash and extend 4.2 miles downstream to Dreamy Draw. In this reach the proposed construction will be a concrete rectangular section. The channel will be open except for covered reaches along Stanford Drive just east of 32nd Street and from east of the Arizona Biltmore Resort to 24th Street.

The proposed tunnel project is oriented more or less west to east beginning at Sta. 865 on the west and extending to Sta. 945 on the east as shown in Figure 1. In general, two possible tunnel alignments are proposed for consideration; the first of which begins at Sta. 865 and extends for a total length of 5,000 feet to the same end point (the short tunnel alternative) and the second of which extends for the entire 8,000 feet (the long tunnel alternative). All open cut work for the proposed project must take place north of the Arizona Canal in order for the facilities to be correctly positioned to collect surface runoff. Finally, it is important to note that water in the proposed channel will flow from east to west or down station in the completed project. Hence, the water will enter at Sta. 945 and exit at either Sta. 895 for the short tunnel alternative or Sta. 845 for the long tunnel alternative.

1.03 Organization of Report

This report is divided into three chapters following the introduction:

Chapter 2 - Synopsis of Site Conditions -- This chapter provides a summary of site conditions paraphrased from the detailed description given in a report entitled "ACDC Deep Tunnel Study, Site Conditions". The purpose of this chapter is to provide a concise description of the ground in which the tunnel alternatives would be constructed.

Chapter 3 - Design Considerations -- This chapter provides a description of the hydraulic and tunnel design and cost considerations associated with the two tunnel alternatives.

Chapter 4 - Conclusions -- This chapter provides listings of the conclusions that are possible as a result of the work performed for this study. A final section discusses the limitations of the report relative to final design and construction.

2.0 SYNOPSIS OF SITE CONDITIONS

2.01 General

A subsurface investigation program was conducted in order to describe ground conditions along the proposed deep tunnel alignment. This program included gathering information from previous studies, performing geophysical investigations, geologic mapping, test borings and laboratory testing. The results of the work are described in a report entitled "ACDC Deep Tunnel Study, Site Conditions". The following paragraphs briefly summarize ground conditions at the project site.

2.02 Soil Conditions

Soil overlies bedrock at the project site. This soil is crudely stratified and is described as coarse to fine gravel with little to some sand, occasional silt and clay layers, some cobbles and boulders. The vast majority of the soil has been calcified or cemented with calcium carbonate to some degree. The soil was classified into four categories based on degree of cementation as follows:

C^1 Uncemented -- Loose soil particles exhibiting P-wave velocities of 880 to 1,900 feet per second.

C^2 Lightly Cemented -- Slightly cohesive mass that will stand vertically or support an overhang and exhibiting P-wave velocities of 2,500 to 3,850 feet per second.

C^3 Moderately Cemented -- A cohesive mass similar to concrete with P-wave velocities of 4,500 to 6,500 feet per second.

C^4 Well Cemented -- A cohesive mass that is very strong, that frequently fractures through clasts rather than matrix and with P-wave velocities of 7,000 to 11,000 feet per second.

The uncemented soil varied in depth from 0 to 14 below the ground surface. Below this layer all soil was cemented to varying degrees and was capable of standing unsupported in small openings for a period of time. A generalized subsurface profile is shown in Figure 2.

2.03 Rock Conditions

The deepest soil/bedrock contact along the proposed tunnel alignment occurs at El. 1,145 at Sta. 865 and at El. 1,165 at Sta. 932. Hence, on the basis of available information, bedrock exists along the entire proposed tunnel alignment at a depth no greater than 100 feet below ground surface.

In general, bedrock along the proposed tunnel is composed of thick sequences of schist and quartzite. Beginning at the west end schist is encountered from Sta. 865 to Sta. 877 +/- and again from Sta. 893 +/- to Sta. 920 +/-. Conversely, quartzite is encountered from Sta. 877 +/- to Sta. 893 +/- and again from Sta. 921 +/- to the end of the project at Sta. 945. Hence, for all practical purposes, the proposed long tunnel would be located in approximately one-half schist and one-half quartzite bedrock.

The schist is described as fresh to slightly weathered, moderately fractured to shattered, fine to coarse grained, thin to massive bedded, slightly to strongly foliated, highly siliceous, micaceous or quartz-rich schist with locally interbedded quartz-pebble conglomerate. The schist exhibited P-wave velocities of 8,000 to 12,500 feet per second. Laboratory tests on samples of schist yielded unconfined compressive strengths of 5,496 and 9,131 psi and point load indices of 130, 130.5 and 262 psi.

The quartzite is described as fresh to slightly weathered, blocky to shattered, fine to coarse grained, thick to massive bedded, slightly foliated metaorthoquartzite with locally interbedded phyllite and quartz-mica schist. The quartzite frequently exhibits large extremes in composition and texture over short distances. Shear zones and quartz veins are present with the quartzite and individually may be several feet thick. The quartzite exhibited P-wave velocities of approximately 10,200 to 16,000 feet per second. Laboratory tests on samples of quartzite yielded unconfined compressive strengths of 32,609 and 26,087 psi and point load indices of 319, 258, 256 and 206 psi.

Although not observed in the test borings or surficial exposures in the immediate vicinity of the ACDC alignment, Precambrian greenstone dikes are known to dissect the schist and quartzite deposits. Thin dikes of Tertiary basaltic rock also occur infrequently in the general area. Contacts between the two bedrock units are gradational and much interlayering and lensing occurs.

2.04 <u>Groundwater Conditions</u>

The groundwater table was not encountered in any test boring made for this study. Perched water assumed to be seepage from the Arizona Canal was encountered in test borings DH82-1, DH82-2, and TH80-2 conducted by the Army Corps of Engineers for ACDC Design Memorandum No. 12. The Arizona Canal is unlined in this area and the water was encountered at shallow depth along non-cemented/ cemented alluvial interfaces. Test Boring B-201 conducted by CRSS encountered assumed perched groundwater at a depth of 68 feet where a sharp transition from poorly to well cemented alluvium occurred.

3.0 DESIGN CONSIDERATIONS

3.01 General

The following chapter is divided into two main sections. Section 3.02 discusses open cut construction in light of all available subsurface information and anticipated urban impacts. Proposed alignments are discussed, the cost associated with construction is presented, and an objective evaluation of impacts associated with open cut construction is provided.

Section 3.03 begins with a discussion of the hydraulic considerations relative to getting the water into and out of the tunnel. This is followed by discussions of proposed tunnel alignments and proposed tunnel construction techniques on the basis of observed subsurface conditions. A presentation of costs for the two different proposed methods of tunnel construction is also made.

3.02 Open Cut

Shown in Figure 3 is the proposed alignment for the open cut channel. In general, the proposed channel closely parallels the existing Arizona Canal with some relocation of the canal required near the Squaw Peak Filtration Plant and an open cut required through the parking lot of the Arizona Biltmore Resort. The proposed channel is 36 ft wide and 24 ft deep. Construction activities along the proposed alignment will extend to depths of at least 24 ft below ground surface along the proposed alignment.

Reference to the Subsurface Profile shown in Figure 2, reveals that the depth of excavation proposed above will extend to well through all categories of uncemented to well cemented soil, and will actually encounter bedrock at Sta. 985+/- to Sta. 888 +/- and at Sta. 913 +/-to Sta. 917 +/-. The uncemented and lightly cemented soil can be excavated without blasting, however, the moderately cemented material, well cemented material, and bedrock may require blasting to some degree in order to facilitate excavation. Given in Tables 1 and 2 are summaries of cost for both the short and long channel alternatives. The following categories of cost were identified for the open cut work:

Mobilization,
Excavation,
Temporary Support,
Compacted Fill,
Reinforced Concrete,
Miscellaneous,
Contingencies,
Engineering & Design,
Construction Administration, Operational Costs and
Urban Impacts.

The tables are organized to provide a convenient comparison between the short and long channel alternatives and the short and long tunnel alternatives. Given below are comments about cost factors for the proposed open-cut channel alternatives. Discussion of cost factors for the tunnel alternatives is given in the next section of this report.

Mobilization - Mobilization was established as a lump sum estimate of $500,000.00.

Excavation - As stated above, it is anticipated that excavation for the open-cut will involve some conventional soil excavation, and an additional amount of blasting-assisted excavation. On the basis of available subsurface information a breakdown of 120,000 cy of soil and 220,000 cy of blasting-assisted excavation was established. Unit prices of $3.50/cy and $7.50/cy, respectively, were assigned to these quantities, including the costs of excavation, loading, hauling, and disposal. The estimates assume blasting will be allowed. Excavation costs may increase if blasting is prohibited.

Temporary Support - Because of the limited work area a lump sum estimate of $270,000.00 was allowed for temporary shoring of excavation sidewalls.

Compacted Fill - Compacted fill consists of material placed behind the channel walls after the forms are stripped. Material removed from the original excavation should be suitable for this purpose. A unit price of $3.20/cy was used for this item.

Reinforced Concrete - This item incudes all allowances for forming, reinforcing and placing the bottom, sidewalls and top of the proposed channel. Reinforced concrete to cover the channel from Sta. 899+20 to Sta. 945+45 is included.

Miscellaneous - As with mobilization, a lump sum estimate of $500,000.00 was used for this item.

Contingency - Because of the preliminary nature of subsurface information for the proposed facility, a fifteen percent contingency was considered necessary to cover unknown costs.

Engineering & Design and Construction Administration - These items were estimated at $1,500,000.00 for the short and $2,000,000.00 for the long channel alternatives, respectively.

Operational Costs - Irrigation and maintenance of landscaping, cleaning of the channel, and other estimated general maintenance are estimated at $20,000 and $30,000 annually for the short and long channels respectively. Assuming a 10 percent interest in perpetuity, net present values are $250,000 and $302,000 for operational cost of the short and long channel, respectively.

As can be seen on the estimate sheets, a total of five urban impact items were established for the open cut. These are utility relocations, canal relocation, roads and bridges, right-of-way, and third party impacts. Allowances were established for these items on the basis of our own knowledge of construction requirements and from discussions with representatives of both the Flood Control District and the Corps of Engineers.

The "coordination" aspects of design and construction for the open-cut work may be laborious and time consuming, and delays and project cost increases associated with blasting or ripping of material could be substantial.

On the basis of all available information and all assumptions as discussed above, it is estimated that the cost for constructing the proposed facility from Sta. 895 to Sta. 945 as an open-cut will be on the order of $19,106.00, and $30,961,200, respectively.

3.03 <u>Tunneling</u>

3.3.1 General

There are several important facts that can be gleaned from the Subsurface Profile, Figure 2, relative to construction of a tunnel at the subject site. The first is that the construction of the tunnel will be essentially dry. Although a small amount of perched water was observed in the test borings, this water should have essentially no impact on the cost of construction.

The second fact is that bedrock occurs at no more than 100 ft below ground surface and that it is overlain by relatively good soil. From both the tunnel excavation and ground movement points of view, the moderately to well cemented soil will behave as well as the bedrock. Evidence suggests, that the upper portion of the bedrock is also cemented and should behave quite well. In conclusion, it is recommended that the crown of the tunnel be placed close to the soil/rock contact at the locations of deepest bedrock.

An evaluation of all available information suggests that two methods of tunneling are possible at the subject site. The first would be for a single large opening of the required size as shown in Figure 4. This opening would be built in stages by drilling and blasting. Consideration was given to excavating one large opening with a large tunnel boring machine, but the short length of tunnel and the required size (approximately 34.5 ft-diameter) would make this option much more costly than drilling and blasting excavation. The second approach to tunneling would be for two 24-ft-diameter (finished) tunnels as shown in Figure 5 that would be excavated with a tunnel boring machine. Zones of crushed rock must be anticipated within the rock mass at tunnel level and the installation of pattern ground support will be required for either type of tunnel described above. Some use of shotcrete could also be beneficial, especially for the single large tunnel.

Throughout the remainder of this document the above two types of tunnels are referred to as the "single" and the "double" tunnel alternatives, respectively. Additional information about the advantages and disadvantages of the two approaches to tunneling and about the details of excavation and support for each type of tunnel will be provided in Section 3.3.3 of this report.

As a final note, reference is made to Figures 6 and 7 which show the proposed short and long alignments for the tunnels. These alignments were carefully chosen so as to simplify underground construction activities. All curves are broad and simple and as much straight line tunnel is shown as possible. Some easements will be needed to construct the tunnel along the proposed alignments but the easement work is warranted in the interest of minimizing the cost of tunneling.

3.3.2 Hydraulic Considerations

SEA Consultants performed a study of hydraulic considerations and published a report entitled Concept Hydraulic Analysis for Tunnel Study, Biltmore Area, Arizona Canal Diversion Channel" the text of which is included herewith in Appendix A. In general, the hydraulic study was necessary to size the tunnels and determine how to get the water into and out of the tunnels without damage and at minimum cost for operation and maintenance.

The SEA report concludes that a tunnel is possible for transmitting flood waters and that either a single or a double tunnel could be used. If the double tunnel alternative is selected, then it is recommended that the final inside diameter of the tunnel be established at 24 ft. Please refer to the text of the SEA report in Appendix A for additional information.

3.3.3 Tunneling Methods

Four different tunneling scenarios were considered consisting of either a single or a double tunnel and either a short or a long alternative. Graphic representation of the proposed single and double tunnel cross-sections are given in Figures 4 and 5, respectively. Plan view layouts of the proposed short and long tunnel alignments are given in Figures 6 and 7, respectively. Finally, a summary of costs for all four designs are given in Tables 1 and 2. As given in the tables, the following categories of costs were identified for the tunnels:

Mobilization,
Inlet and Outlet Structures,
Excavation,
Temporary Support,
Compacted Backfill,
Reinforced Concrete,
Sediment Basin,
Intermediate Drainage,
Miscellaneous,
Contingencies,
Engineering & Design,
Construction Administration,
Operational Costs, and
Urban Impacts.

