2/93 DICK PERREAULT

DRAFT

Dick,
this document was produced in draft only.
It may have some superseded information.
Call me if you have any questions
Steve

Flood Control District of Maricopa County
2801 West Durango
Phoenix, AZ 85009

City of Tempe

RIO SALADO WATER RESOURCES MASTER PLAN

Technical Memorandum 4

Salt River Hydraulics

Submitted by:

CH2M HILL

July 1991

Technical Memorandum 1--Data Inventory

Technical Memorandum 2--Permitting Constraints

Technical Memorandum 3--Water Balances

Technical Memorandum 4--Salt River Hydraulics

The purpose of this memorandum is to summarize the hydrology, hydraulics, and sediment transport conditions of the Salt River system and discuss optional materials and methods of dam and hydraulic structure construction.

Technical Memorandum 5--Stormwater Management

Technical Memorandum 6--Aquifer Storage and Recovery

Technical Memorandum 7--Surface Water Development

Technical Memorandum 8--Legal and Institutional Issues

Technical Memorandum 9--Pipelines and Pump Stations

Technical Memorandum 10--Reclaimed Water Recharge

Technical Memorandum 11--Stormwater Facilities

Technical Memorandum 12--Dam and Control Structures

Technical Memorandum 13--Irrigation Systems

Technical Memorandum 14--Lake Systems

Technical Memorandum 15--Legal Issues

TABLE OF CONTENTS

	Page
INTRODUCTION	1
STUDY OBJECTIVES	1
LIMITATIONS	1
HYDROLOGIC AND HYDRAULIC SETTING	**2**
HYDROLOGY	2
HYDRAULICS	3
AGENCY REQUIREMENTS AND CRITERIA	**6**
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY	6
ARIZONA DEPARTMENT OF WATER RESOURCES	6
OTHER AGENCIES	7
DAM DESIGN ALTERNATIVES	**8**
DAM AND GATE ALTERNATIVES	8
INFLOW CONTROL ALTERNATIVES	22
ALTERNATIVE DAM LOCATIONS	23
EVALUATION OF ALTERNATIVES	25
CONSTRUCTION COST OPINIONS	25
LIFE CYCLE COSTS	26
DECISION MATRIX	26
BIBLIOGRAPHY	**29**

APPENDIX A CONSTRUCTION COST ESTIMATES

APPENDIX B COST ANALYSIS OVER DESIGN LIFE

FIGURES

1 Channel Gate Alternative, Inflatable Dam Option 10

2 Channel Gate Alternative, Bottom-Hinged Leaf Gate Option 13

3 Fixed Weir Alternative, Side Channel Spillway with Leaf Gates Option 15

4 Fuse Plug Alternative, Fuse Plug with Leaf Gate Option 19

5 Fuse Plug Alternative, Segmented Fuse Plug Option 21

TABLES

1 Discharge Frequency Values Salt River at Mill Avenue Bridge 3

2 Discharge Frequency Values Indian Bend Wash at the Salt River 3

3 Construction Cost Opinions 25

4 Life Cycle Cost Opinions 26

5 Decision Matrix Rating System 27

6 Parameter Weighing System 27

7 Decision Matrix 28

INTRODUCTION

The focal point of Tempe's Rio Salado Project will be Town Lake. The lake is proposed to be developed by creating an impoundment in the usually dry Salt River channel. The river is currently undergoing the construction of major flood control features including bank protection, grade control structures, and realignment. The design of Town Lake must successfully integrate recreational features of the lake with flood control of the Salt River.

STUDY OBJECTIVES

This document summarizes the hydraulics of the study reach of the Salt River as derived from previous investigations and presents the results of evaluations of the types of dams that could be used to form the lake. The document is intended to lay the groundwork for further investigations in subsequent technical memorandums and to provide a basis for discussion in Workshop 2. Specific objectives of this document include:

- Consolidate and summarize information from the various agencies and the City of Tempe on the Salt River hydraulics issues pertinent to Rio Salado.
- Summarize anticipated design criteria of the relevant regulatory agencies.
- Identify and describe potential impoundment dam types, locations, configurations, and materials.
- Identify options for managing the impacts of sediment transport on structural elements of the lake and grade control structures.
- Provide discussion of flood protection and damage mitigation measures to protect facilities placed in the channel.
- Evaluate potential dam alternatives for potential impacts during the passing of significant flood flows and of bed load materials, aesthetics, and construction and operations and maintenance costs.

LIMITATIONS

The information presented in this memorandum is preliminary. Much of the data used was prepared by others and was not independently verified. The information is provided to evaluate basic design criteria, evaluate feasibility of the Town Lake concept, and provide a basis of comparison for alternative dam concepts.

HYDROLOGIC AND HYDRAULIC SETTING

The Salt River is the primary conveyance for the Salt-Gila Drainage System through metropolitan Phoenix. The design of Town Lake must be considerate of the hydrologic, hydraulic, and sediment transport characteristics of the system to assure that the flood control valve and safety of the river are not compromised. This section describes these characteristics and general design criteria.

The proposed Rio Salado Park includes approximately 6.5 miles of floodway improvements to the Salt River. The Salt River channelization is being completed in two stages. The first stage extends from the western city limit at 48th Street to the Southern Pacific Railroad bridge. This stage was designed by the Arizona Department of Transportation (ADOT) and is currently in the latter stages of construction. The second stage extends the Stage I channelization to the eastern city limit near the McClintock Drive bridge. Stage II was designed by the City of Tempe and is currently under construction. These channel improvements will allow reclamation of the Salt River floodplain for higher-valued land uses, as well as allow restoration of the in-stream habitat.

This section describes the hydrology, hydraulics, and sediment transport characteristics of the Salt River in the Rio Salado project area. Design criteria and assumptions based on these characteristics are also presented.

HYDROLOGY

SALT RIVER

The project hydrology is based on a revised analysis of the Central Arizona Project features by the U.S. Army Corps of Engineers, Los Angeles District and on limited information supplied by the Salt River Project (SRP). The Flood Control District of Maricopa County (FCDMC) is currently preparing a restudy of the Salt River hydrology. This restudy will incorporate the affects of channelization and upstream impoundments on the Salt River and its tributaries. Therefore, the flood data used for the Rio Salado project may be subject to revision upon completion of the FCDMC study or change in reservoir operation procedures by SRP.

The Corps' revised hydrologic analysis examined the impacts on Salt River flooding of eliminating the proposed Cliff Dam on the Verde River, and of significantly increasing water storage behind Roosevelt Dam. The Rio Salado project hydrology is based primarily on the Corps' analysis. Discharge frequency results for existing conditions without Roosevelt Dam improvements and for future conditions with Roosevelt Dam improvements are shown in Table 1.

Table 1 Discharge Frequency Values Salt River at Mill Avenue Bridge		
Frequency (Year)	**Existing Conditions (cfs)**	**Future Conditions (cfs)**
5	40,000	44,000
10	93,000	84,000
20	135,000	110,000
50	160,000	135,000
100	215,000	160,000
200	275,000	190,000
500	330,000	250,000

INDIAN BEND WASH

The Indian Bend Wash will discharge stormwater into the Town Lake. The quantity and frequency of these discharges will import the management of Town Lake. Discharge frequency values for Indian Bend Wash are based on a U.S. Army Corps of Engineers design memorandum for improvements to Indian Bend Wash and are shown in Table 2. Flood peaks on Indian Bend Wash and the Salt River were assumed to be independent.

Table 2 Discharge Frequency Values Indian Bend Wash at the Salt River	
Frequency (year)	**Existing Conditions (cfs)**
10	8,000
25	14,000
50	21,000
100	30,000
SPF	62,000

HYDRAULICS

EXISTING FLOOD HAZARDS

Prior to channelization, the 2,000-feet-wide Salt River 100-year floodplain inundated transportation and utility routes, and extensive urban areas. There are four bridges in the Rio Salado project area: the Southern Pacific Railroad bridge (c. 1885), the Mill Avenue bridge (c. 1930), the Rural Road bridge (c. 1983), and the McClintock Drive

bridge (c. 1984). The Mill Avenue bridge has an arched superstructure with massive wall-type piers. This type of bridge structure obstructs a large portion of the waterway, and causes it to be inundated during major flood events.

The Ash Avenue bridge (c. 1910) is shown in the channelization plans, but was recently demolished. However, a second Mill Avenue bridge is planned for construction just upstream of the existing Mill Avenue bridge. The new bridge will have architecture similar to the existing bridge. Preliminary analysis indicates that the new Mill Avenue bridge will impact the backwater profile similar to the demolished Ash Avenue bridge.

The existing bridges limit the hydraulic capacity of this reach of the Salt River. However, the channelization projects now under construction include features designed to increase conveyance through the bridges as well as alleviate other existing flood hazards.

CHANNELIZATION PLANS

The primary goals of the Rio Salado channelization are to construct a floodway capable of conveying a large magnitude flood event in excess of the 100-year flood, and to stabilize the channel bed and banks. Channelization features include levee bank protection, grade control and drop structures, designed to prevent excessive erosion or changes in the channel alignment.

Stage I channelization downstream of the Southern Pacific Railroad (SPRR) bridge, was designed according to FCDMC criteria. The channel is designed to safely convey a flow of 215,000 cfs with 4 feet of freeboard and a maximum flow of 289,000 cfs with one foot of freeboard.

Stage II channelization upstream of the SPRR bridge was designed according to different FCDMC criteria. The four bridges above the SPRR bridge limit the conveyance of this reach of the river. Therefore the FCDMC required that the channel convey the maximum capacity of the smallest existing bridge openings, i.e., the Rural Road bridge. The Rural Road bridge capacity was maximized by lowering the elevation of Grade Control Structure No. 4 three feet. This increased the hydraulic gradient at the Rural Road bridge and increased the bridge capacity to 250,000 cfs. Therefore, the operational channel design was based on 215,000 cfs with 4 feet of freeboard, or a maximum bridge capacity of 250,000 cfs with one foot of freeboard.

