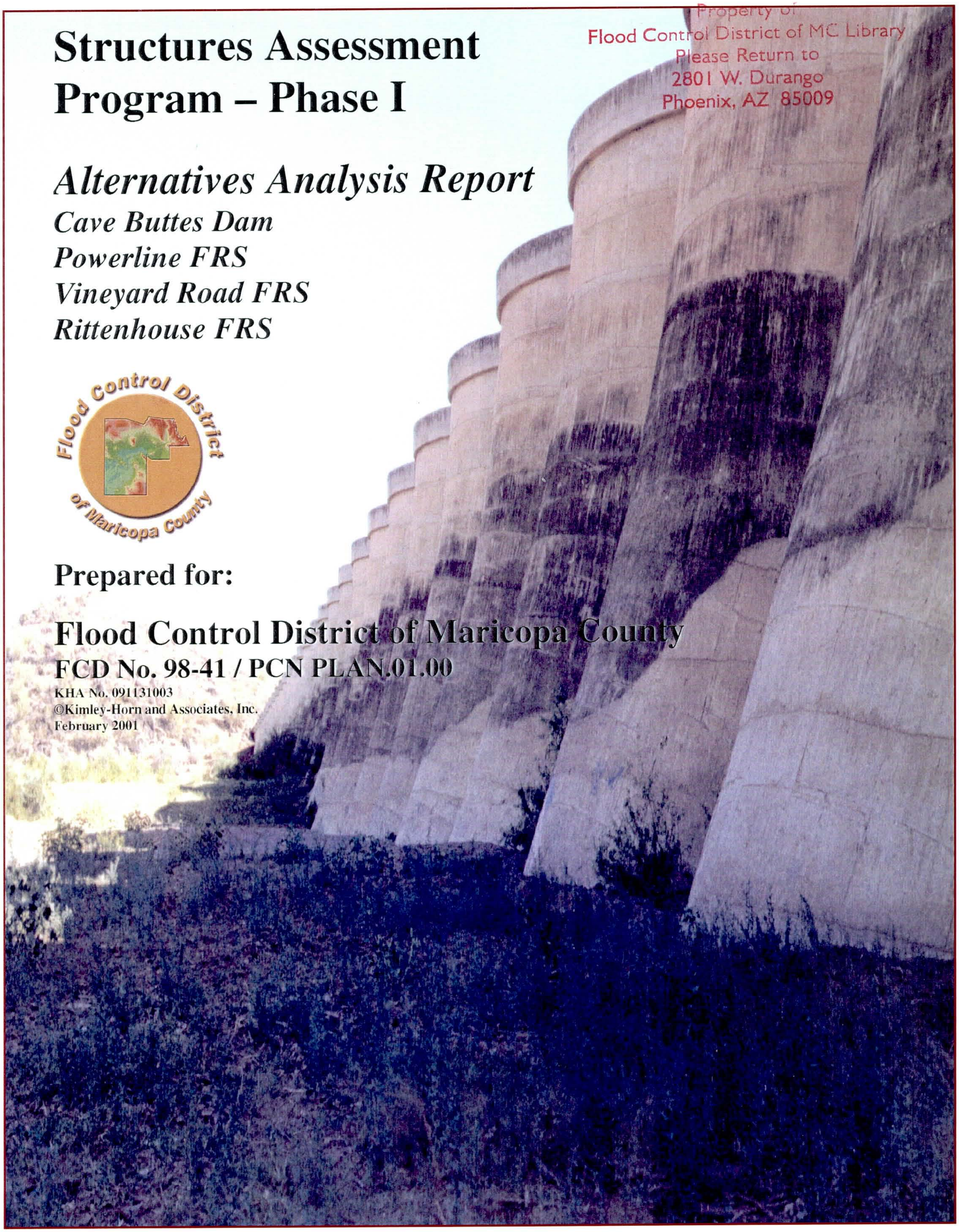

Structures Assessment Program – Phase I

Alternatives Analysis Report

Cave Buttes Dam
Powerline FRS
Vineyard Road FRS
Rittenhouse FRS

Prepared for:

Flood Control District of Maricopa County
FCD No. 98-41 / PCN PLAN.01.00

KHA No. 091131003

February 2001

Kimley-Horn and Associates, Inc.

Tom Renckly - FCDX

To: Tom Johnson - FCDX; Tom Renckly - FCDX; Larry Lambert - FCDX; Joe Tram - FCDX; Ed Raleigh - FCDX
Cc: Tim Phillips - FCDX; Doug Williams - FCDX; William "Wayne" Killgore - FCDX; Russ Miracle - FCDX; Bob Stevens - FCDX; Dick Perreault - FCDX; Amir Motamedi - FCDX; Michael Lopez - FCDX; Dave Johnson - FCDX; George Lindop - FCDX
Subject: Alternative Analysis Report - Cave Buttes Dam, Powerline FRS, Vineyard FRS, Rittenhouse FRS - Phase I Strucutres Assessment Program

The subject report has been finalized.
Copies will be distributed to the following people:
Ed R., Joe T., Mike W., Larry L.
Also I have a copy for loan.

Cave Buttes Dam
No action is planned at this time on alternatives evaluated for Cave Buttes Dam.
Future studies are pending project prioritization at the completion of Phase I Assessments for all 22 dams.

Powerline FRS, Vineyard FRS, Rittenhouse FRS
While ADWR has not identified dam safety deficiencies for these dams, it is generally recognized that significant dam rehabs or dam replacements are likely to be needed for the 3 dams to meet the goals of the Structures Assessment Program.

The Individual Structures Assessment (ISA) Report (distributed previously) for these dams provides an overall assessment of the dams as well as recommendations for more immediate site specific investigations and repairs required for the dams. The more immediate issues identified in the ISA report will be addressed under the Phase II investigation and repair contracts.

These three dams are currently classified as significant and with anticipated downstream development they will become high hazard, possible in the near future.

In addition urbanization encroachment is upon us at the dams. You may recall we had a meeting last year with State Land Dept. to discuss a developers plans at Powerline FRS. The state has asked for a copy of the Alternative Analysis report. Suggest you take a look at exhibit 5 of the report which indicates current District flowage easements for the Apache Junction-Gilbert Watershed Project.
Mike Wilson will be setting up an internal District meeting in two or three weeks to discuss the issues and to formulate the District's approach with the state and the developer. At the meeting I'd also like to discuss the long term plans for these three dams.

Thanks
Tom R.

Powerline FRS, Vineyard FRS and Rittenhouse FRS

STRUCTURES ASSESSMENT PROGRAM – PHASE I

ALTERNATIVES ANALYSIS REPORT

Cave Buttes Dam
Powerline FRS
Vineyard Road FRS
Rittenhouse FRS

FLOOD CONTROL DISTRICT
OF MARICOPA COUNTY
FCD 98-41
PCN PLAN.01.00

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DATE SIGNED Feb 16 2001
ARIZONA, U.S.A.

FEBRUARY 2001

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DATE SIGNED
ARIZONA, U.S.A.

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ACKNOWLEDGEMENTS

ACKNOWLEDGMENTS

Kimley-Horn and Associates, Inc. (KHA) on behalf of the Flood Control District of Maricopa County, extends our collected gratitude and appreciation to all the dam safety agency representatives, staff, individuals, and subconsultants for their review, knowledge, and experience into assisting in making this Alternatives Analysis Report and the Structures Assessment Program a highly successful endeavor.

Kimley-Horn particularly wishes to thank the Steering Committee and Technical Committee for their valued input and guidance to the Structures Assessment Program.

The Steering Committee for the Structures Assessment Program consisted of Federal dam safety agency representatives, Arizona Department of Water Resources- Dam Safety Section, and District division chiefs from Engineering, Planning & Project Management, and Operations and Maintenance. The Steering Committee served in an advisory capacity to the District's project manager and Kimley-Horn and Associates concerning the major findings and recommendations of Phase I of the program. The Steering Committee members are as follows:

- Natural Resources Conservation Service – Mr. Noller Herbert, P.E., State Conservation Engineer, Phoenix, Arizona.
- U.S. Army Corps of Engineers, Los Angeles District – Mr. George Beams, P.E., Director of Engineering and Dam Safety Officer, Los Angeles, California
- Bureau of Reclamation – Mr. Dean Hagstrom, Phoenix, Arizona
- Arizona Department of Water Resources, Dam Safety Section – Mr. Jon Benoist, P.E., Chief of Dam Safety, Phoenix, Arizona
- Flood Control District of Maricopa County
  - Chief of Engineering Division – Mr. Ed Raleigh, P.E.
  - Chief of Planning and Project Management – Mr. Tom Johnson, P.E.
  - Chief of Operations and Maintenance – George Lindop

The Technical Committee served in a technical capacity to the District's project manager and Kimley-Horn and Associates concerning the major findings and recommendations of Phase I of the program. The Technical Committee members are as follows:

- Natural Resources Conservation Service – Mr. Noller Herbert, P.E., State Conservation Engineer, Phoenix, Arizona.
- U.S. Army Corps of Engineers, Los Angeles District – Mr. Ted Ingersoll, P.E.
- Bureau of Reclamation – Mr. Dean Hagstrom, Phoenix, Arizona
- Arizona Department of Water Resources, Dam Safety Section – Mr. Mike Greenslade, P.E., Phoenix, Arizona.
- Flood Control District of Maricopa County
  - Planning and Project Management – Mr. Tom Renckly, P.E., Project Manager for the Structures Assessment Program
  - Engineering – Mr. Joe Tram, P.E., Special Projects Branch Manager

- Operations and Maintenance – Chuck Smith

Kimley-Horn and Associates acknowledges the important contribution of our subconsultant for this report. URS-Greiner/Woodward Clyde and Associates (Mr. John Sikora, Denver, Colorado) provided technical review and assistance in regards to dam engineering and dam technologies.

Kimley-Horn and Associates, Inc. (Phoenix, Arizona) project team members are as follows:

- Principal/Senior Project Manager (QA/QC) – Doug Plasencia, P.E.
- Project Manager and principal technical writer – Robert Eichinger, P.E.
- Senior Engineer – Daniel Sagramoso, P.E.
- Project Engineer and research– Mr. Jon Ahern, P.E.

ALTERNATIVES ANALYSIS REPORT

EXECUTIVE SUMMARY

STRUCTURES ASSESSMENT PROGRAM – PHASE I
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

ALTERNATIVES ANALYSIS REPORT

EXECUTIVE SUMMARY

1.0 Introduction

This Alternatives Analysis Report documents the results of an alternatives analyses for four of the twenty-two Flood Control District of Maricopa County (District) flood control dams. The Alternatives Analysis report is part of Phase I of the Structures Assessment Program, as outlined below.

The purpose of the Alternatives Analysis is to evaluate structural and nonstructural flood control alternatives/measures or solutions: the objective, which is to reduce the District's risk and liability, associated with dam ownership. The structural alternatives evaluated include repair of dams, modification of dams to improve performance, replacement of dams with some other form of structural flood control measure or, modification of the pool so as to eliminate the need for the dam embankment. Nonstructural alternatives include mitigation through flood insurance, acquisition of flowage easements/properties, development of emergency action plans, or some combination of two or more nonstructural solution elements.

The Alternatives Assessment Report is the culmination of a concept investigation and cost estimate for structural and nonstructural measures and alternatives for four District dams – Cave Buttes Dam, Powerline Flood Retarding Structure (FRS), Vineyard Road FRS, and Rittenhouse FRS. These alternatives are primarily conceptually designed to reduce the risk of dam ownership to the District. The following structural and nonstructural measures were used as the basis to develop the project alternatives for each of the dams evaluated as part of this study.

Structural Measures:

a. Repair of currently identified dam safety deficiencies.
b. Upgrade dams to meet future ADWR standards.
c. Modify dams to improve performance.
d. Replace dams with structural features such as basins, floodways, or dams modified to convey flows that provide the same flood control function as the dam.
e. Qualitatively evaluate the protection afforded by the dam to be able to contrast recommendations with a no-dam alternative.

f. All structural solution alternatives shall identify opportunities for multi-use functions, improved aesthetics, environmental enhancement, and potential for partnering with others to accomplish project objectives.
g. One alternative shall consider modifying the pool so as to eliminate the need for the dam embankment.

Non-Structural Downstream Measures (Cave Buttes Dam Only; not included for Powerline, Vineyard Road, and Rittenhouse Flood Retarding Structures because emergency spillway inundation mapping was not available at the time of the study. Evaluation of non-structural downstream measures should be completed after the mapping is available.)

Potential Inundation Areas Downstream of Emergency Spillways and Principal Outlets shall be conducted below Cave Buttes Dam. Measures are:

a. Mitigate Through Flood Insurance
b. Acquire Properties/Flowage Easements
c. Upgrade EAP's
d. Combination of two or more non-structural solution elements

Non-Structural Impoundment Area Measures:

Non-Structural Measures for Impoundment Areas will be conducted for the four dams:

a. Mitigate Through Flood Insurance
b. Acquire Properties/Flowage Easements
c. Develop EAP's
d. Combination of two or more non-structural solution elements

The set of alternatives for each dam were evaluated based on criteria from which to rank the alternatives and determine a "preferred" dam alternative. The alternative that ranks the highest based on assignment of point values from a range of values for each evaluation criteria is the preferred dam alternative for the dam being evaluated. The preferred dam alternative as derived by this analysis is identified only for the purposes of the Phase I Assessments and in no way indicates that this would be the final selection of a project for implementation. Prior to identification of a final preferred alternative detailed Phase II studies and coordination with project stakeholders would be completed. Prior to selection and implementation of a final alternative authorizing processes, documents and agreements would be required.

This Executive Summary of the Alternatives Analysis Report provides a summary of the project features for each of the four District dams examined as part of the Phase I study. The report also summarizes the results of the alternatives analysis for each dam, and provides a concept level evaluation and preliminary costs for each alternative considered.

With the exception of downstream emergency action plans to FEMA standards, the Flood Control District is under no regulatory mandate or otherwise required to implement structural and/or non-structural alternatives evaluated in this report. These alternatives are being evaluated are part of the Structures Assessment Phase I Program to determine feasible measures that may be implemented by the District to accomplish the stated goals of the program

2.0 Structures Assessment Program

In recognition and realization of the changes occurring and associated with flood control dams both on the national and local level, the Flood Control District of Maricopa County (District) has embarked on the **Structures Assessment Program**, the purpose of which is to minimize the risk and liability associated with the District's flood control dams. Since many of the District dams were built, there have been a number of changes, which now need to be addressed. These changes are:

- District dams have aged and some are showing signs of distress,
- Significant urbanization within Maricopa County and adjacent to District dams has occurred and continues at a rapid pace,
- Changes in dam technology and design practices,
- Changes in methodology for determining inflow design flood,
- Significant increase in permit requests for utility and roadway crossings of dams,
- Newly enacted rule changes by the Arizona Department of Water Resources, and,
- Subsidence impacts on District dams due to groundwater pumping.

The Structures Assessment Program will address and assess the District's dam safety program on several fronts including:

- Dam safety inspections/evaluations,
- Emergency Action plans,
- Impoundment areas and spillway channels,
- Improvements to the overall dam safety program,
- Impacts of future dam safety rules and regulation changes,
- Planning studies to evaluate project options, and
- Flood Control District policy evaluation.

The Structures Assessment Program will be conducted in three phases. Phase I will primarily involve:

- Collection of data and inspection of dams,
- Develop dam safety recommendations and priorities, considering changes listed above,
- Perform preliminary alternative analysis studies to modify existing projects to address urbanization related issues, and,
- Evaluate newly enacted ADWR rule changes and District policy issues.

Phase II will primarily involve:

- Perform detailed investigations and analyses as identified by need and priority in Phase I,

- Initiate project planning and authorization activities to correct identified distress issues,
- Implement changes to overall dam safety program and policies, and,
- Perform conceptual design studies and alternative analyses for modification of projects to address urbanization and distress issues.

Phase III will primarily involve:

- Implement projects to correct any identified dam safety concerns. These could include but are not limited to structural modifications, land acquisitions below spillways, and alternative, lower risk solutions,
- Implement approved projects and land acquisitions to address urbanization issues, and,
- Continue long-term dam safety program.

Phase I of the Structures Assessment Program will primarily be an evaluation and study phase. The District has retained Kimley-Horn and Associates to provide services to conduct Phase I evaluations and studies. The first work assignment will focus on four District dams. Evaluations and studies performed for these dams will initiate the Phase I process. It is intended that the first work assignment will be a pilot study from which to establish initial District dam safety policy and programs, and from which to refine engineering and planning methods for the Structures Assessment Program. The dams evaluated in the first work assignment were the Powerline Flood Retarding Structure (FRS), the Vineyard Road Flood Retarding Structure, the Rittenhouse Flood Retarding Structure, and Cave Buttes Dam. This separate Alternatives Analysis report documents the alternatives analysis of these four dams.

A Steering Committee was formed at the inception of Phase I to serve in a dam safety program advisory capacity to the District's project manager concerning the major findings and recommendations of Phase I of the program. The committee consisted of representatives of the District's planning, engineering, and operations functions, Arizona Department of Water Resources Dam Safety Section, Natural Resources Conservation Service, Corps of Engineers, and Bureau of Reclamation. The Steering Committee will review the findings and recommendations of this Summary and provide their input, guidance, and experience to advise and steer the course for enhancing the District's dam safety program.

A Technical Committee also was formed at the inception of Phase I and served in a technical advisory capacity to the District's project manager concerning the major findings and recommendations of Phase I of the program. The technical committee consists of representatives of the District's planning, engineering, and operations functions, Arizona Department of Water Resources Dam Safety Section, Natural Resources Conservation Service, Corps of Engineers, and Bureau of Reclamation. The technical committee will review the full Policy & Program report and provide their input, technical comments, guidance, and experience to enhance dam safety program elements.

3.0 Cave Buttes Dam Alternatives Analysis

The structural concept alternatives for Cave Buttes Dam were primarily formulated to provide a greater degree of operational flexibility of the dam during normal flood and emergency flood operations. The structural alternatives include the following concept measures:

Structural Alternatives:

- No. 1: Low Level Outlet – Dike No. 2. Examine the feasibility at a concept level for providing a low-level outlet in Dike No. 2.
- No. 2 Divert Emergency Spillway Flow to Central Arizona Project (CAP) canal. Utilize CAP canal to carry discharged waters from emergency spillway up to capacity of CAP canal.
- No. 3: Low Level Outlet – Dike No. 3. Divert stormwater from the reservoir pool through low level outlet in Dike No 3.

The nonstructural alternatives include the following concept measures:

Below Dam - Nonstructural Measures:

- No. 4: Mitigate through Flood Insurance
- No. 5: Downstream Flowage Easements
- No. 6 Update Emergency Action Plan

Pool Area - Nonstructural Measures:

- No. 7: Mitigate through Flood Insurance
- No. 8: Acquire Properties/Flowage Easements
- No. 9: Develop Emergency Action Plan for Pool area

4.0 Powerline, Vineyard Road, and Rittenhouse Flood Retarding Structure Alternatives Analysis

The structural concept alternatives for Powerline, Vineyard Road, and Rittenhouse Flood Retarding Structures were formulated to upgrade, modify or enhance performance or operations, or replace the dam with some other structural flood control measure. The structural alternatives include the following concept measures:

Structural Alternatives:

- No. 1: Segmentation: Examine segmenting each dam into two dams each.
- No. 2: Upgrade to high hazard dam: Examine upgrading dams to high hazard dam.
- No. 3: Modifications to improve performance
- No. 4: Basins: Replace the dam with a basin
- No. 5: Levee/floodway system: Link dams to function as a levee/floodway system
- No. 6: Discharge into Central Arizona Project
- No. 7: Upsize Powerline Floodway

The nonstructural concept alternatives for Powerline, Vineyard Road, and Rittenhouse FRS were primarily formulated to reduce the risk and liability associated with ownership of the dam. The nonstructural alternatives include the following concept measures:

Nonstructural Measures for Pool Area:
- No. 8: Mitigate through Flood Insurance
- No. 9: Acquire Properties/Flowage Easements
- No. 10: Emergency Action Plan: Develop EAP to include pool inundation areas.

5.0 Evaluation and Ranking

To assist in evaluating and comparing project alternatives, an evaluation and ranking matrix consisting of eight criteria with a range of point values was developed. The development of the matrix criteria was formulated with the input from both the District and the KHA project team. The matrix was developed with assistance from the District in an attempt to objectively evaluate alternatives for a range of flood control and non-flood control criteria while still emphasizing that the primary purpose of each alternative is to reduce the risk and liability of dam ownership. The matrix is used to rank the alternatives presented and also to use as a guideline for future Phase I dam alternative evaluations.

Table 1. Evaluation Criteria Matrix

Evaluation Criteria	Range Of Point Values
Jurisdictional	1 to 8
Cost	1 to 10
Implementation	1 to 8
Environmental	1 to 8
Multi-Use	1 to 5
Risk And Liability	1 to 15
Compatibility With District Plans	1 to 8
Flood Control	1 to 8

(see Section 5.3 of the main report for further discussion)

6.0 Closing

Table 2 and Table 3 (below) provide the results of the ranking of the set of alternatives for Cave Buttes Dam and the set of alternatives for Powerline, Vineyard Road, and Rittenhouse FRS, respectively. The ranking was based on the criteria and range of point values provided in Table 1 above.

Three structural alternatives for Cave Buttes Dam were developed, evaluated, and rankings assigned based on point values from a set of eight evaluation criteria. The preferred alternative is to construct a low-level outlet in Dike No.2 which when operated would discharge ultimately to the Reach 11 detention dike east of Cave Creek Road. This structural alternative will provide the District operational flexibility in the management of the Cave Buttes Dam reservoir impoundment. In the event of a large storm on the Cave Creek watershed that produces a high volume of runoff to Cave Buttes Dam, the District would be able to discharge impounded floodwaters from the Cave Buttes Dam impoundment and direct the discharges to the Reach 11 reservoir. This alternative works if volume is available in the Reach 11 reservoir, little to no inflow is coming into the Reach 11 dam, and agreements are reached between the District, Bureau, and the CAP.

Table 2. Cave Buttes Dam Alternatives Ranking.

Alternative No.	Total Point Score	Rank
1. Low-level outlet Dike No. 2	37	1 -Structural
2. Floodway from spillway to CAP canal	33	
3. Low-level outlet Dike No. 3	32	
4. Flood Insurance - Downstream	20	
5. Acquire Properties/Flowage Easements Downstream	34	
6. Develop Emergency Action Plan Downstream	44	1-Nonstructural Downstream
7. Flood Insurance - Pool Area	22	
8. Acquire Properties/Flowage Easements Pool Area	30	
9. Develop Emergency Action Plan Pool Area	47	1-Nonstructural Pool Area

Six nonstructural alternatives for below and above Cave Buttes Dam were developed, evaluated, and rankings assigned based on point values from a set of eight evaluation criteria. The preferred nonstructural alternative for both downstream and upstream is to develop a site specific emergency action plan. This alternative could be combined with limited purchase of properties and/or easements within floodprone areas for the full PMF. In this manner, the District would regulate development within the inundation limits for the full PMF both upstream and downstream.

The investigation of the purchase of flood insurance for the Cave Buttes Dam pool area and downstream area included a review FEMA's Flood Insurance Manual (May 2000) and discussions with the Flood Insurance Administration regarding flood insurance coverage. The review of the Manual and discussions with FEMA indicates that FEMA offers flood insurance coverage on a property by property basis. Area coverage is not available through the FIA flood insurance program. One concept recently discussed in Washington, D.C. is the idea of residual risk flood insurance for areas protected by flood control structures. In concept this would lead to low insurance rates, perhaps allowing for group verses individual policies. KHA urges that the District promote this concept with the FIA and professional associations to gain support for legislative initiatives.

Seven structural alternatives for Powerline, Vineyard Road, and Rittenhouse FRS were developed, evaluated, and rankings assigned based on point values from a set of eight evaluation criteria. The seven alternatives were applied to the three dams as a set since the dams are operationally and functionally linked. The preferred structural alternative is to upgrade the three dams to high hazard dams capable of safely passing the full PMF. The second preferred alternative, construction of detention basins, was not preferred due to the high cost of land acquisition and construction costs. Several structural alternatives are not compatible with current District planning studies for the East Maricopa Floodway, Powerline Floodway, and Queen Creek wash.

Table 3. Powerline, Vineyard Road, and Rittenhouse FRS Alternatives Ranking.

Alternative No.	Total Point Score	Rank
1. Segmentation	36	
2. Upgrade to high hazard dam	48	1-Structural
3. Modifications to improve performance	43	
4. Basins	46	2-Structural
5. Levee/floodway system:	33	
6. Discharge into Central Arizona Project	34	
7. Upsize Powerline Floodway	35	
8. Flood Insurance - Pool Area	43	
9. Acquire Properties/Flowage Easements Pool Area	35	
10. Develop Emergency Action Plan Pool Area	49	1-Nonstructural

Three nonstructural alternatives for the pool areas of Powerline, Vineyard Road, and Rittenhouse FRS were developed, evaluated, and rankings assigned based on point values from a set of eight evaluation criteria. The preferred nonstructural alternative for the pool area is to develop a site specific emergency action plan. This alternative could be combined with limited purchase of properties and/or easements within floodprone areas for the full PMF (in the event that the upgrade to high hazard dam is promulgated). In this manner, the District would regulate development within the inundation limits for the full PMF around the impoundment area.

Although the preferred alternative for Powerline, Vineyard Road, and Rittenhouse FRS is to upgrade to a high hazard dam, in any case, structural alternative No. 3 - Modifications

- should be implemented regardless of the structural alternative selected for rehabilitation, modifications, or upgrading the three dams.

The preferred structural and nonstructural flood control alternatives evaluated and examined as part of this study should assist the District in the management of their risk and liability associated with the dams under consideration. The goal of the alternatives study was to identify a set of flood control measures, both structural and nonstructural, that could potentially reduce risk and liability associated with dam ownership. The preferred alternatives, based on the assignment of point values and ranking, should meet this important District goal.

Table 4 (following page) provides a summary of the structural and nonstructural flood control alternatives for Cave Buttes Dam. Table 5 provides a summary of the structural flood control alternatives for Powerline, Vineyard Road, and Rittenhouse flood retarding structures. Table 6 provides a summary of the nonstructural flood control alternatives for Powerline, Vineyard Road, and Rittenhouse flood retarding structures.

Table 4. Cave Buttes Dam Summary of Structural and Nonstructural Alternatives.

Structural Alternative Description	Elements Of Alternative	Estimate Of Alternative Cost	Alternative Ranking
Low Level Outlet In Dike No. 2	RCB 10-ft by 6-ft gated Capacity 750 cfs Trap Channel 12-ft btm Concrete lined @0.005 ft/ft 13,360 feet long	$ 2.1m	(1)
Divert to Central Arizona Project Canal from Emergency Spillway	Concrete Trap Channel (2 segments) Capacity 3,000 cfs 1. Upstream Btm width 24 ft 3000-ft Depth 6-ft 2. Downstream Btm width 24 ft 3300-ft Junction Structure with twin steel leaf gates 12-ft by 12-ft ea.	$ 1.9 m	(2)
Divert from Reservoir Pool through Low Level Outlet In Dike No. 3	Twin 8-ft x 4-ft RCB gated Capacity 100 cfs Trap earth-lined channel 500-ft long 10-ft bottom	$ 132 k	(3)
Non-Structural Alternative Description	**Elements of Alternative**	**Estimate of Alternative Cost**	
Below Dam Update Emergency Action Plan	Prepare Emergency Action Plan per FEMA 64 guidelines and requirements of ADWR	$20k - $30k	(1)
Below Dam Acquire Properties/ Downstream Flowage Easements	Acquire easements for PMF limits outside 100-year No. acres = 200	$9m	(2)
Below Dam Mitigate Through Flood Insurance	Coverage $100,000/dwelling unit No. of acres = 644	Annual Premium = $298k 30-year Premium = $8.9m	(3)
Pool Area Develop Emergency Action Plan	Prepare Emergency Action Plan per FEMA 64 guidelines and requirements of ADWR	$20k - $30k	(1)
Pool Area Acquire Properties/Flowage Easements	Acquire easements up to PMF ponding limits No. of acres = 720	$32m	(2)
Pool Area Mitigate Through Flood Insurance	Coverage $100,000/dwelling unit No. of acres = 1000	Annual Premium = $482k 30-year Premium = $14.5m	(3)

Table 5. Powerline (P), Vineyard Road (V), and Rittenhouse (R) FRS Summary of Structural Alternatives.

Structural Alternative Description		Elements Of Alternative	Estimate Of Alternative Cost	Alternative Ranking
Upgrade to High Hazard Dams.	P	Raise dam 4.5 ft and increase emergency spillway to 900-ft	$ 3.05 m	(1) Total cost $11.5 m
	V	Raise dam 4.9 ft and increase emergency spillway to 900-ft	$5.75 m	
	R	Raise dam 4.3 ft and increase each (2) emergency spillway to 450-ft	$ 2.69 m	
Basins. Replace dams with Basins.	P	5-ft deep; 8,000-ft long; 4,400-ft wide	$ 57.9 m	(2) Total cost $127.5 m
	V	5-ft deep; 26,00-ft long; 1,400-ft wide	$ 24.4 m	
	R	5-ft deep; 14,000-ft long; 2,400-ft wide	$ 45.2 m	
Modify Dam to improve Performance (add sills; Erosion control)	P V R	Concrete Control sill (4 to 4.5-ft deep) Abutment Slope Protection (D_{50} 1.0 to 1.6-ft) Trashrack Modification	$ 0.66 m	(3) Total cost $ 0.66 m
Segmentation. Segment Structures into smaller "dams" segments or cells	P	Segment = 6,000 ft with 6-ft dia equalization culvert and 6-ft by 6-ft floodgate	$ 2.5 m	(4) Total cost $ 4.04 m
	V	Segment = 2,000 ft with 6-ft dia equalization culvert and 6-ft by 6-ft floodgate	$ 0.54 m	
	R	Segment = 2,900 ft with 6-ft dia equalization culvert and 6-ft by 6-ft floodgate	$1.0 m	
Increase Capacity of Powerline Floodway		Channel Capacity 4,000 to 6,000-cfs; Concrete lined rectangular, 52-ft bottom width; depth = 5-ft, length = 9.1 miles	$ 13.2 m	(5) Total cost $13.2 m
Discharge into the Central Arizona Project canal. Provide low-level outlets for each dam to CAP canal.	P	Twin 7-ft dia RCP gated outlet; length = 210-ft. Discharge = 900 cfs	$ 174 k	(6) Total cost $0.67 m
	V	Twin 7-ft dia RCP gated outlet; length = 210-ft. Discharge = 900 cfs	$ 269 k	
	R	Twin 7-ft dia RCP gated outlet; length = 210-ft. Discharge = 900 cfs. Floodway channel = 310-ft	$ 225 k	
Levee/Floodway System. Replace dams with levees and floodways.		Modify the dams into a contiguous levee system with upstream floodway. Discharge to Sonoqui Detention dike.	$88.3 m	(7) Total Cost $88.3 m

Table 6. Powerline (P), Vineyard Road (V), and Rittenhouse (R) FRS Summary of Non-Structural Alternatives.