Given below are comments about cost factors for the proposed tunnel alternatives.

Mobilization - As with open-cut work, mobilization was established as a lump sum item of $500,000.00. It should be noted that the extra cost to mobilize various types of specialized tunneling equipment is included in the unit cost for excavation.

Inlet and Outlet Structures - Contrary to open cut work, the tunnels will require fairly large structures to direct the water into and out of the tunnel. In order to provide sufficient room for tunneling activities it is highly probable that both of the shafts will be excavated as single, large, rectangular openings at the beginning of construction. Depending on the results of model studies and on other considerations, these structures may be completed either as large boxes or as large circular shafts. Unit price allowances of $80/cy for excavation and $180/cy for reinforced concrete were established for this work.

Excavation - Two types of excavation are possible for the tunnels; drilling and blasting or tunnel boring machine (TBM). For the purpose of this study an average unit price allowance of $65/cy for the drilling and blasting operation was established. For TBM tunnels, the cost of tunneling is almost always expressed as a cost per lineal foot. For comparison purposes, however, it was decided to reduce this number to a cost per cubic yard excavation of $50. Multiplying this unit price by the proposed cross-sectional volume of the single tunnel results in a unit cost per foot of TBM excavation of $983.

Temporary Support - In order to advance either type of tunnel through the ground, it will be necessary to support the ground as the tunnel advances. On the basis of available subsurface information, it is anticipated that the drilled and blasted tunnel would be excavated in stages and supported with rock bolts, mesh and shotcrete as the opening is enlarged. For TBM tunnels, a combination of rock bolts, mesh, and steel straps and/or channels is proposed. Given in the tables are the proposed unit price allowances for various tunnel support elements.

Compacted Fill - A minimal amount of compacted fill will be required for the tunnel options in the vicinity of the shafts. As with open-cut construction, a unit price allowance of $3.20/cy was established for compacted fill assuming that the originally excavated soil can be used for this purpose.

Sediment Basins - The trappings of sediment will be accomplished by slightly enlarging the Cudia City Wash Sediment Basin and constructing a new sediment basin for 32nd Street. A lump sum cost of $30,000 was estimated for this item. Additionally $200,000 was included under Right-of-Way for land acquisition.

Intermediate Drainage - Collection of surface drainage along the tunnel alignment was included for a lump sum cost of $1,600,000. This proposed system would consist of storm drain pipe, catch basins, manholes and an 8 foot diameter drop shaft at Sta. 920+00.

Reinforced Concrete - Upon the completion of excavation, it will be necessary to line the tunnels with reinforced concrete. For the drilled and blasted alternative, the entire concrete invert section of the tunnel would be placed with provision for laying rail support for the forms that would be used to place the sidewall and crown sections of the tunnel. For the TBM tunnel a circular form would be manufactured that would be slipped through the tunnels as the concrete is placed. A unit price allowance of $200/cy for all tunnel linings was established.

Miscellaneous - A lump sum allowance of $750,000 was established for miscellaneous items for the tunnels. This item is larger than the $500,000 item established for the open cut work because of the need to provide chlorination and/or pumping facilities for the tunnel.

Contingency - As with the open cut work a contingency of 15 percent to cover unknown costs was established for the tunneling work.

Engineering & Design and Construction Administration - These items were estimated at $1,500,000 and $2,000,000 for the short and long tunnel alternatives. These numbers are the same as those used for the short and long channel alternatives. In reality, it will cost approximately the same amount of money to design and to monitor construction of any of the proposed methods of approach.

Operational Costs - Either of the proposed tunnel alternatives will require operation and maintenance costs for pumping and chlorination. After each flood event some time would be needed and some out-of-pocket cost required to pump and/or to treat water in the tunnel. Assuming an annual allowance of $65,000 for this item and a cost of money of ten percent in perpetuity, results in a net present value of $650,000 for operational costs for the tunnel option.

As can be seen from the estimate sheets, cost allowances for the urban impacts associated with tunneling are minimal. For the short and long tunnel no money was allowed for utility or canal relocations or for roads and bridges. For the short and long tunnel alternatives, allowances of $200,000 and $700,000 were provided for right-of-way acquisition.

Under third party impacts, classifications of moderate and low were given to the drilled and blasted and TBM alternatives, respectively. Drilling and blasting will result in ground and air-borne vibrations, dust, fumes, and other construction activities that nearby residents will find aggravating. In addition, it may be necessary to work from both ends of the tunnel if the construction schedule is an important consideration for the work. Hence, although the total amount of impact from a drilled and blasted tunnel is less than that associated with open-cut work, it is still far from no impact at all.

For TBM's, the level of third party impact associated with the subject project is minimal. All work will take place from a single shaft and little blasting vibration at the shafts only will be associated with the work. In addition, the ground through which the tunnels will be excavated is dry and stable so that the potential for disturbance to overlying or adjacent structures is minimal.

The total estimated cost for the open cut channel from Sta. 945 to Sta. 895 is $19,106,000. Comparative numbers for the long tunnel alternatives are $30,914,885 and $29,062,235 for the drilled and blasted and TBM methods of tunneling, respectively.

As reported previously, the total estimated cost for the open cut channel from Sta. 945 to Sta. 865 is $30,961,200 Comparative numbers for the long tunnel alternatives are $43,912,000 and $41,199,735 for the drilled and blasted and TBM methods of tunneling, respectively.

4.0 CONCLUSIONS

4.01 Conclusions

Given below are the primary conclusions that can be drawn from work to date on the subject project.

1. It is possible to use a tunnel for diversion of storm-related flows in the vicinity of the Arizona Biltmore Resort.

2. Sufficient work was performed during this and previous studies to layout and to estimate the costs for various methods of approach to construction.

3. Ground conditions in the vicinity of the Arizona Biltmore Resort are acceptable for tunnel construction. Soil at the site is generally cemented and above the water table. Rock is also cemented near the top and fairly competent throughout most of the alignment. Some zones of crushed rock can be anticipated with either proposed method of tunneling.

4. Either a single large tunnel excavated and supported in stages or two 24-ft-diameter (finished) tunnels excavated by tunnel boring machine can be used for the proposed facility.

5. Open cut construction of the proposed facility will cost $19,106,000 for the short alternative and approximately $30,961,200 for the long alternative, with allowances for utility and canal relocations, road and bridge construction, and right of way acquisition.

6. The short tunnel will cost anywhere from $29,062,235 to $30,914,885 depending on the method of excavation of the tunnel. Reduced cost allowances for utility and canal relocations, road and bridge construction, and right-of-way acquisition are included in this estimate. The third party construction impacts for this method of approach would vary from low to moderate for tunnel construction to very high for open cut work.

7. A long tunnel at the proposed site will cost either $43,912,000 for the single drilled and blasted method of approach or $41,199,735 for two TBM tunnels. No allowances in this estimate are necessary for utility or canal relocations, road and bridge construction, or right-of-way acquisition. Third party construction impacts associated with tunneling are classified as moderate to low for the drilled and blasted and TBM methods of approach, respectively.

4.02 Report Limitations

It is pointed out that cost allowances given herein are provided on the basis of the inherent assumptions that contracting requirements for the project will be prudent and reasonable and that bidding activities for the project will be competitive. Unusual construction requirements or a tight construction market at the time of bidding could cause escalation of the estimates provided herein. However, even if the absolute magnitude of estimates is inappropriate for some reason, the relative values of cost should remain fairly constant and should provide a legitimate comparison of the various methods of approach to construction of the diversion facility.

It is also pointed out that all discussion provided herein is based on limited information about final design requirements and, in particular, ground surface conditions. Additional test borings and other forms of subsurface exploration will be required prior to final design and construction of the proposed facility.