SEDIMENTATION

Analysis of the affects of Salt River sedimentation is based on information supplied by the City of Tempe, and assumes that the Rio Salado channelization is completed. The FCDMC is preparing a sediment transport master plan for the Salt River. This study will provide more complete data regarding sedimentation.

The Salt River in Tempe has undergone significant rapid change during the past 30 years. Recent channelization has narrowed the floodplain and increased conveyance. The bed elevation has dropped 10 to 15 feet since the 1960s. Because of this degradation the bed has become armored with cobble-sized sediment. Construction of numerous bridges and grade control structures has also altered the natural regime of the river. Natural habitat and vegetation has largely been eliminated in the riverbed.

Sand and gravel mining has drastically altered sediment supply in the project reach and is responsible for much of the channel change. An estimated 20 million tons of sediment have been removed from the riverbed by in-channel sand and gravel mining. As a result, the river flows at a sediment deficit. The Rio Salado channelization will decrease downstream sediment supply by an additional 1 million tons over the life of the project. Several grade control structures are included in the channelization plans in order to mitigate the affects of the sediment deficit on adjacent reaches.

The Rio Salado project will have a minimal impact on downstream reaches. The dam will be designed to pass sediment for at least the lower recurrence interval floods. Any potential deficit caused by the dam would be small compared to the sediment deficit created by in-stream mining and channelization. Furthermore, grade control structures, in addition to the grade control provided by the dam itself, will mitigate slope changes initiated by the project.

Project features subjected to flooding should be designed to withstand scour. General scour depths of 6 to 14 feet are predicted in the project area. Pier scour depths of 12 to 22 feet are possible.

Performance of specific alternatives with respect to sedimentation are discussed in the Design Alternatives section.

AGENCY REQUIREMENTS AND CRITERIA

The proposed dam and channel modifications must not significantly limit the ability of the channel improvements to function as a flood control facility, in accordance with design criteria set forth by several regulatory agencies. Coordination meetings with regulatory agencies were held to discuss specific design criteria, as outlined below.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY (FCDMC)

The FCDMC has regulatory authority and maintenance responsibility for the Salt River in the project area. The FCDMC has stipulated several design requirements. First, the FCDMC has stated that the Rio Salado project channel improvements and proposed dam should not reduce the capacity of the channel to less than 250,000 cfs with one foot of freeboard. Second, the proposed project also should not affect the capacity or maintenance requirements of the Indian Bend outfall structure. Third, the FCDMC also requires access to the toe of the levee bank protection in the lake reach for inspection after major flood events. This inspection is required before the dam can be raised and the lake refilled. Finally, the FCDMC would prefer the normal pool elevation of the lake to be about one foot below the top of the cement stabilized alluvium terrace. FCDMC will require submittal of the dam design for their review. The FCDMC's normal review period is 4 weeks.

ARIZONA DEPARTMENT OF WATER RESOURCES (ADWR)

ADWR's Division of Dam Safety has jurisdiction over dams that are either 25 feet or more in height or that store more than 50 acre-feet. The proposed Rio Salado dam will be 13 to 16 feet high and store approximately 2,400 acre-feet, and will therefore be a jurisdictional dam.

ADWR categorizes dams into low, significant, and high hazard potential classifications and into small, medium, and large dams within each hazard potential classification. The Rio Salado dam could be placed in any of the hazard potential classifications depending on the interpretation of the amount of urban development downstream of the dam and the potential for economic loss in the event of a dam failure. Within any of the hazard potential classifications, it will be a small dam.

Based on this preliminary evaluation, ADWR has indicated certain potential design criteria for the dam. First, the inflow design storm could be less than the Probable Maximum Flood (PMF) and could possibly be the 100-year flood event. Second, a fully dynamic modeling of dam performance during different operational and failure

scenarios may not be necessary if a simpler model gives satisfactory results. Third, a formal application for an ADWR permit is required. Finally, ADWR recommends that they be kept informed of the progress of the design of the dam before a formal permit application is filed.

Formal application to ADWR's Division of Dam Safety will include submittal of plans and specifications, a design report, and an application fee based on the total cost of the dam. The permit review process could take two to six months from the date of formal application. Construction may not begin without written approval of the Director of ADWR. ADWR will make periodic inspections of the project during construction. After construction is complete, the application fee will be recomputed using the final total cost of the dam with any overpayment refunded to the owner, or any underpayment paid to ADWR. Total cost of the dam includes materials and construction costs plus engineering fees.

ADWR will perform annual safety inspections of the dam for the first few years after construction and then at less frequent intervals as conditions warrant. If the lake is emptied, ADWR needs to be notified so they can inspect the dam.

OTHER AGENCIES

The U.S. Army Corps of Engineers and the Arizona Department of Transportation were also contacted regarding design criteria and review requirements. While these agencies will review aspects of the Rio Salado project, their review will not specifically address design of the dam as discussed in this technical memorandum. Specifically, requirements for a Corps of Engineers Section 404 permit were discussed in Technical Memorandum 2.

DAM DESIGN ALTERNATIVES

DAM AND GATE ALTERNATIVES

The potential dam and gate alternatives that were considered for the Rio Salado project include three basic configurations as follows:

- Channel Gates
- Fixed Weirs
- Fuse Plugs

The alternatives considered for each basic configuration include tainter gates; bascule or bottom-hinged leaf gates; inflatable dams, both water- and air-filled; ogee crest weirs; labyrinth weirs; and fuse plug configurations with sections set at different blowout elevations and with mechanical gates for passing low frequency flows. In addition, two alternate locations for the dam were evaluated.

Several alternatives were eliminated after preliminary evaluation. Tainter gates were not selected for more detailed consideration because in order to have a 23-foot clear channel, the 16-foot-high gates would have to be mounted on a superstructure that would be significantly larger than for any other gated option. Water-filled inflatable dams were eliminated from further consideration because large pumps are required to inflate the dams. Furthermore, the fail-safe deflation of water-filled dams requires more extensive piping and a charged siphon which requires significantly longer deflation times than that required by air-filled dams. Labyrinth weirs were eliminated after preliminary consideration because they would be required to operate in a submerged condition. When submerged, labyrinth weirs act as conventional broad-crested weirs, and would require a channel width of about 2,000 feet.

CHANNEL GATES

Channel gates would be installed across the entire channel width. Three different options were considered: a 16-foot-high inflatable dam; a 13-foot-high inflatable dam on a 3-foot-high concrete sill; and bottom-hinged leaf gates. Each option is discussed in detail below.

16-Foot-High Inflatable Dam

This option consists of a 16-foot-high inflatable dam mounted on a grade control structure. The dam would exceed the height of the tallest existing inflatable dam in the United States by 3 feet. The dam was evaluated assuming it consisted of four 210-foot-long dam segments, with three intermediate piers. Each pier would be 18 feet high, and have a 5 foot top width and 1:1 side slopes. The dam configuration is shown in

Figure 1. Each dam segment should be independently operable to allow flexibility for low flow and sediment passage and to be able to exercise each segment for maintenance checks. The dam could be overtopped by about 6 feet without inducing instability or exceeding resistance to overturning forces during the 5-year frequency flow without lowering the dam.

For this conceptual evaluation, the inflatable dam is assumed to be founded on a reinforced concrete slab, 16 inches thick, and 30 feet wide anchored to an existing grade control structure. Because the top width of Grade Control Structure No. 4 is approximately 8 feet, considerable modification of the grade control structure would be required to provide an adequate foundation. Geotechnical options include constructing a wider concrete or appropriate RCC foundation adjacent to the existing structure, or founding the 30-foot-wide slab partly on compacted alluvium and providing a seepage cutoff wall to prevent undermining of the slab. Some densification of the soil may be required to provide a uniform foundation. Anchor bolts would be used to connect the new concrete slab to the existing grade control structure.

The inflatable dam will have three intermediate reinforced concrete piers. The piers may be founded on the existing grade control structure or on the 16-inch slab. They will have to be anchored to provide stability against overturning. No widening of the existing channel is anticipated for this option.

The hydraulic impacts of the dam and intermediate piers were modeled using the Corps of Engineers HEC-2 backwater model developed by CRSS Commercial Group Inc. for the Rio Salado Channelization Study. The HEC-2 model indicates that with the dams fully deflated, the water surface elevation at the dam site is increased less than 0.1 feet. At the Southern Pacific Railroad bridge, located 200 feet upstream of the proposed dam site, the water surface elevation is decreased by 0.1 feet. The backwater profile was essentially unchanged at the Rural Road bridge.

Some of the key advantages and disadvantages of this option relative to fixed weir and fuse plug options are outlined below. Quantitative evaluation of this option relative to other design options is summarized in Table 7.