Non-Structural Alternative Description	Elements Of Alternative		Estimate Of Alternative Cost	Alternative Ranking
Develop Emergency Action Plan to FEMA 64 guidelines	P	EAP for both pool area and downstream area	\$20k - \$30k	(1)
	V	EAP for both pool area and downstream area	\$20k - \$30k	
	R	EAP for both pool area and downstream area	\$20k - \$30k	Total Cost \$ 60k - 90 k
Mitigate through Flood Insurance	P	610 acres (uninhabitable structures only)	Annual Premium \$1,500 30-year Premium \$45,000	(2)
	V	637 acres (uninhabitable structures only)	Annual Premium \$1,500 30-year Premium \$45,000	
	R	660 acres (uninhabitable structures only)	Annual Premium \$1,500 30-year Premium \$45,000	Total cost \$ 135 k
Acquire Properties/Flowage Easements	FCD already owns or leases sufficient lands. Option to purchase pool areas (total 2,000 acres)		\$100 m	N/A

Section 1.0 Introduction

1.1 Authorization

The Alternatives Analysis Report was prepared by Kimley-Horn and Associates, Inc. (KHA) under authorization by the Flood Control District of Maricopa County (District) through the scope of work for the Structures Assessment Program-Phase I, Work Assignment No. 1 (Contract FCD 98-41). Kimley-Horn and Associates retained URS Greiner Woodward-Clyde, and Geological Consultants to assist with the preparation of the elements of Work Assignment No. 1.

1.2 Purpose

The Alternatives Analysis Report documents an alternatives study for each of the Work Assignment No. 1 structures. The purpose of the Alternatives Analysis is to evaluate structural and nonstructural flood control alternatives that could potentially reduce District risk and liability associated with dam ownership. The structural alternatives evaluated include repair of dams, modification of dams to improve performance (including operational performance), replacement of dams with some other form of structural flood control measure(s) and, modification of the pool so as to eliminate the need for the dam embankment. Nonstructural alternatives include mitigation through flood insurance, acquisition of flowage easements/properties, development of emergency action plans, or some combination of two or more nonstructural solution elements.

With the exception of downstream emergency action plans to FEMA standards, the Flood Control District is under no regulatory mandate or otherwise required to implement structural and/or non-structural alternatives evaluated in this report. These alternatives are being evaluated are part of the Structures Assessment Phase I Program to determine feasible measures that may be implemented by the District to accomplish the stated goals of the program

The Alternatives Analysis Report is a companion report to two other major reports under FCD 98-41. These two other reports are the Policy and Program Report and the Individual Structures Assessment Report.

The purpose of the Program and Policy Report is threefold: (1) to document and discuss the present status of the District's dam safety program and policies; (2) benchmark the District's dam safety program and policies against other established dam safety programs; and (3) to recommend changes and revisions to the District's dam safety program and policies to bring the District up to current state of practice and set a framework/direction for future District needs and requirements.

The purpose of the Individual Structures Assessment Report is twofold: (1) to assess the existing condition of Cave Buttes Dam, Powerline, Vineyard Road, and Rittenhouse

Flood Retarding Structures; and (2) to recommend actions for further investigations/monitoring of the structures and develop work plans to repair signs of distress in the structures.

1.3 Scope

The Alternatives Assessment Report is the culmination of a concept investigation and opinion of probable costs for structural and nonstructural measures and alternatives for four District dams – Cave Buttes Dam, Powerline Flood Retarding Structure (FRS), Vineyard Road FRS, and Rittenhouse FRS. These alternatives are primarily conceptually designed to reduce the risk of dam ownership to the District. The alternatives analysis was founded in the scope of work for Work Assignment No. 1, Task 5.0 – Alternatives Analysis. Under Task 5.0, Kimley-Horn and Associates evaluated at a concept level the following potential alternative measures:

Structural Measures:

a. Repair of currently identified dam safety deficiencies.
b. Upgrade dams to meet future ADWR standards.
c. Modify dams to improve performance.
d. Replace dams with structural features such as basins, floodways, or dams modified to convey flows that provide the same flood control function as the dam.
e. Qualitatively evaluate the protection afforded by the dam to be able to contrast recommendations with a no-dam alternative.
f. All structural solution alternatives shall identify opportunities for multi-use functions, improved aesthetics, environmental enhancement, and potential for partnering with others to accomplish project objectives.
g. One alternative shall consider modifying the pool so as to eliminate the need for the dam embankment.

Non-Structural Downstream Measures (Cave Buttes Dam Only; not included for Powerline, Vineyard Road, and Rittenhouse Flood Retarding Structures because emergency spillway inundation mapping was not available at the time of the study. Evaluation of non-structural downstream measures should be completed after the mapping is available.)

Potential Inundation Areas Downstream of Emergency Spillways and Principal Outlets shall be conducted below <u>Cave Buttes</u> Dam. Measures are:

a. Mitigate Through Flood Insurance
b. Acquire Properties/Flowage Easements
c. Upgrade EAP's
d. Combination of two or more non-structural solution elements

Non-Structural Impoundment Area Measures

Non-Structural Measures for Impoundment Areas will be conducted for the four dams:

a. Mitigate Through Flood Insurance
b. Acquire Properties/Flowage Easements
c. Develop EAP's
d. Combination of two or more non-structural solution elements

The concept alternative measures considered for development and evaluation for each dam was formulated in an Alternatives Analysis concept development meeting held between the District and KHA on February 16, 2000. The concept measures or alternatives were documented in a KHA memorandum dated March 16, 2000 to the District.

1.4 Report Organization

The Alternatives Analysis Report is organized into seven sections plus appendices.

Section 1.0 – Introduction: Provides the project authorization, purpose, scope, and report organization.

Section 2.0 – Structures Assessment Program Background: Provides a general discussion of the Structures Assessment Program and the three phases of the program.

Section 3.0 - Alternatives Analysis for Cave Buttes Dam

Section 4.0 – Alternatives Analysis for Powerline, Vineyard Road, and Rittenhouse Flood Retarding Structures.

Section 5.0 – Evaluation and Ranking

Section 6.0 – Closing: Provides closing comments for the Alternatives Analysis Report

Section 7.0 - References

Section 2.0 Structures Assessment Program Background

2.1 General

The Flood Control District of Maricopa County (District) recently celebrated their fortieth anniversary by renewing their mission and commitment to continued excellence in reducing flood risks for the people of Maricopa County by providing comprehensive flood and stormwater management services. As part of their continued mission, the District has embarked on a Structures Assessment Program, the primary objective of which is to minimize the risk and liability associated with District flood control dams.

The District owns, operates and maintains twenty-two dry flood control dams and is mandated by state and federal law to assure the safety of these structures. The District has initiated a program called the Structures Assessment Program to assess and evaluate these structures (or dams – used interchangeably) and related features due to an ever-increasing urbanized environment and to assure continued compliance with current standards and guidelines. The situation faced by the District is that the same population protected by the dams can be at risk in the unlikely event of dam failure. The District is seeking measures that provide flood control and that properly manage long term risk. The Structures Assessment Program is intended to address issues related to urbanization and dam safety as well as to enhance and improve the District's ongoing Dam Safety Program.

The purpose of the Structures Assessment Program is to minimize risk and liability associated with the District's flood control dams. Since many of the District dams were built, there have been a number of changes, which now need to be addressed. These changes are:

- Structures have aged and some are showing signs of distress,
- Significant urbanization has occurred and continues at a rapid pace,
- Changes in dam technology and design practices,
- Changes in methodology for determining inflow design flood,
- Significant increase in permit requests for utility and roadway crossings of dams,
- Newly enacted rule changes by the Arizona Department of Water Resources (ADWR), and,
- Subsidence impacts due to groundwater pumping.

The Structures Assessment Program will address and assess the District's dam safety program on several fronts including:

- Dam safety inspections/evaluations,
- Emergency Action plans,
- Impoundment areas and spillway channels,
- Improvements to the overall dam safety program,
- Future rules and regulation changes,

- Planning studies to evaluate project options, and
- Flood Control District policy evaluation.

The Structures Assessment Program will be conducted in three phases. Phase I will primarily involve:

- Collection of data and inspection of dams,
- Develop dam safety recommendations and priorities, considering changes listed above,
- Perform preliminary alternative analysis studies to modify existing projects to address urbanization related issues, and,
- Evaluate newly enacted ADWR rule changes and District policy issues.

Phase II will primarily involve:

- Perform detailed investigations and analyses as identified by need and priority in Phase I,
- Initiate project planning and authorization activities to correct identified distress issues,
- Implement changes to overall dam safety program and policies, and,
- Perform conceptual design studies and alternative analyses for modification of projects to address urbanization and distress issues.

Phase III will primarily involve:

- Implement projects to correct any identified dam safety concerns. These could include things like structural modifications, land acquisitions below spillways, and alternative, lower risk solutions,
- Implement approved projects and land acquisitions to address urbanization issues, and,
- Continue long-term dam safety program.

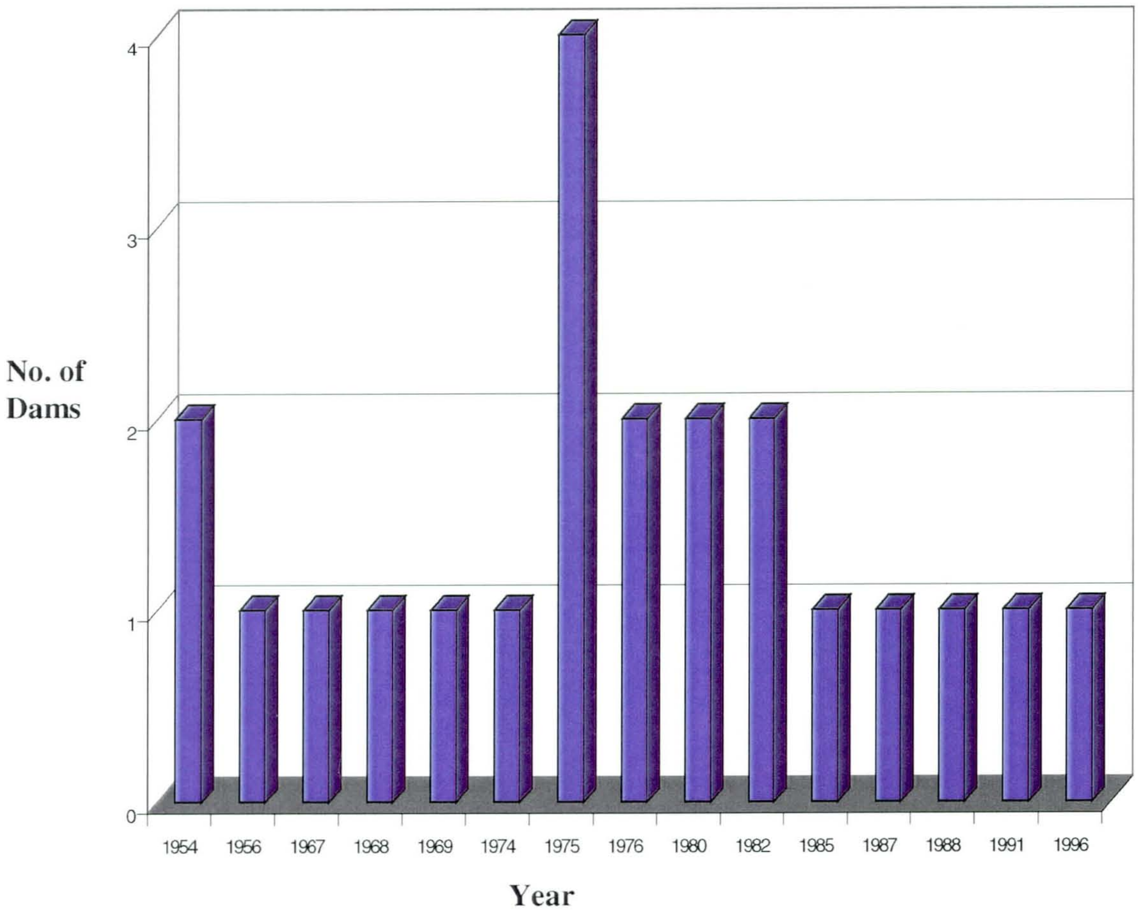
Phase I of the Structures Assessment Program is primarily an evaluation and study phase. The District has retained Kimley-Horn and Associates to provide services to conduct Phase I evaluations and studies. The first work assignment focussed on four District dams. Evaluations and studies performed for these dams will initiate the Phase I process. It is intended that the first work assignment will be a pilot study from which to establish initial District dam safety policy and programs, and from which to refine engineering and planning methods for the Structures Assessment Program. The dams evaluated in the first work assignment were the Powerline Flood Retarding Structure (FRS), the Vineyard Road Flood Retarding Structure, the Rittenhouse Flood Retarding Structure, and Cave Buttes Dam.

A steering committee serves in an advisory capacity to the District's project manager concerning the major findings and recommendations of Phase I of the program. The committee consists of representatives of the District's planning, engineering, and operations functions, Arizona Department of Water Resources Dam Safety Section, Natural Resources Conservation Service (NRCS), Corps of Engineers, and Bureau of Reclamation (USBR).

2.2 Structures Opportunities and Challenges

The Flood Control District owns, operates, and maintains twenty-two flood control dams. The dam impoundments are normally dry and only experience reservoir ponding in response to rainfall/runoff within their respective watersheds. Figure 2-1 illustrates the number of District flood control dams constructed year by year.

Figure 2-1. District Dams Constructed by Year.



The conditions under which the District dams were originally designed and constructed are somewhat different from the conditions experienced today. Many structures were originally built to protect rural, small watersheds and agricultural farmlands from flooding. Today, these same structures are now providing flood control benefits to an urban environment. Urbanization has been and is continuing to encroach upon the downstream areas of the structures as well as into and around the impoundment area reserved for the pool reservoir. The increased urbanization increases the chances for loss of life or significant economic damages in the event of a dam failure. An example of encroachment of urbanization is provided in Figures 2-2 and 2-3 for Adobe Dam.

In addition to the aging of dams and urbanization challenges, the dam safety regulatory environment has undergone changes as well. Dam safety rules, regulations, and design criteria and requirements, through changes in dam technology and dam safety experience, have been strengthened since the time the structures were originally planned and constructed. Many of the changes in dam safety regulations were retroactive and sometimes conflict with the original design of the existing dam. Changes in dam safety regulations may increase the hazard classifications of some dams from the original classification.

The existing small watershed dams were planned and constructed originally to provide, as the primary purpose, flood control benefits. In today's environmentally sensitive awareness, the structures, reservoir areas, and downstream conveyance corridors are being looked upon for further and expanded multi-use opportunities. These opportunities include recreation corridors, riparian and wildlife habitat enhancement, groundwater recharge, and educational opportunities.

The local situation and conditions appear to mirror national trends, however there are some local challenges as well. The District is faced with the same challenges experienced at the national level, but on a localized level. These include aging of dams, urbanization, and, changing dam safety regulations. Figure 2-2 provides an illustration of the encroaching urbanization at Adobe Dam. Figure 2-3 shows a ground level photograph immediately downstream of Adobe Dam.

Some of the District dams within the next 10 to 15 years will be reaching the end of their original design life. This does not necessarily mean that the dams have reached the end of their useful life, but it does point to the need for increased major maintenance activities and the need to initiate planning for the potential replacement of function. Many of these structures are showing the effects of aging and changes from the environment such as subsidence due to groundwater. Typical effects included increased sedimentation, deterioration of concrete structures, and settlement and cracking of earthen embankments.

Recent inspections of several District dams have revealed transverse and/or longitudinal cracks on the dams slopes or crests. Examination of dam safety records indicate that these same structures have had a history of cracking, crack investigations, and crack repairs. Earth fissures associated with ground subsidence have been documented in the vicinity of several District dams.

Figure 2-2. Aerial photograph of Adobe Dam showing urbanization encroachment on the downstream toe and reservoir pool area.

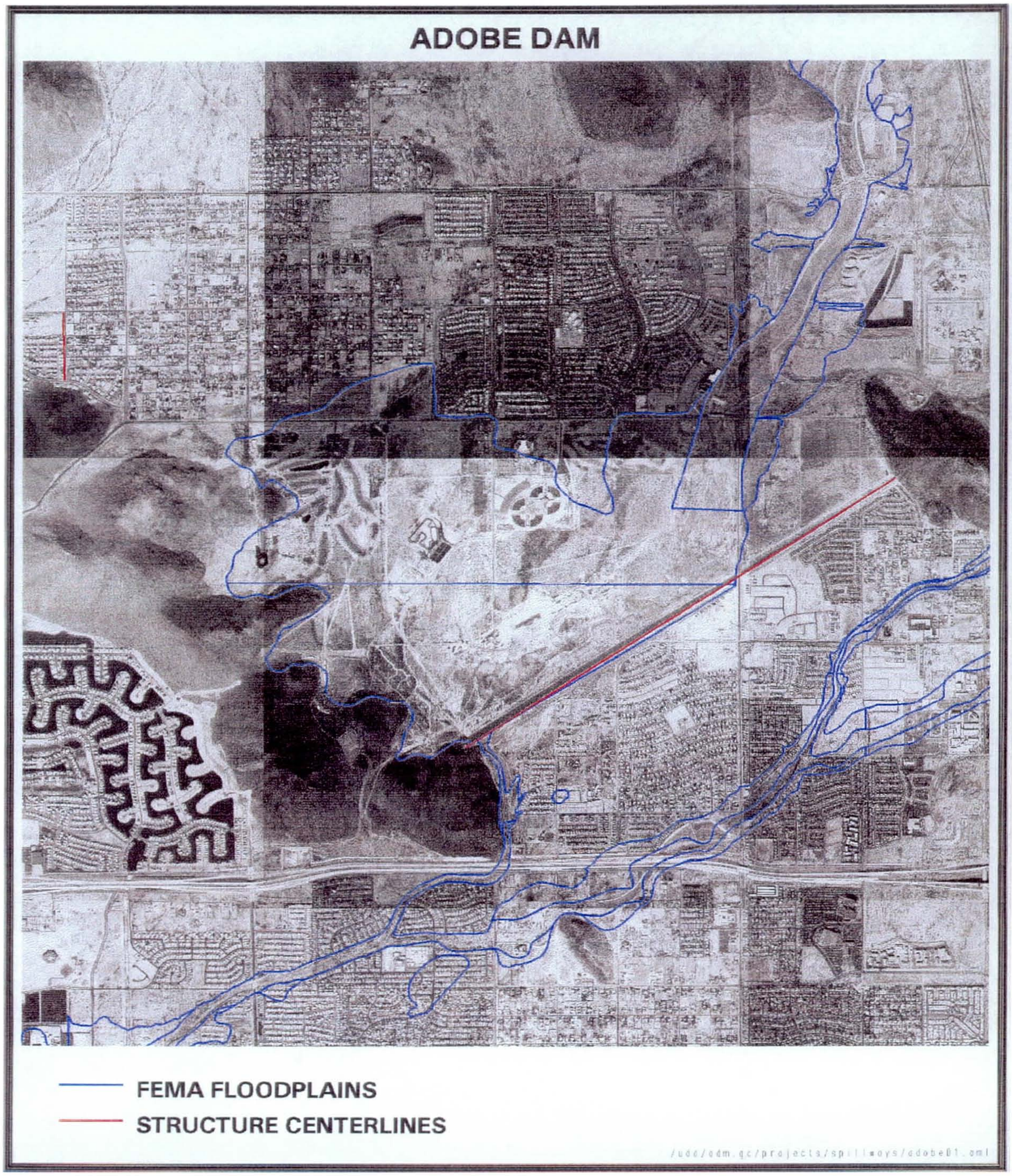


Figure 2-3. Ground level photograph downstream of Adobe Dam showing homes built adjacent to downstream toe of dam.

Opportunities facing the District now and in the near future will be the development of a strong dam safety program and a commitment of District resources to the goals of the Structures Assessment Program, commitment of qualified personnel with the capabilities to carry out the Structures Assessment Program and enhanced dam safety program, application of new dam technologies including incorporating the results of research and development from the Corps, Bureau of Reclamation, FEMA, and NRCS, and application of risk-based methodologies to dam safety.

One of the more important opportunities for the District, as part of their Structures Assessment Program, is the evaluation and assessment of each of their twenty-two flood control dams and associated features. The assessment of each structure will be conducted based upon a technical review of each structure's dam safety documentation and upon an extensive examination of the existing field conditions found at each dam. Ultimately, recommendations will be developed for further actions and investigations in regards to dam safety for each of the District's dams.

Section 3.0 Alternatives Analysis for Cave Buttes Dam

This section of the Report documents the concept structural and nonstructural measures/alternatives evaluated as part of the Cave Buttes Dam alternatives analysis. The purpose of the alternatives analysis for Cave Buttes Dam is to examine measures, alternatives, or actions that may be taken by the District to reduce the risk and liability associated with ownership of the dam. The concept alternatives for Cave Buttes Dam were formulated in a meeting between the District and KHA on February 16, 2000 and documented in the meeting minutes and in a KHA memorandum dated March 16, 2000.

The structural concept alternatives for Cave Buttes Dam were primarily formulated to provide a greater degree of operational flexibility of the dam during normal flood and emergency flood operations. A full description of the structural alternatives is provided in Section 3.2. The structural alternatives include the following concept measures:

Structural Alternatives: (Reservoir Operations during flooding event)

- Low Level Outlet – Dike No. 2. Examine the feasibility at a concept level for providing a new principal spillway or a low-level outlet in Dike No. 2.
- Divert Emergency Spillway Flow to Central Arizona Project (CAP) canal. Utilize CAP canal to carry discharged waters from emergency spillway up to capacity of CAP canal.
- Low Level Outlet – Dike No. 3. Divert stormwater from the reservoir pool through low level outlet in Dike No 3.

The nonstructural concept alternatives for Cave Buttes Dam were primarily formulated to reduce the risk and liability associated with ownership of the dam. A full description of the nonstructural alternatives is provided in Section 3.3. The nonstructural alternatives include the following concept measures:

Below Dam - Nonstructural Measures:

- Mitigate through Flood Insurance
- Downstream Flowage Easements
- Update Emergency Action Plan

Pool Area - Nonstructural Measures:

- Mitigate through Flood Insurance
- Acquire Properties/Flowage Easements
- Develop Emergency Action Plan for Pool area

The formulation, discussion, evaluation, and presentation of the Cave Buttes Dam alternatives are presented later in this section. The following discussion provides a brief description, purpose, and physical characteristics of the dam and associated features.

3.1 Description of Cave Buttes Dam

The Cave Buttes Dam is part of the U.S. Army Corps of Engineers "New River and Phoenix City Streams" regional flood control project. The project included construction of four earthfill dams designed to provide standard project flood protection (Dreamy Draw Dam, Cave Buttes Dam, Adobe Dam, and New River Dam); the construction of 17.3 miles of channelization along the Arizona Canal (Arizona Canal Diversion Channel – ACDC) designed to intercept 100-year frequency flood flows; acquisition of flowage easements; and floodplain management below the dams.

3.1.1 Purpose of Dam

The purpose of Cave Buttes Dam is to provide flood and erosion control protection for Cave Creek Wash. Cave Buttes Dam was designed to retain the Standard Project Flood and the emergency spillway inflow design flood is the probable maximum flood.

The reservoir behind the dam is 1,820 acres with a capacity of 46,600 acre-feet. A permanent pool will not be retained in the reservoir, instead, the dam and reservoir are designed to trap floodwater and store it only for as long as it takes to release it slowly and safely downstream. Reservoir capacity is then restored to handle a future flood.

The emergency spillway is located 2,000 feet west of the west abutment of the main dam. Construction of the dam and appurtenant structures was completed in October 1979.

3.1.2 Dam Location

Cave Buttes Dam is located on Cave Creek Wash in Maricopa County, Arizona. Cave Buttes Dam is located on Cave Creek Road about 17 miles north of downtown Phoenix and less than a mile downstream of the existing Cave Creek Dam. The project consists of the main dam structure, a detached emergency spillway, three dikes, and an overlook structure. Figure 3-1 provides a location map of Cave Buttes Dam.

Located upstream of the dam is the non-operational Cave Creek Dam. This dam is within the impoundment pool reserved for Cave Buttes Dam. Cave Creek Dam is a concrete multi-arched dam. One of the three gates for the principle spillway has been removed. The other two gates have been permanently raised in the full open position.

Cave Buttes Dam is classified as a large, high hazard dam. The U.S. Army Corps of Engineers completed construction of the dam and dikes in October 1979. The drainage area for Cave Buttes Dam is 191 square miles.

3.1.3 Physical Features

Cave Buttes dam is a rolled earth-filled zoned structure. The length of the dam is 2,275 feet with a maximum height of 190 feet and a crest width of 20 feet. The reservoir capacity is 46,600 acre-feet with a maximum water surface elevation of 1678.1 feet. The dam was designed with 5 feet of freeboard. The peak design inflow is 54,000 cfs and the design outflow is 500 cfs from the principal spillway. The slopes of the dam are protected with cobble riprap both upstream and downstream. The main dam is accessible by using Cave Creek Dam Road off of Cave Creek Road. Access is controlled by a padlocked gate. The maximum recorded impoundment for Cave Buttes reservoir is 17,592 acre-feet with a stage of 75.9 feet at the dam (January 11, 1993). The upstream and downstream slopes are lined with riprap cobbles and stone.

Dike No. 1 is located just east of the main dam between a saddle created by two rock outcrops. The length of the dike is 935 feet and is also a rolled earthfilled zoned structure. The slopes are protected with cobble riprap both upstream and downstream. The crest width is 20 feet. The dike is designed to contain the full pool reservoir.

Dike No. 2 is located east of the main dam. Primary access to Dike No. 2 is off of Cave Creek Road, one-quarter mile south of Jomax Road. Access is by a padlocked gate. Dike No. 2 is also a zoned earthfilled rolled structure. The length of Dike No. 2 is 9,005 feet and has a crest with of 20 feet. The slopes are protected with cobble riprap both upstream and downstream. Cave Creek Road ramps up and over Dike No. 2. The road essentially bisects the dike.

Dike No. 3 is located approximately 2.6 miles north of the main dam. Primary access to Dike No. 3 is by an existing dirt road that skirts east of the Old Cave Creek Dam. Dike No. 3 is an earthfilled structure. The length of Dike No. 3 is approximately 3,200 feet and has a crest with of 20 feet. The slopes are protected with cobble riprap both upstream and downstream.

The principal spillway in an ungated concrete structure 7.5 ft by 7.5 ft square with a 45-inch concrete outlet pipe approximately 548 feet long. The trash rack is located on the upstream inlet. The structure has several square orifice opening in the walls and is open on the top (with a debris screen). The outlet of the principal spillway discharges into a constructed channel through a outlet structure. An energy dissipater is located on the downstream end of the concrete outlet structure. A pedestrian/inspection bridge spans the outlet channel.

The detached emergency spillway was excavated into rock and is located 2,000 feet west of the main dam. The spillway is approximately 540 feet wide. A concrete sill spans the width of the spillway and is located approximately one-third of the way into the spillway. The sill crest elevation is 1657.1 feet (MSL).

Station monuments are located along the crest of the main dam and training dikes. A series of staff gages are located on the upstream east groin of the main dam. A water level recorder house is located on the east end crest of the main dam. The gage well is located in the proximity of the toe of the east upstream groin and the inlet of the principal spillway. Settlement monuments are located along the crest of the dam, Dike No. 1, and dike no 2. Reference monuments are located at the rock outcrops at the abutments for the main dam, Dike No. 1, and Dike No. 2. A review of the as-built drawings indicate that Dike No. 3 has neither settlement monuments or reference monuments.

Table 3-1 provides a summary of the physical structure data for Cave Buttes Dam.

Table 3-1. Cave Buttes Dam Structure Physical Data.