RP01775.030

Tables

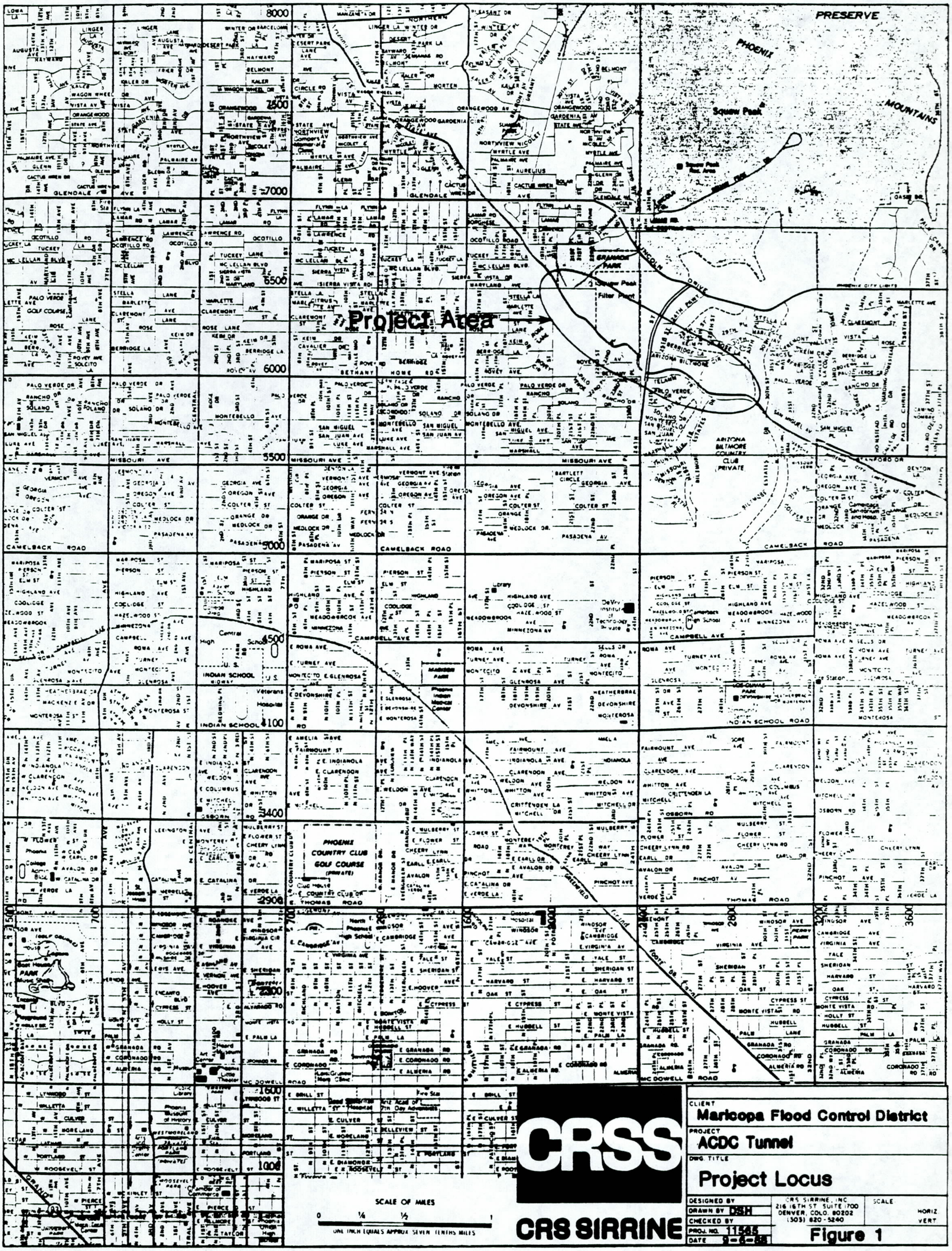
TABLE 1
ACDC PROJECT
MARICOPA COUNTY FLOOD CONTROL DISTRICT
COST ESTIMATE

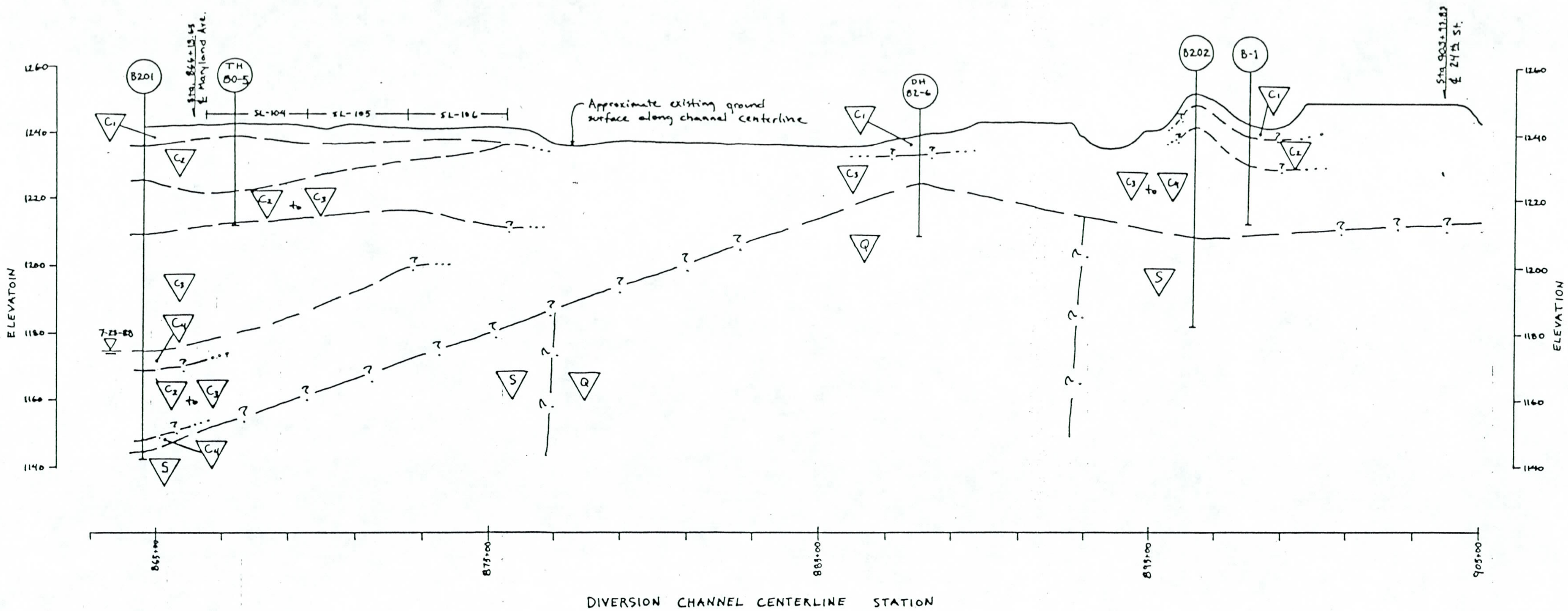
ITEM NO.	DESCRIPTION	SHORT CHANNEL ALTERNATIVE STA. 895+00 to STA. 945+00 COST BASIS	AMOUNT	SHORT TUNNEL ALTERNATIVE SINGLE HORSESHOE SHAPED TUNNEL STA. 895+00 to STA. 945+00 COST BASIS	AMOUNT	SHORT TUNNEL ALTERNATIVE TWIN CIRCULAR BORE TUNNELS STA. 895+00 to STA. 945+00 COST BASIS	AMOUNT
1	MOBILIZATION	LUMP SUM	$500,000	LUMP SUM	$500,000	LUMP SUM	$500,000
2	INLET & OUTLET STRUCTURES:						
	EXCAVATION	N.A		10,000 CY @ $80/CY	$800,000	9,000 CY @ $80/CY	$720,000
	REINFORCED CONCRETE	N.A		2,200 CY @ $180/CY	$396,000	2,000 CY @ $180/CY	$360,000
3	EXCAVATION:						
	COMMON	120,000 CY @ $3.50/CY	$420,000	3,000 CY @ $7.50/CY	$22,500	3,000 CY @ $7.50/CY	$22,500
	BLASTING	220,000 CY @ $7.50/CY	$1,650,000	5,000 LF @ 36 CY/LF @ $65/CY	$11,700,000	10,000 LF @ 20 CY/FY @ $50/CY	$10,000,000
4	TEMPORARY SUPPORT:						
	SHORING	LUMP SUM	$270,000	NA		NA	
	ROCK BOLTS & MESH	NA		5000 FT @ 37 LF/FT @ $7/LF	$1,295,000	10,000 FT @ 10LF/FT @ $7/LF	$700,000
	SHOTCRETE	NA		5,000 FT @ 40 SF/FT @ $5/SF	$1,000,000	NA	
	CHANNELS & STRAPS	NA		NA		10,000 FT @ 15LF/FT @ $12/LF	$1,800,000
5	COMPACTED FILL	125,000 CY @ $3.20/CY	$400,000	2,000 CY @ $3.20/CY	$6,400	2,000 CY @ $3.20/CY	$6,400
6	REINFORCED CONCRETE	40,000 CY @ $180/CY	$7,200,000	5000 LF @ 6 CY/LF @ $200/CY	$6,000,000	10,000 LF @ 3 CY/LF @ $200/CY	$6,000,000
7	SEDIMENT BASIN	NA	$0	LUMP SUM	$30,000	LUMP SUM	$30,000
8	INTERMEDIATE DRAINAGE	NA	$0	LUMP SUM	$1,600,000	LUMP SUM	$1,600,000
9	MISCELLANEOUS	LUMP SUM	$500,000	LUMP SUM	$750,000	LUMP SUM	$750,000
	SUBTOTOAL CONSTRUCTION		$10,940,000		$24,099,900		$22,488,900
10	CONTINGENCIES(15%)		$1,641,000		$3,614,985		$3,373,335
	GRAND TOTAL CONSTRUCTION		$12,581,000		$27,714,885		$25,862,235
11	ENGINEERING & DESIGN		$1,500,000		$1,500,000		$1,500,000
12	CONSTRUCTION ADMINISTRATION		$1,500,000		$1,500,000		$1,500,000
	TOTAL		$15,581,000		$30,714,885		$28,862,235
13	NET PRESENT VALUE - OPERATIONAL COST:		$200,000		$650,000		$650,000
14	URBAN IMPACT:						
	UTILITY RELOCATIONS		$1,200,000		$0		$0
	CANAL RELOCATION		$600,000		$0		$0
	ROADS AND BRIDGES		$1,725,000		$0		$0
	RIGHT OF WAY		$0		$200,000		$200,000
	THIRD PARTY IMPACTS		VERY HIGH		MODERATE		LOW
	GRAND TOTAL		$19,106,000		$30,914,885		$29,062,235

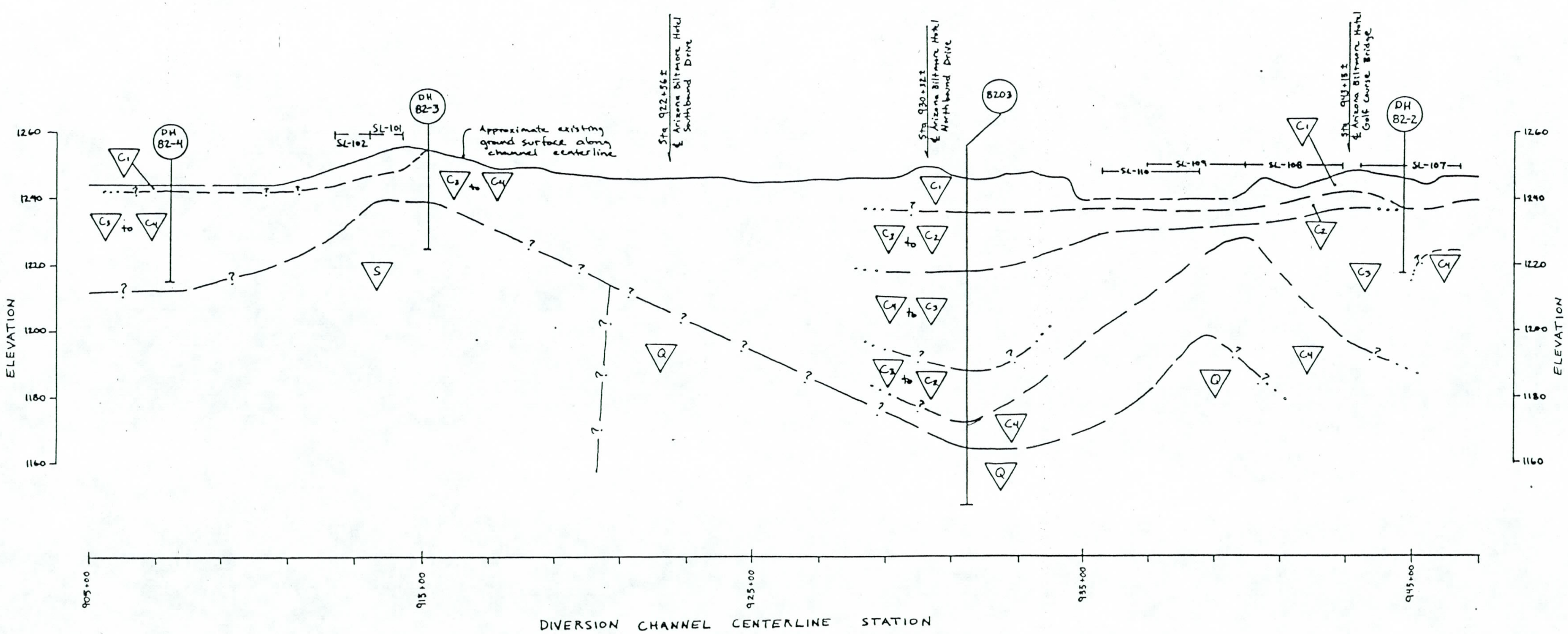
TABLE 2
ACDC PROJECT
MARICOPA COUNTY FLOOD CONTROL DISTRICT
COST ESTIMATE

ITEM NO.	DESCRIPTION	LONG CHANNEL ALTERNATIVE STA. 865+00 to STA. 945+00 COST BASIS	AMOUNT	LONG TUNNEL ALTERNATIVE SINGLE HORSESHOE SHAPED TUNNEL STA. 865+00 to STA. 945+00 COST BASIS	AMOUNT	LONG TUNNEL ALTERNATIVE TWIN CIRCULAR BORE TUNNELS STA. 865+00 to STA. 945+00 COST BASIS	AMOUNT
1	MOBILIZATION	LUMP SUM	$500,000	LUMP SUM	$500,000	LUMP SUM	$500,000
2	INLET & OUTLET STRUCTURES:						
	EXCAVATION	NA		10,000 CY @ $80/CY	$800,000	9,000 CY @ $80/CY	$720,000
	REINFORCED CONCRETE	NA		2,200 CY @ $180/CY	$396,000	2,000 CY @ $180/CY	$360,000
3	EXCAVATION:						
	COMMON	140,000 CY @ $3.50/CY	$490,000	3,000 CY @ $7.50/CY	$22,500	3,000 CY @ $7.50/CY	$22,500
	BLASTING	360,000 CY @ $7.50/CY	$2,700,000	7,500 LF @ 36 CY/FY @ $65/CY	$17,550,000	15,000 LF @ 20 CY/FY @ $50/CY	$15,000,000
4	TEMPORARY SUPPORT:						
	SHORING	LUMP SUM	$270,000	NA		NA	
	ROCK BOLTS & MESH	NA		7,500 FT @ 37 LF/FT @ $7/LF	$1,942,500	15,000 FT @ 10LF/FT @ $7/LF	$1,050,000
	SHOTCRETE	NA		7,500 FT @ 40 SF/FT @ $5/SF	$1,500,000	NA	
	CHANNELS & STRAPS	NA		NA		15,000 FT @ 15LF/FT @ $12/LF	$2,700,000
5	COMPACTED FILL	140,000 CY @ $3.20/CY	$448,000	2,000 CY @ $3.20/CY	$6,400	2,000 CY @ $3.20/CY	$6,400
6	REINFORCED CONCRETE	56,000 CY @ $180/CY	$10,080,000	7500 LF @ 6 CY/LF @ $200/CY	$9,000,000	15,000 LF @ 3 CY/LF @ $200/CY	$9,000,000
7	SEDIMENT BASIN	NA	$0	LUMP SUM	$30,000	LUMP SUM	$30,000
8	INTERMEDIATE DRAINAGE	NA	$0	LUMP SUM	$1,600,000	LUMP SUM	$1,600,000
9	MISCELLANEOUS	LUMP SUM	$500,000	LUMP SUM	$750,000	LUMP SUM	$750,000
	SUBTOTAL CONSTRUCTION		$14,988,000		$34,097,400		$31,738,900
10	CONTINGENCIES(15%)		$2,248,200		$5,114,610		$4,760,835
	GRAND TOTAL CONSTRUCTION		$17,236,200		$39,212,010		$36,499,735
11	ENGINEERING & DESIGN		$2,000,000		$2,000,000		$2,000,000
12	CONSTRUCTION ADMINISTRATION		$2,000,000		$2,000,000		$2,000,000
	TOTAL		$21,236,200		$43,212,010		$40,499,735
13	NET PRESENT VALUE - OPERATIONAL COST:		$302,000		$650,000		$650,000
14	URBAN IMPACT:						
	UTILITY RELOCATIONS		$2,550,000		$0		$0
	CANAL RELOCATION		$1,200,000		$0		$0
	ROADS AND BRIDGES		$2,975,000		$0		$0
	RIGHT OF WAY		$3,000,000		$700,000		$700,000
	THIRD PARTY IMPACTS		VERY HIGH		MODERATE		LOW
	GRAND TOTAL		$30,961,200		$43,912,010		$41,199,735

Figures


PRESERVE
PHOENIX
MOUNTAINS
Squaw Peak
Project Area
Arizona Biltmore Country Club Private
Phoenix Country Club Golf Course (Private)
Glendale
Missouri Ave
Camelback Road
Indian School Road
Thomas Road
McDowell Road
SCALE OF MILES
0 1/4 1/2 1
ONE INCH EQUALS APPROX SEVEN TENTHS MILES
CRSS
CRS SIRRINE
CLIENT
Maricopa Flood Control District
PROJECT
ACDC Tunnel
DWG. TITLE
Project Locus
DESIGNED BY
DRAWN BY DSH
CHECKED BY
PROJ. NO.
DATE
CRS SIRRINE, INC.
216 16TH ST. SUITE 1700
DENVER, COLO. 80202
(303) 820-5240
SCALE
HORIZ.
VERT.
Figure 1