Advantages:

- Dam should perform well during anticipated flood events
- Dam backwater affects minimal
- Construction and design not complex because:
  - Long spans require fewer piers
  - Partial foundation exists
- Operations and Maintenance less complicated because:
  - Dam deflates without electrical power
  - Rubber material withstands sand erosion in high velocity flooding
  - Less sediment trapped by dam

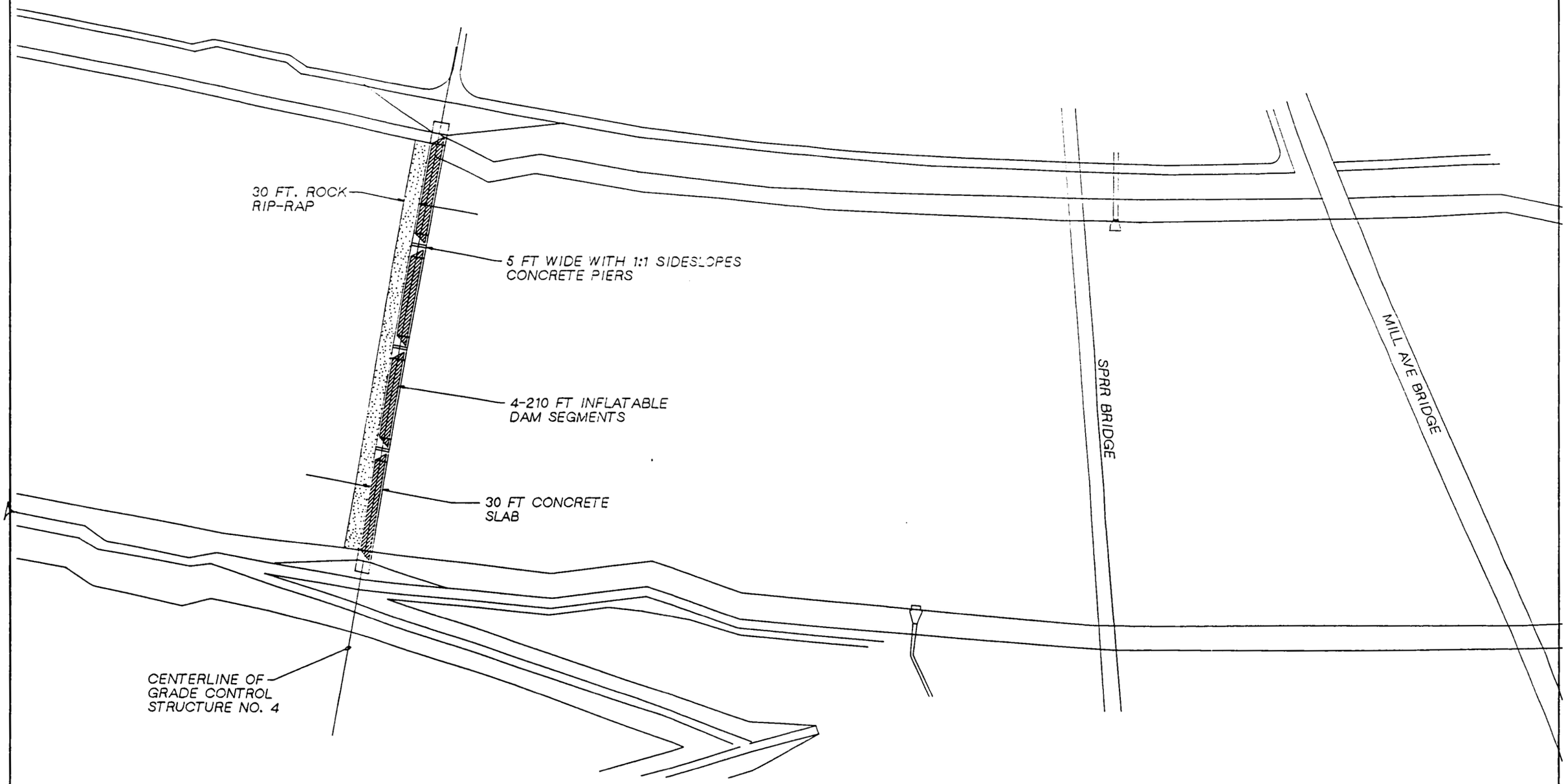

FIGURE 1
RIO SALADO
CHANNEL GATE ALTERNATIVE
INFLATABLE DAM OPTION

PHX31608.FS

Disadvantages:

- Long material delivery time for rubber dam (7 to 12 months)
- Untested design parameters, including:
  - Dam height exceeds tallest U.S. installation
  - Project design life of rubber bag unproven
  - Some potential for vandalism by knife cuts

13-Foot-High Inflatable Dam on 3-foot Sill

This option consists of a 13-foot-high inflatable dam on top of a 3-foot sill to achieve the total desired height of 16 feet. At 13 feet, the installation would equal the tallest inflatable dam in the United States. To build the 3-foot-high sill, substantial modifications to the existing grade control structure would be required including widening the structure to anchor the inflatable dam to a 30-foot-wide, 16-inch-thick slab.

Geotechnical considerations for the 13-foot dam are similar to the 16-foot dam. The top of the grade control structure would have to be widened to provide an adequate base for the 30-foot-wide sill. However, due to the mass of the concrete sill below the inflatable dam, this option is more stable than the 16-foot-high inflatable dam.

The HEC-2 model used previously to evaluate the backwater effects of the 16-foot-high inflatable dam was modified to include a 3-foot higher bottom elevation through the dam section. The model was run using a flowrate of 250,000 cfs with the dam fully deflated. The HEC-2 model indicates that 200 feet upstream of the proposed dam site, the dam causes an increase in the water surface elevation of about 0.7 feet; an increase of 0.2 feet at the Southern Pacific Railroad bridge; and an increase of less than 0.1 feet at the Rural Road bridge.

Some the key advantages and disadvantages of this option relative to the 16-foot inflatable dam are outlined below. Relative to the fixed weir and fuse plug options, the advantages of the 13-foot inflatable dam are comparable to the 16-foot dam. Quantitative evaluation of this option relative to other design alternatives is summarized in Table 7.

Advantages:

- Height of inflatable bag field-tested in U.S.
- More stable than 16-foot dam

Disadvantages:

- Required 3-foot sill causes:
  - More extensive foundation than 16-foot dam
  - Sediment trapping in lake

- Increase backwater affect upstream

Bottom-Hinged Leaf Gates

This option consists of placing seventeen 50-foot-wide bottom-hinged leaf gates with 5-foot-thick intermediate piers across the entire channel width. The existing channel would need to be widened about 100 feet to reduce the head loss resulting from the width of the gates and piers. This would require major modification of the completed bank protection. The dam configuration is shown in Figure 2.

The hydraulic cylinders used to raise and lower the leaf gates are normally installed in the bed of the channel. This enables intermediate cylinders to be placed between the piers to increase the gate width. However, the sediment load in the Salt River requires that the hydraulic cylinders be placed above the gates on the intermediate piers. This reduces the maximum gate width to 50 feet.

Geotechnical design parameters include anchoring and providing foundations for hydraulic structures. The existing grade control structure base will need to be widened to accommodate the concrete slab under the leaf gates. Alternatively, soil under the concrete slab could be densified prior to placement of the slab. A slab approximately 30 feet wide would be required. The piers supporting the gates must be properly anchored to the existing grade control structure or the concrete slab to provide stability against overturning and settling. The gates could leak water if the foundation settles. Reinforced concrete piers must be constructed at both abutments of the dam.

The gate openings, intermediate piers, and channel widening were coded into the HEC-2 model. The model was run using a flowrate of 250,000 cfs with the gate lowered into the full open position. The channel widening downstream of the proposed dam caused an increase in the water surface elevation of about 0.4 feet at the downstream face of the dam. Two hundred feet upstream the dam caused an increase in the water surface elevation of about 1.0 feet; an increase of 0.3 feet at the Southern Pacific Railroad bridge; and an increase of 0.1 feet at the Rural Road Bridge. Two hundred feet upstream of the proposed dam, the water surface was approximately 1.5 feet below the top of the levee. Therefore, this dam configuration has more than the required one foot of freeboard at the 250,000 cfs flowrate.

Some the key advantages and disadvantages of this option are outlined below. Quantitative evaluation of this option relative to other design alternatives is summarized in Table 6.

Advantages:

- Field-tested gate technology is:
  - Less complex to design than many of the options considered
  - Familiar to review agencies