ITEM NATDAM ID AZ1007 STATE ID 07.58	PHYSICAL DATA
Drainage Area	191 sq mi
Dam (rolled earthfill)	
Crest elevation	1679.1 ft
Maximum height above streambed	109
Crest length	2,260 ft
Freeboard	5.0 ft
Emergency Spillway (detached)	
Crest elevation	1657.1 ft
Crest length	510
Elevation of max water surface	1674.1
Principal Outlet Works (ungated conduit)	
Diameter of conduit	3.75 ft (45 in)
Length	528 ft
Intake Elevation	1560.7 ft
Saddle Dike No. 1	
Crest Length	930 ft
Maximum height above existing ground	39 ft
Saddle Dike No. 2	
Crest Length	9,035 ft
Maximum height above existing ground	55 ft
Saddle Dike No. 3	
Crest Length	3,245 ft
Maximum height above existing ground	10
Reservoir Area at spillway crest	1,820 ac
Capacity (gross) at spillway crest	46,600 af
Storage allocation below spillway crest	
Flood control (net)	40,900 af
Sedimentation	5,700 af
Standard Project Flood (Reservoir Design Flood)	
Total volume	42,200 af
Peak inflow	54,000 cfs
Peak outflow	486 cfs
Drawdown time	48 days
Probable Maximum Flood (PMF = IDF)	
Total volume	122,000 af
Peak inflow	172,000 cfs
Peak outflow	100,600 cfs
Drawdown time	61 days
Hazard Classification	**High**
Size of Dam	**Large**

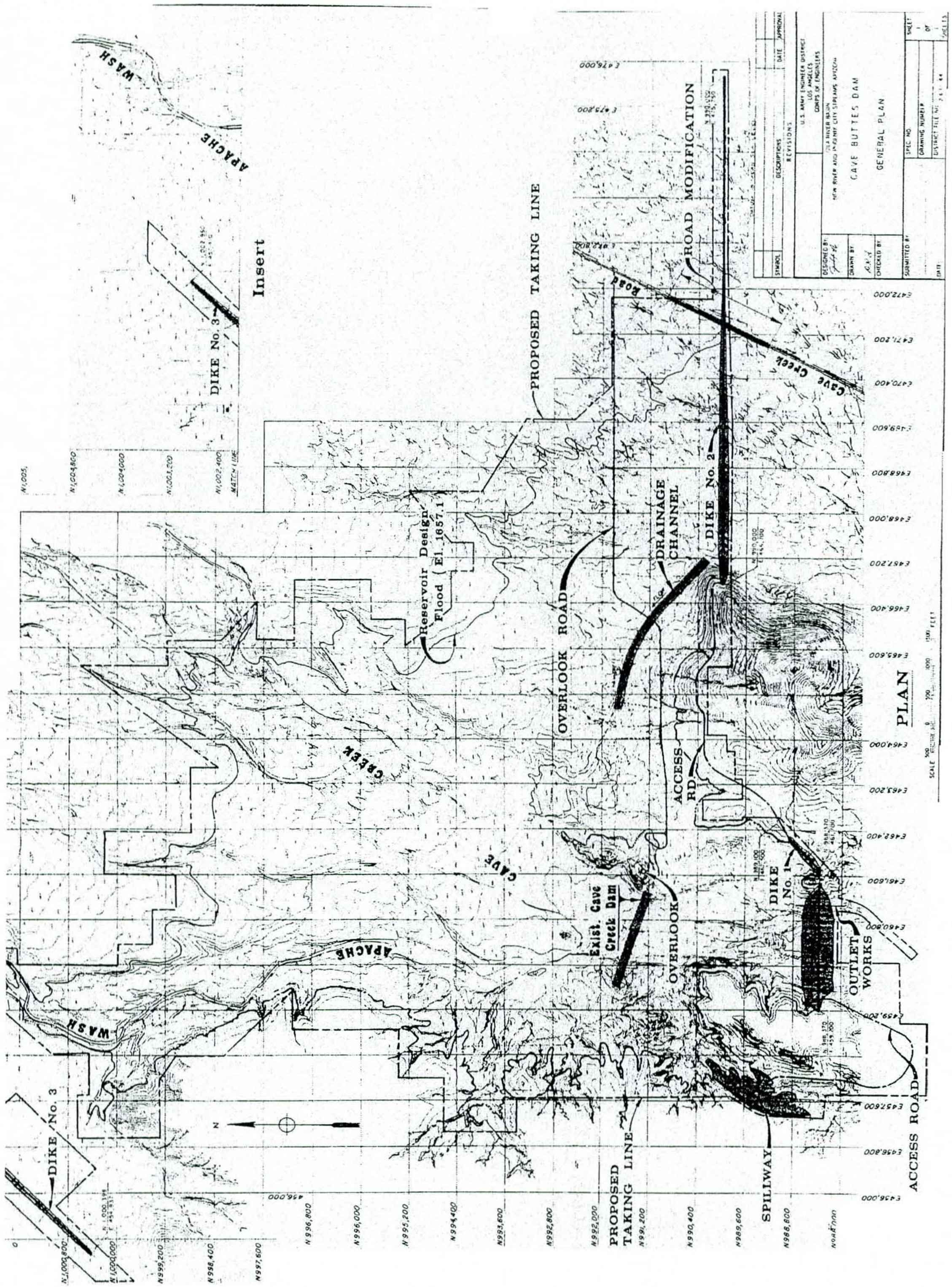


Figure 3-1. Cave Buttes Dam Location Map.

3.1.4 Cave Buttes Dam Operations

In the event of a forecast for measurable rainfall that may produce significant runoff on the Cave Creek Wash watershed and hence potentially an impoundment at the dam, the Flood Control District initiates the flood response actions outlined in their "Flood Emergency Response Manual". The manual is the District's standard operating procedure, the purpose of which is to provide an outline of the duties and responsibilities for District personnel to act upon during significant rainfall events and/or flood emergencies. The procedure provides an overview of how the District as a whole will respond to significant rainfall events and floods emergencies, and therefore does not outline procedures for specific personal tasks and responsibilities. According to the manual, the District division managers will assign personal tasks and responsibilities as necessary.

The manual outlines the tasks and required response actions, both in summary and in detailed format, for the Chief Engineer, Operations and Maintenance Division, Engineering Division, Planning and Project Management Division, Land Management Division, and Information Technology Division in response to a flood emergency. It should be noted that the flood emergency response manual is geared for all of the District structures including drains, floodways, major river corridors (Salt, Verde, Gila, and the Agua Fria Rivers), District dams, and for non-District dams (Town of Fountain Hills structures).

The manual contains a list of District emergency telephone and cellular numbers of District personnel. This list was last updated on July 1, 1999. Also included in the manual are selected bridge capacities and closure data on major rivers, Salt River and Gila River flow travel time tables, District dam and detention basin rating curves, and a listing of USGS stream gages with names and locations at pertinent sites.

Maricopa County has been divided into twelve observation areas for purposes of flood monitoring. Each of the twelve areas has specific instructions for observations so that District staff members who may not have been trained on a specific structure will know what to look for at each site in their area. Primary and secondary observation points have been identified, and team members are to proceed to these sites in the order noted in the manual or as dictated by local flooding. Area staff assignments require teams to observe these areas on two twelve-hour shifts during an emergency. Table 1 of the Flood Emergency Response Manual contains a listing of the flood observation points (primary points) for each of the twelve areas. The manual provides detailed observation instructions for each team assigned to each of the twelve areas of the County.

Initial emergency flood response is based on radar and rainfall information as it provides the most lead-time and is the best available data. The District's real-time telemetry system consists of a network of rain and stream gages. The system has the capability of sounding alarms when preset rainfall intensity levels are exceeded.

Once rainfall alarms are activated and a thunderstorm or flash flood warning/watch has been issued the rainfall and runoff from the event is monitored and evaluated. The sources of information used for this purpose come from telemetry staff gages, field observations of staff gages, and stage/discharge rating curves.

The District maintains and operates a number of flood control structures, including dams, throughout the County. The Flood Warning/Data Collection Branch monitors water depths behind the dams and through the principal outlet throughout an event. All of the dams have telemetered stage gages, which measure impoundment depth. ALERT alarm, O&M notification levels, and Emergency Operations Center notification levels are set for each dam. Table 4 of the manual lists the emergency notification elevations at each of the District dams (as of July, 1999).

The District's flood emergency response is linked and coordinated with the Maricopa County Department of Emergency Management (DEM). The Department has prepared an Emergency Operation Plan (EOP) for Maricopa County that not only includes response and actions in the event of flooding and dam failures, but also covers catastrophes due to aircraft crashes, earthquakes, fires and explosions, hazardous materials incidents, and national security emergencies. Annex G of the EOP specifically addresses storms and floods. The annex includes a discussion on the dissemination of weather data and information, water release warning procedures, dam failure scenarios for the major dams in the vicinity of the Phoenix metropolitan area (e.g. Salt and Verde River dams; Aqua Fria River), and for each major District dam. The appendices to Annex G provide individual information and descriptions for each of the District dams. This information includes the location of the dam, a physical description of the structure, and the purpose of the dam (i.e., provide flood protection). Included with each appendix is gross inundation mapping that shows what areas of Maricopa County that could experience flooding due to failure of a specific dam. The appendices outline the specific tasks and actions that the District will follow in the event of a dam failure, the actions and responses of the DEM, and the notification procedures. The appendices provide a written description of the area to evacuate in the event of a dam failure.

The U.S. Army Corps of Engineers has prepared an "Operation, Maintenance, Repair, Replacement, and Rehabilitation Manual" (OMRRR) for the Phoenix, Arizona and Vicinity (including New River) flood control project that includes Cave Buttes Dam. Part IV of the manual addresses flood operation procedures for Corps structures that are an element of the Phoenix, Arizona and Vicinity flood control project. The operation program, which specifies flood-operation procedures, consists of four phases: pre-stormflow, initial stormflow, final stormflow, and post stormflow. Each phase is characterized by a degree of mobilization or demobilization – a patrol procedure which includes inspection, operation of field facilities such as gates and staff gages, and any immediate maintenance, and a reporting requirement. Part IV outlines the pertinent information for each of the four phases.

During a flooding event when an impoundment occurs at the Cave Buttes Dam, the dam operates in a passive manner. In other words the principal spillway discharges impounded floodwater behind the dam at a rate that depends on the stage of impoundment. There is no operating gate on the 45-inch concrete pipe principal spillway. Minimal District operations and maintenance intervention is required for the operation of the dam and spillways during an event. Operations during a flooding event is mainly a monitoring mode activity.

The emergency spillway is also ungated due to the nature of the dam being a dry flood control dam. The discharge rate from the emergency spillway will depend on the impoundment stage and depth of flow within the spillway channel.

During a standard project flood (SPF), the reservoir would be fully impounded to an elevation of approximately the spillway crest elevation (1657.1-ft). As water recedes within the reservoir, water detained behind Dike No. 2 flows toward the main dam via a constructed drainage channel that connects the Dike 2 impoundment area with the main dam impoundment pool. Dike No. 3 was constructed at a topographic saddle. This dike functions to contain the IDF pool and prevents discharge out of the pool during the IDF and into an adjacent watershed.

There are no other passive facilities or methods in-place to withdraw impounded water stored within the reservoir area for Cave Buttes Dam. No permanent water storage or pool is allowed within the impoundment area according to the ADWR operating license issued to the District for the dam.

3.2 Structural Alternatives

The structural alternatives for Cave Buttes Dam were developed and formulated in a planning meeting held between the District and KHA. The structural alternatives were developed to examine the feasibility of expanding or increasing the operational flexibility of the dam and associated facilities during flooding events. The primary purpose of increasing the operational flexibility of Cave Buttes Dam and reservoir is to evacuate the impoundment as quickly (and safely) as possible, utilize secondary spillway outlets to direct impounded water to other regional adjacent flood control structures, and gain flexibility in the management of flows from the contributing upstream watershed. The structural flood control alternatives that were formulated for concept development and analysis for Cave Buttes Dam are listed below and are described in detail in the following subsections.

- Low Level Outlet in Dike No. 2.
- Divert to Central Arizona Project Canal from Emergency Spillway.
- Divert Stormwater from Reservoir Pool through Low-Level Outlet in Dike No. 3.

3.2.1 Low Level Outlet in Dike No. 2

This structural alternative examines the feasibility at a concept level for providing a low-level outlet in Dike No. 2. The operational flexibility with this alternative is the alternative would allow the District to directly discharge impounded water from behind Dike No. 2 instead of waiting for the impoundment behind Dike No. 2 to reach a depth sufficient to flow over to the main dam and hence add to the filling of the main reservoir. In this fashion flood pool volume in the main reservoir would not be filled and volume could be available for potential follow-up storms. With a low-level outlet in Dike No. 2, the District would have the flexibility to either continue operations as normal under flood conditions or divert flows out from Dike No. 2. Conceptually, the low-level outlet in Dike No. 2 would discharge into a newly constructed channel that would then outlet flows to the Bureau of Reclamation CAP Reach 11 embankment dam and reservoir (Paradise Valley Detention Dikes).

Under present conditions, floodwaters impounded by Dike No. 2 are conveyed to the main impoundment area directly north of the main dam through a drainage bypass channel (see Figure 3-1). Installation of new low-level outlet structure in Dike No. 2 would reduce the amount of floodwaters released through the principal and the emergency spillways of the main dam during a major flood event. Floodwaters would be directed to the Reach 11 embankment dams. A benefit to the CAP canal is that this alternative may reduce the potential of damage to the CAP from flood flows released through the Cave Buttes Dam emergency and principal spillways.

The location of the proposed outlet structure is provided in Appendix A, Exhibit A, and Exhibit B and is shown to be located near the right abutment of Dike No. 2. The location of the outlet structure was selected on what appears as a historical drainage channel that existed prior to the construction of Dike No. 2. This was confirmed after reviewing the USGS quad map (Union Hills, photo-revised 1981). A review of the as-built profile of Dike No. 2 indicates that the low point of the existing ground is at the right abutment for Dike No. 2.

The capacity of the new outlet structure was selected to match the outlet capacity of a Bureau of Reclamation Reach 11 outlet structure. The Bureau of Reclamation Reach 11 dams have a total of four 750-cfs gated outlets that serve as the primary spillways for the Reach 11 dams. These dams do not have emergency spillways. In the event of a storm event of such a magnitude that would warrant a release from any of the Bureau outlets, the outlets would discharge into the CAP canal.

A gated box culvert outlet structure approximately 10-ft span by 6-ft high would need to be constructed to have a design discharge capacity of 750-cfs. The flows released from the new outlet structure would be conveyed in a newly constructed flood channel. Exhibit B shows the proposed alignment of the channel. The channel would be constructed along or replace the existing natural drainage wash and cross under Cave Creek Road near Pinnacle Peak Road. The flood channel would parallel the east side of Cave Creek Road to terminate at Reach 11. The flood channel would be trapezoidal in shape, concrete-lined, with a 12-foot bottom width, and 1.5 (H) to 1 (V) side slopes. The slope of the channel from the USGS maps was determined to be approximately 0.005 ft per foot. The depth of flow in the channel for 750-cfs is 3.2-ft.

The estimated cost of a low-level outlet in Dike No. 2 is approximately $2.1 million. Cost estimate data is provided in Appendix A.

The addition of Cave Buttes Dam floodwaters to the Reach 11 impoundment area (Dike No. 1) would need to be approved the Bureau of Reclamation and the Central Arizona Project (CAP). A regional flood control operations plan would most likely need to be formulated between the District, the Bureau, and the CAP. Discussions with the Bureau of Reclamation on the design event for Dike No. 1 and Dike No. 2 of the Reach 11 dams indicate that the two Dikes provide flood protection in excess of the 200-year storm. The remaining two dikes (Dikes No. 3 and No. 4) were sized to retain the full PMF volume from the respective watersheds.

3.2.2 Divert Emergency Spillway Flows to Central Arizona Project Canal

This structural alternative examines the feasibility at a conceptual level of discharging floodwaters from the Cave Buttes Dam emergency spillway to the CAP canal. The CAP canal is located approximately 1.5 miles south of the dam. Under this alternative the CAP canal would be utilized to carry floodwaters up to the capacity of the canal. The capacity of the CAP canal according to the Bureau of Reclamation is 3,000 cfs.

At the present time there is no constructed channel downstream of the emergency spillway that has the capacity to handle the full PMF outflow from the dam. There is a CAP canal overchute structure at the Cave Creek crossing, but this structure was designed with a capacity for passing the 100-year discharge from the principal spillway of the dam. This structure is severely undersized to handle a PMF discharge from the emergency spillway. The study "Delineation of Spillway Flows for Cave Buttes Dam" conducted by Michael Baker Jr. Engineers in October 1996 indicates that severe ponding will occur on the upstream side of the CAP canal as a result of PMF discharge from the emergency spillway. The study concluded that overtopping of the canal from the PMF event will most likely occur (although this condition was not investigated as part of the study).

This alternative was formulated to allow the partial diversion of floodwaters from emergency spillway into an already constructed conveyance facility instead of overtopping the canal and potentially discharging into the downstream urbanized areas. Discharging into the (evacuated) CAP canal for a least some of the spillway flows may allow more time for emergency action response and reduce the probability of overtopping the CAP canal. It is not the purpose of this analysis neither to evaluate the potential lessened overtopping of the canal from emergency spillway discharges nor to quantitatively evaluate diverting up to 3,000 cfs into the CAP canal on inundation limits, flow, and ponding depths. It is assumed that once flows are diverted into the CAP up to 3,000 cfs, no additional flows can enter the canal, from either from District or Bureau facilities (Reach 11), and flows above this quantity from the dam continue in an inundation manner such as depicted in the Baker study.

Under this alternative, for Cave Buttes Dam emergency spillway flows of 3,000 cfs or less would outlet to the CAP. This is not to say that any event that produces a discharge in the emergency spillway would discharge 3,000 cfs to the canal. The alternative provides the District the option of delivering discharges up to the capacity of the CAP canal. Several constraints regarding the operation of the CAP under flooding conditions at Cave Buttes Dam and the Reach 11 dams must be identified and resolved before this alternative could be realized institutionally.

A similar discharge operation currently exists that is utilized by the Bureau of Reclamation for their Reach 11 dams. The Bureau of Reclamation owns the Reach 11 dams (also known as the Paradise Valley Detention Dikes) that are just on the north side of the CAP. Reach 11 of the CAP extends from Cave Creek Road to east of Pima Road. The Reach 11 dams were constructed to protect the CAP from floodwaters generated from the upstream watershed.

The Reach 11 dams were constructed with a total of four-gated outlet structures and no emergency spillways. The outlet structures were each sized to convey 750-cfs through the gates. The flows from the outlet structures are discharged directly into the CAP canal. The standard operating procedures of the outlets under flooding conditions in the impoundments of Reach 11 were established by the Bureau and are published in the standard operations plan for Reach 11. The canal requires to be drained in order for the Reach 11 outlets to discharge to the canal. Briefly, this plan includes:

- The CAP canal is drained by diverting inflow at the upstream Aqua Fria River diversion outfall. The remaining water in the Reach 11 canal downstream of the Aqua Fria River outfall is drained to the Salt River/CAP interconnect..
- Once the canal is drained (or relatively drained), the Reach 11 dam outlet gates are opened, based on depth of water in the reservoirs,
- The outlet structures discharge 750-cfs each to the canal for a total discharge of 3,000-cfs. The CAP canal design capacity is 3,000-cfs.
- The CAP canal serves as the primary outfall for the Reach 11 dams. The dams have no other outfall.

This structural alternative for the Cave Buttes Dam would essentially require the same CAP canal evacuation steps to occur in order for emergency spillway flows to be discharged into the CAP canal. The procedure is already in place to drain the CAP canal in order for the canal to accept floodwaters from the Reach 11 dams. What remains to be constructed under this alternative is a floodway channel with a capacity of 3,000-cfs from the emergency spillway to the CAP canal and a confluence structure at the canal for flows from the channel to enter the CAP canal.

Initial hydraulic analysis indicates that for a capacity of 3,000-cfs, a trapezoidal concrete-lined channel would have to be constructed with a bottom width of approximately 12-ft for a flow depth of 4.6-ft. The slope of the channel used for the analysis was 0.02 ft per foot. This is a fairly steep channel that results in a high flow velocity. A stepped channel may be required upon additional detailed analysis to break the channel grade between a relatively steep upper reach to a relatively flatter lower reach. An alignment was assumed to allow the channel to discharge into the CAP just east of the 7th Street crossing of the CAP canal. The invert of the CAP at the 7th Street bridge is approximately 1494-ft. A check of datums used between the Bureau as-builts of the CAP canal and the aerial mapping for the Baker study should be conducted. It appears that both sets of mapping agree fairly well.

A confluence structure would be required to join the trapezoidal channel with the CAP canal. Two types of structures appear to be feasible – either closed (pipe or box culvert types) and open (open channel types). The CAP canal north embankment is higher than natural ground at the canal. An open type confluence structure would require breaking through the embankment and the CAP canal lining. Floodgates would replace the CAP canal lining at the point of confluence and would be closed under normal operating conditions in the CAP canal. When the canal is drained under emergency conditions, the floodgates on the confluence structure would be opened to allow floodwaters from the Cave Buttes emergency spillway to enter the CAP canal.

The estimated construction cost for diverting emergency spillway flows to the CAP canal is approximately $1.9 million. Cost estimation data is provided in Appendix B.

The concept of addition of Cave Buttes Dam floodwaters to the CAP canal would need to be approved the Bureau of Reclamation and the Central Arizona Project, who are responsible for the operation and maintenance of the Reach 11 dams and the canal. A regional standard operations and flood control emergency operations plan would most likely need to be formulated between the District, the Bureau, and the Central Arizona Project in order to institute this alternative.

There may not be a conflict, for localized storms, for allowing floodwaters to enter the CAP canal at the 7th Street location. However, for larger regional storms which cover both the Cave Buttes Dam watershed and the Reach 11 watershed, capacity in the CAP canal would be an issue and most likely the Bureau would have priority over the District.

3.2.3 Low-Level Outlet in Dike No. 3

This structural alternative examines the feasibility at a conceptual level of discharging floodwaters from the Cave Buttes Dam reservoir through a proposed new low-level outlet located in Dike No. 3. The operational flexibility with this alternative is that it allows the District to directly discharge impounded water from the Cave Buttes reservoir through Dike No. 3 and into the watershed for Skunk Creek. As stated above, Dike No. 3 lies in a topographic saddle that delineates the Cave Creek watershed from the Skunk Creek watershed. Skunk Creek is impounded by another District earth embankment dam – Adobe Dam.

This alternative could allow the District the flexibility to manage the reservoir pool(s) for Cave Buttes Dam (and Adobe Dam). Diverting flows to Skunk Creek and hence to Adobe Dam allows the District to have flood storage volume available in Cave Buttes

Dam in the event of extremely large flood events on the Cave Creek watershed and not on the Skunk Creek watershed.

The disadvantage of this alternative is operational. The low-level outlet in Dike No. 3 would be used only in very extreme flooding events (such as the PMF). The flood pool inundation limits for the SPF do not reach Dike No. 3, only the PMF limits. The cost of this alternative, given the low probability that the low-level outlet may never be used, may make this structural alternative unfeasible.

Dike No. 3 is located approximately 2.6 miles north of the main dam (see Figure 3-1). This structure was designed and constructed to detain the full PMF flood pool and not to spill impounded water into the adjacent Skunk Creek watershed.

A gated twin concrete box culvert outlet structure approximately 8-ft span by 4-ft high would need to be constructed to have a design discharge capacity of 100-cfs. The design constraint at this location is that the maximum water surface in the Cave Buttes reservoir pool associated with the PMF is 1671.1-ft. The invert of the twin box culverts was estimated to be 1669.5-ft. The maximum headwater is therefore 1.6-ft. The SPF flood pool elevation (1657.1-ft) is below the invert of the outlet culverts. The top of the Dike is at elevation 1679.1-ft.

The estimated construction cost for a low-level outlet in Dike No. 3 is approximately $132 thousand. Cost estimation data is provided in Appendix C.

The flows released from the new outlet structure would be conveyed in a newly constructed short segment of earth flood channel. Exhibit A shows the proposed location and alignment of the channel. The channel would be constructed to allow released floodwaters from Cave Buttes reservoir to drain toward Skunk Creek. The channel would transition to eventually reduce flow depths such that flows would spread out and become overland flow.

As a modification of this alternative the District may consider a fuse-plug in Dike No. 3 instead of a gated outlet structure. Or, as another modification, the District may simply breach the Dike. The downstream flood channel would still be constructed under these modifications. No further evaluation of the fuse-plug or breach concepts are provided in this report.

3.3 Nonstructural Alternatives

The nonstructural alternatives for Cave Buttes Dam were developed and formulated in a planning meeting held between the District and KHA. The nonstructural alternatives were developed to reduce the risk and liability associated with the operations of the dam during a flooding event at the dam. The nonstructural alternatives or measures were formulated to address risk exposure to the District at a dam for the area immediately downstream of the dam and the impoundment area at a dam. Descriptions of the concept nonstructural measures/alternatives are provided below and are grouped into measures downstream of Cave Buttes Dam and measures at the reservoir pool area.

A. Below Dam - Nonstructural Measures

The area south of the CAP canal downstream of Cave Buttes Dam is highly urbanized. However, the area between the Cave Buttes Dam to the CAP canal is presently somewhat vacant. Light industrial land use occurs west of Cave Creek Wash and north of the CAP canal. Residences have been constructed along Cave Creek Dam Road, Cave Creek Road, and Pinnacle Peak Road. The "below dam" nonstructural flood control alternatives that were formulated for concept development and analysis for Cave Buttes Dam are listed below and are described in detail in the following subsections.

- Mitigate Through Flood Insurance
- Downstream Flowage Easements
- Update the Emergency Action Plan

B. Pool Area - Nonstructural Measures

The pool area of Cave Buttes Dam is presently vacant land. One structure exists within the pool area. This structure is the Old Cave Creek Dam. A model airplane recreational facility is located north of Dike No. 2 within the pool area limits. The "pool area" nonstructural flood control alternatives that were formulated for concept development and analysis for Cave Buttes Dam are listed below and are described in detail in the following subsections.

- Mitigate Through Flood Insurance
- Downstream Flowage Easements
- Develop Emergency Action Plan

3.3.1 Below Dam - Mitigate Through Flood Insurance

This nonstructural alternative examines the feasibility at a concept level of obtaining flood insurance for the area between Cave Buttes Dam and the CAP canal. The concept is the District would obtain flood insurance on a property by property basis for structures within the area inundated by the full PMF downstream of the emergency spillway to the CAP canal. The report titled "Delineation of Spillway Flows for Cave Buttes Dam" conducted by Michael Baker Jr. Engineers in October 1996 provides the full PMF inundation limits downstream of the emergency spillway to the CAP canal.

The FEMA flood insurance rate map (FIRM Map No. 04013C1210 F September 30, 1995) indicates the limits of the 100-year floodplain and floodway for Cave Creek Wash downstream of Cave Buttes Dam to the CAP. This panel was based on the 100-year floodplain/floodway delineation of Cave Creek Wash conducted by Burgess & Niple in March 1991 for the District. The 100-year discharge for this segment of Cave Creek Wash is based on the outflow from the principal spillway of Cave Buttes Dam.

FEMA flood insurance rates are based on FIRM map zones and zone designations. The zones indicate which premium rates would be applicable for the structure(s) in question. The area between Cave Buttes Dam and the CAP canal falls within two zone designations. These zones include Zone AE – which includes the 100-year floodplain for Cave Creek Wash and Zone X.

From review FEMA's Flood Insurance Manual (May 2000) and from discussions with the Flood Insurance Administration regarding flood insurance coverage indicates that FEMA offers flood insurance coverage on a property by property basis. Area coverage is not available through the FIA flood insurance program. One concept recently discussed in Washington, D.C. is the idea of residual risk flood insurance for areas protected by flood control structures. In concept this would lead to low insurance rates, perhaps allowing for group verses individual policies. We would urge that the District promote this concept with the FIA and professional associations to gain support for legislative initiatives.

Limits of flood insurance coverage for residential structures is $250,000 and for commercial properties is limited to $500,000. The FIA did indicate that private sector flood insurance can provide flood insurance coverage on an area basis as well as a schedule of structures basis (can have a variety of structures covered under the same policy). Many commercial interests purchase private sector flood insurance as opposed to FEMA/FIA insurance flood insurance due to the coverage limits of the FEMA program.

A number of factors are considered in determining the premium for flood insurance coverage. These factors include:

- Amount of coverage purchased
- Location (Flood Zone)
- Age of the structure/building
- Building occupancy
- Design of the building
- For buildings in Special Flood Hazard Areas, elevation of the building
- Buildings eligible for special low-cost coverage at a pre-determined, reduced premium rate are single-family and 1 – 4 family dwellings located in zones B, C, and X.

The average coverage and premium data as of May 1, 2000 for flood insurance is provided in Table 3-2.

Table 3-2. Average Flood Insurance Coverage and Premium.

Occupancy Type	Regular Program	
	Coverage	Premium*
Single Family	$124,300	$570
Two to four family	$101,700	$524
Other residential	$85,900	$665
Non-residential	$218,600	$1,514

* Premium values are based on Pre-FIRM Special Flood Hazard Area rates and includes Federal Policy Fee & Expense Constant. Date as of May 2000.

If a building or structure is located in a low-risk area, which is a B, C, or X zone on the current flood insurance rate map for the area, the building may be eligible for the Preferred Risk Policy. The policy covers both the building (residence and contents) with one premium, which can be as little as $106 per year. The savings are about 30% of the standard application premium costs if a Preferred Risk Policy is purchased. For example, the premium rate under the Preferred Risk Policy for $100,000 coverage and $25,000 for contents (without basement on a residential building) is $221 per year. Under the standard application the premium rate for $100,000 coverage is $351 per year.

The Michael Baker Jr. report titled "Delineation of Spillway Flows for Cave Buttes Dam" (October 1996) delineated the flooding inundation limits for the 1/3, 2/3, and full PMF discharge downstream of the emergency spillway to the CAP canal. The limits of the full PMF discharge downstream of the emergency spillway delineated by Baker covers an area of approximately 644 acres.

The FEMA flood insurance rate map (FIRM Map No. 04013C1210 F September 30, 1995) provides the 100-year floodplain inundation limits for Cave Creek Wash below Cave Buttes Dam to the Central Arizona Project canal. FIRM designation Zone AE inundation area is fairly small compared to the inundation limits for the full PMF from the dam to the CAP canal. The 100-year Cave Creek Wash floodplain covers an area of approximately 85 acres. The other zone designation within the PMF limits includes Zone X. The approximate number of acres within Zone X is 559acres.

Table 3-3 provides the approximate costs of flood insurance for the area downstream of the Cave Buttes Dam emergency spillway to the CAP canal. The annual premiums assume $100,000 of flood insurance coverage for a single family residence and two dwelling units per acre.

Table 3-3. Approximate Flood Insurance Costs Downstream of Cave Buttes Dam.

Zone	No. Acres	Dwelling Units	Unit Premium	Annual Premium	30-Year Premium
AE	85	170	$301	$51,170	$1,535,100
X	559	1,118	$221	$247,078	$7,412,340

3.3.2 Below Dam - Downstream Flowage Easements

This nonstructural alternative examines the feasibility at a concept level of obtaining flowage easements downstream of the emergency spillway to the CAP canal. As stated previously in the description of the structural alternatives for Cave Buttes Dam, there is no floodway downstream of the Cave Buttes Dam emergency spillway to handle emergency spillway discharges. The inundation limits for the full PMF spillway discharge have been delineated in the report titled "Delineation of Spillway Flows for Cave Buttes Dam" conducted by Michael Baker Jr. Engineers in October 1996.

The District is required through a cooperative agreement with the Corps of Engineers to maintain the 100-year floodway and flood fringes for Cave Creek from downstream of Cave Buttes Dam to the Arizona Canal Diversion Channel (ACDC).

The District owns or leases lands downstream of Cave Buttes Dam and the CAP canal. The District owns these lands by fee title or leases land from the State of Arizona (State Lands Department). Land ownership between Cave Buttes Dam and the CAP canal was reviewed in comparison to the full PMF inundation limits.

Exhibits A and B illustrates the FCD land ownership/leased land between Cave Buttes Dam and the CAP canal and the limits of the full PMF inundation. Exhibit B indicates that the District owns or leases some of the lands that are inundated by the full PMF spillway discharge. As a result, it appears that the District does not need to purchase flowage/ponding easements for lands which are owned or leased by the District. However, there are lands inundated by the full PMF that are outside FCD ownership. These lands are primarily along the north bank of the CAP canal from 6,000 feet west of the 7th Street crossing of the canal to approximately 4,000 feet east of the Cave Creek Wash overchute structure.

In order to limit the type of structures within the ponding limits of the full PMF that are outside FCD lands, the District may consider purchasing ponding easements along the north bank of the CAP canal. The easement would be written to stipulate that habitable structures would not be allowed within the limits of the easement. The approximate number of PMF ponding acres outside FCD lands along the north bank of the CAP canal is approximately 200 acres. The purchase of ponding easements would be a one-time expenditure and was assumed to cost $45,000 per acre. The cost of the ponding easement would be approximately $9 million.