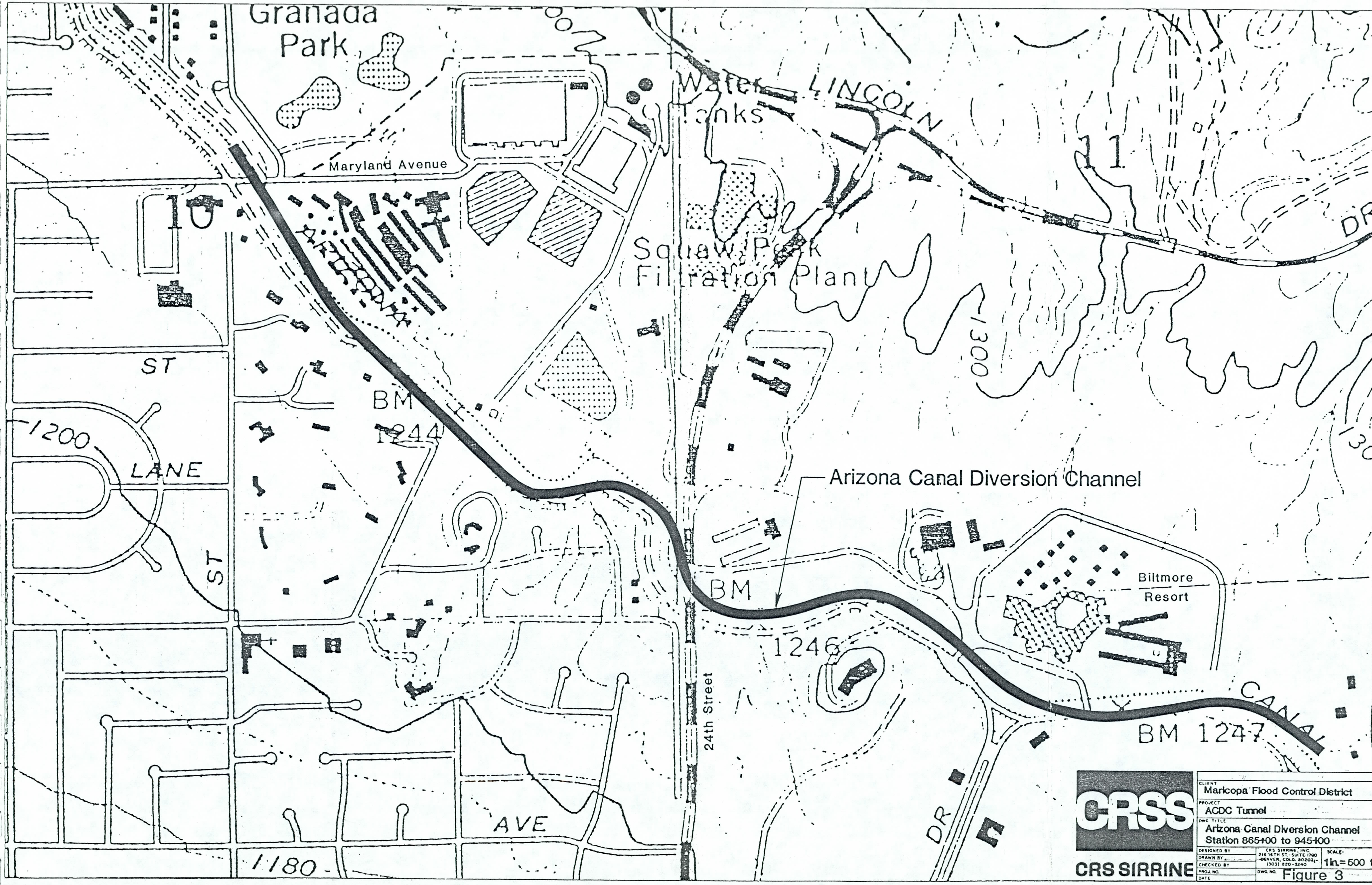
LEGEND

- C_1 Colluvium, non-cemented
- C_2 Colluvium, lightly cemented
- C_3 Colluvium, moderately cemented w/ occasional lightly or well cemented zones
- C_4 Colluvium, well cemented, "rock-like"
- S Schist, poorly to well foliated
- Q Quartzite, massive to slightly foliated
- Contact, approximate or gradational
- Contact, inferred
- Contact, uncertain
- Groundwater encountered

NOTES:
1. Horizontal to vertical scale exaggeration is 10 to 1.
2. Contacts between schist and quartzite bedrock units are based on outcrops in project vicinity and are very gradational with much interlayering and lensing.
3. Water encountered in test boring B-201 was perched. Possible seepage from Arizona Canal.
4. For more detail on subsurface profile see individual boring logs or seismic profiles.
5. Borings TH 80-5, DH B2-2,3,4+6 conducted by U.S. Army Corps of Engineers. Test borings B201, B203 and B205 conducted by CRSS July 1988, boring B-1 conducted by CRSS February 1987. Seismic Lines SL-101 through SL-110 conducted by CRSS June 1988.

CRSS
CRS SIRRINE

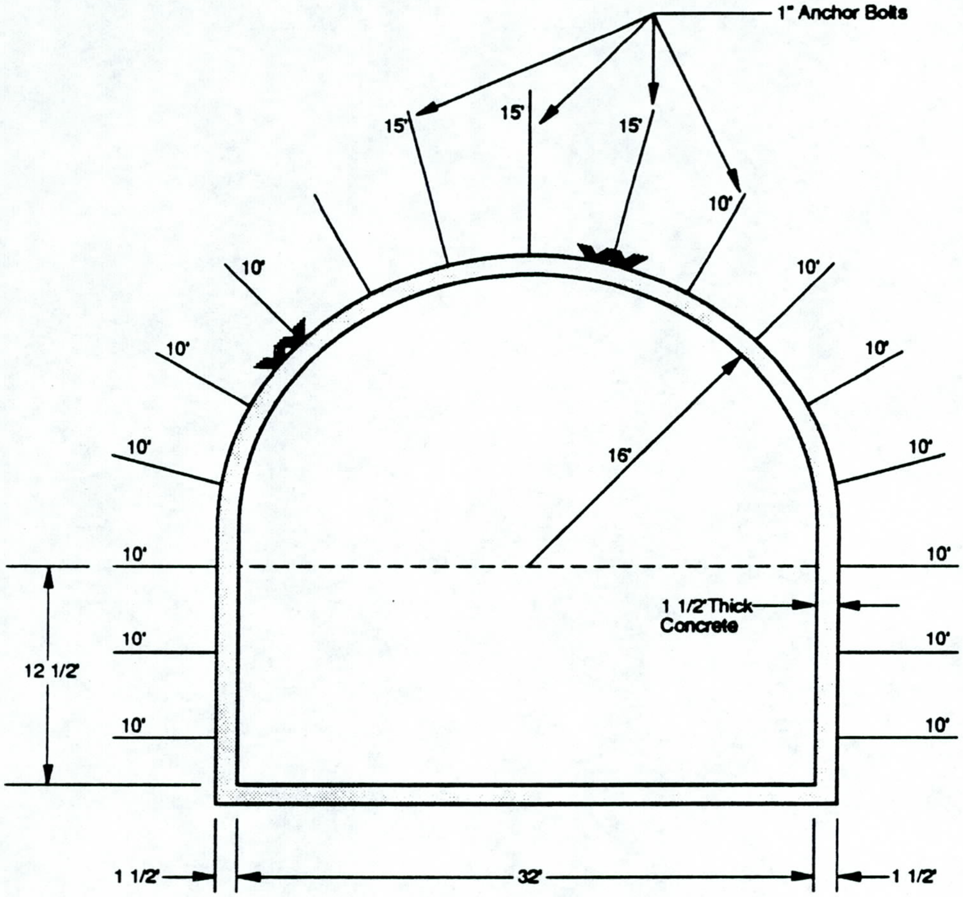
CLIENT	Maricopa County Flood Control District
PROJECT	Arizona Canal Diversion Channel Tunnel
DWG. TITLE	Subsurface Profile
DESIGNED BY	DSH
DRAWN BY	DSH
CHECKED BY	
PROJ. NO.	11563.24
DATE	8-3-88
	CRS SIRRINE, INC. 216 16TH ST. - SUITE 1700 DENVER, COLO. 80202 (303) 820-5240
Original Scale	1" = 200' HORIZ. 1" = 20' VERT
DWG. NO.	Figure. 2



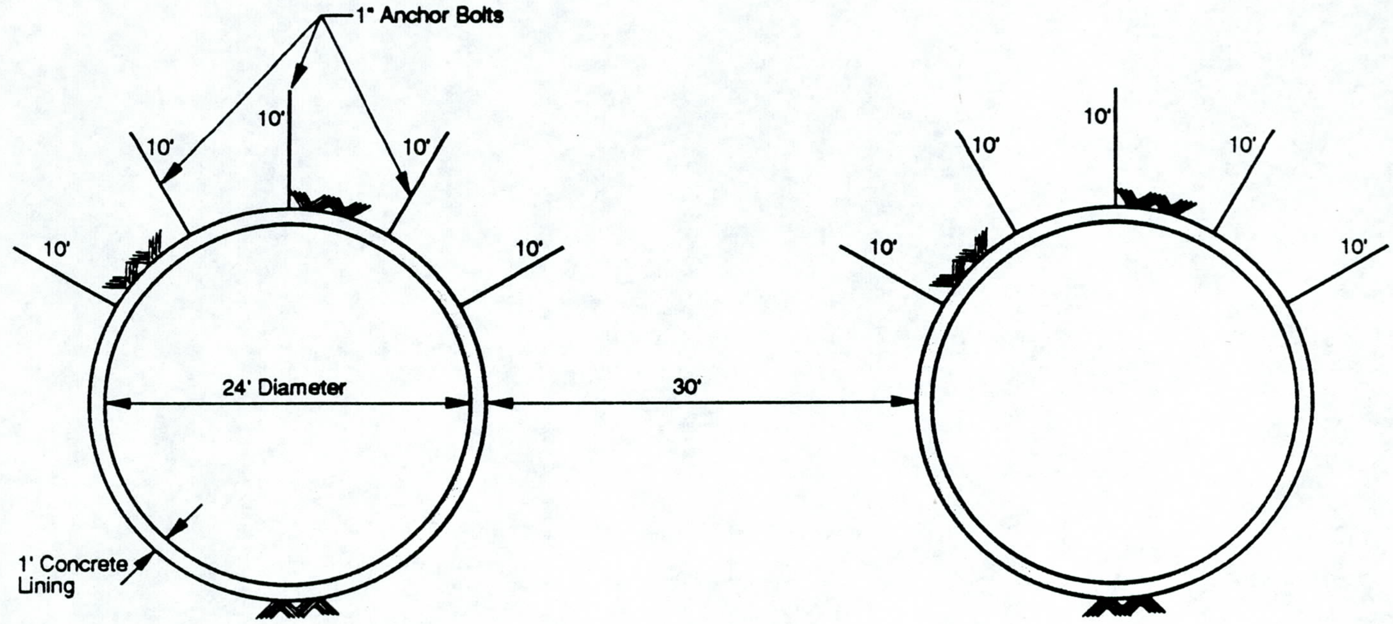
CRSS

CRS SIRRINE

CLIENT	Maricopa Flood Control District
PROJECT	ACDC Tunnel
DWG. TITLE	Arizona Canal Diversion Channel Station 865+00 to 945+00
DESIGNED BY / DRAWN BY / CHECKED BY / PROJ. NO. / DATE	CRS SIRRINE, INC. 216 16TH ST. - SUITE 1700 DENVER, COLO. 80202 (303) 820-5240
SCALE	1 in.= 500 ft.
DWG. NO.	Figure 3

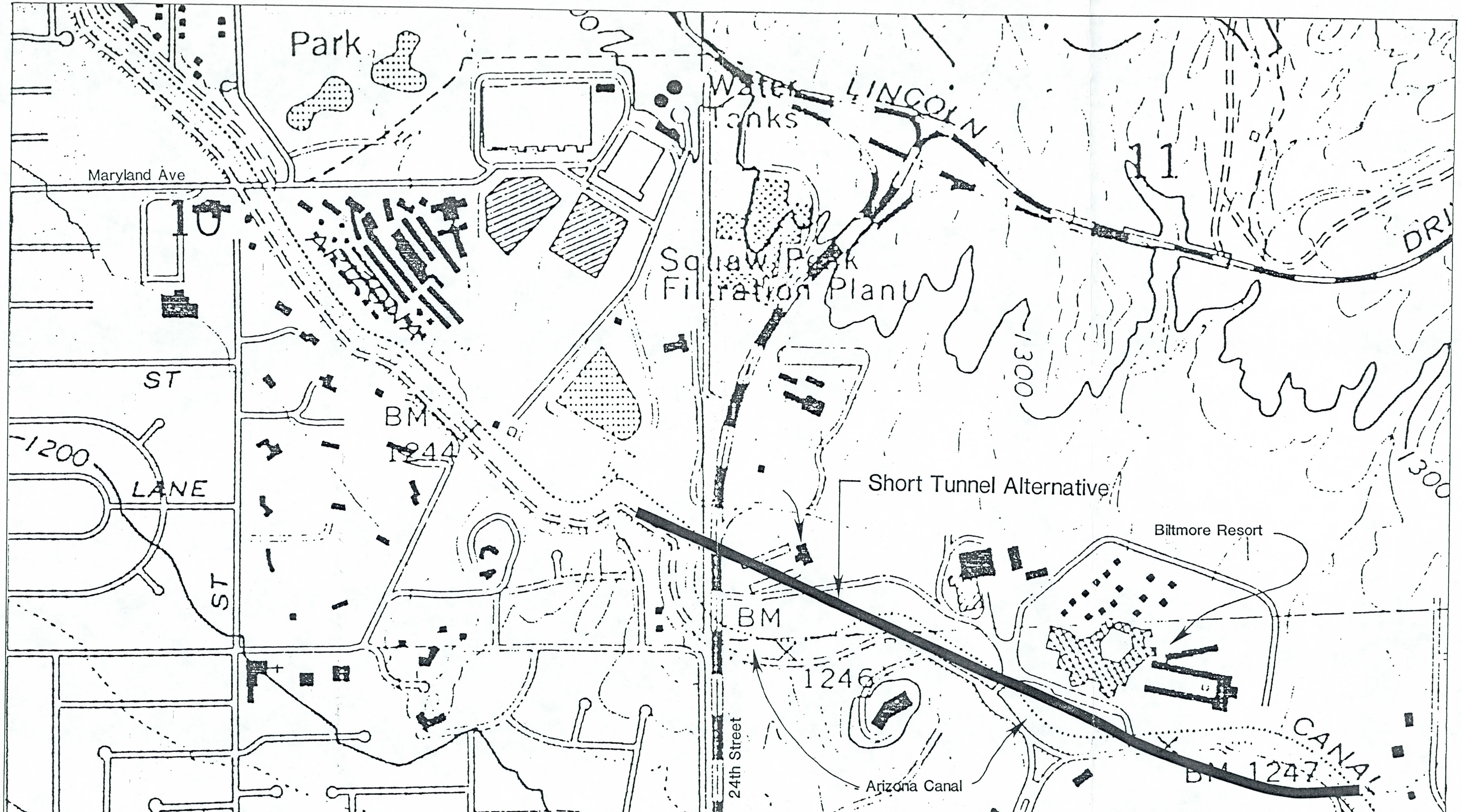


Typical Configuration - Conventional Excavation		Job No. 11565.00	Date 10/88
CRSS	Project *ACDC Deep Tunnel Study*	Designed by GS	Scale none
	Client *Maricopa County Flood Control District*	Drawn by MC	Figure 4



Note:
Both tunnels are identical.