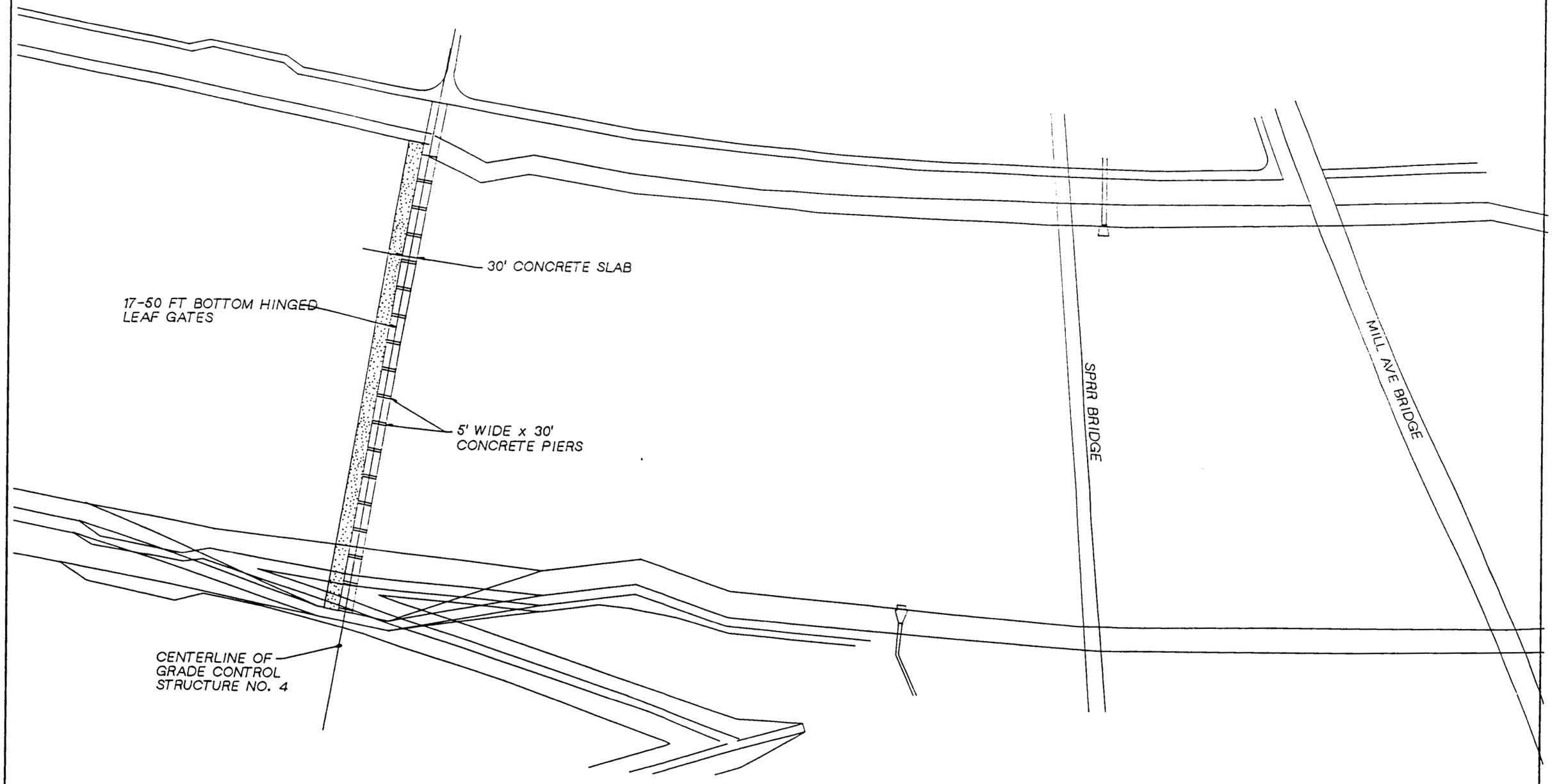

FIGURE 2
RIO SALADO
CHANNEL GATE ALTERNATIVE

BOTTOM HINGED LEAF GATE OPTION

PHX31608.FS

CH2MHILL

- Gates withstand flooding
- Dam backwater affects acceptable

Disadvantages:

- Construction complicated by:
  - Numerous required intermediate piers
  - Required channel widening
  - Long delivery time for gates (8 months)
  - Extensive foundation requirements
- Operations and maintenance complicated by:
  - Rigid gates damaged by sand abrasion in floods
  - Debris accumulation can prevent lowering of gates
  - Foundation settlement may cause gate leakage

FIXED WEIRS

Several configurations of fixed weirs were considered. Since the available head across the weir would be small, the weir length needed to pass the design flowrate of 250,000 cfs would be large. This weir length eliminates the possibility of placing a fixed weir across the channel perpendicular to the flow direction. Therefore, side channel weirs were considered in combination with gates and fuse plugs to keep the dam size within the limits of the existing channel. The fixed weir configurations evaluated cannot be constructed at the western dam site because of its proximity to the Priest Avenue bridge.

Side Channel Spillway with Leaf Gates

This option consists of two side channel weirs extending downstream on each side of six 50-foot-wide bottom-hinged leaf gates placed in the center of the channel (Figure 3). Each side channel weir has an ogee-shaped crest and is 1,070 feet long. The side channel entrances at the leaf gates are 335 feet wide. The leaf gates are sized to pass the Salt River 5-year flow of 44,000 cfs flowing 16 feet deep. The weir is sized to pass the remaining flow that does not pass through the gates with a maximum upstream water surface equal to the top of levee elevation minus one foot of freeboard.

No detailed subsurface information was found on which to evaluate the foundation depth required for the side channel weirs. Substantial erosion protection will be required at the base of the weirs in the downstream channel to ensure the stability of the structure. The leaf gates will be supported by a 30-foot-wide reinforced concrete slab as described previously. The piers for the gates could be founded on the modified grade control structure as described previously.

The HEC-2 computer program cannot directly model the hydraulics of the side weir configuration. Therefore, to evaluate the potential backwater affects of the dam, a

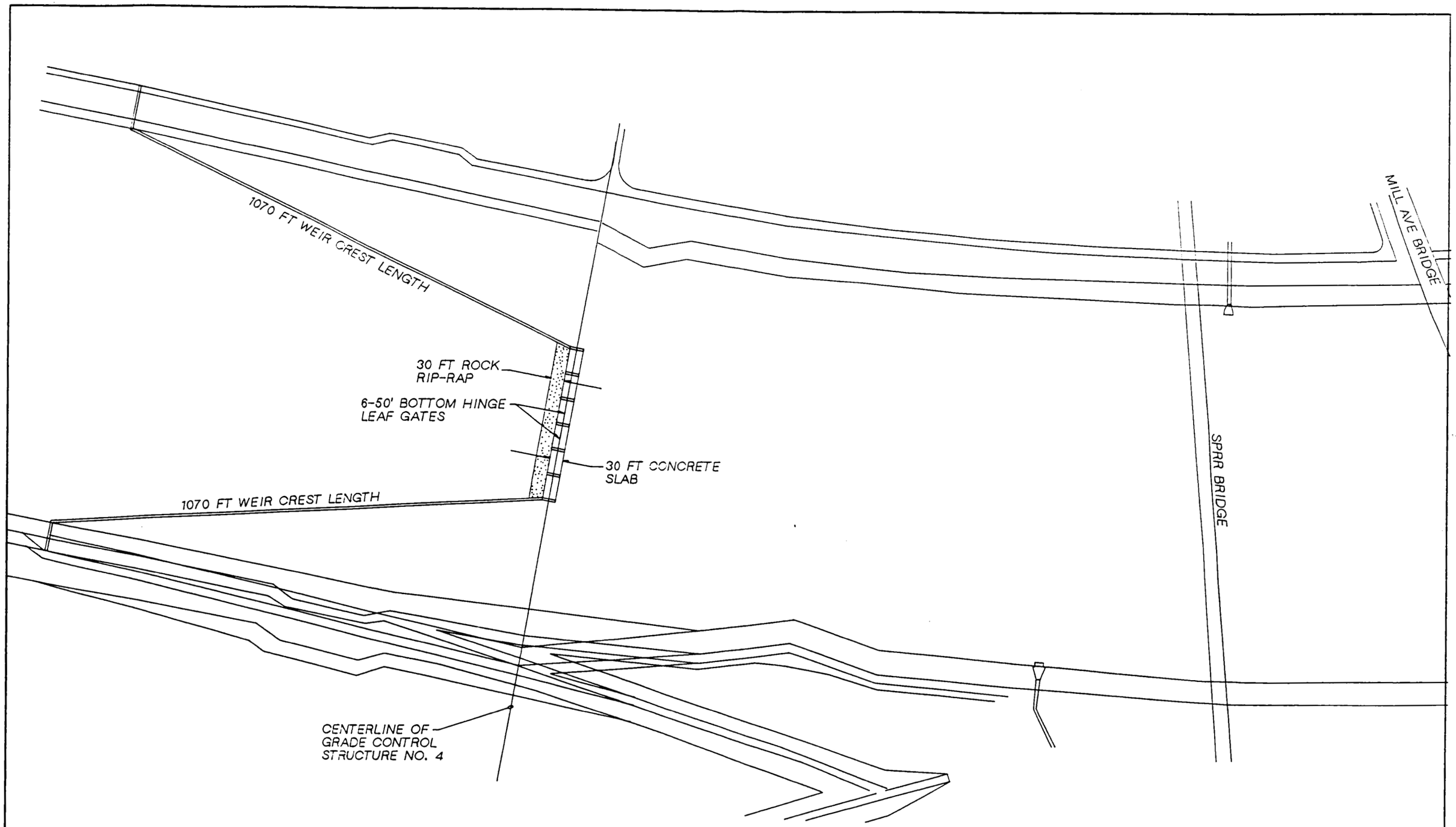

FIGURE 3
RIO SALADO
FIXED WEIR ALTERNATIVE

SIDE CHANNEL SPILLWAY
WITH LEAF GATES OPTION

PHX31608.FS

CH2M HILL

HEC-2 profile was computed using a starting water surface of one foot below the top of levee. This starting elevation corresponds to the maximum water surface elevation over the weir at 250,000 cfs. The HEC-2 model indicated a water surface elevation increase of about 1.4 feet two hundred feet upstream of the dam; an increase of 0.8 feet at the Southern Pacific Railroad bridge; and an increase of 0.4 feet at the Rural Road bridge. The increase in water surface elevation at the Rural Road bridge would probably impinge on the low chord of the bridge, which is unacceptable to the FCDMC. To properly evaluate the performance of the side channel spillway and gates, a physical model would need to be built and tested. This modeling and testing may produce information necessary to reduce the head required over the weir enough to alleviate the unacceptable backwater effects at the Rural Road bridge.

Some the key advantages and disadvantages of this option outlined below. Quantitative evaluation of this option relative to other design alternatives is summarized in Table 7.

Advantages:

- Weirs withstand overtopping
- Aesthetic opportunities include:
  - Increased lake frontage vs. inflatable dams
  - Weirs may be fitted with waterfalls or other features
  - Weirs may be constructed with native materials

Disadvantages:

- Construction and design complicated by:
  - Backwater affects may exceed conveyance limits
  - Physical modeling of weir required
  - Extensive foundation required
  - Cannot be constructed at western site due to size and right-of-way construction
- Operations and maintenance complicated by:
  - Sediment trapped in side channels

Side Channel Spillway with Fuse Plug

This option is the same as the Side Channel Spillway with Leaf Gate option except the bottom-hinged leaf gates are replaced with a fuse plug. The weir would pass all low frequency flood flows up to and including the 100-year discharge of 160,000 cfs. The fuse plug would blow out for flows in excess of the 100-year flow. The weirs would be designed to pass 250,000 cfs with one foot of freeboard. The backwater effects at the bridges would be the same as for the previous option. This option would also require physical modeling to accurately evaluate its performance.