3.3.3 Below Dam – Update Emergency Action Plan

This nonstructural alternative examines the feasibility at a concept level of updating the existing flood response action plan that was developed by the District. The District does not have an individual emergency action plan for Cave Buttes Dam and reservoir area that meets FEMA dam safety guidelines and requirements. The District utilizes their "Flood Emergency Response Manual" developed for all District structures, floodways, and levees to provide standard operating procedures for flood response.

The District has prepared a "Delineation of Spillway Flows for Cave Buttes Dam" (October 1996) that delineates the downstream flooding inundation limits for the 1/3, 2/3, and full PMF discharge from the spillway. The limits of this study, however, stopped at the north embankment of the CAP. Several conclusions regarding the results of the spillway inundation study indicate that the CAP would be overtopped for the various flows investigated. Notes prepared on the exhibits that accompany the spillway delineation report state that "flows crossing the CAP will cause flooding downstream of the CAP". The notes further state that "no attempt was made under this (October 1996) study to determine the flood limits downstream of the CAP".

The District also conducted a dambreak analysis and delineated the limits of the dambreak inundation area downstream of the dam. The study is documented in the report titled "Dambreak Analysis of Cave Buttes Dam" (April 1990). The results of the study have delineated the dambreak floodwave inundation limits from Cave Buttes Dam to the Arizona Canal Diversion Channel (ACDC) approximately 11.6 miles.

An individual emergency action plan needs to be prepared for the Cave Buttes Dam, reservoir area, and appurtenant structures (Dikes No. 1, 2, and 3; emergency spillway, and principal spillway). The plan would be inclusive: all elements of the dam and reservoir area would be incorporated as part of the plan. An estimate of the approximate costs to prepare an EAP for Cave Buttes Dam ranges from $20,000 to $35,000.

The report titled "Policy and Program Report" (April 2000) prepared by Kimley-Horn and Associates, Inc. recommended that the District prepare individual emergency action plans for each of the District dams in their inventory. The "Policy and Program Report" (Section 5.5) provided guidelines of what to include as part of the emergency action plans in order to meet minimum standard of care and FEMA dam safety guidelines. ADWR requires that all jurisdictional dams have an emergency action plan on file with the Department. The plan may be prepared in-house by District staff or the District may use outside engineering consultant services for this task.

The "Policy and Program Report" not only provided recommended elements to include in the emergency action plan but also provided a schedule for updating the plan and the conditions triggering an update. The Report also discussed different levels of exercising the emergency action plan such as orientation seminars, drills, tabletop exercises, functional exercises, and full-scale exercises.

The plan needs to identify the potential of overtopping of the CAP canal by discharges from the emergency spillway. An inundation map downstream of the CAP from overtopping may be similar to the dambreak inundation limits for Cave Buttes Dam. However, this condition needs to be examined and is beyond the scope of this report.

3.3.4 Pool Area – Mitigate Through Flood Insurance

This nonstructural alternative examines the feasibility at a concept level of obtaining flood insurance for the area between the PMF ponding limits and the SPF ponding limits. The District currently owns the property upstream of the dam to the elevation of the SPF. The concept is the District would obtain on a property by property basis flood insurance policy for the area inundated by the full PMF above the elevation of the SPF.

The FEMA flood insurance rate map (FIRM Map No. 04013C1210 F September 30, 1995) indicates the limits of inundation upstream of the Cave Buttes Dam and Dike No. 2. These limits are for the SPF event and are set to the elevation of the emergency spillway crest (1657.1-ft). The full PMF ponding limits are based on a maximum water surface elevation of 1671.1-ft.

The SPF inundation pool (Cave Buttes Dam reservoir pool) depicted on the FIRM panel is designated as Zone A. The zones outside the SPF inundation pool are designated Zones X and A. The PMF limits are incorporated within these map zones.

Section 3.3.1 provided a brief review of the premium rates for flood insurance for a typical coverage amount of $100,000 for a single-family residence. The area between the SPF ponding limits and the full PMF ponding limits is approximately 1,000 acres. This area is delineated on the FIRM panel as consisting of Zone A and Zone X flood zones. The approximate split between the Zone A and Zone X is 25% and 75%, respectively. Given the previous rates and dwelling (potential) units per acre provided in Section 3.3.1, the amount of flood insurance premiums required to insure the lands outside the SPF and within the PMF limits is provided in Table 3-4.

Table 3-4. Approximate Flood Insurance Costs for Pool Area of Cave Buttes Dam.

Zone	No. Acres	Dwelling Units	Unit Premium	Annual Premium	30-Year Premium
A	250	500	$301	$150,500	$4,515,000
X	750	1,500	$221	$331,500	$9,945,500

3.3.5 Pool Area – Acquire Properties/Flowage Easements

This nonstructural alternative examines the feasibility at a concept level of acquiring lands or properties that the District does not currently own or lease that is between the elevation of the SPF and the PMF. The District owns the lands upstream of the Cave Buttes Dam to the limits of the elevation of the standard project flood (SPF). The Corps of Engineers required the District to purchase necessary lands for the construction of Cave Buttes Dam and the inundation limits for the SPF. The ponding limits of the SPF is the area upstream of the dam covered by the area delineated by the SPF full pool elevation. This elevation is also the elevation of the crest of the emergency spillway (1657.1-ft). The SPF inundation limits are depicted on FEMA flood insurance rate map FIRM Map No. 04013C1210 F (September 30, 1995). The District, however, does not own all lands up to the inundation limits of the PMF (or elevation 1671.1-ft). The PMF ponding limits are depicted on Exhibit A and B (including the SPF ponding limits).

The other option under this nonstructural alternative is for the District to obtain ponding easements for the area between the full PMF limits and the full SPF limits. Costs are prepared that indicated the costs of land purchase versus costs for ponding easements.

The PMF ponding limits extends beyond the current FCD land boundaries as depicted in Exhibit A. New FCD land limits were conceptually delineated to incorporate the full PMF ponding limits. The amount of new lands that the District would need to acquire beyond the lands the District already owns is approximately 720 acres. At a unit cost of $50,000 per acre, the total land cost to purchase lands to completely include the full PMF pool is $36,000,000. The cost of a ponding easement is approximately ninety percent of the full purchase cost or approximately $32.4 million (or $45,000 per acre).

3.3.5 Pool Area – Develop Emergency Action Plan

This nonstructural alternative examines the feasibility at a concept level of preparing an emergency action plan for the pool area. The pool area for the purposes of this alternative includes the full PMF ponding limits.

As stated above, the District does not have an individual emergency action plan for Cave Buttes Dam and reservoir meeting FEMA dam safety guidelines. This includes the impoundment area for the SPF and the PMF (IDF). Presently, the District is considering providing recreational and landscaping elements within the limits of the impoundment area. There already exists a model airplane facility just north of Dike No. 2 as well as casual hiking and biking trails. The City of Phoenix is master planning recreational elements in the area of the Cave Buttes Recreational Area.

As stated above, an individual emergency action plan needs to be prepared for the Cave Buttes Dam, reservoir area, and appurtenant structures (Dikes No. 1, 2, and 3; emergency spillway, and principal spillway). The plan would be inclusive: all elements of the dam and reservoir area would be incorporated as part of the plan.

The report titled "Policy and Program Report" (April 2000) prepared by Kimley-Horn and Associates, Inc. recommended that the District prepare individual emergency action plans for each of the District dams in their inventory. The Report (Section 5.5) provided guidelines of what to include as part of the emergency action plans in order to meet minimum standard of care and FEMA dam safety guidelines. ADWR requires that all jurisdictional dams have an emergency action plan on file with the Department. The plan may be prepared in-house by District staff or the District may use outside engineering consultant services for this task.

The "Policy and Program Report" not only provided recommended elements to include in the emergency action plan but also provided a schedule for updating the plan and the conditions triggering an update. The Report also discussed different levels of exercising the emergency action plan such as orientation seminars, drills, tabletop exercises, functional exercises, and full-scale exercises.

Specific elements to include as part of the section of the emergency action plan covering the pool area are warning signs within and around the pool area explaining to the public of what to do in the event of inflows into the reservoir area. Evacuation routes should be displayed on the warning signs. This is a particularly important element because of the anticipated level of recreational facilities being planned for the Cave Buttes Recreational Area and the potential of Old Cave Creek Dam becoming a national historical landmark.

3.3.6 Evaluation and Ranking

The evaluation and ranking of the structural and nonstructural flood control alternatives for Cave Buttes Dam is presented in Section 5.0 of this report.

Section 4.0 Alternatives Analysis for Powerline, Vineyard Road, and Rittenhouse Flood Retarding Structures

This section of the Report documents the concept structural and nonstructural measures/alternatives evaluated as part of the Powerline, Vineyard Road, and Rittenhouse Flood Retarding Structures alternatives analysis. The purpose of the alternatives analysis for Powerline, Vineyard Road, and Rittenhouse Flood Retarding Structures is to examine measures, alternatives, or actions that may be taken by the District to potentially reduce the risk and liability associated with ownership of the dams. The concept alternatives for Powerline, Vineyard Road, and Rittenhouse Flood Retarding Structures were formulated in a meeting between the District and KHA on February 16, 2000, documented in meeting minutes, and a KHA memorandum dated March 16, 2000.

The structural concept alternatives for Powerline, Vineyard Road, and Rittenhouse Flood Retarding Structures were formulated to reduce the risk and liability associated with ownership of the dams and to upgrade, modify or enhance performance or operations, or replace the dam with some other structural flood control measure. The structural alternatives include the following concept measures:

Structural Alternatives:
- Segmentation: Examine segmenting each dam into two dams each. Segmentation of the dam will follow roadway alignments for east/west crossings.
- Upgrade to high hazard dam: Examine upgrading dams to high hazard dams.
- Modifications to improve performance
- Basins: Replace the dam with a basin
- Levee/floodway system: Link dams to function as a levee/floodway system
- Discharge into Central Arizona Project
- Upsize Powerline Floodway/East Maricopa Floodway
- Alternatives to include multi-use opportunities

The nonstructural concept alternatives for Powerline, Vineyard Road, and Rittenhouse FRS were primarily formulated to reduce the risk and liability associated with ownership of the dam. The nonstructural alternatives include the following concept measures:

Nonstructural Measures for Pool Area:
- Mitigate through Flood Insurance
- Acquire Properties/Flowage Easements
- Emergency Action Plan: Develop EAP to include pool inundation areas.

The formulation, discussion, evaluation, and presentation for Powerline, Vineyard Road, and Rittenhouse FRS alternatives are presented later in this section. The following discussions provides a brief description, purpose, and physical characteristics of the flood retarding structures and associated features.

4.1 Description of Powerline FRS

The Powerline FRS is a structural plan element of the Watershed Work Plan for the Apache Junction – Gilbert Watershed, Maricopa and Pinal Counties, Arizona. The Watershed Work Plan was prepared by the Natural Resources Conservation Service (NRCS; formerly the Soil Conservation Service, SCS) in January 1963. The watershed heads in the southwest-facing slopes of the Superstition Mountains and drains onto a wide alluvial fan on which valuable agricultural, urban and commercial developments have been constructed. The total Apache Junction – Gilbert Watershed watershed is approximately 140 square miles in area. The watershed is one of three for which concurrent planning efforts were conducted by the NRCS at the request of the District. The northernmost watershed is the "Buckhorn-Mesa", the central watershed is the "Apache Junction – Gilbert", and the southern watershed is the "Williams-Chandler".

4.1.1 Purpose of Dam

The Powerline FRS is one of two structural measures designed and constructed under the Apache Junction-Gilbert Watershed Work Plan. The other structural measure is the Powerline Floodway. The purpose of the Powerline FRS is to provide flood and erosion control benefits for downstream developments (agriculture, commercial and urban areas). The design function of the Powerline FRS will control runoff from floods up to and including the 100-year event.

4.1.2 Dam Location

Powerline FRS is located off Ironwood Road, south of Baseline Road, about 35 miles east of downtown Phoenix and approximately five miles south of the town of Apache Junction. Figure 4-1 provides a location map of Powerline FRS. The project consists of the FRS structure and an emergency spillway. The project is part of the Apache Junction-gilbert Watershed Protection and Flood Prevention Project, which includes the Rittenhouse and Vineyard flood retarding structures. The Flood Prevention Project was prepared, designed, and constructed by the U.S. Department of Agriculture, Natural Resources Conservation Service.

The reservoir behind the FRS is 456 acres with a capacity of 4,194 acre-feet (according to the as-built construction plans). A permanent pool will not be retained in the reservoir, instead, the FRS and reservoir are designed to trap floodwater and store it only for as long as it takes to release it slowly and safely downstream. Reservoir capacity is then restored to handle a future flood.

The emergency spillway is located adjacent to the south abutment of the FRS. Construction of the FRS and appurtenant structures was completed in March 1967.

4.1.3 Physical Features

Powerline FRS is a rolled earthfill structure. The length of the FRS is 13,398 feet with a maximum height of 21 feet and a crest width of 14 feet. The reservoir capacity is 4,194 acre-feet with a maximum water surface elevation of 1583.3 feet. The FRS was designed with 4.8 feet of freeboard and 175 acre-feet of sediment storage (50-year). Powerline FRS is accessible off Ironwood Road with access controlled by a padlocked gate. The maximum recorded impoundment for Powerline reservoir is 952 acre-feet with a stage of 11.0 feet at the FRS (January 11, 1993).

The principal spillway is an ungated 36-inch diameter concrete pipe approximately 156 feet long. The design outflow is 203 cfs from the principal spillway. The trash rack is located on the upstream inlet. The outlet of the principal spillway discharges into a constructed channel through a outlet structure. An energy dissipator is located on the downstream end of the concrete outlet structure.

The emergency spillway was excavated into earth and is located adjacent to the south abutment of the FRS. The spillway is approximately 600 feet wide with a capacity of 16,600 cfs. The spillway crest elevation is 1583.8 feet.

The inflow design flood under ADWR rules and regulations is the ½ PMF.

Station markers are located along the downstream crest of the FRS. A series of staff gages is located on the upstream slope adjacent to the principal spillway. Settlement monuments are located along the crest and downstream toe of the FRS.

A central filter drain was constructed in the Powerline FRS embankment in June 1991.

Table 4-1 provides a summary of the physical structure data for Powerline FRS. The following are definitions of the terms emergency spillway hydrograph (ESH) and freeboard hydrograph (FBH). These terms are identified in Table 4-1. The terms are derived from the NRCS document "TR-60: Earth Dams and Reservoirs" (October 1985).

- Emergency Spillway Hydrograph - is the hydrograph used to establish the dimensions of the emergency spillway
- Freeboard Hydrograph - is the hydrograph used to establish the minimum settled elevation of the top of the dam. It is also used to evaluate the structural integrity of the spillway system.

Table 4-1. Powerline Flood Retarding Structure Physical Data.

ITEM NATDAM ID AZ00082 STATE ID 11.02	PHYSICAL DATA
Drainage Area	47.1 sq mi
Storage Capacity	
Sediment	175 af
Floodwater	4019 af
Total	4194 af
Surface Area	
Sediment Pool	88 ac
Floodwater Pool	456 ac
Volume of Fill	936,000 cy
Elevation Top of Dam	1589.1 ft
Maximum Height of Dam	21.0 ft
Length of Dam	2.54 mi
Freeboard	4.8 ft
Emergency Spillway	
Inflow Design Flood (ADWR)	½ PMF
Crest Elevation	1583.3 ft
Bottom Width	600 ft
Type	Earth-lined
Percent Chance of Use	1
Av. Curve No. Condition II	81.9
Emergency Spillway Hydrograph	
Storm Rainfall (6 hr)	3.5 in
Storm Runoff	0.68 in
Spillway Capacity	16,600 cfs
Freeboard Hydrograph	
Storm Rainfall (6 hr)	7.0 in
Storm Runoff	2.29 in
Principal Spillway	
Diameter of Conduit	36-in rcp
Length of Conduit	156 ft
Capacity at Elev Emergency	203 cfs
Time to release	10 days
Capacity Equivalents	
Sediment Volume	0.07 in
Detention Volume	1.49 in
Spillway Storage	1.77 in
Class of Structure	**B (NRCS)**
Hazard Classification (ADWR)	**Significant**
Size of Dam (ADWR)	**Medium**

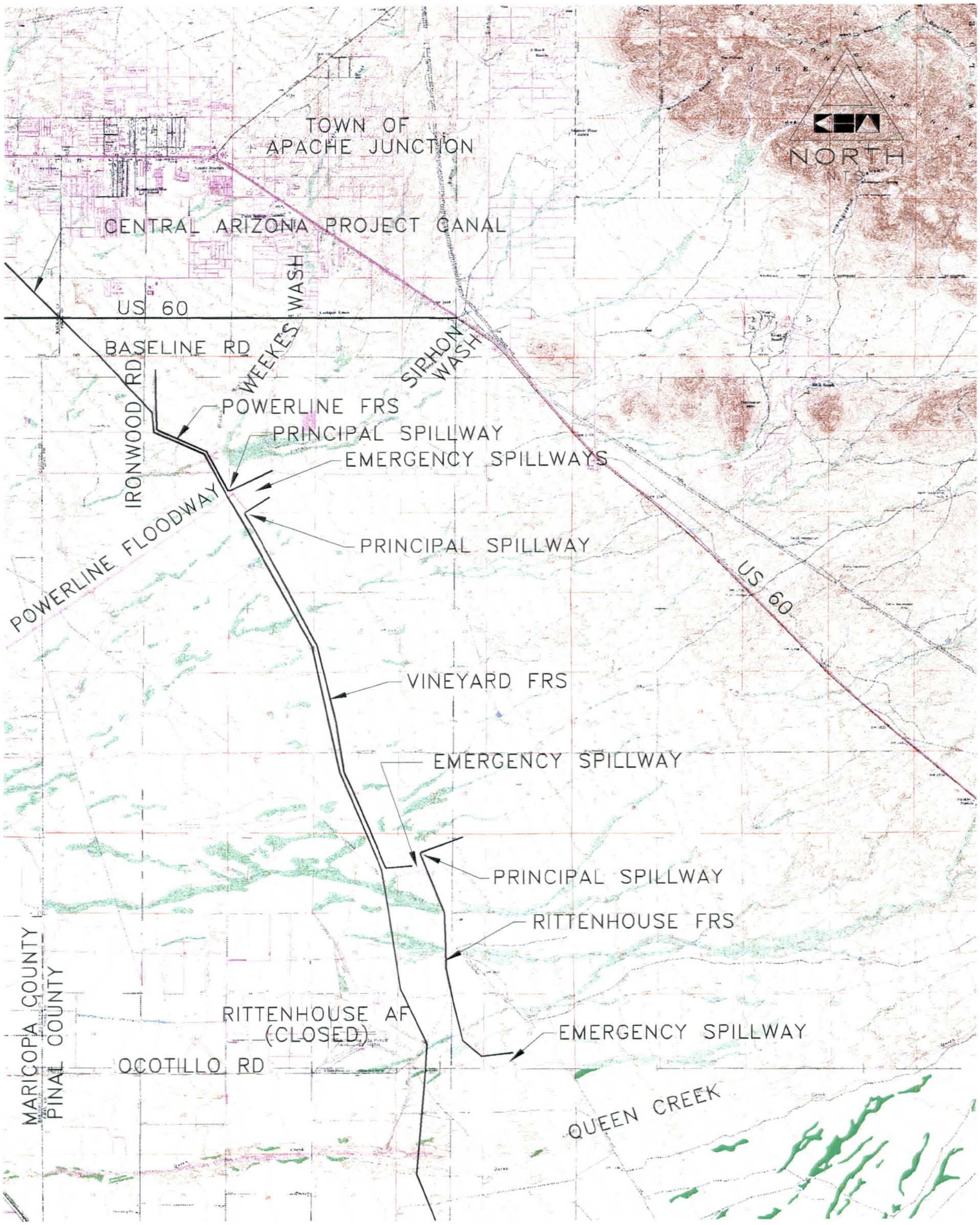


Figure 4-1. Powerline, Vineyard Road, and Rittenhouse FRS Location Map.

4.1.4 Powerline, Vineyard Road, and Rittenhouse FRS Operations

The operation of Powerline, Vineyard Road, and Rittenhouse flood retarding structures are similar and are therefore presented and discussed in this subsection for Powerline FRS. All three dams have a principal and emergency spillway except that Vineyard Road FRS has two emergency spillways. The principal spillways for both Powerline and Vineyard Road FRS discharge into a common channel called the Powerline Floodway. The floodway is located near the outlet of the principal spillway for Powerline FRS (see Figure 4-1). The floodway crosses over the Central Arizona Project (CAP) through a structure called an overchute. The principal spillway for Rittenhouse FRS discharges into the impoundment reservoir area behind Vineyard Road FRS. Rittenhouse FRS functions as a cascading reservoir (see Figure 4-1). Therefore, discharges from all three principal spillways will eventually flow into the Powerline Floodway. The Powerline FRS emergency spillway and Vineyard Road FRS right emergency spillway are located adjacent to each other as depicted on Figure 4-1. There are no defined downstream watercourses for which to discharge emergency spillway flows from each structure. Should a discharge occur of any significant flow in any of the four emergency spillways, floodwaters would flow overland toward the CAP. The potential exists for ponding to occur on the upstream embankment of the CAP and if discharge flows are great enough, flows could enter the CAP, and/or flood over the CAP potentially breaching the CAP canal.

In the event of a forecast for measurable rainfall that may produce significant runoff on the watersheds and hence potentially an impoundment at the dam, the Flood Control District initiates the flood response actions outlined in their "Flood Emergency Response Manual". The manual is the District's standard operating procedure the purpose of which is to provide an outline of the duties and responsibilities for District personnel to act upon during significant rainfall events and/or flood emergencies. The procedure provides an overview of how the District as a whole will respond to significant rainfall events and floods emergencies, and therefore does not outline procedures for specific personal tasks and responsibilities. According to the manual, the District division managers will assign personal tasks and responsibilities are necessary.

The manual outlines the tasks and required response actions, both in summary and in detailed format, for the Chief Engineer, Operations and Maintenance Division, Engineering Division, Planning and Project Management Division, Land Management Division, and Information Technology Division in response to a flood emergency. It should be noted that the flood emergency response manual is geared for all of the District structures including drains, floodways, major river corridors (Salt, Verde, Gila, and the Agua Fria Rivers), District dams, and for non-District dams (Town of Fountain Hills structures).

The manual contains a list of District emergency telephone and cellular numbers of District personnel. This list was last updated on July 1, 1999. Also included in the manual are selected bridge capacities and closure data on major rivers, Salt River and

Gila River flow travel time tables, District dam and detention basin rating curves, and a listing of USGS stream gages with names and locations at pertinent sites.

Maricopa County has been divided into twelve observation areas for purposes of flood monitoring. Each of the twelve areas has specific instructions for observations so that District staff members who may not have been trained on a specific structure will know what to look for at each site in their area. Primary and secondary observation points have been identified, and team members are to proceed to these sites in the order noted in the manual or as dictated by local flooding. Area staff assignments require teams to observe these areas on two twelve-hour shifts during an emergency. Table 1 of the Flood Emergency Response Manual contains a listing of the flood observation points (primary points) for each of the twelve areas. The manual provides detailed observation instructions for each team assigned to each of the twelve areas of the County.

Initial emergency flood response is based on radar and rainfall information as it provides the most lead-time and is the best available data. The District's real-time telemetry system consists of a network of rain and stream gages. The system has the capability of sounding alarms when preset rainfall intensity levels are exceeded.

Once rainfall alarms are activated and a thunderstorm or flash flood warning/watch has been issued the rainfall and runoff from the event is monitored and evaluated. The sources of information used for this purpose come from telemetry stage gages, field observations of staff gages, and stage/discharge rating curves.

The District maintains and operates a number of flood control structures, including dams, throughout the County. The Flood Warning/Data Collection Branch monitors water depths behind the dams and through the principal outlet throughout an event. All of the dams have telemetered stage gages, which measure impoundment depth. ALERT alarm, O&M notification levels, and Emergency Operations Center notification levels are set for each dam. Table 4 of the manual lists the emergency notification elevations at each of the District dams (as of July, 1999).

The District's flood emergency response is linked and coordinated with the Maricopa County Department of Emergency Management (DEM). The Department has prepared an Emergency Operation Plan (EOP) for Maricopa County that not only includes response and actions in the event of flooding and dam failures, but also covers catastrophes due to aircraft crashes, earthquakes, fires and explosions, hazardous materials incidents, and national security emergencies. Annex G of the EOP specifically addresses storms and floods. The annex includes a discussion on the dissemination of weather data and information, water release warning procedures, dam failure scenarios for the major dams in the vicinity of the Phoenix metropolitan area (e.g. Salt and Verde River dams; Aqua Fria River), and for each major District dam. The appendices to Annex G provide individual information and descriptions for each of the District dams. This information includes the location of the dam, a physical description of the structure, and the purpose of the dam (i.e., provide flood protection). Included with each appendix is gross inundation mapping that shows what areas of Maricopa County that could experience flooding due failure of a specific dam. The appendices outline the specific tasks and actions that the District will follow in the event of a dam failure, the actions and responses of the DEM, and the notification procedures. The appendices provide a written description of the area to evacuate in the event of a dam failure.

During a flooding event where an impoundment occurs at Powerline FRS, the dam will operate in a passive manner. In other words the principal spillway discharges impounded floodwater behind the dam at a rate that depends on the stage of impoundment. There is no operating gate on the concrete pipe principal spillway. Very little District operations and maintenance intervention is required for the operation of the dam and spillways during an event. District site operations during a flooding event is usually a monitoring mode activity.

The emergency spillway is also ungated due to the nature of the dam being a dry flood control dam. The discharge rate from the emergency spillway will depend on the depth of flow within the control section of spillway channel. During a 100-year flood, the reservoir would be fully impounded to an elevation of approximately the spillway crest elevation. Previous hydrologic studies for Powerline FRS indicate that there will be a spill from the emergency spillway as a result of the 100-year event. The operations for Vineyard Road FRS and Rittenhouse FRS during a flooding event as similar as just described for Powerline FRS except during a 100-year event no spills occur in their respective emergency spillways.

There are no other passive facilities or methods in-place to withdraw impounded water stored within the reservoir area for Powerline, Vineyard Road, and Rittenhouse FRS. No permanent water storage or pool is allowed within the impoundment areas according to the ADWR operating license issued to the District for each of the dams.

4.2 Description of Vineyard Road FRS

The Vineyard Road FRS is a structural plan element of the Watershed Work Plan for the Williams-Chandler Watershed Protection and Flood Prevention Project, Maricopa and Pinal Counties, Arizona. The Watershed Work Plan was prepared by the Natural Resources Conservation Service (NRCS; formerly the Soil Conservation Service, SCS) in January 1963. The watershed heads in the southwest-facing slopes of the Superstition Mountains and drains onto a wide alluvial fan on which valuable agricultural, urban and commercial developments have been constructed. The total watershed is approximately 52.1 square miles in area. The watershed is one of three for which concurrent planning efforts were conducted by the NRCS at the request of the District. The northernmost watershed is the "Buckhorn-Mesa", the central watershed is the "Apache Junction – Gilbert", and the southern watershed is the "Williams-Chandler".

4.2.1 Purpose of Dam

The purpose of the Vineyard Road FRS is to provide flood and erosion control benefits for downstream developments (agriculture, commercial and urban areas). The Vineyard

Road FRS was designed to control runoff from floods up to and including the 100-year event.

4.2.2 Dam Location

Vineyard Road FRS is located about 35 miles east of downtown Phoenix and seven miles southeast of the town of Apache Junction. Figure 4-2 provides a location map of Vineyard Road FRS. The project consists of the FRS embankment structure, two emergency spillways, and a principal spillway. The project is part of the Williams-Chandler Watershed Protection and Flood Prevention Project, which includes the Rittenhouse and Vineyard Road flood retarding structures. The Flood Prevention Project was prepared, designed, and constructed by the U.S. Department of Agriculture, Natural Resources Conservation Service.

The spillway crest reservoir behind the FRS is 840 acres with a capacity of 4,310 acre-feet (according to the as-built construction plans). A permanent pool will not be retained in the reservoir, instead, the FRS and reservoir are designed to trap floodwater and store it only for as long as it takes to release it slowly and safely downstream. Reservoir capacity is then restored to handle a future flood. The sediment pool capacity is 178 acre-feet at an elevation of 1566.2.

The emergency spillways are located adjacent to the left and right abutments. Construction of the FRS and appurtenant structures was completed in July 1968.

4.2.3 Physical Features

Vineyard Road FRS is a rolled earthfill structure. The length of the FRS is 28,829 feet with a maximum height of 16.5 feet and a crest width of 14 feet. The reservoir capacity is 4,310 acre-feet at a water surface elevation of 1574.8 feet. The FRS was designed with 4.7 feet of freeboard. The FRS is accessible by using Ironwood Road to an access control gate south of Baseline Road. The maximum recent recorded impoundment for Vineyard Road reservoir is 897 acre-feet with a stage of 5.9 feet at the FRS (January 16, 1993).

The principal spillway is an ungated 56-inch diameter concrete pipe approximately 100 feet long. The design outflow is 368 cfs from the principal spillway. The trash rack is located on the upstream inlet. The outlet of the principal spillway discharges into a constructed channel through an outlet structure. An energy dissipator is located on the downstream end of the concrete outlet structure.

The two emergency spillways are excavated into earth and are located adjacent to the left and right abutments. Each spillway is 300 feet wide. The spillway crest elevation is 1574.8 feet (MSL).

Station markers are located along the downstream crest of the FRS. A series of staff gages is located on the upstream slope adjacent to the principal spillway. Settlement monuments are located along the crest and downstream toe of the FRS.

The as-built plans for the Vineyard Road FRS indicate that five irrigation outlets were constructed. These outlets are located at Stations 129+90 (24-inch rcp), 152+40 (18-inch rcp), 251+00 (24-inch rcp), 276+80 (18-inch rcp), and Station 321+40 (24-inch rcp). The outlets included inlet and outlet structures, gates, stem guides, operator wheels on the crest, and trash racks on the inlet. These irrigation outlets were subsequently abandoned. None of the five irrigation outlets were found in the field as depicted in the as-built drawings of Vineyard FRS. The inlet and outlet structures appear to have been removed and the conduit left in-place. The inlets of the conduits were filled with grout.

A central filter drain, without finger drain outlets, was constructed in the Vineyard Road FRS embankment in July 1983.