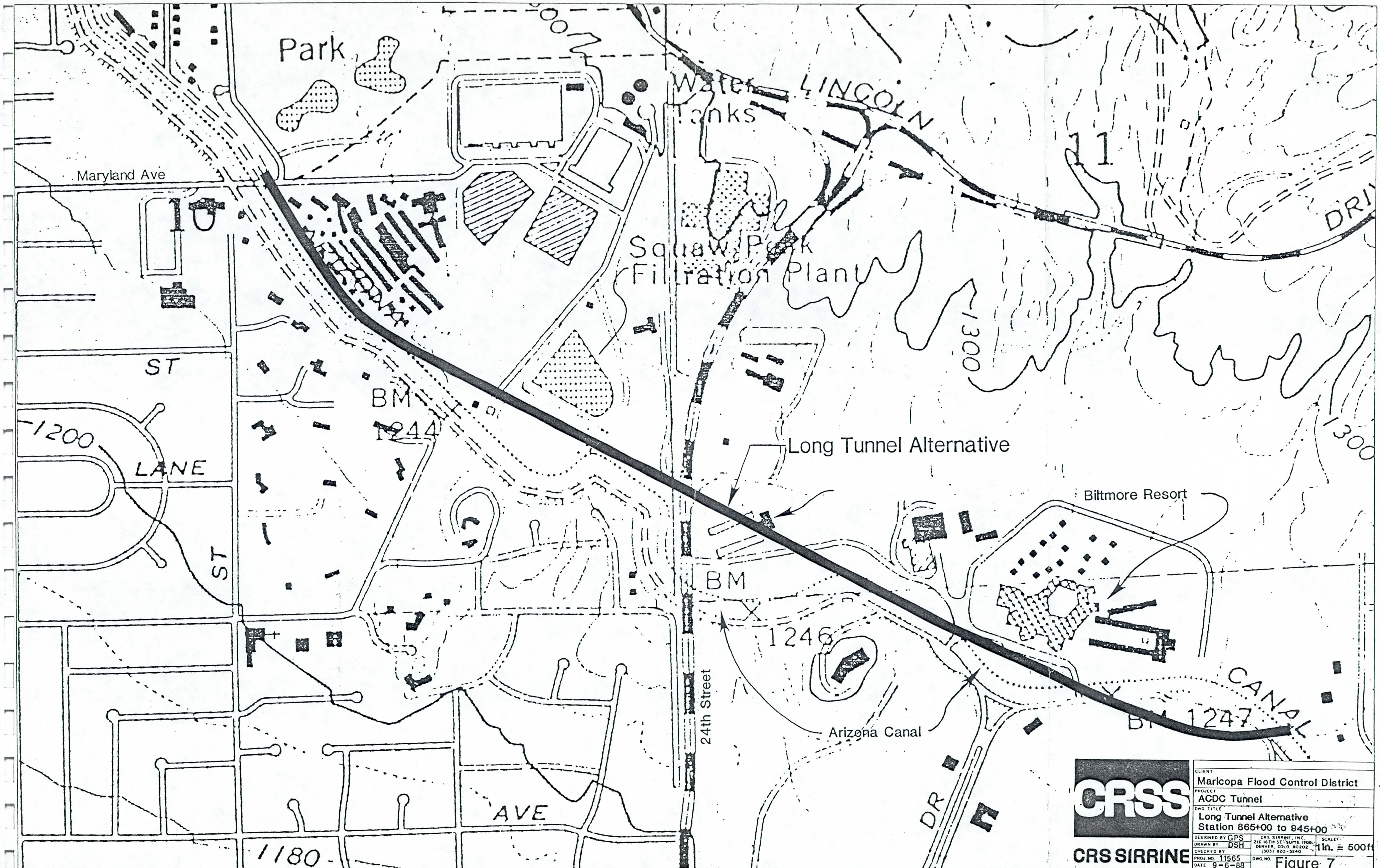
Typical Configuration - Tunnel Boring Machine Excavation		Job No. 11565.00	Date 10/88
CRSS	Project: *ACDC Deep Tunnel Study*	Designed by GS	Scale none
	Client: *Maricopa County Flood Control District*	Drawn by MC	Figure 5



CRSS

CRS SIRRINE

CLIENT	Maricopa Flood Control District
PROJECT	ACDC Tunnel
DWG. TITLE	Short Tunnel Alternative Station 895+00 to 945+00
DESIGNED BY / DRAWN BY / CHECKED BY	CRS SIRRINE, INC. 216 16TH ST.-SUITE 1700 DENVER, COLO. 80202 (303) 820-5240
SCALE:	1 in.=500 ft.
PROJ. NO. / DATE	DWG. NO. Figure 6


Park
Water Tanks
LINCOLN
DRI
11
10
Maryland Ave
Squaw Peak Filtration Plant
1300
ST
BM 1244
1200
LANE
ST
Long Tunnel Alternative
Biltmore Resort
BM
1246
24th Street
Arizona Canal
CANAL
BM 1247
AVE
DR
1180
CRSS
CRS SIRRINE
CLIENT
Maricopa Flood Control District
PROJECT
ACDC Tunnel
DWG TITLE
Long Tunnel Alternative
Station 865+00 to 945+00
DESIGNED BY GPS
DRAWN BY DSH
CHECKED BY
PROJ. NO. 11565
DATE 9-6-88
CRS SIRRINE, INC.
216 16TH ST. SUITE 1700
DENVER, COLO. 80202
(303) 820-5240
SCALE
1 in. = 500 ft
DWG. NO.
Figure 7

Appendix A

SEA Consulting Engineers, Inc.
Concept Hydraulic Analysis for Tunnel Study
Biltmore Canal Area
Arizona Canal Diversion Channel

(Text Only)

CONCEPT

HYDRAULIC ANALYSIS

FOR

TUNNEL STUDY

BILTMORE AREA

ARIZONA CANAL DIVERSION CHANNEL

January, 1989

SIGNED 2/9/89

Arizona U. S. A.

CONCEPT HYDRAULIC ANALYSIS
FOR
TUNNEL STUDY
BILTMORE AREA
ARIZONA CANAL DIVERSION CHANNEL

TABLE OF CONTENTS

LIST OF FIGURES

LISTING OF APPENDICES

EXECUTIVE SUMMARY

In March of 1987 CRS Sirrine was commissioned to analyze several alternatives to the Arizona Canal Diversion Channel design proposed by the Corps of the Engineers in the Biltmore area of Reach 4 of the project. In May of 1987 SEA, Inc. was commissioned to perform an independent hydrologic study in the Cudia City Wash area contributing to the flow in Reach 4 (the headwater Reach) of the ACDC project. CRS's study revealed the need for further concept design study for a deep tunnel alternative to the project in the Biltmore area. SEA's hydrologic study substantiated the Corps of Engineers design flow of 8300 cfs in this area. The study presented herewith is the hydraulic investigation performed by SEA as part of the concept design commissioned by Maricopa County Flood Control District for the Biltmore area.

Several alternatives were investigated for the tunnel configuration, inlet/outlet structures and other physical engineering parameters involved in the hydraulics of producing a tunnel 100 ft. below the surface in the Biltmore area for approximately 7400 linear feet.

The basic recommendation of this analysis is to operate the tunnel in a charged condition, namely filled in an orderly fashion and operated full. In order to accomplish this a chlorination system is recommended to keep the water in the tunnel from becoming septic. A pumping system is recommended for evacuating the tunnel, for maintenance and other reasons. The tunnel itself should be located approximately 100 ft. below the surface of the ground in order to minimize the probability of constructibility problems given the geology in the area. Bored twin tunnels of 24 ft. inside diameter with approximately a 1-foot concrete lining are recommended. Approximate dimensions for a sedimentation basin are included.

The need for an elaborate inlet structure to dissipate energy is not necessary because of the method of filling and maintaining charge through the tunnel. Several inlet structures were investigated for the option of operating the tunnel dry. Economics of the situation warrant a charged operational condition. The only modification to the channel design as proposed by the Corps of Engineers would be a slight increase in the side walls of the channel for approximately 2900 ft. upstream. Trash racks would be required at the inlet control structure. All of these characteristics would be further investigated in a model study which we strongly recommend in conjunction with the preliminary design, which should follow this concept study.

1 INTRODUCTION

The Arizona Canal Diversion Channel is a project conceived and developed by the Corps of Engineers, Department of the Army. The concept of the ACDC is to capture excess stormwater from the mountains north of the City of Phoenix and divert them in a primarily westerly direction to the major river systems of the valley, via New River. The main ACDC channel originates at the base of the Camelback Mountains, flows westerly to where it converges with a waterway called Skunk Creek in Peoria, Arizona, which in turn continues its westerly direction to merge with the New River and ultimately the Aqua Fria River.

The channel is composed of several reaches, each varying in physical characteristics. As might be expected, the channel increases in capacity and therefore size as it flows westerly. Our project is in Reach 4, which constitutes the headwaters of the channel. In this Reach the overland characteristics are primarily highly urbanized, high density residential developments. Our study area constitutes approximately STA 896+00 to 947+00 and is referred to as the Biltmore area.

There are several governmental entities involved in the ACDC project development. These are primarily the Corps of Engineers, Maricopa County Flood Control District, and in this Reach the City of Phoenix. Other municipal entities involved downstream are the City of Glendale, the City of Peoria and the City of Surprise, Arizona. As might be expected the Arizona Department of Water Resources is also greatly interested in the development of this project.

The purpose of this study is to develop a conceptual design and cost estimate for providing a deep tunnel alternative to the open channel design proposed by the Corps of Engineers for the above mentioned Reach. The alignment of the tunnel concept will approximate the alignment of the current Arizona Canal in the area and lie immediately north of that structure.

In March of 1987 CRS Sirrine was commissioned to perform a study entitled "Arizona Canal Diversion Channel Tunnel Alternative Feasibility Study" for this section of the Reach. In this study gravity flow tunnel alternatives to the covered channel were analyzed. During the course of this investigation another tunnel alternative was defined that offered potential cost effective savings. The study recommended the performance of a conceptual design study to further define the costs and technical feasibility of that alternative. The alternative CRS Sirrine identified was the deep tunnel alternative, which is the subject of this report.

In August of 1988, SEA was commissioned to perform a hydraulic concept study on this deep tunnel alternative in conjunction with CRS Sirrine concept report on the project. The following report represents that study.

The area of investigation in this concept hydraulic analysis lies primarily in developing the physical parameters, i.e. size and geometry of the structures involved in getting the flow from the ACDC, underground to the deep tunnels and through the Biltmore area. SEA's study included hydraulic calculations involving design discharge, analysis of inlet and outlet structures, hydraulic calculations of flows, flow quantity, discharge, velocity, energy dissipation and sediment transport. The primary characteristics to be investigated included tunnel diameter and configuration, vertical fall, energy dissipation, head loss, excess head loss, trash racks, safety, and operational characteristics, all in coordination with the Maricopa County Flood Control District.

During the course of the study several alternatives were analyzed from an engineering standpoint to determine feasibility. A thorough literature search was performed to develop basic concepts and approaches. One of the first questions that arose was the actual tunnel configuration. Several alternatives were investigated including a single tunnel, a double tunnel, and the traditional horseshoe shaped tunnel. The conventional blasted tunnel option would produce a horseshoe shaped tunnel nominally 25 by 40 feet. The boring machine option would involve twin tunnels of approximately 25 ft. diameter each.

After observing the geological configuration in the area, it was determined that a depth of approximately 100 ft. would be optimal from a constructibility standpoint. The operation of the tunnel itself was also investigated. Two options existed; operating the tunnel in a pumped dry condition, and operating in a charged full condition. Several options were also investigated for the inlet works primarily single versus double inlet, as well as several type of inlet structures; vortex, helical, drop, etc. The need for a sedimentation basin and trash racks was also investigated.