The fuse plug will be approximately 100 feet wide at the base and will have a crest width of approximately 10 feet. The material for the fuse plug embankment must be

granular, but have enough cohesion so that it is relatively impermeable when the lake is full. A silty sandy core would likely be used with drainage and filter zones. Protection of the upstream slope of the fuse plug could be provided by a layer of gunite. Three-inch minus aggregate could be raked into the gunite as it is placed to provide energy dissipation for waves in the reservoir. Other wave protection materials are also possible.

The earthfill fuse plug is founded partially on the 8-foot-wide existing grade control structure. Some densification of the soil under the fuse plug may be required to provide a uniform foundation. Reinforced concrete piers with an upstream wing wall would be required at both abutments to protect the fuse plug. The piers must extend the full width of the fuse plug. Densification of soil under the piers will be required to proved a uniform foundation for the concrete. Geotechnical considerations for the side channel weir described previously apply to this option.

Some the key advantages and disadvantages of this option relative to the side channel with leaf gate alternative are outlined below. Comparison of the side channel spillway with fuse plug option with channel gate and fuse plug alternatives is similar to the side channel spillway with leaf gates. Quantitative evaluation of this option relative to other design alternatives is summarized in Table 7.

Advantages:

- Extends lake frontage

Disadvantages:

- Design is more complicated than the leaf gate option because:
  - Review agencies are less familiar with fuse plugs
  - Physical modeling required to assess hydraulic performance
  - Unacceptable backwater affects
  - Cannot be constructed at western site
- Operations and maintenance more complex because:
  - Fuse plugs wash out in 100-year flood or greater
  - Fuse failure may deposit sediment in channel downstream
  - Major reconstruction required after fuse plug blowout
  - Requires periodic removal of sediment trapped in lake

FUSE PLUGS

A fuse plug dam is an earthen structure designed to be stable until overtopped by flooding, then to fail (blow out) in a predictable manner. After blow out, the structure would not obstruct passage of extreme floods. The structure would then be rebuilt following the flood event.

Two options were considered using fuse plugs as the major component of the dam:

- Fuse plug with leaf gates
- Segmented fuse plug with different blow-out elevations

These options are discussed in more detail below.

Fuse Plug with Leaf Gates

This option consists of nine 50-foot-wide bottom-hinged leaf gates with 5-foot-wide intermediate piers and a 550-foot-long fuse plug. The leaf gates were sized to pass the Salt River 20-year flowrate of 110,000 cfs. The fuse plug would blow out for flows in excess of 110,000 cfs. The channel would have to be widened about 200 feet to accommodate the gates and fuse plug. The dam configuration is shown in Figure 4.

As described previously, the existing grade control structure would be widened to accommodate the 30-foot-wide concrete slab which would support the leaf gates. In addition, the same considerations described for the piers previously also apply to this option. Settlement of the leaf gate foundation may cause leakage, and periodic adjustment of the seals may be required. Finally, a reinforced concrete pier will be constructed to serve as the south dam abutment where the channel is widened.

The south side of the dam will consists of an earth fill fuse plug, similar to that described previously. A reinforced concrete pier with a wing wall is needed between the southernmost leaf gate pier and the earthfill fuse plug to prevent erosion of the fuse plug. Soil must be densified prior to construction of the wing wall to provide uniform foundation conditions. The considerations given for the previous fuse plug option also apply to this option.

The fuse plug and gates were coded into the HEC-2 model. The model was run using a flowrate of 250,000 cfs and with the gates down in the full open position. The fuse plug was assumed to be completely washed out so the model reflected a clear channel. The dam caused an increase in the water surface elevation of 0.7 feet about 200 feet upstream of the dam, but only a negligible increase at the Southern Pacific Railroad bridge and the Rural Road bridge.

Some of the key advantages and disadvantages of this option are outlined below. Quantitative evaluation of this option relative to other design alternatives is summarized in Table 7.

Advantages:

- Dam backwater affects acceptable
- Sediment not trapped in lake during design flood

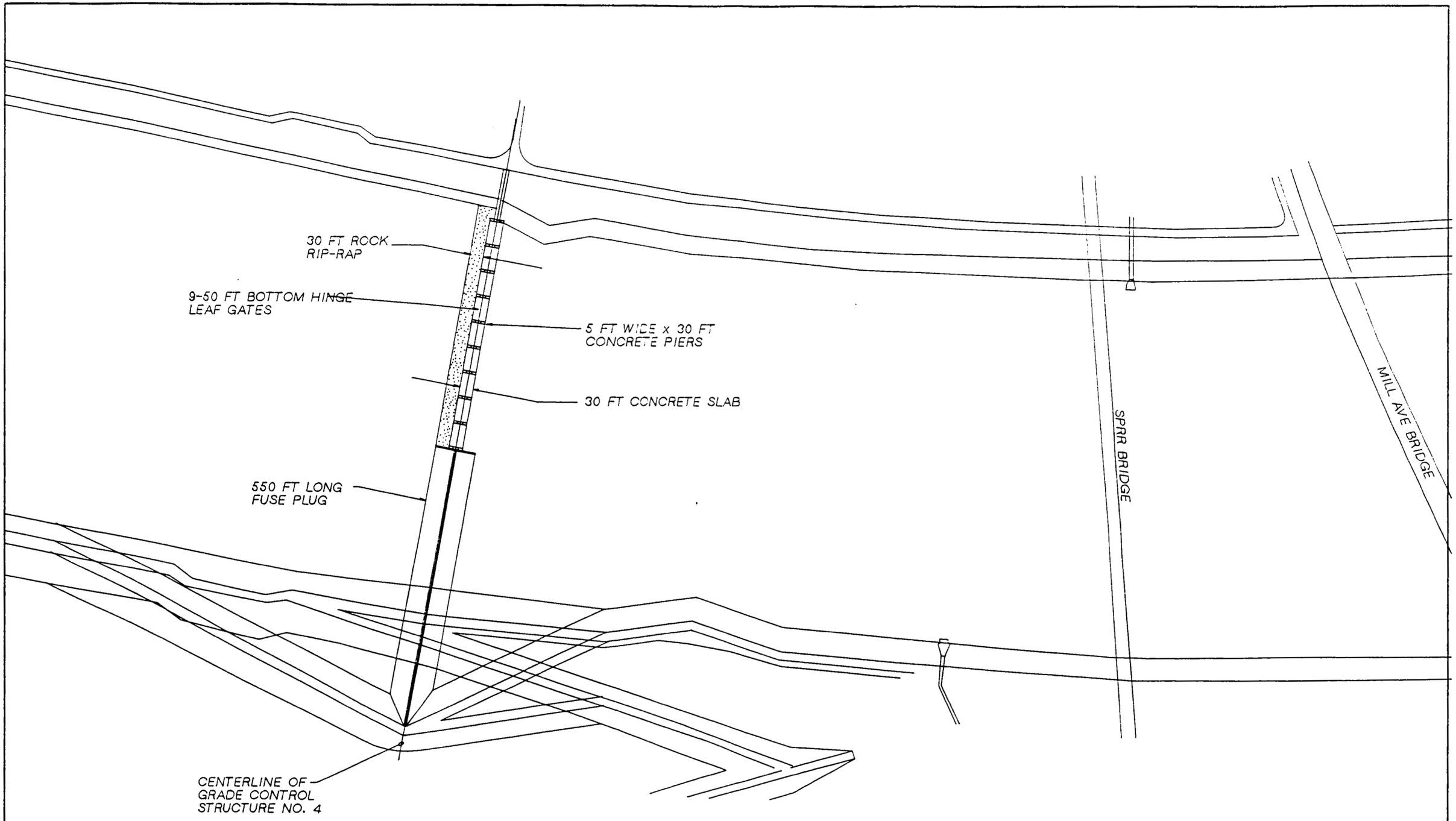

FIGURE 4
RIO SALADO
FUSE PLUG ALTERNATIVE

FUSE PLUG WITH LEAF GATE OPTION

PHX31608.FS

Disadvantages:

- Cost of construction high
- Operations and maintenance complicated by:
  - Fuse plug reconstruction required after blowout
  - Sediment from plug blowout deposited in channel downstream
- Design and construction complex due to:
  - Required channel widening
  - Regulatory agencies unfamiliar with fuse plugs
  - Extensive foundation work required
- Fuse plug blowout perceived as failed dam
- Lake would be empty during reconstruction

Segmented Fuse Plug

This option consisted of a segmented fuse plug with four 50-foot gates to pass the Salt River 5-year flow of 44,000 cfs. The fuse plug would consist of three 200-foot-long segments set to blow out at increments of 40,000 cfs. The fuse plugs would be completely removed at the 100-year flowrate of 160,000 cfs. The last two fuse plug segments must be designed to blow out with high tailwater in the downstream channel. The dam configuration is shown in Figure 5. The dam's backwater effect at 250,000 cfs would be small because of the clear channel in the fuse plug section of the dam and only four leaf gates in the channel.

Geotechnical considerations for this option are similar to the fuse plug with leaf gates option, except the fuse plug is divided into three segments, and the leaf gate section is shorter. The same considerations described previously for the fuse plug and leaf gate options apply here. In addition, a reinforced concrete pier with a wing wall parallel to the dam axis must be placed between each fuse plug segment to prevent erosion of the second and third fuse plug after the first plug has eroded.

Some the key advantages and disadvantages of this option relative to the fuse plug with leaf gate option are outlined below. Relative to channel gate and fixed weir alternatives, the segment fuse plug is comparable to the fuse plug with leaf gates. Quantitative evaluation of this option relative to other design alternatives is summarized in Table 7.