Table 4-2 provides a summary of the physical structure data for Vineyard Road FRS. The following are definitions of the terms emergency spillway hydrograph (ESH) and freeboard hydrograph (FBH). These terms are identified in Table 4-2. The terms are derived from the NRCS document "TR-60: Earth Dams and Reservoirs" (October 1985).

- Emergency Spillway Hydrograph - is the hydrograph used to establish the dimensions of the emergency spillway
- Freeboard Hydrograph - is the hydrograph used to establish the minimum settled elevation of the top of the dam. It is also used to evaluate the structural integrity of the spillway system.

Table 4-2. Vineyard Road Flood Retarding Structure Physical Data.

ITEM NATDAM ID AZ 00084 STATE ID 11.11	PHYSICAL DATA
Drainage Area	52.1 sq mi
Storage Capacity	
Sediment	178 af
Floodwater	4132 af
Total	4310 af
Surface Area	
Sediment Pool	150 ac
Floodwater Pool	840 ac
Volume of Fill	1,154,400 cy
Elevation Top of Dam	1579.5 ft
Maximum Height of Dam	16.50 ft
Length of Dam	5.46 mi
Freeboard	4.7 ft
Emergency Spillway	
Inflow Design Flood (ADWR)	½ PMF
Crest Elevation	1574.8 ft
Bottom Width	600 ft*
Type	Earth-lined
Percent Chance of Use	1
Av. Curve No. Condition II	82
Emergency Spillway Hydrograph	
Storm Rainfall (6 hr)	3.5 in
Storm Runoff	0.67 in
Spillway Capacity	12,800 cfs
Freeboard Hydrograph	
Storm Rainfall (6 hr)	7.5 in
Storm Runoff	2.51 in
Principal Spillway	
Diameter of conduit	56-in rcp
Length of conduit	100 ft
Capacity at Elev Emergency	368 cfs
Time to release	10 days
Capacity Equivalents	
Sediment Volume	0.07 in
Detention Volume	1.45 in
Spillway Storage	1.30 in
Class of Structure	**B (NRCS)**
Hazard Classification (ADWR)	**Significant**
Size of Dam (ADWR)	**Medium**

*Two 300-ft spillways

4.3 Description of Rittenhouse FRS

The Rittenhouse FRS is a structural plan element of the Watershed Work Plan for the Williams-Chandler Watershed Protection and Flood Prevention, Maricopa and Pinal Counties, Arizona. The Watershed Work Plan was prepared by the Natural Resources Conservation Service (NRCS; formerly the Soil Conservation Service, SCS) in January 1963. The watershed heads in the southwest-facing slopes of the Superstition Mountains and drains onto a wide alluvial fan on which valuable agricultural, urban and commercial developments have been constructed. The total watershed is approximately 47.7 square miles in area. The watershed is one of three for which concurrent planning efforts were conducted by the NRCS at the request of the District. The northernmost watershed is the "Buckhorn-Mesa", the central watershed is the "Apache Junction – Gilbert", and the southern watershed is the "Williams-Chandler".

4.3.1 Purpose of Dam

The purpose of the Rittenhouse FRS is to provide flood and erosion control benefits for downstream developments (agriculture, commercial and urban areas). The Rittenhouse FRS was designed to control runoff from floods up to and including the 100-year event.

4.3.2 Dam Location

Rittenhouse FRS is located about 35 miles east of downtown Phoenix and five miles south of the town of Apache Junction. Figure 4-2 provides a location map of Rittenhouse FRS. The project consists of the FRS structure and an emergency spillway. The project is part of the Williams-Chandler Watershed Protection and Flood Prevention Project, which includes the Powerline and Vineyard flood retarding structures. The Flood Prevention Project was prepared, designed, and constructed by the U.S. Department of Agriculture, Natural Resources Conservation Service.

The reservoir behind the FRS is 660 acres with a capacity of 4,060 acre-feet. A permanent pool will not be retained in the reservoir, instead, the FRS and reservoir are designed to trap floodwater and store it only for as long as it takes to release it slowly and safely downstream. Reservoir capacity is then restored to handle a future flood. The sediment pool capacity is 175 acre-feet at an elevation of 1587.5.

The emergency spillway is located adjacent to the south abutment of the main FRS. Construction of the FRS and appurtenant structures was completed in 1969.

4.3.3 Physical Features

Rittenhouse FRS is a rolled earth-filled structure. The length of the FRS is 19,008 feet with a maximum height of 24.3 feet and a crest width of 14 feet. The reservoir capacity is 4,060 acre-feet with a maximum water surface elevation of 1597.6 feet. The FRS was designed with 4.7 feet of freeboard. The FRS is accessible by using Ocotillo Road east of Vineyard Road. A padlocked gate controls access. The maximum recorded impoundment for Rittenhouse reservoir is 359 acre-feet with a stage of 11.0 feet at the FRS (January 11, 1993).

The principal spillway is an ungated 33-inch concrete pipe approximately 145 feet long. The design outflow is 143 cfs from the principal spillway. The trash rack is located on the upstream inlet structure. The outlet of the principal spillway discharges into a constructed channel through a outlet structure. The outlet structure includes an energy dissipator.

The emergency spillway was excavated into earth and is located adjacent to the left abutment. The spillway is approximately 600 feet wide and has a capacity of 12,800 cfs. The spillway crest elevation is 1597.6 feet (MSL).

Station markers are located along the downstream crest of the FRS. A series of staff gages is located on the upstream slope adjacent to the principal spillway. Settlement monuments are located along the crest and downstream toe of the FRS.

Two irrigation outlets were constructed as part the Rittenhouse FRS. The outlets are located at Stations 69+50 and 156+00 and include inlet and outlet structures. The inlet structure includes a gate and trash rack. The operator wheel is located at the crest with the stem of the gate cradled on the upstream slope. The conduits are both 24-inch reinforced concrete pipes. The irrigation outlets discharge into downstream washes.

A central filter drain with rock/gravel finger drains was constructed in Rittenhouse FRS in May 1979.

Table 4-3 provides a summary of the physical structure data for Rittenhouse FRS. The following are definitions of the terms emergency spillway hydrograph (ESH) and freeboard hydrograph (FBH). These terms are identified in Table 4-3. The terms are derived from the NRCS document "TR-60: Earth Dams and Reservoirs" (October 1985).

- Emergency Spillway Hydrograph - is the hydrograph used to establish the dimensions of the emergency spillway
- Freeboard Hydrograph - is the hydrograph used to establish the minimum settled elevation of the top of the dam. It is also used to evaluate the structural integrity of the spillway system.

Table 4-3. Rittenhouse Flood Retarding Structure Physical Data.

ITEM NATDAM ID AZ0085 STATE ID 11.12	PHYSICAL DATA
Drainage Area	47.7 sq mi
Storage Capacity	
Sediment	175 af
Floodwater	3875 af
Total	4060 af
Surface Area	
Sediment Pool	118 ac
Floodwater Pool	660 ac
Volume of Fill	798,800 cy
Elevation Top of Dam	1602.3 ft
Maximum Height of Dam	24.3 ft
Length of Dam	3.6 mi
Freeboard	4.7 ft
Emergency Spillway	
Inflow Design Flood (ADWR)	½ PMF
Crest Elevation	1597.6 ft
Bottom Width	600 ft
Type	Earth-lined
Percent Chance of Use	1
Av. Curve No. Condition II	80
Emergency Spillway Hydrograph	
Storm Rainfall (6 hr)	3.5 in
Storm Runoff	0.66 in
Spillway Capacity	12,800 cfs
Freeboard Hydrograph	
Storm Rainfall (6 hr)	7.5 in
Storm Runoff	2.51 in
Principal Spillway	
Diameter of Conduit	33-in rcp
Length of Conduit	145 ft
Capacity at Elev Emergency	143 cfs
Time to release	30 days
Capacity Equivalents	
Sediment Volume	0.07 in
Detention Volume	1.47 in
Spillway Storage	1.04 in
Class of Structure	**B (NRCS)**
Hazard Classification	**Significant**
Size of Dam	**Medium**

4.4 Structural Alternatives

The flood control structural alternatives for the Powerline, Vineyard Road, and Rittenhouse FRSs were developed in a planning meeting held between the District and KHA. The structural alternatives were formulated to upgrade, modify or enhance the performance and/or operations, or replace the dam with some other structural flood control measure. The structural flood control alternatives that were formulated for concept development and analysis for the Powerline, Vineyard Road, and Rittenhouse FRSs are listed below and are described in detail in the following subsections.

- Segmentation
- Upgrade To High Hazard Dam
- Modifications to Improve Performance
- Replace Dams With Detention Basins
- Levee/Floodway System
- Utilization of Central Arizona Project Canal
- Increase the Capacity of Powerline Floodway

A. Previous and On-going District Studies

Several District studies and plans have been completed and other District investigations and studies are on-going which may have a direct impact and impose planning constraints on the structural alternatives developed as part of this alternatives analysis for Powerline, Vineyard Road, and Rittenhouse FRS. The study area for the previous and on-going studies includes the region of east Mesa from the CAP canal to the EMF. This area includes major drainageways such as the EMF, Queen Creek, Sanoki Wash, Rittenhouse Channel, Powerline Floodway, and others. A brief outline of these studies is provided as follows:

East Maricopa Floodway Capacity Assessment Study (HNTB 1999) - This study evaluated the conveyance capacity of the entire EMF for the existing conditions 100-year 24-hour SCS design discharge and the 100-year 24-hour future conditions. The study also determined the conveyance capacity of the EMF under bank-full conditions. The study indicates that the existing capacity of the EMF does not meet the design capacity. As a matter of fact, under existing conditions, the EMF is substantially overtaxed. The design capacity of the EMF downstream of the Powerline Floodway confluence is 6,500-cfs. Under existing conditions, the total flow to the EMF downstream of the Powerline Floodway confluence is 11,456-cfs.

East Mesa Area Drainage Master Plan (Dibble and Assoc. 1998) - This study was initiated in order to provide flood protection to the east Mesa area. The study determined the existing and future conditions hydrology for the east Mesa area for planning purposed, identified drainage problems, and proposed drainage factilites to address current and future flooding problems. The study provided structural flood control recommendations for drainage improvements. One of the recommendations was to re-align the Powerline Floodway along the north perimeter of Williams Gateway Airport. The relocated floodway would extend to the proposed Elliot Channel adjacent to the proposed SanTan Freeway. The capacity of the relocated Powerline Floodway is 3,731-cfs at the freeway and 2,932-cfs along the north perimeter of the airport.

Queen Creek Area Drainage Master Study (Wood and Assoc. 1991) - This study was implemented to identify stormwater problems in the Queen Creek area and provide a master drainage plan to alleviate these problems. The study's limits are bound by Goldmine and San Tan Mountains to the south, the EMF to the west, the CAP canal to the east, and the Powerline Freeway to the north.

Queen Creek/Sanoki Wash Hydraulic Master Plan (Huitt-Zollars March 2000) - This study is formulating drainage improvements for use by local municipalities as a guide for future development in the area. The study is evaluating alternative drainage improvements to Queen Creek wash from the county border with Pinal County to the EMF. Under existing conditions, the study indicates that the 100-year flood in Queen Creek wash at the county line is 3,150-cfs. Under the "no-detention" alternative, the 100-year flow in the wash is 3,240-cfs at Power Road. Several "detention" alternatives were investigated as part of the study. These alternatives examined several off-line and on-line detention basins. The 100-year flow is reduced under the detention alternatives to 2,750-cfs at Power Road.

East Maricopa Floodway Capacity Mitigation Study Report (Huitt-Zollars February 2000) - This study integrates the above studies into both a hydrologic model and a hydraulic model for the EMF. The study proposes several alternatives for structural improvements to the EMF and for watershed and EMF detention basins. The preferred structural flood control alternative for EMF improvements included an off-line detention basin along the Powerline Floodway west of Ellsworth Road. The existing conditions 100-year 24-hour flows getting to the Powerline Floodway at the EMF confluence is, according to this study, 7,340-cfs. Under the preferred alternative (with detention basin of 892 acre-feet along Powerline Floodway), the 100-year flow getting to the Powerline Floodway is reduced to 4,710-cfs.

Preliminary East Maricopa Capacity Mitigation and Multi-Use Corridor Study (Alternative 1) (Collins/Pina May 2000) - The purpose of this study was to provide three alternatives on a design/concept level for infrastructure improvements to the EMF. These alternatives include channel improvements and potential areas for inline and offline detention basins. Each alternative has been formulated so that the EMF can convey the 100-year peak design discharge with the required freeboard. According to this study, as a matter of note, the Powerline Floodway has a 100-year design discharge of approximately 3,000-cfs.

4.4.1 Segmentation

This structural alternative examines the feasibility at a concept level for segmenting the Powerline, Vineyard Road, and Rittenhouse dams. Segmentation basically divides the dam impoundment area of each dam into two semi-separate pools or cells. Segmentation of the structures is an alternative that will allow for a reduction in flood damages in the case of a potential dam breach. Under this alternative, only a portion of the impoundment is released as compared to an unsegmented impoundment.

For all three structures, segmentation will be along section lines or where potential future roadway crossings may be constructed. The concept for the dividing "segment" (in the case of a roadway crossing) would be an earthfill embankment that would begin at grade on the downstream side, rise up and over to match the top of the dam, and then drop on the upstream side such that the end of the segment is located beyond the delineated pool area. However, for this alternative only the upstream portion of the segment will considered.

A major constraint associated with this alternative includes assuring that the original design detention volume for the dam is maintained among the two pool/cell segments. An equalization culvert would need to be constructed in the earth embankment of the segment. The equalization culvert would require a gate structure to isolate one pool/cell segment of a dam from the other pool segment under emergency situations. A second constraint is the proposed method of draining the "isolated" pool. If it becomes necessary to close the gate on the equalization culvert, and if the emergency or breach occurs in the portion of the dam with the principal spillway and that pool is rapidly emptied, then question remains of how to discharge water from the isolated pool segment. The answer to this question depends on the nature, location, and extent of the breach in the other pool segment. ADWR may require that each pool segment be provided with an individual principal spillway in order to meet licensing requirements. However, for the purposes of this alternative it is assumed that the gated equalization culvert would regulate flows discharged from the isolated pool by incrementation of the gate opening. In this fashion, no new principal spillways would be required.

The inflow design flood (IDF) for Powerline, Vineyard Road, and Rittenhouse FRS currently is the 1/2 PMF. Hydrologic analysis and design of the segmentation for each of the dams would require that the equalization culverts be sized correctly to handle the IDF. This analysis would be based on a time-variant tailwater condition for each dam (i.e., the equalization culverts would be sized based on the differential head between the two pool segments under two conditions - filling and draining). The approach to the hydrologic analysis would be to delineate the contributing watersheds to each pool segment. The analysis would be required to investigate the operation of the "split" pool system in safely passing the IDF through the principal and emergency spillway.

The equalization culverts would be constructed with a floodgate. The gate would only operate during an urgent or emergency condition where the stability and integrity of the dam embankment is threatened. The gate is only to be used in the event it is necessary to isolate the reservoir into two pools. Otherwise the gate stays open. The urgent or emergency condition would be associated with impoundment of stormwater and (underlining added for emphasis) observation of piping from the main embankment.

Previous investigations downstream of the Powerline FRS area, in the vicinity of Hawk Rock, have discovered the presence of earth fissures. The Bureau of Reclamation has repaired one of these earth fissures. This earth fissure, called the Junkers Fissure, is on alignment, if the fissure were to break through the repair, to intersect the CAP and possibly the west end of the Powerline FRS. While earth fissure prediction (location, extent, alignment, rate of progression) methods are somewhat subjective the potential exists that the fissure in question could someday intersect and breach the Powerline structure. Segmentation for the Powerline FRS would be one proactive method of fissure intervention. The embankment segment would be on an east-west alignment, for the purposes of this alternative, with Guadalupe Road. Guadalupe Road is a major mile arterial in several east Valley cities and someday may extend into Pinal County. The upstream embankment segmentation would extend beyond the ponding limits at an elevation higher than the pool elevation. Modifications under this segmentation alternative include adding an earth segment and a gated culvert. These modifications are graphically illustrated in planform in Exhibit C (located in the map pockets in the back of this report).

Segmentation for the Vineyard Road FRS would occur on alignment with the extension of Ray Road. This location would essentially divide the structure in half. The same constraints exist for Vineyard FRS as discussed above for Powerline FRS. The upstream embankment segmentation would extend beyond the ponding limits at an elevation higher than the pool elevation. Modifications under this segmentation alternative include adding an earth segment and a gated equalization culvert. These modifications are graphically illustrated in planform in Exhibit C.

Segmentation for the Rittenhouse FRS could potentially occur on alignment with either the extension of German Road or Queen Creek Road. There is no mile street that divides the Rittenhouse embankment basically in half. For the purposes of this discussion, the segmentation alignment will follow German Road. The reason for selecting German Road as the alignment for the embankment segment is that this divides the reservoir pool into a 1/3 north pool and a 2/3 south pool. The 2/3 south pool could potentially be drained to discharge to Queen Creek. Allowing this volume to drain to Queen Creek instead of to the Vineyard Road FRS reduces the volume of floodwaters to the Powerline Floodway. The reduction of floodwaters to the Powerline Floodway will assist in alleviating capacity problems in the Powerline Floodway and the East Maricopa Floodway.

The same constraints exist for Rittenhouse FRS as discussed above for both Powerline and Vineyard Road FRS. The upstream embankment segmentation would extend beyond the ponding limits at an elevation higher than the pool elevation. Modifications under this segmentation alternative include adding an earth segment and a gated equalization culvert. These modifications are graphically illustrated in planform in Exhibit C.

The length of earth embankment segment for Powerline, Vineyard Road, and Rittenhouse FRS was determined from examination of USGS quadrangle maps and plotting the impoundment ponding limits to the elevation of the emergency spillway. The segment starts at the elevation of the top of the dam at the dam itself. A minimum of three feet of freeboard was utilized from the ponding elevation (emergency spillway crest) to the top of the segment. Table 4-4 summarizes the elevations of the top of dam and emergency spillway with the approximate length of segment for each structure.

A review of the as-built plans for the three dams indicates the each structure was constructed with an upstream "borrow" channel. The channel was used to obtain borrow material to construct the dam embankment and to act as a low flow channel to direct water to the principal spillways. The minimum channel bottom width is 50-ft with 3:1 side slopes. The equalization culvert in the earth segment was sized based on this low flow channel configuration. After the size of the equalization culvert was determined for each dam, a floodgate was selected that would be constructed on the upstream side of the culvert. When the gate is closed, the gate would block water from flowing from one pool segment into the other pool segment. Table 4-5 provides the volume of earth embankment material estimated to constructed the segment, the size and configuration of the low-flow culvert, and size of floodgates. The table also provides an estimate of the construction costs for each segment alternative. Appendix E provides back-up data for the cost estimate of this structural alternative.

Table 4-4. Top of Dam and Emergency Spillway Elevations.

Structure	Elevation Of Top Of Dam (ft)	Elevation Of Emergency Spillway (ft)	Difference (ft)	Length Of Segment (ft)
Powerline	1589.1	1583.3	5.8	6,600
Vineyard Road	1579.5	1574.8	4.7	2,000
Rittenhouse	1602.3	1597.6	4.7	2,900

Table 4-5. Segmentation Structural Alternative Approximate Cost.

Structure	Volume of Embankment (cy)	Size Of Culvert	Floodgate Size	Approximate Cost ($)
Powerline	212,000	6-ft dia RCP	6-ft by 6-ft	$ 2.5 m
Vineyard Road	40,000	6-ft dia RCP	6-ft by 6-ft	$ 0.54 m
Rittenhouse	81,000	6-ft dia RCP	6-ft by 6-ft	$ 1.0 m

4.4.2 Upgrade To High Hazard Dam

Under the newly enacted ADWR rules and regulations, all three dams would be classified as intermediate size, significant hazard dams. The inflow design flood (IDF) for all three structures is the 1/2 PMF. It is anticipated that the downstream hazard rating of the three structures may change from significant to high hazard due to encroaching urbanization. The new rules state that future conditions must be considered when evaluating the downstream hazard potential. In the case of future land use conditions, for a high hazard dam, the IDF may vary between the 1/2 PMF and the full PMF due to the projected growth of urbanization in Maricopa and Pinal Counties downstream of the dams. At the present time the District is considering upgrading the structures to high hazard. The upgrade to high hazard classification will require additional evaluation by the District beyond what is presented in this study.

In a previous District hydrologic study of Powerline FRS, the District concluded that the Powerline FRS emergency spillway would spill from the 100-year event. The District study recommended that as a remedy and as a future District action, to upgrade the Powerline FRS to a **high hazard dam** that would safely pass the **full PMF**. In another study conducted to determine the inundation limits from a dambreak analysis, for all three structures in their existing condition, the study concluded that all three dams would be overtopped by the PMF event. For the purposes of this structural alternative, all three dams were evaluated against the full PMF, following in line with the District recommendation for Powerline FRS to be upgraded to a high hazard dam. However, at the present time, the 1/2 PMF may be sufficient as the IDF through the structures.

Under the new rules, since these are existing dams that would potentially undergo major alterations and modifications, ADWR would require that the new rules and regulations be applied as though the dams were new dams. Therefore, the requirements for new significant and high hazard dams regarding embankment stability factors and seismic criteria would apply (although not evaluated as part of this study). The ADWR design requirements for new significant and high hazard potential dams is found in Arizona Administrative Code Title 12, Chapter 15, Article 12, specially, **R12-15-1216 "Dam Design Requirements for New High, Significant, and Low Hazard Potential Dams".**

The upgrade of the three dams would require either:
1. Raising the dam embankment, or
2. Enlarging the reservoir impoundment pool volume, or
3. Upsizing the principal and/or emergency spillways, and
4. Examination of downstream constraints (Powerline Floodway capacity) and hazard potential, or
5. A combination of the above first three items with item no. 4.

There are many possible combinations of structural measures that could be investigated that would provide reasonable dam performance for a high hazard structure. However, it is not the scope of this alternative analysis to optimize the configuration of the dam and spillways based the many possible iterations of principal and emergency spillway configurations, dam height, and reservoir volumes. In order to keep the analysis for upgrading to a high hazard dam as simple as possible, it is assumed that all physical features of each dam will remain the same as existing except that the dam height and emergency spillway crest width will be increased to safely pass the IDF (full 6-hr PMF). In this fashion the only structural modifications to the dam is the height of the embankment and/or width of emergency spillways. No upsizing of the reservoir impoundment was considered as part of this analysis. The alternative analysis incorporated a minimum of three-feet of freeboard from the top of the dam to the maximum pool water surface. The elevation of the crest of the emergency spillway was not changed over as-built elevations.

The basis for the analysis of this structural alternative is the hydrologic and dambreak study conducted by James M. Montgomery (August 1989) for Powerline, Vineyard Road, and Rittenhouse FRS. The JMM study examined the performance of each dam against the full PMF. The results of the study indicated that each dam, under existing structure conditions, would be overtopped by the full PMF. The electronic HEC-1 models for the JMM study could not be located within District files. The District courteously reproduced the JMM HEC-1 models for the 100-year storm events and provided these models to KHA.

KHA reviewed the FCD model inputs, executed the HEC-1 models with the FCD input data files and duplicated the results of the JMM study for the 100-year event.

KHA modified the 100-year models and converted these models to match the JMM HEC-1 input files for the 6-hour PMP models. KHA executed the modified models for the 6-hour PMP and reproduced the results of the JMM report. KHA then adjusted the 6-hour PMP models to correct several discrepancies found the JMM 6-hr PMP models. The JMM report stated that the reservoir routing for the PMP events would start with an initial condition that the reservoirs were full to the elevation of the emergency spillway crest. A review of the JMM models indicated that the reservoir initial conditions started the reservoir routing at a pool elevation much lower than the elevation of the emergency spillway crest. KHA also adjusted the orifice equation exponent in the reservoir routing routine from 1.5 to 0.5 (SL record for Rittenhouse and Vineyard FRS).

KHA took the adjusted PMP models and modified the top of dam elevations and emergency spillway widths to safely pass the 6-hr PMF. The dams were raised and spillways widened to obtain three-feet of freeboard. It should be noted that additional reservoir routing iterations will be required to optimize top of dam elevation, spillway widths, and reservoir storage capacity. It is recommended that new topographic mapping be developed to assist future analyses. Table 4.6 summarizes the results of the HEC-1 models for passing the full 6-hr PMF with three-feet of freeboard.

Table 4.6. Results of Upgrading to High Hazard Dams (IDF = full PMF).

Structure	Top Of Dam (ft)		Emergency Spillway Width (ft)		Max Water Surface (ft)		Overtopping Depth (ft)	
	As-built	KHA Model	As-built	KHA Model	JMM Model	KHA Model	JMM Model	KHA Model
Powerline	1589.1	1593.4	600	900	1590.03	1590.40	0.93	0
Vineyard	1579.5	1584.4	600	900	1579.93	1581.35	0.43	0
Rittenhouse	1602.3	1606.6	600	900	1602.84	1603.56	0.54	0

Table 4.7 provides an approximate cost for upgrading Powerline, Vineyard Road, and Rittenhouse FRS to high hazard dams capable of passing the full PMF with three feet of freeboard. The embankment costs are based on the existing location and embankment section of each flood retarding structure.

Table 4-7. Upgrade to High Hazard Structural Alternative Approximate Cost.

Structure	Volume of Embankment (cy)	Emergency Spillway Grading (sy)	Approximate Cost ($)
Powerline	260,720	34,600	$ 3.05 m
Vineyard Road	516,608	35,800	$ 5.75 m
Rittenhouse	226,171	34,600	$2.69 m

4.4.3 Modifications to Improve Performance

This structural alternative/measure examines on a conceptual level the feasibility of structural modifications to the dams and/or associated features that would enhance the performance of the dam from an operational, performance, and maintenance aspect. The structural modifications are based upon a review of previous technical studies for each dam and from a detailed field inspection conducted in October 1999. The structural modifications apply to each of Powerline, Vineyard Road, and Rittenhouse FRS. These modifications are based on existing conditions found at each dam site and are modifications other than those identified in the Individual Structures Assessment Report (e.g., finger drains to the central filter; transverse crack repairs; and normal maintenance activities). The modifications are for the purposes of this alternative study are:

1. Construct a concrete sill in the control section of the emergency spillway for each dam. The concrete sill will function to arrest headcut propagation from erosion due to discharges in the earthen emergency spillways. The concrete sills will also function as control weirs for discharges into the emergency spillways. Rating curves for the emergency spillways may be developed based on the length and elevation of the concrete control sill.
2. Provide erosion protection around the abutment ends that are adjacent to the emergency spillways. At the present time, the abutments located adjacent to the emergency spillway channels are unprotected from potentially erosive flows in the spillway. The type of erosion protection evaluated consists of rock riprap placed on graded slopes. A geotextile fabric would be placed on the embankment slopes prior to placement of the rock riprap. The riprap would be sized based on the velocity of maximum discharge in the spillway.
3. Evaluate trash rack opening size for principal spillways. The existing trash rack opening size appears to be conservatively large considering the types of debris and size of debris within the impoundment areas and the diameter of the principal spillways. The largest normal type debris was found to be tree limbs from dead palo verde or mesquite trees. This was confirmed with District O&M staff. Occasionally, landscape trimmings and miscellaneous construction debris is dumped illegally within the reservoir pool area.

The emergency spillways for Powerline, Vineyard Road, and Rittenhouse FRS are limited service spillways. A limited service spillway is designed to operate very infrequently, and with the knowledge that some degree of erosion or damage will occur during operation. When the facility does operate, the following conditions should be attained:

1. The spillway flow and/or resulting erosion will not endanger the dam or dam foundation.
2. The control of the discharge will remain at the predetermined control section and will not be lost due to erosion.
3. There will be sufficient time available after a spillway use event to evaluate the resultant conditions and perform repairs or reconstruction prior to the next event.

A positive discharge control section is required for a limited service spillway. The section should be permanently fixed either in a rock cut or by construction of a concrete sill structure. The simplest type of control structure is a flat concrete slab with sidewalls, placed at a break in grade that will result in critical depth on the control section. The control section may be located to provide a long spillway channel with a large portion of the channel at a subcritical slope. This is done to ensure that the erosion, or head cutting, will start downstream from the subcritical slope and that the channel length is maximized, in order to maximize the material to be eroded and the time that will be required for the erosion to reach the control section.

A review of the as-built construction plans for Powerline, Vineyard Road, and Rittenhouse FRS emergency spillways show that the structures were not constructed with concrete control structures. The control sections are located at the upstream end of the emergency spillways and are compacted earth benched sills. The control sections or crests of the emergency spillways are shown to be one-foot above a level upstream approach section. The compacted earth bench sills should be replaced with a concrete sill and upstream and downstream concrete slabs. The depth of the concrete sills should be determined based on the predicted depth of scour downstream of the sill or upon a headcut analysis. For the purposes of this structural enhancement, the sill depth was based on the predicted scour below a channel drop (grade control structure). Table 4-8 provides a summary of the sill depth computations.

Table 4-8. Control Section Sill Depth Summary.

Structure		Emergency Spillway Design Discharge (Cfs)	Spillway Width (Ft)	Unit Discharge (Cfs/Ft)	Drop Height (Ft)	Downstream Depth Of Flow (Ft)	Scour Depth (Ft)	Control Sill Depth (Ft)
Powerline		16,600	600	28	1.0	3.22	3.4	4.5
Vineyard Road	N	6,400	300	21	1.0	3.18	2.9	4.0
	S	6,400	300	21	1.0	3.04	3.0	4.0
Rittenhouse		12,800	600	21	1.0	2.69	3.1	4.5

The left abutment of Powerline FRS, both abutments for Vineyard Road FRS, and the left abutment for Rittenhouse FRS all terminate at the approach channels to the emergency spillways. The as-built plans for all three structures indicates that the approach channel lies on a constructed bend to direct flows around the end of the abutments. The potential exists that the abutments could be eroded due to flows in the approach channel. An evaluation of the approach flow velocity against the type of compacted earth embankment of the dam was conducted in order to estimate the size of rock required for erosion control at the abutments. Table 4-9 provides a summary of the required rock riprap size to be placed on the abutment slopes for erosion protection. Note that the District has an on-going structure slope erosion repair and maintenance program. It is recommended that the abutment protection be incorporated as part of that program.

Table 4-9. Abutment Slope Riprap Size.