In looking at the option of operating in a dry condition, the problem of the inlet structure was heavily investigated. Several alternatives were examined. The vortex drop inlet configuration was determined to be most feasible for operation in this condition. All of these hydraulic considerations were developed based on studies performed by the Corps of Engineers for design flow of 8300 cfs and in conjunction with ongoing investigations on concept design by CRS Sirrine.

2 METHODOLOGY

2.1 Literature Review

The subject of inverted siphon design has not been extensively considered in the engineering literature. Short references that describe the general theory can usually be found in hydraulic text books, although they are not usually accompanied by detailed design information. Much of the available literature is historical.

Hinds (1928) provides a very complete discussion of inlet and outlet transitions for siphons and flumes. This discussion is very design-oriented, containing worked examples and performance information. Hinds relates results for inverted siphons with flow rates in the range of a few hundred cubic feet per second, and is generally concerned with shallow undercrossings. The gradual transitions and relatively low slopes required for these applications are not directly applicable to the current study.

Other design information is contained in the publications of the Bureau of Reclamation. For example, the 1967 Design Standards Number 3, Canals and Related Structures, contains design information and worked examples for inverted siphons and tunnels. Flows in this document range from 25 to 925 cfs. As in the Hinds report, the information is more suited to gradual transitions and shallow undercrossings.

Basic information on air entrainment in inverted siphons is provided by Kalinske and Robertson (1943). This document includes the results of laboratory and field studies regarding the formation of hydraulic jumps in closed conduits, together with a physical discussion. Consideration of the hydraulic jump is of obvious importance in the design of an inverted siphon.

A number of tunneling applications have been made in Europe. An important recent contribution to tunnel flow literature comes from Sweden's Royal Institute of Technology (Czarnota, 1986). Using model studies, field instrumentation, and theoretical analysis, hydraulic losses in tunnels were evaluated. Czarnota found that bored tunnels typically experience head losses similar to pipe conduit losses, while tunnels constructed using conventional blasting methods typically experience considerably higher losses. The difference results from the variation in cross-section usually present in blasted tunnels. This variation results in alternating acceleration and deceleration of the flow through the tunnel, and can create head losses as high as 20% of the total losses.

It may, on first consideration, seem somewhat odd that the literature on the subject of inverted siphon design is so scarce. It is important to remember, however, that applications of inverted siphons are generally quite different. Each application has to be designed within particular constraints that make it unique. Often, a model study is made as a part of the design process to test theoretical design. Such a design process does not lend itself to handbook or chart design.

2.2 Hydraulic Design of Tunnel

At concept level, hydraulic design of the tunnel had to be completed at a very general level. The literature search provided some guidance on design procedures through worked examples. Further input was provided through telephone and personal interviews with hydraulic engineers with extensive experience in tunnel applications. From the accumulation of these ideas and opinions, design concepts were formulated for several options.

Once the concepts were available, head losses were estimated. Head loss estimates for tunnel flow were made using procedures outlined in the Bureau of Reclamation Design Standards Number 3 (1967). Frictional losses within the tunnel were evaluated using friction slopes developed using the Manning Equation. The Darcy Weisbach equation is generally used for closed conduit flow, as it allows for frictional resistance to change with the fluid viscosity, density, or velocity. In this application, temperature changes should not be extreme, and the Reynold's Number of the flow places it in the range where friction factor becomes insensitive to changes in Reynold's Number. A comparison was made between the two equations, and the resulting head losses were very similar. Because of ease of application, precedent in design literature, and similarity to Darcy-Weisbach results, the Manning Equation was used for friction calculations.

So-called minor losses, that is the losses occurring due to bends, expansions, contractions, etc., were evaluated using standard minor loss coefficients which are applied to the velocity head of the flow. Coefficients were obtained from the Bureau of Reclamation manual and from Morris and Wiggert (1972). There is some question of the applicability of such coefficients, which generally have been developed in very small pipe conduits, to a 24 foot diameter tunnel. It can be argued that the very large flows in the tunnels will react differently than smaller flows in a small pipe just as a result of scale differences. It can also be argued, however, that boundary layer thicknesses in the larger tunnel would encompass a smaller percentage of the total flow area in the larger tunnel than in the smaller pipe. Since losses are largely a result of boundary layer activity, the losses in the tunnel would probably be less than in the small pipes; application of standard head loss coefficients would therefore be conservative.

In either case, for this concept level design, use of standard coefficients for evaluating minor losses was judged a reasonable method for estimating the magnitude of these losses and comparing different options for tunnel design. Certainly in final design of the tunnel, model studies would be utilized to more definitely determine the head losses. Such studies were neither economically feasible nor prudent for concept design.

2.3 Inlet and Outlet Hydraulics

Estimation of inlet and outlet losses is even more difficult than estimation of losses in the tunnel. The flow system at these locations is extremely complex. The water carried by the ACDC must undergo transitions from open channel flow to pressure flow, from a rectangular channel to a circular tunnel, and must complete a sharp turn to effect the approximately 100 foot drop to the tunnel body. At the outlet, the water carried by the tunnel must reverse this process.

Since the situation is so complex, no definitive answer can be given to the question of the attendant head losses. The only reasonable way to give such an answer would involve model studies. Therefore, the challenge is to estimate the losses in some meaningful fashion so that different alternatives can be evaluated.

The method for making this estimate followed the Bureau of Reclamation examples as a guide. Frictional losses through the transitions were estimated using average friction slopes developed from the Manning Equation. Other losses were evaluated using head loss coefficients. These coefficients were obtained from the Bureau of Reclamation Manual and from estimates made by experienced researchers in tunnel inlet design.

The total losses for each alternate considered were computed as the sum of the tunnel losses and the inlet and outlet losses. This value could be compared to the difference between the invert elevations of the proposed ACDC to estimate the improvements required to accommodate the headwater requirements of the tunnel.

Provisions for combining the surface runoff in the Reach with the tunnel will have to be provided also. This could most easily be accomplished by a network of storm drains to pass this flow to the outlet end of the tunnel. The exact configuration of such a system would appropriately be developed as a part of the final design for the tunnel.

3 DISCUSSION OF ALTERNATIVES

Alternative design concepts were compared based upon the criteria of economy, hydraulic performance, and constructibility. The purpose of these evaluations was to identify, as well as analyze the most promising alternatives at concept level. These most promising alternatives provide a starting place for final design activities. In this section of the report, the alternatives will be discussed in a general way, and the factors which led to the elimination of concepts will be discussed. The recommended alternative will be explained in more detail in the next section.

3.1 Single vs. Double Tunnel

There were two possible configurations for the tunnel conduit identified in the tunnel feasibility investigation. These are a single, conventional blasted tunnel and a pair of bored tunnels. The conventional blasted tunnel option would produce a horseshoe shaped tunnel, nominally 25' x 40'. The tunnel boring machine option would produce two parallel circular tunnels, each nominally 26' in diameter.

A preliminary evaluation of the single tunnel option was performed at the beginning of this study. From the standpoint of hydraulic head, the preliminary evaluation indicated that the single tunnel option could be sized such that head losses were comparable to the double tunnel option, including allowances for variability in cross section in the blasted tunnel. However, the constructibility of a blasted tunnel in this urban environment is highly questionable. Therefore, no in-depth analysis was completed on the single tunnel option.

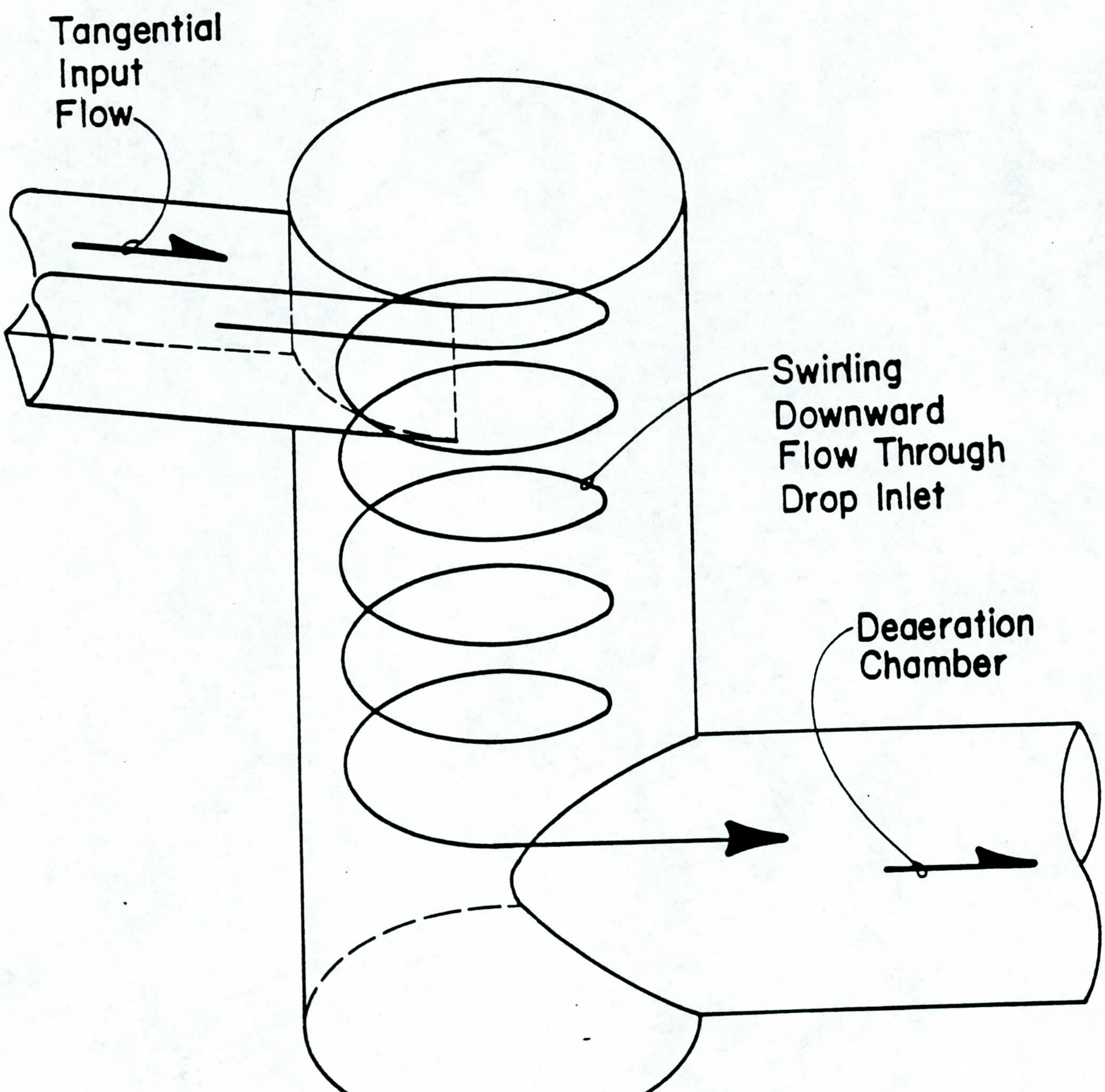
3.2 Pumped Dry vs. Charged Condition

The inlet design for the tunnel presented very unique challenges. The tunnel basically operates in two drastically different modes during a storm event. In the early part of the storm, as the rising limb of the hydrograph reaches the tunnel, allowances must be made to dissipate the energy released as the water falls the 100 feet to the main tunnel shaft. Once the tunnel is full, energy takes on a different importance; every effort must be made to minimize hydraulic losses in order to reduce upstream improvements to accommodate required headwater.

Understandably, it was very difficult to identify a design concept which was sensitive to these two contradictory purposes. The design would have to be capable of creating head losses until the tunnel fills, after which time it would have to produce minimal head losses.

The most promising idea was proposed by Dr. S.C. Jain, from the Iowa Institute of Hydraulics. Dr. Jain has extensive experience with modeling of inlet structures for tunnel applications. He suggested the use of a vortex drop inlet structure. The vortex drop inlet structure (Figure A1) is a simple system consisting of a vertical drop shaft with tangential input. The tangential input flow forces the water to swirl around the outside of the drop shaft, dissipating energy through side friction all the way down. At the bottom, a deaeration chamber is provided before the entrance flow actually reaches the tunnel.

This design has been used successfully in other applications around the country. Dr. Jain felt that it was workable in this application also, and provided estimates of



Schematic Drawing
of Vortex Drop Inlet

FIGURE A1

sea

factors to use in evaluating the head losses. The drop inlet structure would operate as an energy dissipator while the tunnel was filling. Once the system became charged, the inlet and deaeration chamber would act as expansions and contractions in a pipe.

The most severe design condition is the pressurized portion of the tunnel operation. This condition will have been achieved by the time the design flow rate, which is the peak flow rate for the storm event, reaches the tunnel. The vortex drop inlet system was therefore evaluated for this condition.

Many other drop structures might possibly serve for the inlet structure of the tunnel spillway. Some examples of other possibilities are drop structures with bars to break up the flow, sloping inlets with dentations to dissipate energy, helical inlets, as well as conventional dissipation schemes and such as steps, blocks, etc. Only the vortex drop structure was considered in concept design because it was felt to be the best in the full flow condition and because considerable information was available to analyze it.

Another option which was considered was to remove half of the operating cycle by keeping the tunnel full at all times. This eliminates the need for energy dissipation structures, such as a vortex drop structure, necessary for filling the tunnel, because the tunnel could be filled in a controlled fashion immediately after construction. Maintaining a charged condition for the tunnel at all times greatly simplifies the inlet transition because the tunnel operates as an inverted siphon at all times; velocities are therefore kept low and the tunnel acts as a pipe. Dr. Jain's experience leads him to conclude that a deaeration chamber would not be required for the charged condition. The design can be completed in a way which is sensitive to the most critical peak flow condition and which holds down upstream improvement requirements.