Advantages:

- Dam backwater effects acceptable

Disadvantages:

- Operations and maintenance more complicated because:
  - Fuse plug segments blow out more frequently
  - Fuse plug blow outs require major reconstruction

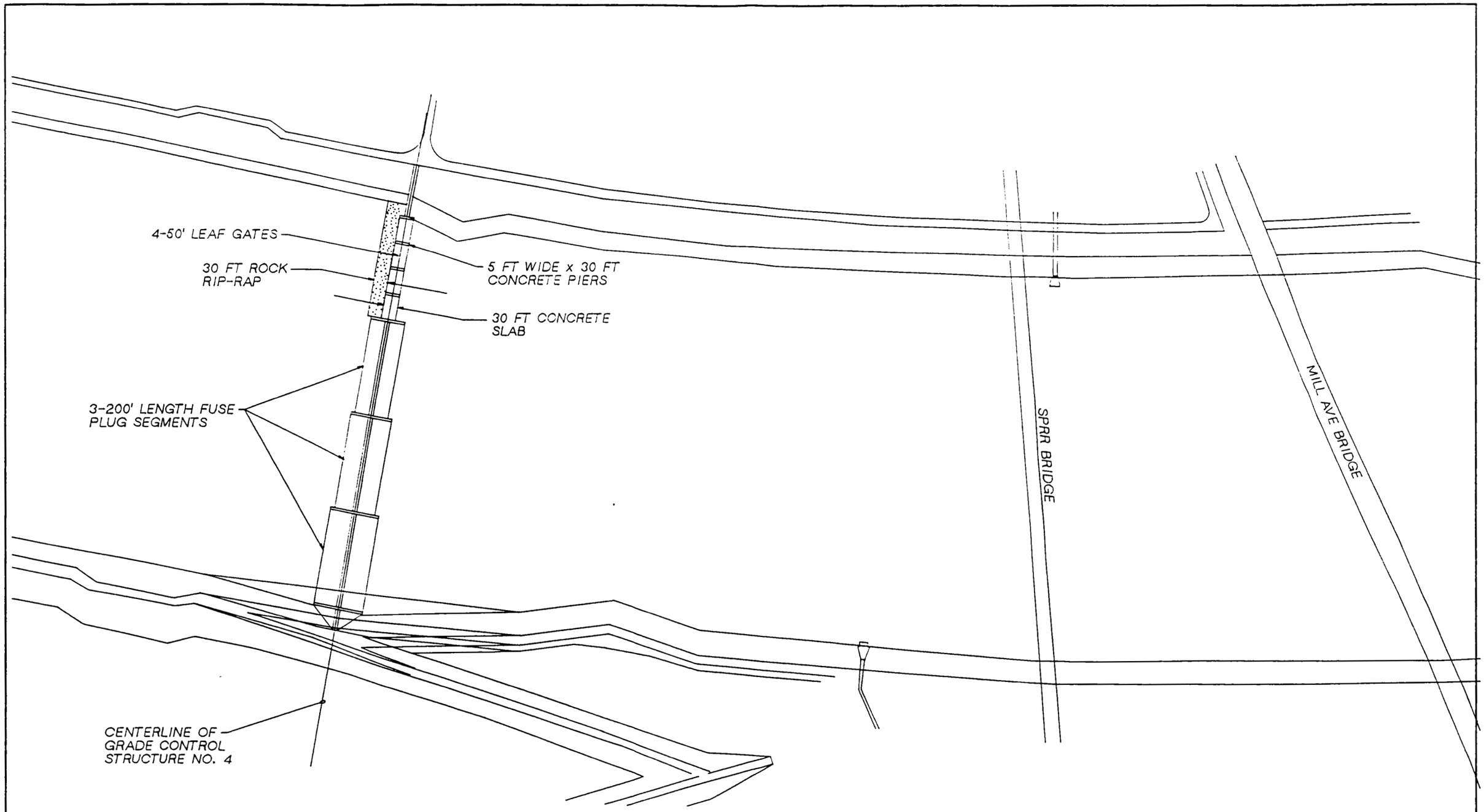

FIGURE 5
RIO SALADO
FUSE PLUG ALTERNATIVE

SEGMENTED FUSE PLUG OPTION

PHX31608.FS

- Design is more complex because:
  - Last two segments must blow out at high tailwater
- Public perception of "dam failure" during blow out
- Lake would be empty during reconstruction

INFLOW CONTROL ALTERNATIVES

The water quality of runoff from the Price Road Drain and Indian Bend Wash may not be compatible with water quality requirements for the lake. Sediment from the Salt River during smaller, more frequent flows may also have an unacceptable impact on the lake. Therefore, two options were developed to restrict water and sediment supplied to the lake. The options include a channel dam at McClintock Drive and a diversion dike within the lake. These options are described below.

CHANNEL DAM AT McCLINTOCK DRIVE

This option places a 5.5-foot-high inflatable dam on the upstream side of the McClintock drop structure. The inflatable dam will have three 275-foot-wide segments with intermediate piers. The lake impounded by this dam could be used to store and infiltrate the flood flows out of the Price Road Drain and keep the flows from entering the lake. The dam will also impound and store 3.3 days of sediment load from the Salt River during the 5-year flow of 44,000 cfs. These sediments could be removed from the channel after flow ceases. This option will not affect flows from Indian Bend Wash unless the western dam site is selected.

DIVERSION DIKE

This option consists of a cement- stabilized alluvium dike placed on the northern side of the channel from the proposed dam site to just upstream of the Indian Bend Wash confluence. The dike crest would be 3 feet below the normal lake elevation. The dike would be about 100 feet from the north bank and 13 feet high at the dam site and about 650 feet from the north bank and 4 feet high at the Indian Bend Wash confluence. The dike would feather into the south bank just upstream of the Indian Bend Wash confluence. The dike would direct the 10-year flowrate of 8,000 cfs from Indian Bend Wash around the main body of the lake and out of one or several of the end gates in the dam. The lake level would be lowered 3 feet while this occurred. The sediment load in the Salt River would be directed into the channel formed by the diversion dike and the north bank, and exit out the dam gates. If the lake is to be used for boating, the submerged dike would present a hazard for boats and would need to be marked and restricted for access.

ALTERNATIVE DAM LOCATIONS

Two alternate dam sites were considered. The eastern site, originally proposed by the City of Tempe, is located approximately 500 feet downstream of the Mill Avenue bridge. The western dam site is approximately 600 feet upstream of the Priest Avenue bridge. Hydraulic performance of each of the design alternatives would differ slightly at the western location. The configurations of the alternatives were assumed to be the same at both sites except as noted. The advantages and disadvantages of moving the dam to the western site closer to Priest Avenue are outlined below.

Advantages:

- Bedrock shallower at western site, therefore:
  - Foundations conditions better
  - Groundwater control easier
- Dam backwater effects may dissipate below bridge reach
- Upstream dam can be positioned to:
  - Mitigate Price Drain/Indian Bend Wash water quality
  - Mitigate Price Drain/Indian Bend Wash sedimentation
- Reduces utility conflicts to allow for more gradual channel transition near dam

Disadvantages:

- No existing grade control structure for dam foundation
- Lake will only be one foot deep at McClintock Drive

GEOTECHNICAL CONSIDERATIONS

The geotechnical evaluation is based on reports supplied by the City of Tempe, and assumes that the Rio Salado channelization is completed as designed. Depth to bedrock information is based information prepared for Technical Memorandum 6. The following geotechnical considerations were analyzed to evaluate alternative dam configurations:

- Subsurface conditions, including:
  - Depth to bedrock
  - Foundation conditions
- Design parameters, including:
  - Stability of the dam
  - Condition of existing grade control structure
  - Seepage under the dam
  - Depth of scour
  - Downstream erosion protection
  - Required channel widening, if necessary

- Construction issues, such as:
  - Pumping or diversion of water during construction

Depth to bedrock appears to be approximately 40 to 60 feet in the vicinity of Grade Control Structure No. 4, the location of the eastern (original) dam site. Depth to bedrock is slightly less at the western site (Priest Road). Based on borings drilled in the channel, it is anticipated that the soil overlying bedrock will be typical alluvial deposits and will consist primarily of sandy gravel with some cobbles. Layers of sand and silt are also present. Soil borings should be performed at the location of the proposed dam to confirm the depth to bedrock and to adequately characterize subsurface conditions.

Several design parameters should be evaluated in more detail prior to final design. First, Grade Control Structure No. 4 should be evaluated using boring and/or coring techniques. Second, because the cement-stabilized alluvium is very new, its ability to support the dam abutments and cut off lateral seepage should be studied. Third, because channel excavations will be required to construct any of the project alterations, pumping or diversion of surface or groundwater may be a problem during construction. Finally, scour protection should be provided for a length of approximately 30 feet downstream of the proposed dam depending on scour depth and design velocities.

EVALUATION OF ALTERNATIVES

Each alternative was evaluated on the basis of probable construction cost and other advantages and disadvantages as discussed in the following paragraphs. These factors are presented in a decision matrix at the end of this section.

CONSTRUCTION COST OPINIONS

Construction cost opinions were determined from an estimate of quantities of materials, manufacturer's recommendations and cost of materials, discussions with contractors with dam construction experience, and CH2M HILL's database on local and nationwide costs of materials and construction. These cost opinions are order-of-magnitude estimates to be used to compare the various alternatives and not to set actual construction budgets. The estimates of probable construction cost for each alternative are shown in Table 3. Detailed cost opinions are provided in Appendix A.

Table 3
Construction Cost Opinions

Alternative	Cost
Channel Gates	
Inflatable Dam--16 ft. high	$6,280,000
Inflatable Dam--13 ft. high with 3 ft. sill	$5,450,000
Bottom-Hinged Leaf Gates	$12,000,000
Fixed Weirs	
Side Channel Weir with Leaf Gates	$8,280,000
Side Channel Weir with Fuse Plug	$4,560,000
Fuse Plug	
Fuse Plug with Leaf Gates	$7,120,000
Segmented Fuse Plug	$3,500,000
Conveyance Alternatives	
Channel Dam at McClintock Drive	$1,950,000
Diversion Dike	$2,820,000

The options of cost shown, and any resulting conclusions on project financial or economic feasibility or funding requirements, have been prepared for guidance in project evaluation and implementation from the information available at the time the opinion was prepared. The final costs of the project and resulting feasibility will depend on actual labor and material costs, competitive market conditions, actual site conditions, final project scope, implementation schedule, continuity of personnel and engineering, and other variable factors. As a result, the final project costs will vary from the opinions of cost presented herein. Because of these factors, project feasibility, benefit/cost ratios, risks, and funding needs must be carefully reviewed prior to making specific financial decisions or establishing project budgets to help ensure proper project evaluation and adequate funding.