Structure		Emergency Spillway Design Discharge (Cfs)	Spillway Width (Ft)	Depth Of Flow (Ft)	Velocity (Ft/S)	Rip-Rap Size (D_{50}) (Ft)*
Powerline		16,600	600	3.22	8.5	1.6
Vineyard Road	N	6,400	300	3.18	6.57	1.0
	S	6,400	300	3.04	6.87	1.0
Rittenhouse		12,800	600	2.69	7.85	1.4

* (from Figure 9.1 City of Tucson Standards Manual for Drainage Design and Floodplain Management, December, 1989)

The principal spillways for each dam are equipped with a trash rack as part of the inlet structure. Trashracks are provided where debris protection is required to prevent clogging of the principal spillway. Trashracks are designed to retain debris of such size and type of material that could result in clogging of the inlet of the principal spillway. Trashracks should be designed for safe operation with 50 percent clogging (Corps of Engineers). The average velocity of flow through a clean trashrack is not to exceed 2.5 feet per second under the full range of stage and discharge (NRCS, TR-60). Velocity is to be computed on the basis of the net area of opening through the rack (NRCS, TR-60).

The existing trashracks are ten-feet in diameter and are constructed of eighteen vertical 3-inch diameter steel pipes that are spaced on two-foot centers and two horizontal steel strap plates spaced 6-feet apart. The full opening area of a window cell of the trashrack is 12-square feet (6-ft long by 2-foot wide).

KHA conducted a field examination of the structures and appurtenant features in October 1999. KHA observed that debris produced from the upstream reservoir area and watershed consisted primarily of dead tree limbs, tree stumps, sticks, shrubs, trash (bottles, cans, styrofoam, paper), and discarded vehicle tires. The car and truck tires were of sufficient size to be retained on the trashracks.

The trashrack openings appear to be adequate in size based on the criteria of 50 percent clogging and the size of debris material found upstream of the trashracks for full stage impoundment. However, a third horizontal steel strap plate is recommended be added to the riser tower and located midway between the floor of the inlet and the first strap. This third strap is for more frequent flooding events than the reservoir design event.

The approximate cost to construct the concrete control sill per dam is $122,000. The approximate cost to place abutment slope protection is approximately $19,000 per abutment protected. Powerline and Rittenhouse FRS abutment slope protection would cost $19,000 each while Vineyard Road FRS would cost $38,000 total (for two abutments to be protected). The cost of the additional strap on the trash racks of the principal spillways was considered incidental. The total estimated cost for the modifications for Powerline, Vineyard Road, and Rittenhouse FRS is $210 thousand, $239 thousand, and $210 thousand, respectively (see Appendix G).

4.4.4 Replace Dams With Detention Basins

This structural alternative/measure examines on a conceptual level the feasibility of totally replacing the dam with detention basins. The embankment of the detention basins would be sized with a maximum height of 6-feet with a storage capacity sufficient for the 100-year event. The 6-foot criteria would make the detention basins exempt from ADWR jurisdiction.

A constraint associated with this alternative is the reservoir pool drawdown time. Maricopa and Pinal County drainage ordinances require that detention basins be evacuated within 36-hours. The drawdown times for the existing dams and reservoirs are longer than 36-hours. A drainage ordinance variance may be required to allow a longer drawdown time for the detention basin alternative. If a variance could not be obtained and granted, then the potential exists for large capacity outflow structures on the detention basins. The peak outflows from each of the detention basins could be greater than the existing peak outflows from each of the respective principal spillways.

There is a potential downstream impact as well if a drainage variance could not be obtained. It is assumed for the purposes of this alternative that the Powerline Floodway will be the primary outfall for the basins. The capacity of the floodway cannot be exceeded by the discharges from the detention basins. One potential remedy to help alleviate the problem of overtaxing the Powerline Floodway would be to direct Rittenhouse detention basin flows south to Queen Creek instead of into a Vineyard Road detention basin.

For the purposes of this structural alternative, it is assumed that the detention basins replacing the dams will not be required to meet the 36-hour drawdown time criteria. In this manner the reservoir and outlets for the basins can be designed to discharge peak flows from the basins at or less than the current outflow peak discharges from the dams.

Basically, the alternative would replace the dams with detention basins. The dam embankments would be lowered to a maximum height of 6-feet and the reservoir areas expanded to detain the 100-year event at a maximum depth of impoundment of 5-feet.

The existing volumes of floodwaters detained for the 100-year event for Powerline, Vineyard Road, and Rittenhouse FRS are 4,019 acre-feet (af), 4,112-af, and 3,875-af, respectively. The reservoir areas for each of the three dams were evaluated to detain these same detention volumes but at shallower depth. One-foot of freeboard was assumed thus allowing a maximum depth of water in the detention pond of 5-feet. The reservoir area required to detain the volumes of water at a maximum depth of 5-feet for Powerline, Vineyard Road, and Rittenhouse FRS are approximately 804-acres, 823-acres, and 775-acres, respectively. The existing impoundment areas are 610-acres, 637-acres, and 660-acres for Powerline, Vineyard Road, and Rittenhouse FRS, respectively. The basin inverts will essentially match the inverts of the principal spillways, however, a slight basin floor slope is required for positive drainage. Table 4-10 provides the detention basin dimensions assuming a maximum depth of ponding of 5-feet.

Table 4-10. Approximate Detention Basin Dimensions.

FRS	100-Year Capacity (Af)	Max Depth (Ft)	Average Length (Ft)	Required Width (Ft)	Principal Spillway Invert (Ft)
Powerline	4,019	5	8,000	4,400	1563.0
Vineyard Road	4,112	5	26,000	1,400	1563.0
Rittenhouse	3,875	5	14,000	2,400	1578.2

The existing principal spillways for Powerline, Vineyard Road, and Rittenhouse FRS are reinforced concrete pipes, the diameters of which are 36-inch, 54-inch, and 33-inches, respectively. The inverts of these principal spillways are 1563-ft, 1563-ft, and 1578.2-ft, respectively.

The approximate construction costs for detention basins are $57.9 million, $24.4 million, and $45.2 million for Powerline, Vineyard Road, and Rittenhouse FRS, respectively. Appendix H provides back-up data for the estimated construction costs for this alternative.

4.4.5 Levee/Floodway System

This structural alternative/measure examines on a conceptual level the feasibility of converting the dams into a levee system. The dams would be linked together to form a very long contiguous levee embankment. The levees would convey 100-year event floodwaters south to discharge to the Sonoqui Detention Dike instead of detaining/impounding water behind the dams. Since water would not be impounded behind the levees, the levees would not be subject to ADWR dam safety rules and regulations.

One of the primary constraints associated with this alternative is the local topography of the area adjacent to the dams. Powerline FRS directs flows southward to it's principal spillway. The question is with Vineyard Road FRS and Rittenhouse FRS. Floodwater impounded behind Rittenhouse FRS discharges through it's principal spillway into the reservoir pool for Vineyard Road FRS. The local topography at Rittenhouse and Vineyard Road FRS is to drain slowly northward toward Powerline Floodway. This alternative would require redirection of Vineyard and Rittenhouse floodwaters south (against existing grade) to discharge to the Sonoqui Detention Dike. What makes this alternative potentially feasible is that the CAP canal located adjacent to the dams also flows in the direction (southward) that the levee system would direct flows (to Sonoqui Detention Dike).

Another very important constraint is the Sonoqui Detention Dike and the ability to accept the flows from the levee system. The Sonoqui Detention Dike was designed and constructed by the Bureau of Reclamation to provide flood protection to the Central Arizona Project Canal, Reach 3. The structure is located south of Rittenhouse FRS (see Exhibit D located in the map pocket in the back of this report) and is approximately 7.3 miles long according to the construction plans of the dike. The top of dike crest elevation is 1585-ft, with a crest width of 16-ft, and is approximately 18-ft in height. The dike was designed to detain the 100-year and PMF storm events. The 100-year pool elevation is 1580-ft (storage 8,424-af) and the PMF pool elevation is the top of the dike. The dike is approximately 165-ft offset (east) of the CAP canal. The primary outlet works is located on the alignment with Queen Creek and consists of an ungated four 72-inch diameter culverts with concrete inlet and outlet structures. The outlet also has a baffle block concrete apron to dissipate energy from outlet releases. The construction plans do not indicate if an emergency spillway was constructed with the structure. The 100-year peak inflow into the Sonoqui Detention Dike is 16,240-cfs with a discharge of 1,113-cfs. The PMF inflow is 37,016-cfs (storage of 13,095-af) and the discharge is 2,296-cfs.

The Sonoqui Detention Dike and reservoir would require modifications in order to accept additional storm flows from the Powerline, Vineyard Road, and Rittenhouse levee system. The modifications could include increasing the height of the dike and increasing the detention volume capacity of the structure. Under this structural alternative, the Rittenhouse levee and the Sonoqui dike would be relocated and constructed closer to the CAP canal to create a link thereby creating the inflow point from the new levee system with the dike. Additional hydrologic analysis will be required to develop a hydrologic model for flows routed from the proposed levee system to be combined with flows at the Sonoqui Detention Dike. Modification of the Sonoqui structure and reservoir and the additional hydrologic/hydraulic analysis will require further examination beyond the level of this concept alternative analysis should this particular structural measure be explored in greater detail in future studies. Another consideration includes either maintaining the current discharge rating for the primary outlet structure or increasing the discharge capacity in account of the additional inflows from the proposed levee system.

The proposed levee system will require an upstream parallel floodway to convey floodwaters to the Sonoqui structure. A concept earthlined floodway beginning at the southern end of Powerline FRS and terminating at Sonoqui was evaluated. It was assumed for the cost estimate that the floodway would be constructed in excavation. Flow velocities based on predicted flowrates in the floodway were examined for erosion compatibility with earthlined channels.

A modification of the levee system alternative would be to incorporate Vineyard Road FRS and Rittenhouse FRS into the levee system and leave Powerline FRS as is. The Vineyard Road and Rittenhouse levee system would discharge to Sonoqui. Powerline FRS would still function as a dam and discharge into the Powerline Floodway. This modification may warrant further examination at a future time if the "levee" alternative appears to require additional investigations. No further study of this modification, beyond mention, will be conducted as part of this analysis.

Basically, this alternative would link together Powerline, Vineyard Road, and Rittenhouse FRS into one very long levee that would convey floodwaters to the Sonoqui Detention Dike, which in turn, discharges to Queen Creek wash. The levees would not fall under the jurisdiction of ADWR dam safety regulations as long as floodwaters are conveyed and there is less than 50-acre feet of impoundment or storage. A floodway channel would be constructed on the upstream side of the levee system in order to convey the more frequent floods southerly to the Sonoqui structure. The levee/floodway system would be designed to convey the 100-year 24-hour flood event.

The dams would be reduced in height and the abutments at Powerline (left abutment), Vineyard Road (both abutments) and Rittenhouse FRS (both abutments) would be removed. The material removed from the embankments/abutments and the floodway excavation would be used to link the dams together and extend the levee south to join with the relocated Sonoqui structure.

A preliminary hydraulic analysis was conducted to estimate the size of the floodway that would parallel to the proposed levee system. The flowrates for sizing the floodway were based on the total peak inflows to each of Powerline, Vineyard Road, and Rittenhouse flood retarding structures (100-year event). The floodway would be an earthlined trapezoidal channel that would transition in bottom width from approximately 540-feet at Powerline FRS downstream to 800-feet at the Sonoqui structure. The floodway channel is conceptually designed to handle 12,000-cfs at Powerline to 34,800-cfs at Sonoqui. Flow velocities in the floodway are conducive to an earthlined channel (velocities range from 2.8 feet-per-second to 3.8 feet-per-second). Table 4-11 summarizes the levee/floodway the approximate costs (See Appendix I for a detailed cost breakdown which includes cost assumptions for land acquisition).).

Table 4-11. Opinion of Probable Construction Costs for Levee/Floodway Structural Alternative.

Description	Cost ($)
Abutment Removals	$ 772,000
Remove Rittenhouse FRS	$ 3,254,000
Link Powerline and Vineyard FRS	$ 653,000
Construct Rittenhouse Levee	$ 2,500,00
Construct Floodway(s)*	$ 46,000,000

(* assumes floodways are all in excavation)

4.4.6 Utilization of Central Arizona Project Canal

This structural alternative/measure examines on a conceptual level the feasibility of utilizing the CAP canal to accept discharges of stored floodwaters from behind Powerline, Vineyard Road, and Rittenhouse FRS. Benefits to the District under this alternative is that it allows the impoundments to be evacuated more rapidly and reduces the potential of use of the limited service emergency spillways. The CAP benefits from this alternative in that the CAP may be spared from potentially damaging flows discharging from the emergency spillways.

This concept is similar to the method of evacuation of floodwaters from behind the Bureau of Reclamation's Reach 11 dam. The CAP is located immediately downstream of the toe of the dam. The standard operations plan for the CAP under this scenario allows the CAP to be drained by diverting upstream CAP flows into the Agua Fria River and downstream into the Salt River through the Salt River/CAP interconnect. Once the CAP is drained floodwaters impounded in Reach 11 may be discharged into the CAP canal.

A similar method to discharge floodwaters into the CAP canal was evaluated for Powerline, Vineyard Road, and Rittenhouse FRS. This alternative assumes that discharges from the dam's principal spillways will discharge as currently operated (discharge into the Powerline Floodway). The concept for this alternative would be to construct gated outlets (similar to the Reach 11 outlets) in the structure embankments. The outlets would be, under flooding or emergency conditions, opened to discharge floodwaters into an evacuated CAP canal. The CAP would convey these floodwaters to Queen Creek or to downstream CAP water users.

The alternative provides the District operational flexibility to discharge floodwaters under flooding or emergency situations. The new outlets in combination with the existing principal spillways would drain the impoundments in less time than if the principal spillways were operating alone. This alternative could lessen the frequency of use of the emergency spillways for each dam and/or reduce the flooding impacts from emergency spillway discharges. If floodwaters could be drained from the impoundments quicker, storage volume is subsequently made available for additional inflows under emergency conditions.

The primary constraint to make this alternative a feasible measure is to develop a procedure to empty or evacuate the CAP canal in a sufficient enough time to allow the dams to discharge to the canal. The CAP water would be diverted out of the canal at the upstream end of this canal reach at the Salt River. The CAP has the capability shut off flow in the canal at the confluence of the canal with the Salt River. This step would circumvent flows entering the CAP canal section that will be used for flood flow conveyance. Next a method of emptying the canal downstream of the dams would have to be devised. The CAP presently has no method for emptying the canal just downstream of the dams. Queen Creek wash crosses over the CAP canal through four 72-inch diameter steel culverts. This is the most logical location to construct a diversion structure to empty the CAP canal. The diversion structure (Queen Creek/CAP interconnect) would also function to release floodwaters that are conveyed by the canal into Queen Creek. Ultimately, flows discharged from the principal spillways to the Powerline Floodway and flows discharged to Queen Creek wash from the CAP canal would reach the EMF and then the Gila River.

Three new gated outlets would be constructed under this structural alternative - one for each flood retarding structure. The outlet structures would be similar to the outlets provided in the Reach 11 dams constructed by the Bureau of Reclamation. The outlet structures, under operating conditions, would discharge directly to the evacuated CAP canal. The location of the new outlet structures would be located adjacent to the principal spillways. Each new gate outlet would be sized for one-third the capacity of the CAP canal adjacent to the dams. The capacity of the CAP canal (Reach 2) adjacent to the structures is 2,750-cfs. For the purposes of the culvert hydraulics analysis, a CAP capacity of 2,700- cfs was used. Therefore, each outlet was sized for 900-cfs.

A preliminary culvert hydraulic analysis was conducted to determine a culvert size for the gated outlets for each dam. The results of the analysis indicate that twin 7-foot diameter reinforced concrete pipes per outlet are sufficient to pass the required 900-cfs. Each gated outlet will require an inlet and outlet structure in addition to the twin pipe culvert. The inlet structure will have an hydraulically/manually operated slide gate to allow floodwaters to enter the twin culverts. Due to the offset distance of the CAP from Rittenhouse FRS, an open channel floodway segment was included as part of the gated outlet from the dam. The floodway would begin at the outlet structure of the twin 7-foot diameter culverts and terminate at the CAP canal. The length of the floodway is approximately 300-feet, trapezoidal in section, concrete-lined, and approximately 4-feet deep. Table 4-12 (below) provides an estimated cost for the gated outlet structures. Appendix J provides back-up cost estimate data for this alternative.

A CAP/Queen Creek interconnect structure may be required under this alternative. Presently, there is no mechanism to drain the CAP canal just downstream of the dams. The interconnect would drain the canal to Queen Creek and also discharge floodwaters from the three dams. It is beyond the scope of this alternative to conceptually develop a CAP/Queen Creek interconnect structure. The costs reflected in Table 4-12 does not include an interconnect structure. However, an interconnect structure may not be required as long as there is a means of shutting off the flow in the canal upstream of the dams. This could occur at the CAP/Salt River interconnect. Once flow is shutoff upstream, the CAP canal can drain downstream, and therefore the dams can discharge into the canal.

Table 4-12. Concept Gated Outlets and Cost Estimate.

FRS	Gated Outlet Configuration	Length Of Culvert (ft)	Length Of Floodway (ft)	Estimated Construction Cost ($)
Powerline	Twin 7-foot dia RCP gated	210	N/A	$ 174k
Vineyard	Twin 7-foot dia RCP gated	350	N/A	$ 269k
Rittenhouse	Twin 7-foot dia RCP gated	100	310	$ 225k

If a CAP/Queen Creek interconnect were included, this alternative assumes that Queen Creek wash and the EMF will have the capacity to service the flood discharges from the three dams (2,700-cfs). No further capacity investigation is provided in this study for Queen Creek wash or the EMF.

This alternative will require approval from the Bureau of Reclamation and the Central Arizona Project. The Bureau does not own any dams within Reach 2 of the CAP and therefor will not be discharging floodwaters into the canal as would be done in Reach 11.

4.4.7 Increase the Capacity of Powerline Floodway

This structural alternative/measure examines on a conceptual level the feasibility of increasing the capacity of the Powerline Floodway from Powerline FRS to the East Maricopa Floodway. Presently the floodway serves as the principal flood conveyance structural facility for the Powerline, Vineyard Road, and Rittenhouse FRS. Under current standard operating conditions, the principal spillways from all three structures directly or indirectly discharge to the Powerline Floodway. The principal spillway for Rittenhouse FRS discharges directly into the impoundment for the Vineyard Road FRS. The Vineyard Road FRS and Powerline FRS principal spillways discharge into the Powerline Floodway.

The existing Powerline Floodway is approximately 8.7 miles long and consists of a concrete-lined trapezoidal flood channel. The bottom width ranges from 6-feet to 8-feet wide and the depth ranges from 4-feet 9-inches to 6-feet 6-inches deep. The approximate capacity of the existing floodway is 600 cfs at the CAP canal and 3,140-cfs at the confluence with the East Maricopa Floodway. The existing right-of-way for the floodway is 66-ft (from construction as-builts) for most of the length of the floodway.

The Powerline Floodway discharges into the East Maricopa Floodway (EMF) at Ray Road at the northwest corner of Williams Gateway Airport. The EMF was constructed to provide flood protection for development in the East Valley. The EMF channel is more than 27 miles long and is located parallel to the Roosevelt Water Conservation District irrigation canal from Princess Basin (above Brown Road in Mesa), across Hunt Highway, then westerly through Pinal County and into the Gila River. The structure is a compacted earth channel, approximately 200-feet wide and ranging in depth from 8 to 12-feet. The EMF spans three watershed projects: Buckhorn-Mesa, Apache Junction-Gilbert, and the Williams-Chandler watershed projects. The design flow of the EMF at the confluence with the Powerline Floodway is approximately 6,500 cfs. The design flow of the EMF at the Gila River confluence is 8,100-cfs.

This structural alternative examines the feasibility of increasing the capacity of the Powerline Floodway to approximately 23,000-cfs. This new capacity was based on the combined capacity from the emergency spillways from both Powerline FRS and the right emergency spillway for Vineyard Road FRS. Note that Vineyard Road FRS has two emergency spillways – a right spillway and a left spillway. Both spillways are of identical capacity and have a combined capacity of 12,800 cfs. The Powerline FRS emergency spillway capacity is 16,600 cfs.

The choice to use the design capacity of the emergency spillways instead of the IDF was based on review of the most recent hydrologic study for Powerline, Vineyard Road, and Rittenhouse FRS. In an August 1989 hydrologic study conducted for each of these dams, James M. Montgomery (JMM) Engineers investigated each dam's performance against the 25-, 50-, and 100-year events. JMM also investigated dam performance against the full PMF (both the 6-hour and 72-hour PMF). JMM routed the 25-, 50-, and 100-year storms through each reservoir and dam assuming that the reservoir was initially empty. JMM did not investigate the ½ PMF. Since no study could be located that evaluated the ½ PMF (the current ADWR IDF for each dam) and since the full PMF overtops each dam according to JMM results, and since little to no spills occurs in the emergency spillways under 100-year flood events, the choice to use emergency spillway design capacity as the new capacity for the Powerline Floodway was clear for the purposes of this alternative.

Spillway capacities were taken from the JMM study. This alternative at least provides a floodway to match the capacity of the Powerline FRS and right spillway of Vineyard Road FRS. It is assumed under this alternative that the left spillway of Vineyard Road FRS and emergency spillway for Rittenhouse FRS continues normal (existing) operations. No modifications to these spillways are investigated as part of this alternative.

This alternative will provide a floodway corridor from Powerline FRS to the East Maricopa Floodway that will have the capacity to handle emergency spillway flows from both Powerline FRS and the right emergency spillway of Vineyard Road FRS. As stated previously, there is no defined downstream channel, wash, or drainage facility that is capable of handling discharges from the emergency spillways (see Section 4.1.4).

Modifications to the Powerline Floodway under this alternative would include widening/deeping the existing trapezoidal channel and providing for a new CAP overchute for the emergency spillway flows. Constraints on this alternative include right-of-way acquisition, widening existing bridge/roadway crossings along the floodway, and the downstream capacity of the EMF from the confluence of Powerline Floodway to the Gila River.

The District should consider as a future action the purchase of land for a large regional detention basin. The detention basin would be located at the former General Motors Proving Grounds. The large detention basin would help resolve regional drainage problems and capacity constraints for both the Powerline Floodway and the EMF downstream of the Powerline/EMF confluence. No further consideration of this alternative is provided as part of this study. This alternative may be examined further as part of the EMF mitigation study and the East Mesa ADMP.

Basically, this alternative will increase the capacity of the Powerline Floodway from the CAP to the EMF. The existing capacity of Powerline Floodway at the CAP is approximately 600-cfs and increases to 3,140-cfs at the EMF.

Initially, as discussed above, it was proposed to evaluate the floodway based on a capacity of 23,000-cfs, which is the combined peak capacity discharge from the Powerline FRS and Vineyard Road FRS right emergency spillway. This design peak discharge was re-evaluated based on the examination of the results of the previous District studies presented in Section 4.4 A. The District's previous studies have indicated that the design capacity of the EMF (Reach 2 - Hunt Highway to Chandler Heights) is approximately 8,100-cfs, substantially lower than the 23,000-cfs. The District studies are attempting to mitigate the capacity problems in the EMF for the 100-year 24-hour storm. Hence, the objective is to reduce peak discharges to the EMF, not increase the flows. In light of these findings, the Powerline Floodway will be evaluated based on the potential maximum and average channel slope from the CAP canal to the EMF and right-of-way constraints.

A hydraulic analysis was conducted to determine the maximum and average channel capacity for an upsized Powerline Floodway. The hydraulic analysis was based on assuming a maximum and average channel slope of 0.0052 ft/ft and 0.003 ft/ft, respectively (slopes derived from as-built plans of Powerline Floodway). The analysis assumed a concrete-lined rectangular section, 66-foot right-of-way, and a 12-foot wide maintenance road. The maximum channel bottom width under this condition was 52-feet. With a depth of 5-feet, the maximum and average channel capacity is 6,220-cfs and 4,200-cfs, respectively. This channel flows supercritical.

For the purposes of this alternative, a rectangular concrete-lined channel with a capacity of 4,000-cfs to 6,000-cfs was assumed. The increase in Powerline Floodway channel capacity would allow the principal spillways for all three dams to be increased in size and allow a greater discharge to occur into the Floodway and hence into the EMF.

The major constraints on this alternative is the existing right-of-way, the Powerline Floodway/EMF confluence, bridge crossings, side channel/drain inlets into the Powerline Floodway, and the capacity of the EMF downstream of the Powerline Floodway. An advantage of this alternative over the CAP alternative is the potential availability of capacity. This alternative has a greater discharge capacity than the CAP canal alternative (which is limited to 2,700-cfs). The estimate of the approximate costs to upsize the Powerline Floodway is $13.2 million. This cost excludes modifications required to bridge crossings, utilities, additional right-of-way, and modifications to the EMF confluence with the Powerline Floodway.

4.5 Nonstructural Alternatives

The nonstructural flood control (impoundment/pool area) alternatives for the Powerline, Vineyard Road, and Rittenhouse FRSs were developed and formulated in a planning meeting held between the District and KHA. The nonstructural flood control alternatives that were formulated for concept development and analysis for the Powerline, Vineyard Road, and Rittenhouse FRSs are listed below and are described in detail in the following subsections.

- Mitigate Through Flood Insurance
- Acquire Properties/Flowage Easements
- Develop Emergency Action Plan

4.5.1 Pool Area – Mitigate Through Flood Insurance

This nonstructural alternative examines the feasibility at a concept level of obtaining flood insurance for the impoundment areas for Powerline, Vineyard Road, and Rittenhouse FRS.

The FEMA flood insurance rate maps (FIRM Map No. 0400770300C, Effective Date: August 15, 1983; 0400770325C, Effective Date August 15, 1983; and 040077 0125 D, Effective Date: March 5, 1990) delineates special flood hazard areas and base flood elevations for various streams and washes in the vicinity of the Powerline, Vineyard Road, and Rittenhouse FRS. The maps do not show the dams or the impoundment ponding limits or zone designations created by the three structures and the pool areas. The inundation pools would most likely be designated as Zone A. The zones outside the inundation pool are designated Zones C and A. It is interesting to point out that the Magma FRS is indicated on map panel 325C. This structure is located south of Rittenhouse FRS and Queen Creek wash. Magma FRS is of the same vintage as Powerline, Vineyard Road, and Rittenhouse FRS.

This nonstructural alternative examines the feasibility at a concept level of obtaining flood insurance for the pool areas for Powerline, Vineyard Road, and Rittenhouse FRS. The District does not own the property (pool area) upstream of the dam, however the District does have flowage easements from State Lands for the pool areas. The concept is the District would obtain on a property by property basis flood insurance policy for the areas inundated by the as-built pool.

The approximate limits for the as-built pools for each FRS are provided in Exhibit C and D (located in the map pocket in the back of this report). The as-built pool elevations for Powerline, Vineyard Road, and Rittenhouse FRS are 1583.3-ft, 1574.8-ft, and 1597.6-ft, respectively. The inundation pools would most likely be designated as Zone A. The zones outside the inundation pools are designated Zones C and A. The number of acres of inundation for Powerline, Vineyard Road, and Rittenhouse FRS reservoir pools are approximately 610, 637, and 660 acres, respectively.

Section 3.3.1 provided a brief review of the premium rates for flood insurance for a typical coverage amount of $100,000 for a single-family residence. Given the previous rates and dwelling units per acre provided in Section 3.3.1, the amount of flood insurance premiums required to insure the lands within the pool area is provided in Table 4-13.

Table 4-13. Approximate Zone A Flood Insurance Costs for Pool Areas of Powerline, Vineyard Road, and Rittenhouse FRS.

FRS	No. Acres	Dwelling Units	Unit Premium	Annual Premium	30-Year Premium
Powerline	610	1,120	$301	$337,120	$10,113,600
Vineyard Road	637	1,274	$301	$383,474	$11,504,220
Rittenhouse	660	1,320	$301	$397,320	$11,919,600

The above costs reflect flood insurance premiums for occupable dwelling units. However, the District does not allow permanent habitable structures within the flood pools for any dam. As such the unit premium rates provided in Table 4-13 were taken at 50 percent to develop rates for uninhabitable structures (corrals, recreational facilities, etc.). The density of uninhabitable and insurable structures was assumed to be 10 units per pool area. Therefore, the annual premium per dam was estimated to be $1,500 and the 30-year average premium is $45,000.

A previous District study for Powerline FRS recommended that the structure be upgraded to pass the full PMF. The District should develop, as a future planning consideration, the full PMF ponding limits for Powerline, Vineyard Road, and Rittenhouse FRS. The District would then be able to compare the 100-year ponding limits with the existing District flowage easements and the full PMF ponding limits. Estimates of flood insurance costs could then be developed for the area bounded between the 100-year ponding limits and the full PMF limits.

4.5.2 Pool Area – Acquire Properties/Flowage Easements

This nonstructural alternative examines the feasibility at a concept level of acquiring lands or properties that the District does not currently own (or lease) for all three dams. The District owns the dams but does not own (but leases) the land upstream of the dams that could potentially be inundated by the 100-year event or the IDF. The lands upstream of the dams are State Trust Lands. The District does have flowage easements from State Lands for the impoundment areas for each dam. Exhibit D (located in the map pocket in the back of this report) illustrates the District easements for the structures including the approximate inundation limits for the 100-year and PMF.

No additional flowage easements appear to be required for the pool areas of Powerline, Vineyard Road, and Rittenhouse FRS for both the 100-year and PMF events. This is based on the District's existing easements as depicted on Exhibit D.

The as-built ponding limits are depicted in Exhibit D. The amount of lands that the 100-year ponding limits include is approximately 2,000 acres. At a unit cost of $50,000 per acre, the approximate costs to purchase lands that include the as-built pools is $100 million.

4.5.3 Pool Area – Develop Emergency Action Plan

This nonstructural alternative examines the feasibility at a concept level of preparing an emergency action plan for the pool area. The pool area for the purposes of this alternative includes the as-built ponding limits as depicted on the dam record drawings.

The District does not have an individual emergency action plan for Powerline, Vineyard Road, or Rittenhouse FRS and impoundment areas meeting minimum FEMA dam safety guidelines.

As stated previously, an individual emergency action plan needs to be prepared for the Powerline, Vineyard Road, and Rittenhouse FRS. The plan would be inclusive: all elements of the dam and reservoir area would be incorporated as part of the plan. The costs of preparation of an EAP for each dam would be approximately $20,000 - $30,000.

The report titled "Policy and Program Report" (April 2000) prepared by Kimley-Horn and Associates, Inc. recommended that the District prepare individual emergency action plans for each of the District dams in their inventory. The Report (Section 5.5) provided guidelines of what to include as part of the emergency action plans in order to meet minimum standard of care and FEMA dam safety guidelines. ADWR requires that all jurisdictional dams have an emergency action plan on file with the Department. The plan may be prepared in-house by District staff or the District may use outside engineering consultant services for this task.