When comparing the two options, one must consider the ramifications for tunnel operation. If an energy dissipating inlet were to be provided, the system would be pumped dry after each storm event from which it receives water. This was estimated to occur approximately 16 times each year. If the system is to be maintained in a charged condition, the water standing in the tunnel would require chlorination to prevent biological activity and the problems associated with it. These include odor and health problems associated with the septic conditions which would exist when water becomes anaerobic. Both of these requirements were considered in this study.

If the system is to be pumped dry after each storm event, the number of pumps and subsequent costs would be determined by the time interval selected for removal of the water. The City of Phoenix requires that retention areas be drained in thirty-six (36) hours following a storm event. Utilizing this time interval, and a pumping facility consisting of 150 horsepower pumps operating at 4,000 gpm, it was determined that six (6) pumps would be required. These pumps are a standard type for which parts are readily available. The design and selection of the pumps should be considered very important due to the high lift situation and the potential for cavitation which may exist. The installation cost for the pumping facility is estimated at $160,000 with an annual operation cost of $40,000 based on sixteen (16) evacuation events. The pump system cost could be reduced if the time interval for evacuation is lengthened, however, too long a period could create a septic condition. This detail would be further evaluated during preliminary design. Negotiations with agencies involved would be conducted.

If the system is maintained in a charged condition a chlorination system for the treatment of the water is necessary. The facility would consist of a building with the chlorination equipment, handling equipment and a pipe distribution system to the tunnels. The installation cost for this chlorination facility is estimated at $100,000.00 with an annual operation cost of $25,000.00 based on continual chlorination between storm events and complete re-chlorination for sixteen (16) evacuation events. A minimal pumpout facility would be included with the chlorination facility to provide for removing the water for tunnel maintenance purposes.

3.3 Single vs. Double Inlet and Outlet

If there are to be two tunnel shafts, the ACDC flow must be divided to enter them. This division can occur either above ground or below ground. The above ground option would require the construction of two inlet structures. The below ground option would require only one inlet structure, but would require a division chamber below ground. Both of these possibilities have been considered. The same alternatives occur at the outlet end of the tunnel.

It appears that the single inlet system would cause slightly higher head losses than the double inlet system. In the double inlet system, an inlet transition could be designed so that only one tunnel is used for low flows, which will have beneficial impact on operational costs. Perhaps the biggest advantage of the double inlet system is it allows the use of one tunnel for storm flows while the other tunnel undergoes maintenance activities. Excavation for dual inlets will be provided in the tunnel construction. Therefore, for these reasons, the double inlet system received greater study than the single inlet system.

Based on the result of the inlet analyses, serious consideration was given only to a dual outlet system. This avoids providing a design for an underground transition with its anticipated potential for turbulance and difficulty of construction. Recombination above ground appears at this time to be a better alternative.

3.4 Excess Head Losses

The tunnel system will undoubtably create head losses which are more extreme than the proposed channel. This is fairly easy to see if one imagines that the open channel flow generates no friction across the free surface, while the tunnel flow experiences frictional losses around its entire perimeter. The magnitude of these losses will be discussed in the next section.

There are two options for handling the losses over and above the losses expected for the proposed ACDC Channel. The upstream end can be improved, with berms or increased wall height for example, to accommodate the extra headwater requirements of the tunnel. Alternatively, the invert elevation of the outlet end of the ACDC could be lowered, and then the ACDC could be graded at a shallower slope than is currently proposed out to some distance to match the proposed grades.

Because the bottom slope of the ACDC is much steeper on the inlet end (0.0008 ft/ft)than the outlet end (0.0003 ft/ft), the inlet end is the better place to handle the headwater improvements. Inlet improvements would not have to be carried as far upstream as outlet improvements would downstream, lessening the impact on the proposed construction plans.

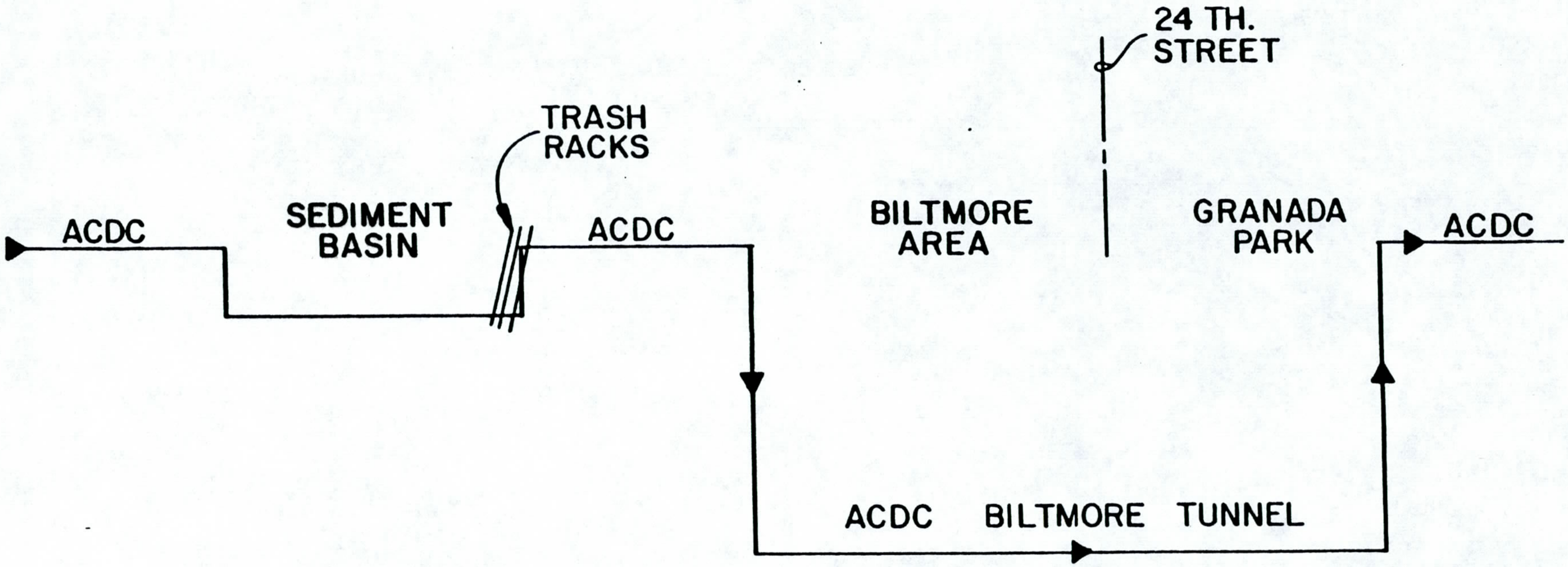
4 DETAILED DISCUSSION OF SELECTED ALTERNATES

The preceding sections have been very qualitative in their descriptions of the alternatives studied. The following section contains more detailed information about preliminary sizes and flows. Each component of the system shown schematically in Figure A2, will be discussed in sequence. Both the pumped and charged systems were considered for head losses in depth, because non-economic considerations could result in one of these options being preferred over the other. Both options are described in detail.

4.1 Sedimentation Basin

The Cudia City Wash Sedimentation Basin, as described in the Corps of Engineers Design Memorandum Number 12, is designed to remove all of the gravel and most of the sand from the Cudia City Wash drainage flow. Between Cudia City Wash and the proposed tunnel inlet, there are two additional sediment sources from small side inflows. These are described as Subbasins 2 and 3 in the Corps of Engineers studies on the Sedimentation Analysis. The accumulated sediment load of the ACDC flow at the tunnel inlet for the design storm was developed from these documents and is tabulated below.

Size	Cudia City Wash Inflow (AF)	S.B. #2 Inflow (AF)	S.B. #3 Inflow (AF)	Total (AF)
Silt and Clay	5.60	2.00	1.95	9.55
Very Fine Sand	0.22	0.60	0.57	1.39
Fine Sand	0.04	0.24	0.23	0.51
Medium Sand	0.01	0.07	0.07	0.15
Coarse Sand	0.00	0.00	0.00	0.00



Schematic Diagram Showing Tunnel Flow System

FIGURE A2

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The fine sand, very fine sand, silt, and clay is all considered wash load in the ACDC, and should remain in suspension throughout the length of the ACDC. In the tunnel, this material might be deposited at the end of a storm event, but will be re-suspended and removed by the next flow event. However, the medium sand is considered bed load in the ACDC, and it would be disadvantageous in terms of maintenance requirements to allow this material into the tunnel.

The resuspension of the wash load material, and the ability of the flow to carry suspended sediment up the outlet end of the tunnel are problems which do not lend themselves to analytical predictions. Transport of cohesive sediments and vertical sediment transport are, at present, poorly understood because of their complexity. At this stage, all that can be done is to point out these limitations. It is recommended that a more detailed analysis of these problems be included early in the preliminary design phase. Also, a slightly exaggerated maintenance schedule should be used at this time for estimation purposes.

A preliminary investigation of the size requirements for such a sediment basin was made. This analysis was based on ideal settling velocity, short circuiting, length to width ratio, and permissible flow through velocity. The trap efficiency and sediment outflow was not evaluated because it was not possible to develop the necessary information for such a study at this level of the design. The results obtained in the analysis are therefore estimates only.

The sediment basin would require approximately 3.5 acres, being no closer to square than 550' long X 275' wide. At least 1 acre-foot of storage below the spillway elevation must be provided. The depth of flow in the basin should be about 15 feet.

Trash racks could be provided at the outlet end of the sedimentation basin. The advantage over placing trash racks immediately at the opening of the tunnel is that velocities will be lower in the sedimentation basin, so that head losses through the racks will be lower. Also note that it is not necessary to place the sedimentation basin in immediate proximity to the tunnel, just as long as it is between the entrance and the next upstream sediment source.

An alternative to placement of a single sediment basin in-line on the ACDC alignment is the placement of a decentralized sediment trapping system. This system would consist of the following elements:

1) Improved efficiency of the Cudia City Wash Sediment Basin to trap all (100%) of the bed load. Modifications to the design of the basin to trap this additional volume are not expected to be extreme. The additional volume would be approximately 8 cubic yards. Additional costs for this volume are considered insignificant for this part of the system.

2) Add a sediment basin for the 32nd Street inflow to trap all the bed load. This would handle all of sub-basin 2 as referenced in the Corps of Engineers report. The approximate size of the basin required would be 65' wide x 125' long x 8' deep.

3) For Sub-basin 3 of the Corps of Engineers report, the conveyance system should be designed to trap all

the medium sand prior to inflow to the tunnel. Alternatively some flows could be routed to the outlet end of the tunnel to join the open channel section.

4.2 Vortex Drop Inlet

Both single inlet and double inlet structure systems were considered. However, for reasons described previously, the single inlet system was eliminated for in-depth scrutiny. The double inlet system would consist of two 15' diameter drop shafts. The tangential input channel at the ground surface will be approximately 4' wide. The channel sides can transition from the ACDC channel section at approximately 17 degrees. A velocity of roughly 20 feet per second will be created in the inflow channel. The inflow channel will have a bottom slope of roughly 27.5 degrees.

The drop inlets will enter deaeration chambers at the elevation of the main tunnel. The deaeration chambers will be roughly 26 feet in diameter and 250 feet long. A 7 foot diameter air vent will be provided about 70 feet from the drop inlet shaft. At the end of the deaeration chamber, a transition to the main tunnel shaft would be provided. This transition could be rounded to avoid extreme head losses. The tunnel shafts would then run to a vertical riser shaft of the same diameter, where the flow would rise to the ACDC invert. An elbow would be provided at the outlet to direct the flow down the ACDC alignment.

A range of tunnel diameters was considered in head loss computations. Tabulated results are included in section 4.4.

4.3 Charged Condition

The charged condition system is considerably less complicated than the pumped system with its energy dissipating inlet. The inlet transition must effect a transition from a rectangular channel to the circular conduits of the inverted siphon, divide the flow, and complete a 90 degree direction change as the water flows vertically down the tunnel. Divergence angles of the channel walls in the flow division would be maintained at about 17 degrees. Adequate allowances for the headwater requirements to overcome the frictional losses of the tunnel must be made.

The tunnels would drop vertically down the 100 feet to the elevation of the main tunnel shafts. The tunnels would then turn 90 degrees and run approximately 7400 feet horizontally to the outlet station. Another 90 degree turn would bring the flow vertically upward to the ACDC. Again, elbows could be provided to direct the flow when it re-enters the ACDC. The entire tunnel system would consist of circular conduits of a single diameter.

The tunnel system could be filled initially and following periodic maintenance by a water pipeline from a source such as the local treatment plant or adjacent irrigation canal.

A range of tunnel diameters was considered for this system. Tabulated head losses are included in section 4.4.

4.4 Tabulated Head Losses

The results of the analyses of the options described above are tabulated below. The proposed ACDC invert elevations at the approximate inlet and outlet stations of the tunnel are 1221.76 and 1216.09 feet, respectively. Thus, the current fall is 5.67 feet. The excess head losses shown below are the head losses over and above this fall. The calculated head losses for each alternate include a 10% increase to allow for the estimations that were made.

	Vortex Drop Structure		Charged Condition	
Tunnel Dia. (ft)	Head Loss (ft)	Excess Head Loss (ft)	Head Loss (ft)	Excess Head Loss (ft)
23.0	10.91	5.24	8.35	2.68
23.5	10.01	4.34	7.38	1.71
24.0	9.31	3.64	6.62	0.95
24.5	8.71	3.04	5.97	0.30
26.0	7.40	1.73	NE	

NE = Not Evaluated

The head losses for the vortex drop structure inlet are considerably more extreme than for the maintained charge system. This is especially significant given that the operational costs for pumping the vortex inlet system are higher than for the maintained charge system. Based upon this information, the maintained charge system is recommended.