LIFE CYCLE COSTS

A cost analysis was performed for each dam and gate alternative to determine the operation, maintenance, rehabilitation, and sediment removal costs over the design life of the dam. Costs were estimated and converted to present worth for comparison. The analysis assumed a design life of 100 years and used the U.S. EPA Office of Municipal Pollution Control 1991 discount rate of 8-3/4 percent for water resources projects. Life cycle costs for the conveyance alternatives were not computed, but will be completed in Technical Memorandum 12. Results of the analysis are shown in Table 4. Detailed estimates of costs are provided in Appendix B.

Table 4
Life Cycle Cost Opinions

Alternative	Cost
Channel Gates	
Inflatable Dam--16 ft. high	$ 7,915,000
Inflatable Dam--13 ft. high with 3 ft. high sill	7,241,000
Bottom-Hinged Leaf Gates	14,499,000
Fixed Weirs	
Side Channel Spillway with Leaf Gates	9,903,000
Side Channel Spillway with Fuse Plug	8,016,000
Fuse Plug	
Fuse Plug with Leaf Gates	8,606,000
Segmented Fuse Plug	4,426,000

DECISION MATRIX

A decision matrix was developed to better evaluate the advantages and disadvantages and the tangible and intangible merits of each alternative. Evaluation parameters are identified and rated for their ability to satisfy the objectives for the project. The rating system is described below.

Each parameter was rated on a five-point scale ranging from favorable to unfavorable. A score of "5" was given if the rating was favorable and a score of "1" was given if the rating was unfavorable. The scale is listed in Table 5.

Table 5 Decision Matrix Rating System	
Rating	**Score**
Favorable	5
Slightly Favorable	4
Neutral	3
Slightly Unfavorable	2
Unfavorable	1

Each parameter was weighted according to its perceived importance to the project. The weighing system is shown in Table 6.

Table 6 Parameter Weighing System	
Weight	**Score**
Very Important	5
Important	4
Neutral	3
Slightly Important	2
Not Important	1

The score for an individual parameter is the product of the rating score and weighing score. The decision matrix is in Table 7.

As summarized in the decision matrix the channel gates alternatives are rated the highest. The 16-foot-high inflatable dam is the highest rated channel gate option. The 16-foot-high inflatable dam performs well during flooding, allows quick recovery of the lake after deflation, does not impact channel hydraulics or sedimentation, is relatively simple technology, and is not as expensive as other alternatives. However, inflatable dams are somewhat susceptible to damage by vandalism, and would require periodic monitoring and inspection.

No fatal flaws were determined for any of the alternatives evaluated. However, the fuse plug options are not as reliable from a geotechnical standpoint as other options. In addition, fuse plug blowouts require considerable construction cost to repair resulting in significant periods when the lake is empty. For these reasons the fuse plug options are not recommended for further consideration.

Table 7
Decision Matrix

Parameter	Parameter Weight	Channel Gates						Fixed Weirs				Fuse Plug			
		Inflatable Dam 16 ft. high		Inflatable Dam 13 ft. high with 3 ft high sill		Bottom Hinged Leaf Gates		Side Channel Spillway with Leaf Gates		Side Channel Spillway with Fuse Plug		Fuse Plug with Leaf Gates		Segmented Fuse Plug	
		Rating	Score	Rating	Score	Rating	Score	Rating	Score	Rating	Score	Rating	Score	Rating	Score
1. Total Cost	5	5	25	5	25	1	5	2	10	3	15	3	15	5	25
2. Aesthetics	4	4	16	4	16	4	16	3	12	3	12	2	8	2	8
3. Sediment Handling	5	5	25	2	10	4	20	1	5	1	5	5	25	4	20
4. Backwater Effects	5	5	25	3	15	3	15	1	5	1	5	4	20	5	25
5. Abrasion Resistance	3	3	9	3	9	2	6	4	12	4	12	3	9	4	12
6. Vandalism Potential	4	1	4	1	4	5	20	5	20	5	20	5	20	5	20
7. Use of Proven Technology	3	3	9	4	12	5	15	3	9	2	6	3	9	2	6
8. Foundation Requirements	4	5	20	4	16	2	8	2	8	2	8	3	12	3	12
9. Modifications to Existing Channel	4	5	20	5	20	1	4	4	16	4	16	1	4	5	20
10. Materials Delivery Time	2	1	2	1	2	2	4	3	6	3	6	3	6	3	6
11. Design Complexity	2	5	10	5	10	5	10	1	2	1	2	2	2	1	2
12. Review Agency Familiarity	2	2	4	2	4	5	10	3	6	2	4	1	2	1	2
13. Maintenance After Major Flood	5	5	25	5	25	5	25	5	25	3	15	2	10	1	5
14. Dam Life	4	2	8	2	8	4	16	5	20	3	12	4	16	2	8
15. Recovery of Lake after Flood	5	5	25	5	25	5	25	5	25	3	15	2	10	1	5
Total			227		201		199		181		153		168		176

BIBLIOGRAPHY

Arizona Department of Water Resources, Oral Communication from Bill Jenkins, Dam Safety Division and Terry Miller, Floodplain Section on April 30, 1991.

CRSS Commercial Group, Inc., Draft Final Report, *Salt River Hydraulic Design, Grade Control Structure No. 4 to McClintock Drive Bridge*, Prepared for City of Tempe, Dept. of Public Works, Tempe, AZ, April, 1990

Flood Control District of Maricopa County, Oral Communication from Dick Perrault on March 5, 1991, and from Dick Perrault and Dave Johnson on April 30, 1991.

Linsely, R.K. and Franzini, J.B., 1972. *Water Resources Engineering,* 2nd Ed., McGraw Hill Book Co., New York.

Thomas Hardick and Assoc., Chandler, Ariz., Geotechnical Investigation for the Salt River Bridge, East Papago Freeway - Section 6, July 31, 1990.

U.S. Army Corps of Engineers, Los Angeles District, *Draft Design Memorandum No. 1, General Design Memorandum Phase III, Project Design for Indian Bend Wash*, January, 1975.

U.S. Army Corps of Engineers, 1982, HEC-2 Water Surface Profiles, Users' Manual.

U.S. Army Corps of Engineers, Oral Communication from Cindy Lester on March 5, 1991.

United States Department of the Interior, Bureau of Reclamation, *Design of Small Dams*, Third Edition, 1987.

APPENDIX A

CONSTRUCTION COST ESTIMATES

INTRODUCTION

This section of the Technical Memorandum documents the qualifications of the presented cost, presented, as well as defines the type of estimates prepared and how the costs of the alternatives were developed. This section also evaluates the sensitivity of the variables that could impact the final cost of the alternatives.

The scope of the costs presented is estimated construction costs for the various design concepts identified by the project team. These concepts include alternative designs for the Rio Salado dam and stormwater management system.

OPINION OF COSTS

The opinions of costs shown, and any resulting conclusions on project financial or economic feasibility or funding requirements, have been prepared for guidance in project evaluation and implementation from the information available at the time the opinion was prepared. The final costs of the project and resulting feasibility will depend on actual labor and material costs, competitive market conditions, actual site conditions, final project scope, implementation schedule, continuity of personnel and engineering, and other variable factors. As a result, the final project costs will vary from the opinions of cost presented herein. Because of these factors, project feasibility, benefit/cost ratios, risks, and funding needs must be carefully reviewed prior to making specific financial decisions or establishing project budgets to help ensure proper project evaluation and adequate funding.

Where alternatives have been compared, only the relative cost differences were evaluated, not the total cost of each alternative. Therefore, unless otherwise indicated, the costs indicated do not necessarily represent the final cost of the project or individual alternatives and are not suitable for determining project feasibility or level of funding required.

BACKGROUND AND DEFINITIONS

The American National Standards Institute (ANSI) and the American Association of Cost Engineers (AACE) define a cost estimate as "an evaluation of all costs of the elements of a project or effort as defined by an agreed-upon scope." Cost estimating accuracy is dependent on an accurately defined project scope, which in turn depends on the amount of data available to characterize the site. As a project progresses and more data become available, the selected remedy can be better defined, thereby making it possible to increase the estimating accuracy. The ANSI and AACE list three levels of cost estimating accuracy to reflect the increasing definition of a remedy at three points

in a project. Without more data and a better definition of each alternative, estimating accuracy cannot be increased.

The three levels of cost estimating accuracy are as follows:

Order of Magnitude

This an approximate estimate made without detailed engineering data. Some examples would be: an estimate from cost capacity curves, an estimate using scale up or down factors, and an approximate ratio estimate.

Budget Estimate

"Budget" in this case applies to the owner's budget, and not to the budget as a project control document. A budget estimate is prepared with the use of flow sheets, layouts, and equipment details.

Definitive Estimate

As the name implies, this is an estimate prepared from defined engineering data. The engineering data include as a minimum, fairly complete plot plans and elevations, piping and instrumentation diagrams, one line electrical diagrams, equipment data sheets and quotations, structural sketches, soil data and sketches of major foundations, building sketches, and a complete set of specifications. This category of estimate covers all types from the minimum described above to the maximum definitive type, which would be made from "Approved for Construction" drawings and specifications.