The "Policy and Program Report" not only provided recommended elements to include in the emergency action plan but also provided a schedule for updating the plan and the conditions triggering an update. The Report also discussed different levels of exercising the emergency action plan such as orientation seminars, drills, tabletop exercises, functional exercises, and full-scale exercises.

Specific elements to include as part of the section of the emergency action plan covering the pool area are warning signs within and around the pool area explaining to the public of what to do in the event of inflows into the reservoir area. Evacuation routes should be displayed on the warning signs.

4.6 Evaluation and Ranking

The evaluation and ranking of the structural and nonstructural flood control alternatives for Powerline, Vineyard Road, and Rittenhouse FRS are presented in Section 5.0 of this report.

Section 5.0 Evaluation and Ranking

This Section of the report provides the methodology to evaluate, rank, and prioritize the project alternatives for each of the flood control dams analyzed as part of this study.

5.1 Evaluation Criteria Matrix

To assist in evaluating and comparing project alternatives, an evaluation and ranking matrix consisting of eight criteria with a range of point values was developed. The development of the matrix criteria was formulated with the input from both the District and the KHA project team. The matrix was developed in cooperation with the District in an attempt to objectively evaluate alternatives for a range of flood control and non-flood control criteria while still emphasizing that the primary purpose of each alternative is to reduce the risk and liability of dam ownership. The matrix is used to rank the alternatives presented in Section 3.0 and Section 4.0 and also to use as a guideline for future dam alternatives evaluations.

In developing an evaluation matrix for assessing the relative merits of the alternatives, the focus was to select a limited number of approximately independent criteria what would cover a reasonable range of factors impacting alternative selection. The creation of the matrix necessarily involves subjective judgement, not only in selecting the individual criterion but also in assigning a range of values for each criterion, as well as the particular value to use in the matrix for a particular criterion and alternative.

The evaluation matrix is presented in Table 5-1. As shown in the table, there are eight criteria with corresponding ranges in point values. Other criteria could be added to the matrix, however, increasing the number of criteria would add to much complexity to the matrix and the evaluation process. In addition, more criteria could lead to criteria having more interrelationships, potentially producing bias in the matrix evaluation results.

Table 5-1. Evaluation Criteria Matrix.

Evaluation Criteria	Range Of Point Values
Jurisdictional	1 to 8
Cost	1 to 10
Implementation	1 to 8
Environmental	1 to 8
Multi-Use and Aesthetics	1 to 5
Risk And Liability	1 to 15
Compatibility With District Plans	1 to 8
Flood Control	1 to 8

5.2 Matrix Application

The matrix is to be used to compare and evaluate the project alternatives that have been described in Sections 3.0 and 4.0. The alternatives are compared or evaluated by assigning a point value for each individual evaluation criterion based on the relative merit of a specific alternative compared to other alternatives within the set of alternatives. The point values for the entire evaluation criterion for a particular alternative are then totaled and a point value assigned to the alternative. The alternative with the highest numerical value compared to other alternatives in the set is the preferred alternative for that set of alternatives for the dam(s). Note, however, that even though an alternative may stand out numerically as the "preferred" alternative over other alternatives, considerations may be given that a combination of one or more alternatives would be most beneficial to the District in reducing risk and liability.

5.3 Evaluation Criteria Discussion

The evaluation criterion covers a wide range of factors that are believed to be the most significant factors when comparing alternatives. Not all criteria are weighted equally. More significant criteria are weighted more by having higher maximum point values. For example, criteria having a maximum point value of 10 will have a slightly more significant impact in the alternative selection than those having a lessor maximum value. The criteria that have been given the most weighting are cost and risk and liability. A discussion of each criterion is provided in the following paragraphs. Table 5-2 (below) provides a guideline to assist in the assignment of point values for each of the evaluation criteria.

Jurisdictional – The jurisdictional criteria reflects the degree of impacts the ADWR dam safety rules and regulations would have on the alternative under evaluation. An alternative that has significant impacts from dam safety rules and regulations (such as upsizing an existing dam to a high hazard dam) would be assigned a low point value as opposed to an alternative that the rules and regulations do not apply. Where the regulations do not apply, the alternative would be assigned a high point value. In this fashion, the jurisdictional criterion has a reverse scoring. The thought is to minimize jurisdictional constraints. Jurisdictional impacts include constraints by other agencies as well including the Corps of Engineers, the Bureau of Reclamation, and the Natural Resources Conservation Service.

Note that when the Corps of Engineers turned over Cave Buttes Dam to the District for operation and maintenance, the Corps retained jurisdiction over modifications to the structure. Caves Buttes Dam is a Congressionally Authorized project. Modification of project features would require a permit from the Corps.

Costs – Preliminary, reconnaissance level capital, engineering/construction management, operation/management, and land costs were developed for each alternative, where applicable. The more costly alternative will be assigned a lower point value, while the lower cost alternative will be assigned a higher value. The extent of engineering facilities included in an alternative is also reflected in the criterion in that more extensive facilities will result in an increased cost. Construction unit costs were prepared based on previous similar earthwork construction projects using Arizona Department of Transportation bid tabulations and other current project bids. Percentages of construction costs were used for estimating engineering/construction management (10%), contingency (25%), and operation/maintenance (10%). Note, operation and maintenance costs are additional to what is expended by the District on the existing structures. Land costs were estimated at $50 thousand an acre and easements at 90% land costs.

Implementation – Alternatives may require outside agency support and approval for project implementation and operations. For example, during emergencies, discharging floodwaters into the CAP canal will require approval and operational concurrence with the Bureau of Reclamation and the Central Arizona Project. Will the CAP and Bureau of Reclamation allow discharge when requested by the District? An alternative measure that is relatively implementable than another alternative would be assigned a higher point value.

Environmental – Alternatives may require an environmental clearance or investigation to evaluate the impact of the alternative on the environment. Alternatives may be constructed and located in high habitat value ecosystems or in sensitive desert areas. Environmental permits may be required for the implementation of a particular alternative (Section 404 permit, etc.). An alternative that is less of an impact to the environment and more readily permittable would be assigned a higher criterion point value than another that is difficult to permit or encroaches on environmentally sensitive areas.

Multi-Use and Aesthetics – Does the alternative offer multi-use/aesthetics improvement opportunities and possible project partners to share in funding improvements? If so, the alternative with the greater multi-use and aesthetics opportunities would be assigned the higher point values. Multi-use opportunities include recreational activities, groundwater recharge, sand & gravel mining, and cultural resources. These opportunities are recognized for the purposes of the evaluation as currently planned and potential multi-use and aesthetics opportunities and improvements.

Risk and Liability - Does the project alternative reduce the risk and liability to the District from owning dams? This criterion must be compared among the other alternatives in the set for the particular dam(s) under consideration. For example, detention basins instead of a dam may be considered to have less of risk and liability exposure to the District. Also, dam modifications to improve performance should also reduce risk and liability. This in turn enhances the District's dam safety program. A project alternative that has a higher reduction in risk and liability would be assigned a high point value as opposed to an alternative that does not have as high of a risk reduction.

Compatibility with District Plans – The District is conducting many planning studies throughout Maricopa County. These planning studies include but are not limited to

Watercourse Master Plans, Area Drainage Master Plans, and Area Drainage Master Studies. The goal of many of the studies is to examine flooding problems and provide flood control solutions (both structural and nonstructural) to alleviate drainage concerns. As an example, the District is currently studying project alternatives to alleviate the capacity constraints of the East Maricopa Floodway. The floodway presently does not have the capacity to meet existing hydrology. A dam project alternative that adds more water to the EMF would not be seen as a favorable flood control measure. Therefore, this alternative would be assigned a low point value.

Level of Flood Control - This criterion allows the ranking of alternatives based on the degree of the flood control measures provided with the project alternative. This includes both structural and nonstructural components. A question to reflect upon for the evaluation of an alternative is: What is the level of flood control provided with the alternative versus the existing level of protection? An alternative that provides a greater degree of flood control (including operations) over existing flood control conditions would be assigned a higher point value than an alternative that provides the same or less level of flood control.

5.4 Evaluation of Cave Buttes Dam Alternatives

Two sets of alternatives were developed in Section 3.0 for evaluation of Cave Buttes Dam. These sets of alternatives are grouped into structural and nonstructural flood control alternatives or measures. The alternatives were developed to reduce the risk and liability to the District of ownership, and operations and maintenance of Cave Buttes Dam. A summary of the two sets of alternatives presented in Section 3.0 for Cave Buttes Dam are provided in Table 5-3 below.

A. Structural Alternatives

The flood control structural alternatives for Cave Buttes Dam are evaluated qualitatively based on the evaluation criteria. A discussion of the three structural alternatives are presented in the following paragraphs. At the end of the discussion point values are assigned to each alternative for each evaluation criteria in tabular format.

Jurisdictional - Cave Buttes Dam is an existing structure that falls under the jurisdiction of ADWR, Office of Dam Safety. The current ADWR rules and regulations as applied to existing flood control dams are stipulated in the rules. Modifications or upgrades to the structure which include structural alternatives 1 and 3 (low-level outlets in Dikes No. 2 and No. 3) would require review and approval by ADWR. Both of these structural alternatives modify the each dike by the construction of the culverts and associated inlet and outlet structures.

Evaluation Criteria	Point Value Descriptions														
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Jurisdictional	Full ADWR Approval	Design Review		Study Review		Inspection Reports		No ADWR review	NA	NA	NA	NA	NA	NA	NA
Cost	(>$10m)				($5 - $7M)					(< $1M)	NA	NA	NA	NA	NA
Implementation	Difficult, many Stakeholders, Politics IGA's		Major public Involvement		Little public Involvement			Easy to implement; District only stakeholder	NA	NA	NA	NA	NA	NA	NA
Environmental	Permanent Impact	EIS/Individual Permit	Nationwide Permit		Temporary Impact			No Impact	NA	NA	NA	NA	NA	NA	NA
Multi-Use/ Aesthetics	No Opportunities	Landscape/ Asthetics	Recreational Cultureal	Sand and Gravel Recharge	Many Multi-use Opportunities	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA
Risk and Liability	No Reduction Risk						Operational And Maintenance Improvements	Flood Insurance	Emergency Action Plans/Flood Warning	Flowage Easements/ Acquisition of Properties			Upgrade to higher Hazard classification		Elimination of Dam Replacement of Flood control function
Compatibility	Compromises System Performance		Meets some goals And objectives			Same goals And objectives		Fully Compatible; Enhances District programs	NA	NA	NA	NA	NA	NA	NA
Flood Control	Less than 100-year protection		100-year		SPF	1./2 PMF		Full PMF Protection	NA	NA	NA	NA	NA	NA	NA

(Note: This table is intended to provide a guideline to the assignment of point values for each of the evaluation criteria).

Table 5-2. Evaluation Criteria Point Value Descriptions.

Table 5-3. Summary of Cave Buttes Dam Alternatives.

<table>
<tr><th colspan="2">Structural Alternatives</th><th colspan="2">Nonstructural Alternatives</th></tr>
<tr><th>No.</th><th>Description</th><th>No.</th><th>Description</th></tr>
<tr><td rowspan="3">1</td><td rowspan="3">Low-Level Outlet in Dike No. 2. Floodway channel from Dike No. 2 to Reach 11 detention dike.</td><td></td><td>Downstream of Dam</td></tr>
<tr><td>1</td><td>Mitigate through flood insurance</td></tr>
<tr><td>2</td><td>Acquire Properties/Flowage Easements</td></tr>
<tr><td rowspan="2">2</td><td rowspan="2">Floodway from emergency spillway to CAP canal. Floodway sized to capacity of CAP canal.</td><td>3</td><td>Update Emergency Action Plan</td></tr>
<tr><td></td><td>Pool Area of Dam</td></tr>
<tr><td rowspan="3">3</td><td rowspan="3">Low-Level Outlet in Dike No. 3. Small earthen floodway on outlet side.</td><td>4</td><td>Mitigate through flood insurance</td></tr>
<tr><td>5</td><td>Acquire Properties/Flowage Easements</td></tr>
<tr><td>6</td><td>Develop Emergency Action Plan</td></tr>
</table>

ADWR dam safety rules would require that the design of the low-level outlets consider and incorporate piping countermeasures for the culvert penetrations. In addition, potential methods of construction of the culvert penetrations would have to be provided to the satisfaction of ADWR. Given that the low-level outlets may not be used in every general storm event to discharge floodwaters (particularly Alternative No. 3), ADWR would still require that a new discharge rating curve for the dam be developed for review and approval.

Alternative 2 may not require approval of ADWR. However, it would be prudent, if the alternative is considered for further investigation, that ADWR be informed of the investigation and their input requested through alternative development. This alternative consists of a new concrete-lined floodway from the terminus of the emergency spillway to the CAP canal. The alternative does not modify the emergency spillway or the spillway discharge rating curve. Essentially, the alternative does not modify, upgrade, or rehabilitate the existing dam, dikes, or spillways. The alternative provides the District a positive means of conveying emergency spillway discharges (up to 3,000-cfs) to the CAP canal.

A permit will be required from the Corps of Engineers for any modifications to the structure, dikes, or emergency spillway. The Corps would require review of plan formulation, construction plan review, as well as any geotechnical supporting studies.

Cost - The approximate total costs for the low-level outlet in Dike No. 2, the floodway below the emergency spillway to the CAP canal, and the low-level outlet in Dike No. 3 are $ 2.1 million, $1.9 million, and $132 thousand, respectively.

Implementation - The greatest impediment to the feasibility of Cave Buttes Dam structural alternatives 1 and 2 is implementation. These alternatives would require review and approval of the Bureau of Reclamation, the Central Arizona Project, and perhaps the City of Phoenix and the Salt River Project.

The Bureau of Reclamation owns the CAP canal. The Central Arizona Water Conservation District (CAWCD) operated and maintains the facilities through a contract with the Federal Government. The primary purpose of the canal is to convey Colorado River water to Central Arizona. The canal was not designed as a flood control facility. Intentional entry of spillway discharges into the canal is not a contingency that the CAP has accommodated. There is one case, however, where this is the exception. The Bureau of Reclamation owns the Paradise Valley Detention Dikes. The only method of evacuating the reservoir pools from behind the dikes is direct discharge into the CAP canal. The Bureau and the CAP have developed a reservoir operations plan that outline the procedures to discharge impounded floodwaters into the CAP canal. The procedure works in conjunction with the blowoff structure at the Salt River Siphon of the CAP canal.

The Bureau of Reclamation would have priority over a District plan such as developed in structural alternative 2. The only method of discharge from the Paradise Valley Detention Dikes is through four 750-cfs outlets. The Detention Dikes do not have any other measure to outlet impounded water such as emergency spillways.

The implementation of alternative 1 and alternative 2 would be highly dependent on the local storm event and distribution of that event over the Cave Buttes Dam watershed and the Paradise Valley Detention Dikes watersheds. The alternatives may be feasible if the storm event was localized on the Cave Buttes Dam watershed and not the Detention Basin's watershed. The District may be required to provide ALERT stations on the watershed for the Paradise Valley Detention Dikes in order to ascertain and predict the level of flooding on the watersheds.

Structural alternatives 1 and 3 increase the operational flexibility in the management of the Cave Buttes Dam pool. The reduction of floodwater levels in the reservoir of the dam would not be solely dependent on the principal spillway alone. It needs to be noted, that structural alternative 2, may experience limited operational service as Dike No. 2 was constructed to contain the Standard Project Flood and the Probable Maximum Flood. Storm events less than these floods may not require that the low-level outlet in Dike No.

2 ever be operated. The same reasoning is especially true for the low-level outlet for structural alternative 3. The only time this outlet would potentially be operational is in the event of the PMF. The SPF ponding limits do not even reach Dike No. 3. It would not be prudent to spend money on an alternative that has a very low probability of utilization.

Environmental - Cave Buttes Dam structural alternative 1 includes a concrete-lined floodway to be constructed within a natural drainage wash. The channel would terminate at the Paradise Valley Detention Dike, segments of which have been designated as a City of Phoenix park and wildlife habitat area. This drainage wash is depicted on the USGS quadrangle maps and appears to have existed prior to the construction of Dike No. 2 (from review of as-builts). The channel would replace a major portion of the upstream segment of the wash. The Corps of Engineers may rule that the wash in the limits of the proposed channel is jurisdictional and therefor require an individual Section 404 permit. As part of the permit application, the Corps would require a least environmentally damaging alternatives analysis. An endangered species determination may be required as part of the Section 404 process. The Bureau of Reclamation may also request a NEPA review and clearance of alternative 1.

Structural alternative 2 may require at a minimum an endangered species determination (Pygmy Owl, in particular). The alignment of the floodway downstream of the emergency spillway courses through an area that has been previously disturbed by sand and gravel mining operations. It is questionable that a Section 404 permit would be required since there appears to be no potentially jurisdictional limits or Waters of the U.S. along the floodway alignment.

Cave Buttes structural alternative 3 would require very minimal environmental considerations. Mitigation because of construction of alternative 3 would may amount to replacement in-kind or on a slightly higher ratio.

Multi-Use and Aesthetics - There is very minimal opportunities for all three Cave Buttes Dam structural alternatives for multi-use purposes. Multi-use opportunities may actually conflict and not desirable for the case of alternatives 1 and 2. Both of these alternatives incorporate dedicated floodways to discharge floodwaters. At the least, trails may be developed along the banklines of the floodways. However, a physical separation (fence) would be required between the floodway and the trails.

Aesthetics could be incorporated as part of the floodways. Landscaping could be installed to screen the floodways particularly in the case of alternative 1, which is somewhat more visual to the public and located in the vicinity of residential developments. Alternatives to concrete lining of the floodways could be explored in further analyses.

Risk and Liability - Structural alternative 3 does little to reduce risk and liability in the current concept. The low-level outlet may never operate since it would take a flood event of the magnitude of the PMF for discharge of impounded floodwaters. Additional hydrologic analysis of alternative 3 regarding routing of impounded water through the low-level outlet would provide a better indication of the contribution and effectiveness of the outlet in flood pool reduction.

Alternative 1 could reduce risk and liability for flooding events (dambreak, emergency spillway flows) from Cave Buttes Dam. The alternative allows the diversion of floodwaters to another dam/reservoir owned by the Bureau of Reclamation. However, the risk and liability issue would require further exploration since basically the alternative would be transferring risk to another structure (not owned by the District). Agreements would have to be promulgated between the District and the Bureau of Reclamation regarding conveying floodwaters from Cave Buttes Dam to the Paradise Valley Detention Dikes and reservoir.

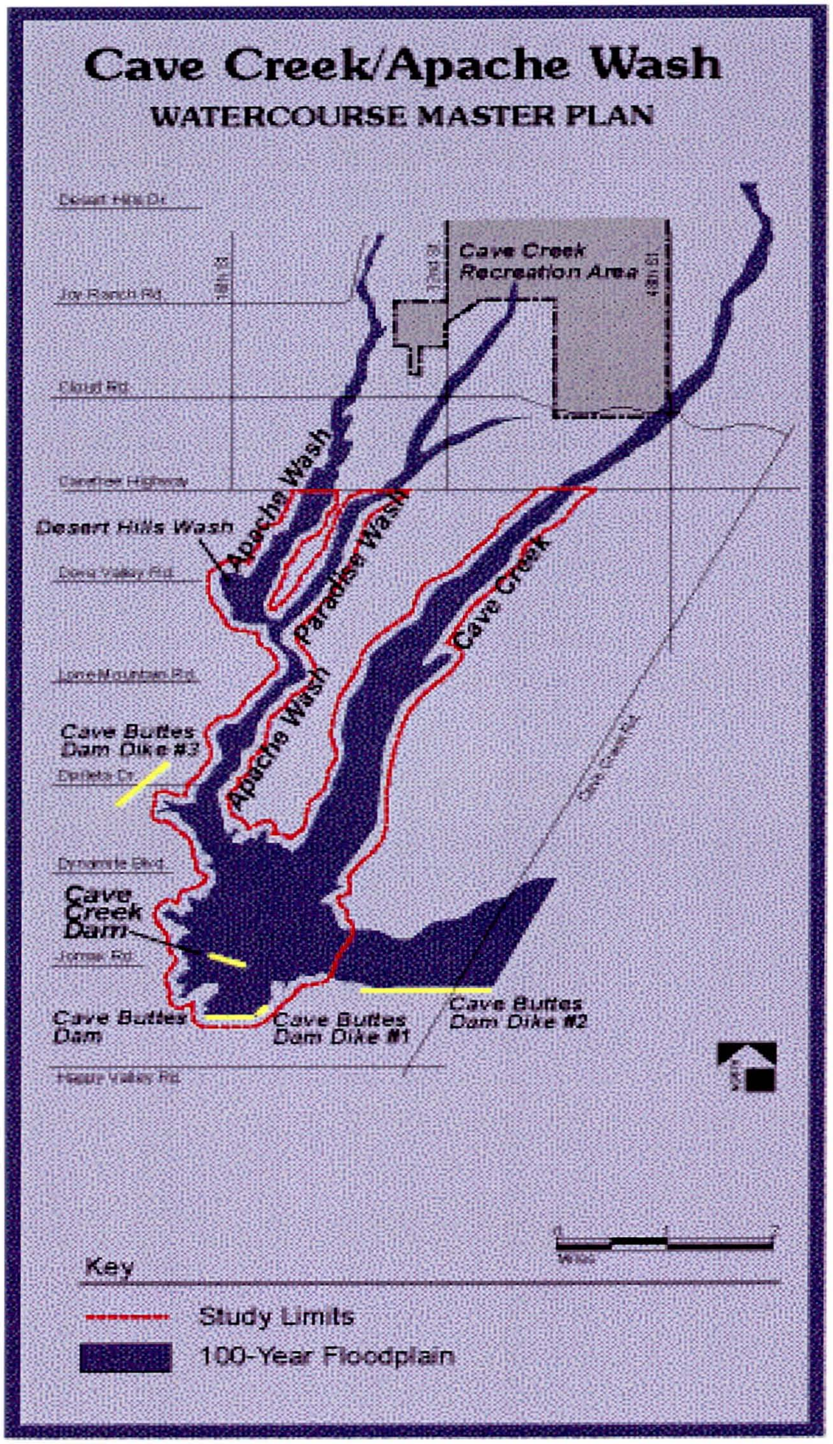
Alternative 2 would reduce risk and liability for the PMF event for Cave Buttes Dam. The emergency spillway would only spill in the event that the flood pool was full and then on top the PMF event would need to occur. The probability of these events occurring simultaneously is very minimal. The advantage of alternative 2 is that it could allow additional lead time in the notification of evacuation and implementation of the emergency action plan. Alternative 2 would allow floodwaters discharged from the emergency spillway (up to 3,000-cfs) to outlet to the CAP canal instead of potentially causing ponding and shallow flooding along the north embankment of the canal (which is the current condition).

Compatibility with District Plans - Upper Cave Creek Watercourse Master Plan. The Flood Control District of Maricopa County and the City of Phoenix are developing a watercourse master plan for Upper Cave Creek and Apache Wash. Based on engineering, environmental and land use considerations, this plan will take a comprehensive approach to flood control and floodplain management.

The study area extends from the Carefree Highway south approximately six miles to the Cave Buttes Dam. This area includes Apache Wash from the Carefree Highway south to where it meets Cave Creek. Additionally, the lower portion of Paradise Wash from the Carefree Highway south to where it joins Apache Wash, and Desert Hills Wash between the Phoenix city limits and Apache Wash is part of the study.

The primary goals of the watercourse master plan are to:
- Protect existing and future residents from the 100-year flood event and possible damages associated with potential lateral migration of the watercourse.
- Consider both structural and non-structural alternatives, with an emphasis on non-structural solutions.
- Minimize future expenditures of public funds for flood control and emergency management.
- Establish a significant open space corridor that meets conscientious and cost effective floodplain management objectives in conjunction with:
  - Preservation of sensitive habitats and cultural resources where possible
  - Maintenance of existing recreational uses; and

   - Allowance for future recreational uses.
- Develop a visionary floodplain management plan that generates widespread support and that can be implemented.



City of Phoenix - Cave Creek Water Reclamation Plant and Discharge Pipeline. The City of Phoenix is constructing the Cave Creek Water Reclamation Plant and discharge pipeline with a maximum capacity of eight million gallons per day (24.5 acre-feet per day). Phoenix will expand the capacity of the plant as development expands within the plant's service area. The discharge pipe delivers treated effluent to be used for turf irrigation and other non-potable uses. During the winter, the demand for treated effluent to users for turf irrigation will be reduced and the plant output remains relatively constant. Therefore, Phoenix will discharge the surplus treated effluent into a tributary of Cave Creek Wash which conveys the treated effluent into the storage pool above Old Cave Creek Dam for disposal by recharge and evaporation. The discharge point is located approximately 1200-feet northeast and upstream of the Old Cave Creek Dam. The discharge point is located outside the PMF ponding limits for Cave Buttes Dam.

City of Phoenix - Cave Buttes Dam Recreation Area. The City of Phoenix and the Flood Control District have entered into an intergovernmental agreement that would allow the City to develop commercial recreational facilities in the Cave Buttes Dam reservoir area for general public use. Discussions with the City of Phoenix Parks and Recreation staff have indicated that planning efforts for the recreation area have just been initiated at the very broadest level. No recreation site plans or programs are available at the time of this alternatives analysis from the City of Phoenix.

All three Cave Buttes Dam structural flood control alternatives would not be adversely impacted by the proposed watercourse master plan, reclamation plant, or the recreational area. As a result of the watercourse master plan the District will realized a reduction in risk and liability from ownership of Cave Buttes Dam. The watercourse master plan will identify and provide flood control protection benefits along Cave Creek Wash to the pool area of Cave Buttes Dam. The master plan will develop erosion set-back limits and establish open spaces to preserve natural wash areas. The plan will allow upstream development to occur in an orderly manner (according to the Master Plan) without encroaching or impacting the upstream floodplain and floodway. The Upper Cave Creek Wash Watercourse Master Plan and the Structures Assessment Program have compatible program elements and goals.

Flood Control - The Cave Buttes Dam was designed to retain the standard project flood. The inflow design flood for the emergency spillway is the probable maximum flood. None of the proposed structural alternatives were conceptualized to either increase or decrease the level of protection provided by Cave Buttes Dam. The low-level outlet alternatives (No. 1 and 3) will provide the District with increased flood-pool operational flexibility. Alternative No. 1 will provide more flexibility than Alternative No. 3. This is because Alternative No. 1 is located in Dike No. 2 and Alternative No. 3 is located in Dike No. 3. Dike No. 3 was constructed to contain the PMF while Dike No. 2 will contain the 100-year and SPF (i.e., Alternative No. 1 could be utilized on a somewhat more frequent basis compared to Alternative No. 3). In fact, the only time when Alternative No. 3 could be utilized is during storm events greater than the SPF.

The floodway alternative (Alternative No. 2) may provide a measure of additional flood control for the area downstream of the emergency spillway to the CAP canal (although this is not the primary purpose of the alternative). This alternative, if constructed, would provide up to a 3,000-cfs floodway from the end of the emergency spillway to the CAP canal. The floodway would contain floodflows instead of spreading out by overland flow. The intent of the alternative is to directly discharge emergency spillway flows (to a maximum of 3,000-cfs) into the CAP canal. The alternative conceivably allows spillway flows to enter the CAP canal to reduce the likelihood of overtopping of the canal or delay the inevitable by some small time increment. The delay may provide some additional time for the evacuation of downstream inhabitants and structures.

The assignment of point values for each structural alternative based on the evaluation of the criteria presented above is presented in Table 5-4. The range of point values for each evaluation criteria was provided previously in Table 5-1.

Table 5-4. Cave Buttes Dam Structural Alternatives Point Values.

Evaluation Criteria	Structural Alternative		
	Low-Level Outlet Dike No. 2	Floodway	Low-Level Outlet Dike No. 3
Jurisdictional	2	4	4
Cost	4	4	6
Implementation	4	2	6
Environmental	6	6	6
Multi-Use and Aesthetics	3	3	1
Risk And Liability	10	8	4
Compatibility With District Plans	4	4	4
Flood Control	4	2	1
Total Points	37	33	32

B. Nonstructural Alternatives

The flood control nonstructural alternatives for Cave Buttes Dam are evaluated qualitatively based on the evaluation criteria. A discussion of the nonstructural alternatives are presented in the following paragraphs. At the end of the discussion point values are assigned to each nonstructural alternative for each evaluation criteria in tabular format.

1. Downstream of Cave Buttes Dam

Jurisdictional - Cave Buttes Dam is an existing structure that falls under the jurisdiction of ADWR, Office of Dam Safety. The current ADWR rules and regulations as applied to existing flood control dams are stipulated in the rules. The rules specifically state that a an owner of an jurisdictional high or significant hazard dam shall have prepared an emergency action plan (EAP) for the dam. The ADWR rules provide minimum requirements for the contents of the plan. An individual EAP for Cave Buttes Dam needs to be prepared as recommended in the "Program and Policy Report" (Kimley-Horn and Associates, Inc., April 2000). ADWR, Office of Dam Safety, does not have rules and regulation constraints for mitigation through flood insurance or the purchase of properties or flowage easements downstream of the dam.

Cost - The approximate costs for mitigation through flood insurance (over 30-years), acquisition of properties, and development of an emergency action plan are $8.9 million, $9.0 million, and $30 thousand, respectively.

Implementation – Updating the EAP for Cave Buttes Dam is the most easily developable and implementable of the three nonstructural downstream alternatives. The EAP can be developed by in-house District staff or out-sourced to an engineering consultant. Several guidelines are available from federal and state dam safety agencies for the development of EAPs. The purchase of flood insurance or the purchase of lands/easements are, however, not as easily implementable as preparing the EAP.

Environmental – Direct environmental constraints cannot be identified for any of the three nonstructural flood control downstream alternatives. A minor indirect benefit may include limiting development within the flood prone area for the full PMF spillway discharge downstream of the dam. If the District purchased or leased lands outside what is already owned/leased by the District within the PMF inundation limits, the District would indirectly be preserving low value desert habitat within these limits.

Multi-Use and Aesthetics – No multi-use and aesthetic opportunities or constraints were identified for the three downstream nonstructural alternatives. All three alternatives will receive a zero value for this evaluation criteria.

Risk and Liability – All three nonstructural downstream flood control alternatives will reduce the District risk and liability with ownership of Cave Buttes Dam. The reduction of risk (or benefit) however is associated with two very extreme probability events. The reduction is realized from either a discharge from the emergency spillway (which occurs for storm events greater than the SPF) or from an unexpected dam failure or dam break associated with reservoir ponding. The purchase of flood insurance downstream of the dam was based on the full PMF inundation limits. The purchase or lease of lands downstream of the emergency spillway was also based on the PMF inundation limits. The emergency action plan, however, is not frequency based, but would be developed based on reservoir stage or rate of rise of ponding in the reservoir. The EAP would realize a risk reduction almost immediately once prepared by the District and approved by ADWR.