4.5 Upstream Improvements

The head losses computed above are in excess of the available fall for the proposed ACDC. The berms or wall height must be increased at the end to contain the extra headwater requirements within the channel of the ACDC. The crest elevation of the berm is set by the headwater requirement, and the berm can run back to the end of the backwater profile. For estimating purposes for the concept design, the backwater profile was calculated using the direct step method. Freeboard has already been included in the ACDC. Only the maintained charge system has been considered following.

Tunnel Dia. (ft)	Berm Length (ft)
23.0	7060
23.5	4820
24.0	2860
24.5	970

4.6 Local Surface Drainage

Local surface drainage will be collected in a proposed storm drain system. Figure 5A indicates a proposed storm drain system design. The proposed system would consist of storm drain pipe, catch basins, and manholes along the present Arizona canal alignment. Storm drain sizes are based on various point inflows along the Canal alignment. The storm drains will direct the local runoff to outfall at both the tunnel inlet and outlet ends and a drop shaft to the tunnel located at approximately Sta. 920+00. It is important that the medium sand be trapped at retarding structures to prevent entrance to the proposed storm drain system and tunnel.

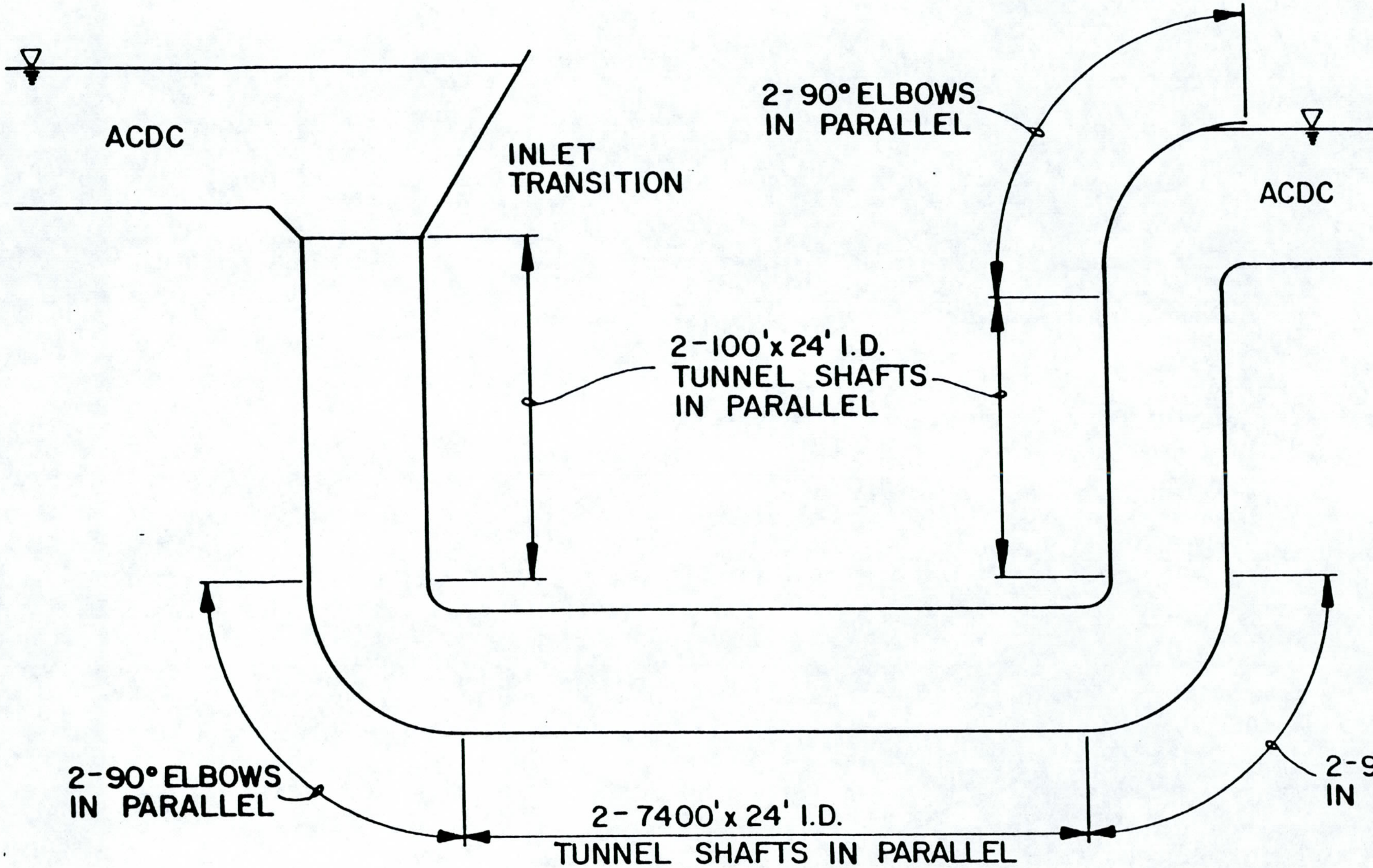
5 RECOMMENDATIONS

The following are recommendations based upon of the hydraulic analysis performed in this concept design study.

Basic configuration of the tunnels should be dual bored circular tunnels, concrete lined with an inside diameter of 24 ft. and a minimum 1.0 ft. of lining. The depth of the tunnel should be approximately 100 ft. below surface to minimize difficulties during construction and take into consideration geologic formations in the area. The recommended alternate is shown on Figures A-3 and A-4.

The operation of the tunnel should be a charged full condition. The tunnel should be filled in an orderly fashion alleviating the need for a very extensive inlet structure. In order to accomplish this, we have included a chlorination system which would keep the water in the tunnel from becoming septic. We have also included pumping facilities to discharge the tunnel water and evacuate the tunnel for maintenance and other reasons.

In analyzing the head losses in these tunnels 24 ft. diameter and 7400 feet in length, and assuming grades outlined by the Corps of Engineers design for Reach 4 in this area, we have determined an excess head water requirement exists of approximately 0.95 feet. In order to account for this, the channel walls upstream of the tunnel structure would have to be increased slightly. This would be accomplished by increasing the channel wall by approximately one foot upstream of the tunnel and tapering down at slope to where it would meet current design grade approximately 2,860 ft. upstream of the inlet to the tunnel. We also feel that this produces



Schematic Drawing of Recommended Alternate

FIGURE A3

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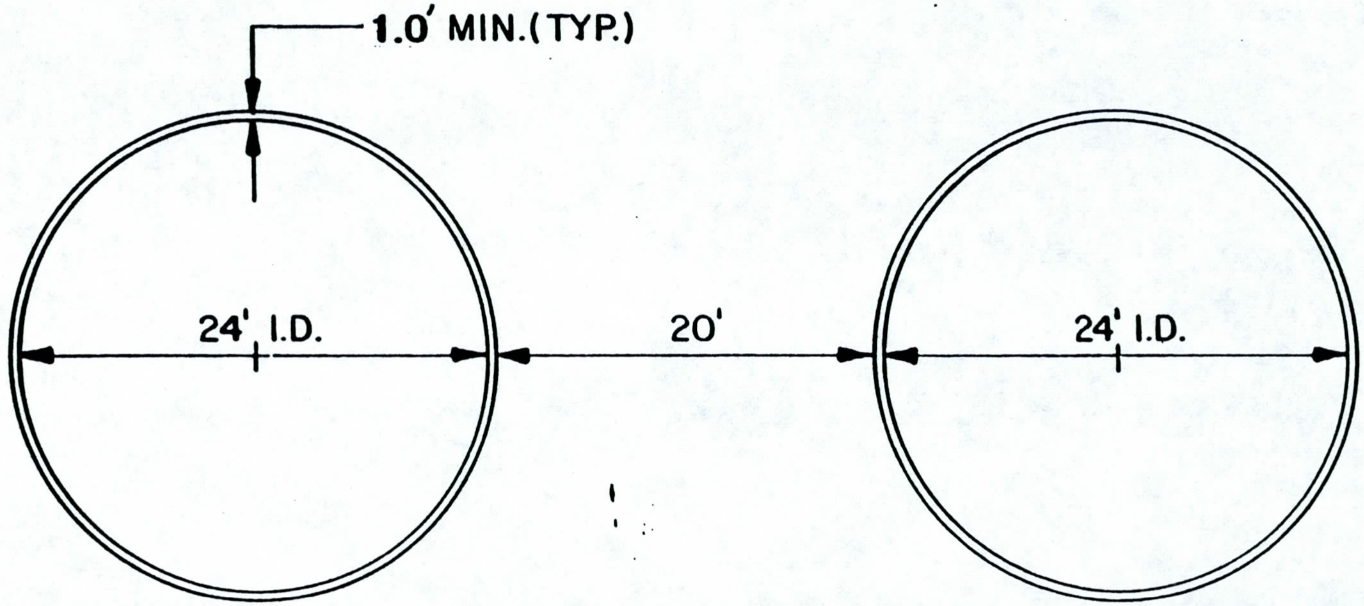

Recommended Tunnel Cross-Section

FIGURE A4

the most economical option for technically producing a tunnel in the area.

A sediment trapping system is required. The system could be a single basin or a decentralized basin at sediment source points depending on the real estate economics.

A storm drain system is recommended to handle the local surface runoff in the tunnel area.

The safety of the tunnel structure must be considered in the final design. The inlet and outlet shafts would be fenced and protected to prevent public access.

The recommendations presented in this report are based upon concept design and assumptions only. It is anticipated that preliminary design be preceded by a model study to further refine design parameters. This would assure verification of these assumptions.

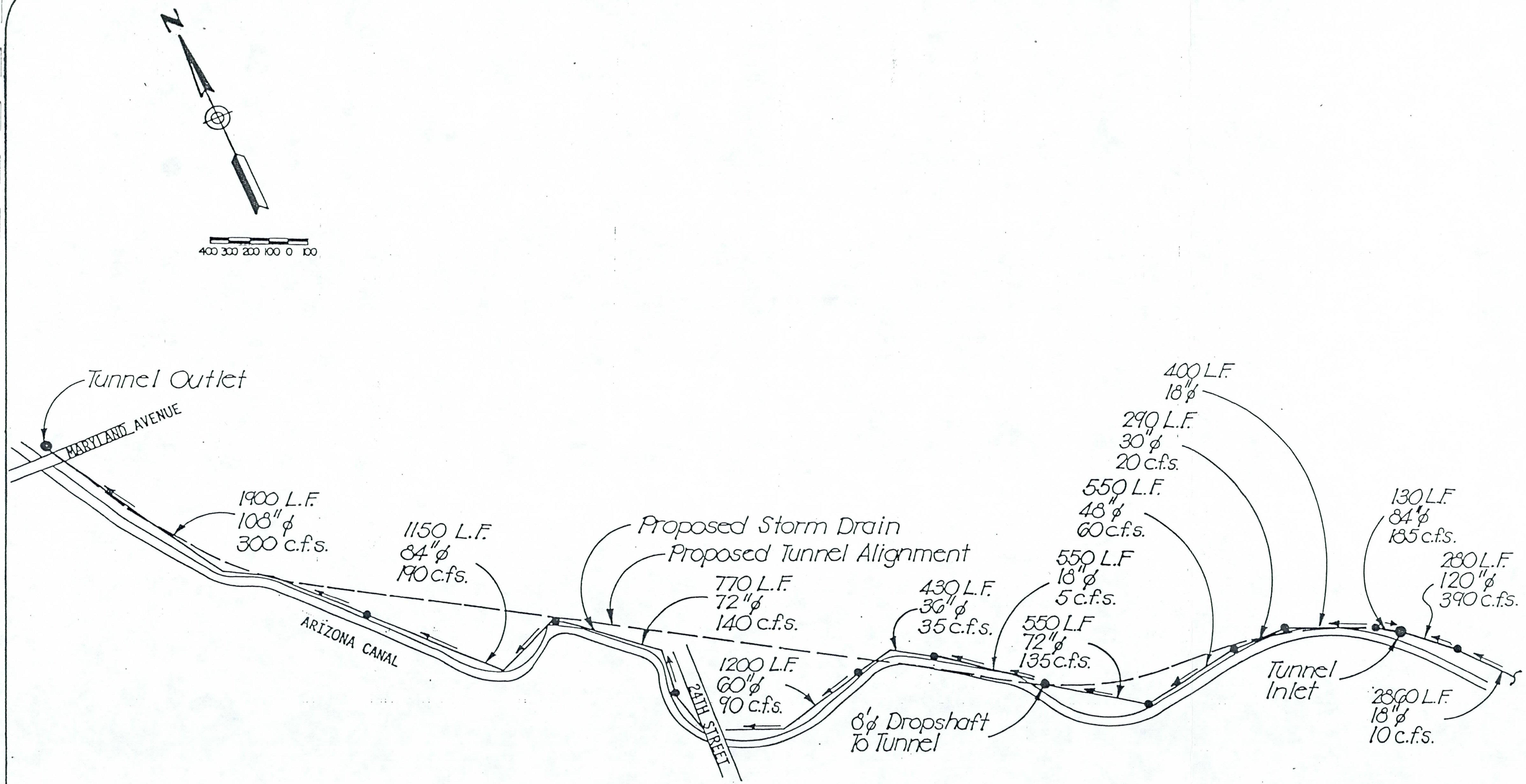

FIGURE A5

PROPOSED STORM DRAIN
FOR LOCAL RUNOFF

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