APPROACH AND DEVELOPMENT OF THE RIO SALADO COST ESTIMATES

The engineering data available for the Rio Salado Dam and Stormwater Management design alternatives reflect **Order of Magnitude** estimates. The data made available for the Rio Salado Dam alternatives consisted of preliminary site layout of dam alignment and major structure locations. Specific equipment and material cost was available from manufacturer's estimates of the preliminary design and do not represent quotes for a detailed design. The data made available for the Stormwater Management alternatives consisted of preliminary alignment of each option. Pipe sizes and type were identified along with an approximate profile of the pipe alignment.

The development of the alternatives began with a quantitative evaluation of the scope. A quantity survey of items and facilities with approximate dimensions and references to size and magnitude were calculated. Assumptions may have been used for the scope of work not defined by dimensions.

The quantified items where then estimated per unit complete for the scope of work defined, or as a lump sum for the entire item. These costs include the following

additional costs that a contractor, subcontractor, or supplier may apply, unless otherwise noted:

- Labor and equipment for installation of the work item
- Overhead for labor
- Profit for both labor and material

Resources for costs and productivity for similar scopes of work included the following sources:

- Historical cost from CH2M HILL data bases
- Manufacturer and supplier recommended costs
- Contractors with similar construction experience
- Cost Data References
  - Means Heavy Construction Cost--1991
  - Means Assemblies Cost Data--1991

All costs developed from these resources have been indexed to the Phoenix area. The costs have also been indexed to current 1991 construction costs. Escalation of costs for construction has not been included for the period from these estimates to the start of actual construction and its duration.

Allowances

Allowances are defined as costs that include other related work necessary to complete a scope of work that has been quantified. The allowance is a percentage that will vary due to the complexity and detail of quantity work itemized during the takeoff period.

Contingencies

Contingencies are defined as costs for unforeseeable elements of costs within the defined project scope. The development of this contingency can be broken down into two categories, bid and scope contingencies. These contingencies are added for unforeseen circumstances which may result in additional costs.

A definition of each category is as follows:

- A **Bid** contingency may cover unknown costs associated with constructing a given project scope, such as adverse weather conditions, strikes by materials suppliers, geotechnical unknowns, and unfavorable market conditions. A bid contingency may vary between 10 and 20 percent and is not necessarily uniform for all alternatives.

- A **Scope** contingency covers scope changes that invariably occur during final design and implementation. Scope contingencies should vary between different alternatives, since they will not share the same

elements of risk. They may be 10 to 25 percent of the cost for projects with known technologies and sites with minimum elements of risks, or much more for state-of-the-art technologies at sites with higher elements of risk.

Other Costs

Costs included in the final development of the alternatives include the contractor's mobilization and demobilization of equipment, as well as bonding and insurance costs that may be required of the contractor to bid the project.

SENSITIVITY ANALYSIS

The level of estimating accuracy is in direct relationship to the data made available to prepare the costs of the alternatives. As a result, assumptions must be made concerning variables, such as subsurface conditions and utility locations, that may create an area of sensitivity that could impact the final cost of an alternative. For reference, these assumptions are documented in the development of the cost for each alternative.

The following describes the sensitivity of the estimated construction costs.

Dam Construction

1. Fabrication, delivery, and installation of dam equipment. For the rubber dam alternative, the technology is relatively new to this part of the country and there is a smaller number of contractors familiar with installation and handling of this system. Fabrication and procurement of this item will have to be scheduled well in advance to meet the construction schedule. The same is also true for the hinged gates and fixed weir alternatives.

2. Preparation below dam foundation. More geotechnical data are required to determine the amount of work necessary for the preparation and development below the dam foundation.

3. Embankment material is assumed to be available on site with minimal haul. However, if such material must be imported and hauled the costs will increase accordingly for each alternative.

Stormwater Management

1. Subsurface conditions along the pipe alignment including the location and number of existing buried utilities in the area of construction will be critical to the productivity of pipe installation.

2. Backfill material and pipe bedding is assumed to be all native material.

This section has presented a qualification to the cost estimates as well as a description of the preparation and development of costs for the alternatives. As the design process continues, more data will be available to better project the construction costs.

Appendix B
COST ANALYSIS OVER DESIGN LIFE

General Assumptions:

Dam Design Life - 100 years

Discount Rate - 8.75% (U.S. EPA Office of Municipal Pollution Control)

Annual Operation and Maintenance Costs - 3% of equipment cost for mechanical components and 1% of total construction cost minus equipment cost for non-mechanical components

Sediment Removal - $3.00 per cubic yard

Rehabilitation Cost of Mechanical Components - 60% of initial equipment costs

Bottom Hinged Leaf Gates will be rehabilitated in 50th year

Inflatable Dam bladders will be replaced in 35th and 70th year

Sediment will be removed every 20 years for options that trap sediment in areas of lake not directly in front of gates or fuse plugs.

Sediment will be removed every 5 years in the side channel spillway with fuse plug option because fuse plug blows out only for flows in excess of 100 year flowrate and there is no other sediment passage mechanism

Fuse plugs will be replaced on a schedule determined by the return interval that they blow out at.

TABLE B-1

INFLATABLE DAM - 16 FT. HIGH

Construction Cost			$ 6,280,000
Annual O & M Cost			
3% of $3,500,000	=	$105,000	
1% of 2,780,000	=	27,800	
		$132,800/yr	
Present Worth O&M Cost			$1,517,000
Rehabilitation of Inflatable Bladders			
Rehabilitation Cost in:			
35 yrs is 3,500,000 x 0.6	=	2,100,000	
70 yrs is 3,500,000 x 0.6	=	2,100,000	
			118,000
Present Worth Rehabilitation Cost			
Total Present Worth Cost			$7,915,000

TABLE B-1 (CONT.)

INFLATABLE DAM - 13 FT. HIGH WITH 3 FT. HIGH SILL

Construction Cost			$5,450,000
Annual O & M Cost			
3% of $2,975,000	=	$ 89,250	
1% of 2,475,000	=	24,475	
		$113,725/yr	
Present Worth O & M Cost			$1,303,000
Rehabilitation of Inflatable Bladders			
Rehabilitation Cost in:			
35 yrs is $2,975,000 x 0.6	=	$1,785,000	
70 yrs is $2,975,000 x 0.6	=	1,785,000	
Present Worth Rehabilitation Cost			$ 98,000
Sediment Removal			
Remove 3 ft. wedge of sediment upstream of 3 ft. high sill every 20 years at a cost of $1,701,000 each time			
Present Worth Rehabilitation Cost			$390,000
Total Present Worth Cost			$7,241,000

TABLE B-1 (CONT.)

BOTTOM HINGED LEAF GATES

Construction Cost			$12,000,000
Annual O & M Cost			
3% of $4,760,000	=	$142,800	
1% of $7,240,000	=	72,400	
		$215,200/yr	
Present Worth O & M Cost			$2,456,000
Rehabilitation of Leaf Gates			
Rehabilitation Cost in 50 years is $4,760,000 x 0.6 = $2,856,000			
Present Worth Rehabilitation Cost			$ 43,000
Total Present Worth Cost			$14,499,000

TABLE B-1 (CONT.)

SIDE CHANNEL SPILLWAY WITH LEAF GATES

Construction Cost			$8,280,000
Annual O & M Cost			
3% of $1,680,000	=	$ 50,400	
1% of $6,600,000	=	66,000	
		$116,400/yr	
Present Worth O & M Cost			$1,330,000
Rehabilitation of Leaf Gates			
Rehabilitation cost in 50 years is 1,680,000 x 0.6 = $1,008,000			
Present Worth Rehabilitation Cost			$16,000
Sediment Removal			
Remove sediment in side channels every 20 years at a cost of $1,210,000 each time			
Present Worth Sediment Removal Cost			$277,000
Total Present Worth Cost			$9,903,000

TABLE B-1 (CONT.)

SIDE CHANNEL SPILLWAY WITH FUSE PLUG

Construction Cost	$4,560,000
Annual O & M Cost 1% of $4,560,000 = $45,600/yr	
Present Worth O & M Cost	$ 522,000
Sediment Removal Remove 5-year return interval sediment load every 5 years at a cost of $1,529,000 each time	
Present Worth Sediment Removal Cost	$2,934,000
Total Present Worth Cost	$8,016,000

TABLE B-1 (CONT.)

FUSE PLUG WITH LEAF GATES

Construction Cost			$7,120,000
Annual O & M Cost			
3% of $2,520,000	=	$ 75,600	
1% of $4,600,000	=	$ 46,000	
		$121,600/yr	
Present Worth O & M Cost			$1,390,000
Fuse Plug Rebuilding			
Rebuild fuse plug every 20 years at a cost of $309,000 each time			
Present Worth Fuse Plug Rebuilding Cost			$ 72,000
Rehabilitation of Leaf Gates			
Rehabilitation Cost in 50 years is $2,520,000 x 0.6 = $1,512,000			
Present Worth Rehabilitation Cost			$ 24,000
Total Present Worth Cost			$8,606,000

TABLE B-1 (CONT.)

SEGMENTED FUSE PLUG

Construction Cost			$3,500,000
Annual O & M Cost			
3% of $1,120,000	=	$33,600	
1% of $2,380,000	=	$23,000	
		$57,400/yr	
Present Worth O & M Cost			$ 656,000
Fuse Plug Rebuilding			
Rebuild first fuse plug segment every 5 years			
Rebuild second fuse plug segment every 10 years			
Rebuild third fuse plug segment every 20 years			
Cost of rebuilding each fuse plug segment is $85,400			
Present Worth Fuse Plug Rebuilding Cost			$ 260,000
Rehabilitation of Leaf Gates			
Rehabilitation cost in 50 years is $1,120,000 x 0.6 = $672,000			
Present Worth Rehabilitation Cost			$ 10,000
Total Present Worth Cost			$4,426,000