Compatibility with District Plans - None of the three downstream flood control alternatives are in conflict with or impacted by the Upper Cave Creek Watercourse Master Plan.

Flood Control - None of the three downstream flood control alternatives were conceptualized to increase or decrease the existing flood control protection provided by Cave Buttes Dam. The alternatives are not reducing the regulatory floodplain limits between the dam and the CAP canal.

Table 5-5. Cave Buttes Dam Nonstructural Alternatives Point Values (Downstream).

Evaluation Criteria	Nonstructural Alternative (Downstream)		
	Mitigate Through Flood Insurance	Acquire Properties/ Easements	Update Emergency Action Plan
Jurisdictional	1	1	8
Cost	4	4	10
Implementation	3	3	8
Environmental	2	6	2
Multi-Use and Aesthetics	1	2	1
Risk And Liability	4	10	9
Compatibility With District Plans	4	4	4
Flood Control	1	4	4
Total Points	20	34	44

2. Pool Area of Cave Buttes Dam

Jurisdictional - Cave Buttes Dam is an existing structure that falls under the jurisdiction of ADWR, Office of Dam Safety. The current ADWR rules and regulations as applied to existing flood control dams are stipulated in the rules. The rules specifically state that a an owner of an jurisdictional high or significant hazard dam shall have prepared an emergency action plan (EAP) for the dam. The ADWR rules provide minimum requirements for the contents of the plan. An individual EAP for Cave Buttes Dam needs to be prepared as recommended in the "Program and Policy Report" (Kimley-Horn and Associates, Inc., April 2000). ADWR, Office of Dam Safety, does not have jurisdictional constraints for mitigation through flood insurance or the purchase of properties or flowage easements downstream of the dam.

Cost - The approximate costs for mitigation through flood insurance (over 30-years), acquisition of properties, and development of an emergency action plan are $14.4 million, $32.0 million, and $30 thousand, respectively.

Implementation – Preparing an EAP for Cave Buttes Dam is the most easily developable and implementable of the three nonstructural pool area alternatives. The EAP can be developed by in-house District staff or out-sourced to an engineering consultant. Several guidelines are available from federal and state dam safety agencies for the development of EAPs. The purchase of flood insurance or the purchase of lands/easements are, however, not as easily implementable as preparing the EAP.

Environmental - Direct environmental constraints cannot be identified for any of the three nonstructural flood control pool area alternatives. A minor indirect benefit may include limiting development within the flood prone area for the full PMF inundation ponding limits. If the District purchased or leased lands outside what is already owned/leased by the District within the PMF inundation limits, the District would indirectly be preserving low value desert habitat within these limits.

Multi-Use and Aesthetics - No multi-use and aesthetic opportunities or constraints were identified for the three pool area nonstructural alternatives. All three alternatives will receive a zero value for this evaluation criteria.

Risk and Liability - All three nonstructural pool area flood control alternatives will reduce the District risk and liability with ownership of Cave Buttes Dam. The reduction of risk (or benefit) however is associated with one very extreme probability event. The reduction is realized from flood events between the SPF and full PMF (inclusive). The purchase of flood insurance in the pool area of the dam was based on the full PMF ponding limits. The purchase or lease of lands upstream of the dam was also based on the PMF ponding limits. The emergency action plan, however, is not frequency based, but would be developed based on reservoir stage and/or rate of rise of ponding in the reservoir. The EAP would realize a risk reduction almost immediately once prepared by the District and approved by ADWR.

Compatibility with District Plans - All three nonstructural pool area flood control alternatives are compatible with the Upper Cave Creek Wash Watercourse Master Plan. This is due to the fact of the common objectives between the Master Plan and the Structures Assessment Program.

Flood Control - None of the three pool area flood control alternatives were conceptualized to increase or decrease the existing flood control protection provided by Cave Buttes Dam. The alternatives are not reducing or increasing the regulatory pool limits upstream of the dam nor are the alternatives changing the existing FEMA flood zone designations.

Table 5-6. Cave Buttes Dam Nonstructural Alternatives Point Values (Pool Area).

Evaluation Criteria	Nonstructural Alternative (Pool Area)		
	Mitigate Through Flood Insurance	Acquire Properties/ Easements	Develop Emergency Action Plan
Jurisdictional	1	1	8
Cost	4	2	10
Implementation	3	3	8
Environmental	1	1	1
Multi-Use and Aesthetics	1	1	1
Risk And Liability	4	10	9
Compatibility With District Plans	6	6	6
Flood Control	2	6	4
Total Points	22	30	47

5.5 Evaluation of Powerline, Vineyard Road, and Rittenhouse FRS Alternatives

Two sets of alternatives were developed in Section 4.0 for evaluation of Powerline, Vineyard Road, and Rittenhouse Flood Retarding Structures. These sets of alternatives are grouped into structural and nonstructural flood control alternatives or measures. The alternatives were developed to reduce the risk and liability to the District of ownership, and operations and maintenance of the dams. A summary of the two sets of alternatives is provided in Table 5-7 on the next page.

A. Structural Alternatives

The flood control structural alternatives for Powerline, Vineyard Road, and Rittenhouse FRS are evaluated qualitatively based on the evaluation criteria. A discussion of the structural alternatives are presented in the following paragraphs. At the end of the discussion point values are assigned to each alternative for each evaluation criteria in tabular format.

Jurisdictional - Powerline, Vineyard Road, and Rittenhouse FRS are existing structures that are under the jurisdiction of ADWR, Office of Dam Safety. The current ADWR rules and regulations as applied to existing flood control dams are stipulated in the rules. Modifications, alterations, or upgrades to the structures, which includes structural alternatives 1, 2, 3, 4, 5, and 6, would require review and approval by ADWR.

Table 5-7. Summary of Powerline, Vineyard Road, and Rittenhouse FRS Alternatives.

Structural Alternatives		Nonstructural Alternatives (Pool Area Only)	
No.	**Description**	**No.**	**Description**
1	Segmentation. Segment Structures into smaller "dams"	1	Mitigate through Flood Insurance
2	Upgrade to High Hazard Dams.	2	Acquire Properties/Flowage Easements
3	Modify Dam to improve Performance (add sills; Erosion control)	3	Develop Emergency Action Plan
4	Basins. Replace dams with Basins.		
5	Levee/Floodway System. Replace dams with levees and floodways.		
6	Discharge into the Central Arizona Project canal. Provide low-level outlets for each dam to CAP canal.		
7	Increase Capacity of Powerline Floodway		

Modifications and upgrade includes alternatives 1, 2, and 3. Dam replacements include alternatives 4 and 5. ADWR would be involved in these two alternatives as these include dam decommissioning and replacement with either detention basins or a levee/floodway system. Structural alternative 6 modifies each dam by the construction of the culverts and associated inlet and outlet structures. ADWR dam safety rules would require that the design of the low-level outlets consider and incorporate piping countermeasures for the culvert penetrations. In addition, potential methods of construction of the culvert penetrations would have to be provided to the satisfaction of ADWR. Given that the low-level outlets may not be used in every general storm event to discharge floodwaters ADWR would still require that a new discharge rating curve for the dam be developed for review and approval.

Alternative 7 may not require approval of ADWR. However, it would be prudent, if the alternative is considered for further investigation, that ADWR be informed of the investigation and their input requested through alternative development. This alternative consists of a new concrete-lined Powerline Floodway. The alternative does not modify the emergency spillways or the spillway discharge rating curves. Essentially, the alternative does not modify, upgrade, or rehabilitate the existing dam, dikes, or spillways.

The alternative provides the District a positive means of conveying principal spillway discharges (up to 4,000-cfs) to the East Maricopa Floodway.

Alternatives 1, 2, 3, 4, 5, and 6 would require ADWR review and approval. ADWR may require that, Alternative 1 - Segmentation, individual principal spillways be incorporated for each pool segment or cell. If it could be demonstrated that the segmentation alternative would not impact the normal operation of the dam and reservoir, a separate principal spillway for the isolated pool may not be necessary. Alternative 3 would require less involvement with ADWR than the other 5 alternatives with ADWR input.

Alternative 5 - Levee/Floodway System would require coordination and review approval from the Central Arizona Water Conservation District and the Bureau of Reclamation. Modifications to increase the capacity of the Sonoqui Detention dike reservoir and modify the structure would have to be approved by these two agencies. The District should be aware that under this alternative, the possibility exits that the CAWCD and the Bureau may request that the District take over ownership, and operations and maintenance of the Sonoqui Detention Dike and reservoir, since the District would be substantially altering the original structure and reservoir characteristics. This action would be contrary to the goals of the alternatives analysis which is to reduce the risk and liability of dam ownership. The District, under the Structures Assessment Program, would not likely take on the ownership of another dam. If the District were to consider ownership, the Bureau would retain jurisdiction over the Sonoqui Detention dike and the dike would also fall under the jurisdiction of ADWR.

If the modifications to Sonoqui result in increased peak discharges from the structure into Queen Creek wash, the Town of Queen Creek may request a design review since the Town is currently in the process of preparing channel improvement plans to Queen Creek as it courses through the Town.

Costs - Table 5-8 (next page) presents a summary of the estimated structural flood control alternative cost estimates. These estimates are considered as planning level costs developed only for the purposes of comparison between alternatives. Appendices E through K provide back-up data for the estimation of costs.

Implementation - The greatest impediment to the feasibility of structural alternative 6 is implementation. This alternative would require review and approval of both the Bureau of Reclamation and the Central Arizona Project (or the Central Arizona Water Conservation District - CAWCD).

The Central Arizona Project operates and maintains the CAP canal. The primary purpose of the canal is to convey Colorado River water to Central Arizona. The canal was not designed as a flood control facility. Intentional entry of spillway discharges into the canal is not a contingency that the CAP has accommodated.

Table 5-8. Opinion of Probable Costs for Powerline, Vineyard Road, and Rittenhouse FRS Structural Flood Control Alternatives.

Structural Alternatives		Cost ($)
1	Segmentation. Segment Structures into smaller "dams"	$ 4.04m
2	Upgrade to High Hazard Dams.	$ 11.5m
3	Modify Dam to improve Performance (add sills; Erosion control)	$ 660k
4	Basins. Replace dams with Basins.	$ 127m
5	Levee/Floodway System. Replace dams with levees and floodways.	$ 88.3m
6	Discharge into the Central Arizona Project canal. Provide low-level outlets for each dam to CAP canal.	$ 670k
7	Increase Capacity of Powerline Floodway	$ 13.2m

There is one case, however, where this is the exception. The Bureau of Reclamation owns the Paradise Valley Detention Dikes. The only method of evacuating the reservoir pools from behind the dikes is direct discharge into the CAP canal. The Bureau and the CAP have developed a reservoir operations plan that outline the procedures to discharge impounded floodwaters into the CAP canal. The procedure works in conjunction with the CAP/Salt River interconnect at the Salt River crossing of the CAP canal. This process was briefly explained in Section 3.0.

The Bureau of Reclamation would have priority over a District plan such as developed in structural alternative 6. The only method of discharge from the Paradise Valley Detention Dikes is through four 750-cfs outlets. The Detention Dikes do not have any other measure to outlet impounded water such as emergency spillways.

Structural alternative 6 increases the operational flexibility in the management of the reservoir pools. The reduction of floodwater levels in the reservoir of the dam would not be solely dependent on the principal spillway alone.

The implementation of alternative 1 could possibly be impacted by available right-of-way and re-engineering the reservoir pools. However, these are somewhat minor constraints.

Alternative 2 would be constructed over the existing structures. Very little to no additional right-of-way would be required. Little downstream development currently exists that would have objections to the aesthetic change in the size of the embankments.

Alternative 3 is easily implementable and it is recommended that these measures be completed regardless of the outcome of the ranking of the structural alternatives. Alternative 4 would be difficult to implement given the required land for construction of the basins. Review of the basin plans would require approval of State Lands, Pinal County, and private landholders within and adjacent to the detention basin pool areas.

Alternative 5 would also be difficult to implement based on the current concept. The Central Arizona Water Conservation District and the Bureau of Reclamation would require design review for modifications to the Sonoqui Detention structure. Operation and maintenance agreements may be required between the District, the Bureau, and the CAWCD. ADWR may require review of any modifications or alterations to the Sonoqui structure.

Alternative 7 would be difficult to implement. There are numerous existing roadway crossings of the Powerline Floodway that would have to be modified to accommodate the upsized floodway. Cooperative agreements and funding from the County and municipal roadway departments would have to be promulgated.

Environmental - Structural alternatives 1, 4, and 5 would have the greatest environmental constraints under the current concepts. All three alternatives would require relatively significant earth disturbing activities. Environmental permitting and review would be required for threatened and endangered species. Alternative 4 would have the most environmental impact followed by alternative 5.

The Corps of Engineers may rule that the reservoir pool areas are jurisdictional and therefor require an individual Section 404 permit. As part of the permit application, the Corps would require a least environmentally damaging alternatives analysis. An endangered species determination may be required as part of the Section 404 process.

Structural alternatives 3 and 7 would require very minimal environmental considerations. Mitigation because of construction of alternative 3 could amount to replacement in-kind or on a slightly higher ratio. Upsizing the Powerline Floodway would have minimal environmental impacts since the alternative is within the same right-of-way as the existing floodway and the new floodway would replace an existing facility.

Multi-Use and Aesthetics - There are multi-use opportunities for structural alternatives 4 and 5. Multi-use opportunities within large detention basins include recreation (parks, ballfields, trails, and equestrian facilities) as well as potential wildlife habitat enhancement, and groundwater recharge. The groundwater table under the Powerline, Vineyard Road, and Rittenhouse FRS has been declining over the last 20- 30 years. The effects of the declining groundwater table has shown manifested itself in local land subsidence and earth fissures. Recharge opportunities with CAP water could be explored with CAP users in conjunction with the detention basin alternative. Alternative 5 (Levee/Floodway) could include a long hiking/jogging/biking and equestrian trails.

Aesthetics could be incorporated as part of the floodways (alternative 5). Landscaping could be installed to screen the floodways particularly in the case of alternative 7, which is somewhat more visual to the public and located in the vicinity of residential developments. Alternatives to concrete lining of the Powerline Floodway could be explored in further analyses.

Minimal multi-use opportunities are identified for the other structural alternatives.

Risk and Liability - All of the structural alternatives in their current concepts, except for alternative 7 and possibly alternative 5, will reduce risk and liability. Alternative 1 (Segmentation) will divide the pool areas for each dam into two smaller pools and reduce the potential volume of water released during a dambreak event. Alternative 2 will upgrade the dams to high hazard dams and safely pass the full PMF through the emergency spillways. Alternative 4 and 5 will completely remove the dams and replace the dams with detention basins or levees in combination with a floodway. However, alternative 5 (levee system) will tie into an existing structure in which the District may be required to take over ownership as well as operations and maintenance. The District would reduce the risk and liability associated with Powerline, Vineyard Road, and Rittenhouse but may pick up liability with ownership of Sonoqui Detention structure. Alternative 6 will provide for a greater degree of operational flexibility by having low-level outlets to the CAP canal. Under this alternative, the District would have the ability to reduce the flood pool relatively quicker than under present operational conditions. This alternative could allow more time for flood emergency response by the District for evacuation of downstream structures and inundation areas.

Alternative 2 would reduce risk and liability for the PMF event for all three dams. The emergency spillways would be widened to accommodate the PMF and obtain the required freeboard. Presently, all three dams would be overtopped by the PMF event.

Compatibility with District Plans - Section 4.4.A presented previous and current on-going District plans and studies in the region around Powerline, Vineyard Road, and Rittenhouse FRS. The most significant of these are the mitigation studies for the EMF. The EMF is presently experiencing severe capacity problems with current hydrologic conditions. The mitigation studies are attempting to identify flood control solutions to alleviate the capacity problems with the EMF and Queen Creek Wash.

The structural alternatives examined as part of this report for the three dams either maintains the existing hydrologic contribution from the dams and or increases the problems for the EMF. Alternative 6 could perhaps reduce the contribution of the dams to the EMF or Queen Creek. This reduction could be realized if the stormwater discharged to the CAP canal were allowed to continue in the canal and be utilized by downstream CAP users and not be discharged directly into Queen Creek. Alternative 5 would be designed so as not to increase the direct contribution of the upstream watershed to Queen Creek wash. This requires that the Sonoqui Detention structure be upsized to handle the flows from the Powerline, Vineyard Road, and Rittenhouse levee systems. Alternatives 1, 2, and 4 would not change the hydrologic contribution from the dams to the downstream watershed under normal conditions.

Flood Control - All three structures were designed to detain the 100-year flood from their respective watersheds. The inflow design flood for the emergency spillway is the half-probable maximum flood using current ADWR criteria. None of the proposed structural alternatives were conceptualized to decrease the design level of protection provided by the dams.

Alternative 1 (Segmentation) will provide the District a greater degree of risk management of the reservoir pools. One pool segment could be isolated from the other pool segment in the event of evident failure of an embankment. Alternative 2 allows the full PMF to be safety passed through the emergency spillways. This alternative provides a greater degree of flood control over existing conditions and over alternative 1.

Table 5-9. Powerline, Vineyard Road, and Rittenhouse FRS Structural Alternatives Point Values.

Evaluation Criteria	Structural Alternative No.						
	1	2	3	4	5	6	7
Jurisdictional	1	1	4	4	3	2	6
Cost	6	5	8	1	2	5	4
Implementation	5	5	8	4	4	2	4
Environmental	4	6	4	6	4	4	4
Multi-Use and Aesthetics	3	5	2	5	4	2	1
Risk And Liability	7	12	7	12	6	8	8
Compatibility With District Plans	6	6	6	8	4	4	4
Flood Control	4	6	4	6	6	7	4
Total Points	36	48	43	46	33	34	35

Alternative 6 (low-level outlets) will provide the District with increased flood-pool operational flexibility. The flood pool could be excavated more rapidly over existing conditions and to a greater degree than that of the other structural modifications (alternatives 1, 2, and 3).

The assignment of point values for each structural alternative based on the evaluation of the criteria presented above is presented in Table 5-9 above. The range of point values for each evaluation criteria was provided previously in Table 5-1.

B. Nonstructural Alternatives (Pool Area)

The flood control nonstructural alternatives for Powerline, Vineyard Road, and Rittenhouse FRS are evaluated qualitatively based on the evaluation criteria. A discussion of the nonstructural alternatives is presented in the following paragraphs. At the end of the discussion point values are assigned to each nonstructural alternative for each evaluation criteria in tabular format.

Jurisdictional - Powerline, Vineyard Road, and Rittenhouse FRS are existing structures that fall under the jurisdiction of ADWR, Office of Dam Safety. The current ADWR rules and regulations as applied to existing flood control dams are stipulated in the rules. The rules specifically state that a an owner of an jurisdictional high or significant hazard dam shall have prepared an emergency action plan (EAP) for the dam. The ADWR rules provide minimum requirements for the contents of the plan. An individual EAP for each dam needs to be prepared as recommended in the "Program and Policy Report" (Kimley-Horn and Associates, Inc., April 2000). ADWR, Office of Dam Safety, does not have jurisdictional constraints for mitigation through flood insurance or the purchase of properties or flowage easements downstream of the dam. The purchase of lands, as opposed to leasing, would require the approval from the State Lands Department.

Cost - The approximate costs for mitigation through flood insurance (over 30-years), acquisition (purchase) of land, and development of an emergency action plan are $135 thousand, $100 million, and $90 thousand, respectively. Purchase of the flood pool areas is a very unlikely District action. The District already leases the flood pool areas and as a matter of fact leases sufficient lands to include the approximate ponding limits of the full PMF (see Exhibit D - located in the map pockets in the back of this report).

Implementation – Preparing an EAP for the three structures is the most easily developable and implementable of the three nonstructural pool area alternatives. The EAP can be developed by in-house District staff or out-sourced to an engineering consultant. Several guidelines are available from federal and state dam safety agencies for the development of EAPs. The purchase of flood insurance or the purchase of lands/easements are, however, not as easily implementable as preparing the EAP.

Environmental - Direct environmental constraints cannot be identified for any of the three nonstructural flood control pool area alternatives. A minor indirect benefit may include limiting development within the flood prone areas for the 100-year inundation ponding limits.

Multi-Use and Aesthetics - No multi-use and aesthetic opportunities or constraints were identified for the three pool area nonstructural alternatives. All three alternatives will receive a minimal value for this evaluation criteria.

Risk and Liability - All three nonstructural pool area flood control alternatives will reduce the District risk and liability with ownership of the three dams. This is primarily based on the fact that the District already leases a substantial amount of land around each of the structures (see Exhibit D - located in the map pockets in the back of this report). Because the District leases so much land behind the existing structures, the District is in the favorable position of directing future land uses in these areas. The emergency action plan is not frequency based, but would be developed based on reservoir stage and/or rate of rise of ponding in the reservoir. The EAP would realize a risk reduction almost immediately once prepared by the District and approved by ADWR.

Compatibility with District Plans - All three nonstructural pool area flood control alternatives are compatible with the previous and ongoing District planning studies in the region. This is due to the fact of the common objectives between the plans to alleviate flooding problems and the Structures Assessment Program - both which are mitigating risk and liability.

Table 5-10. Powerline, Vineyard Road, and Rittenhouse FRS Nonstructural Alternatives Point Values (Pool Area).

Evaluation Criteria	Nonstructural Alternative (Pool Area)		
	Mitigate Through Flood Insurance	Acquire Properties/ Easements	Develop Emergency Action Plan
Jurisdictional	7	4	4
Cost	8	1	9
Implementation	3	2	8
Environmental	6	6	8
Multi-Use and Aesthetics	2	2	1
Risk And Liability	8	12	9
Compatibility With District Plans	6	5	6
Flood Control	3	3	4
Total Points	43	35	49

Flood Control - None of the three pool area nonstructural flood control alternatives were conceptualized to increase or decrease the existing flood control protection provided by the three dams. The alternatives are not reducing or increasing the pool limits upstream of the dam nor are the alternatives changing the existing FEMA flood zone designations. The assignment of point values for each nonstructural alternative based on the evaluation of the criteria presented above is presented in Table 5-10 above.

5.6 Summary of Ranking and Preferred Alternatives

A. Cave Buttes Dam Alternatives

The evaluation and ranking of Cave Buttes Dam structural and nonstructural alternatives was presented and discussed in Section 5.4 above. The preferred structural alternative based on cumulative total points is the low-level outlet in Dike No. 2. The preferred nonstructural below dam alternative is the preparation of an emergency action plan. The preferred nonstructural pool area alternative is also the preparation of an emergency action plan.

All three of these alternatives ranked high on risk and liability compared to the other alternatives. The preparation of the emergency action plans was the lowest cost alternative among all alternatives considered. Table 5-11 (below) provides a summary of the Cave Buttes Dam structural and nonstructural alternatives.

B. Powerline, Vineyard Road, and Rittenhouse FRS Alternatives

The evaluation and ranking of Powerline, Vineyard Road, and Rittenhouse FRS structural and nonstructural alternatives was presented and discussed in Section 5.5 above. The preferred structural alternative is to upgrade the dams to high hazard dams. The preferred nonstructural alternative for the pool area is to develop an emergency action plan.

Although the preferred alternative for Powerline, Vineyard Road, and Rittenhouse FRS is to upgrade to a high hazard dam, in any case, structural alternative No. 3 - Modifications - should be implemented regardless of the structural alternative selected for rehabilitation, modifications, or upgrading the three dams. Table 5-12 and Table 5-13 (below) provides a summary of Powerline, Vineyard Road, and Rittenhouse FRS structural and nonstructural alternatives, respectively.

Table 5-11 Cave Buttes Dam Summary of Structural and Nonstructural Alternatives.

Structural Alternative Description	Elements Of Alternative	Estimate Of Alternative Cost	Alternative Ranking
Low Level Outlet In Dike No. 2	RCB 10-ft by 6-ft gated Capacity 750 cfs Trap Channel 12-ft btm Concrete lined @0.005 ft/ft 13,360 feet long	$ 2.1m	(1)
Divert to Central Arizona Project Canal from Emergency Spillway	Concrete Trap Channel (2 segments) Capacity 3,000 cfs 1. Upstream Btm width 24 ft 3000-ft Depth 6-ft 2. Downstream Btm width 24 ft 3300-ft Junction Structure with twin steel leaf gates 12-ft by 12-ft ea.	$ 1.9 m	(2)
Divert from Reservoir Pool through Low Level Outlet In Dike No. 3	Twin 8-ft x 4-ft RCB gated Capacity 100 cfs Trap earth-lined channel 500-ft long 10-ft bottom	$ 132 k	(3)
Non-Structural Alternative Description	**Elements of Alternative**	**Estimate of Alternative Cost**	
Below Dam Update Emergency Action Plan	Prepare Emergency Action Plan per FEMA 64 guidelines and requirements of ADWR	$20k - $30k	(1)
Below Dam Acquire Properties/ Downstream Flowage Easements	Acquire easements for PMF limits outside 100-year No. acres = 200	$9m	(2)
Below Dam Mitigate Through Flood Insurance	Coverage $100,000/dwelling unit No. of acres = 644	Annual Premium = $298k 30-year Premium = $8.9m	(3)
Pool Area Develop Emergency Action Plan	Prepare Emergency Action Plan per FEMA 64 guidelines and requirements of ADWR	$20k - $30k	(1)
Pool Area Acquire Properties/Flowage Easements	Acquire easements up to PMF ponding limits No. of acres = 720	$32m	(2)
Pool Area Mitigate Through Flood Insurance	Coverage $100,000/dwelling unit No. of acres = 1000	Annual Premium = $482k 30-year Premium = $14.5m	(3)

Table 5-12. Powerline (P), Vineyard Road (V), and Rittenhouse (R) FRS Summary of Structural Alternatives.

Structural Alternative Description		Elements Of Alternative	Estimate Of Alternative Cost	Alternative Ranking
Upgrade to High Hazard Dams.	P	Raise dam 4.5 ft and increase emergency spillway to 900-ft	$ 3.05 m	(1) Total cost $11.5 m
	V	Raise dam 4.9 ft and increase emergency spillway to 900-ft	$5.75 m	
	R	Raise dam 4.3 ft and increase each (2) emergency spillway to 450-ft	$ 2.69 m	
Basins. Replace dams with Basins.	P	5-ft deep; 8,000-ft long; 4,400-ft wide	$ 57.9 m	(2) Total cost $127.5 m
	V	5-ft deep; 26,00-ft long; 1,400-ft wide	$ 24.4 m	
	R	5-ft deep; 14,000-ft long; 2,400-ft wide	$ 45.2 m	
Modify Dam to improve Performance (add sills; Erosion control)	P V R	Concrete Control sill (4 to 4.5-ft deep) Abutment Slope Protection (D_{50} 1.0 to 1.6-ft) Trashrack Modification	$ 0.66 m	(3) Total cost $ 0.66 m
Segmentation. Segment Structures into smaller "dams" segments or cells	P	Segment = 6,000 ft with 6-ft dia equalization culvert and 6-ft by 6-ft floodgate	$ 2.5 m	(4) Total cost $ 4.04 m
	V	Segment = 2,000 ft with 6-ft dia equalization culvert and 6-ft by 6-ft floodgate	$ 0.54 m	
	R	Segment = 2,900 ft with 6-ft dia equalization culvert and 6-ft by 6-ft floodgate	$1.0 m	
Increase Capacity of Powerline Floodway		Channel Capacity 4,000 to 6,000-cfs; Concrete lined rectangular, 52-ft bottom width; depth = 5-ft, length = 9.1 miles	$ 13.2 m	(5) Total cost $13.2 m
Discharge into the Central Arizona Project canal. Provide low-level outlets for each dam to CAP canal.	P	Twin 7-ft dia RCP gated outlet; length = 210-ft. Discharge = 900 cfs	$ 174 k	(6) Total cost $0.67 m
	V	Twin 7-ft dia RCP gated outlet; length = 210-ft. Discharge = 900 cfs	$ 269 k	
	R	Twin 7-ft dia RCP gated outlet; length = 210-ft. Discharge = 900 cfs. Floodway channel = 310-ft	$ 225 k	
Levee/Floodway System. Replace dams with levees and floodways.		Modify the dams into a contiguous levee system with upstream floodway. Discharge to Sonoqui Detention dike.	$88.3 m	(7) Total Cost $88.3 m

Table 5-13. Powerline (P), Vineyard Road (V), and Rittenhouse (R) FRS Summary of Non-Structural Alternatives.

Non-Structural Alternative Description	Elements Of Alternative		Estimate Of Alternative Cost	Alternative Ranking
Develop Emergency Action Plan to FEMA 64 guidelines	P	EAP for both pool area and downstream area	$20k - $30k	(1) Total Cost $ 60k - 90 k
	V	EAP for both pool area and downstream area	$20k - $30k	
	R	EAP for both pool area and downstream area	$20k - $30k	
Mitigate through Flood Insurance	P	610 acres (uninhabitable structures only)	Annual Premium $1,500 30-year Premium $45,000	(2) Total cost $ 135 k
	V	637 acres (uninhabitable structures only)	Annual Premium $1,500 30-year Premium $45,000	
	R	660 acres (uninhabitable structures only)	Annual Premium $1,500 30-year Premium $45,000	
Acquire Properties/Flowage Easements	FCD already owns or leases sufficient lands. Option to purchase pool areas (total 2,000 acres)		$100 m	N/A

6.0 Closing

The purpose of the Alternatives Analysis was to evaluate structural and nonstructural flood control alternatives/measures or solutions: the objective, which is to reduce the District's risk and liability, associated with ownership of dams. The structural alternatives evaluated include repair of dams, modification of dams to improve performance, replacement of dams with some other form of structural flood control measure or, modification of the pool so as to eliminate the need for the dam embankment. Nonstructural alternatives include mitigation through flood insurance, acquisition of flowage easements/properties, development of emergency action plans, or some combination of two or more nonstructural solution elements.

The analysis evaluated potential flood control alternatives that were developed in conjunction with the District and the KHA project team. There are other potential structural alternatives for the dams, however, the structural alternatives that were evaluated were programmed as the alternatives that could provide the District with the greatest degree of management and/or reduction of risk and liability.

Three structural alternatives for Cave Buttes Dam were developed, evaluated, and rankings assigned based on point values from a set of eight evaluation criteria. The preferred alternative is to construct a low-level outlet in Dike No.2 which when operated would discharge ultimately to the Reach 11 detention dike east of Cave Creek Road. This structural alternative will provide the District operational flexibility in the management of the Cave Buttes Dam reservoir impoundment. In the event of a large storm event on the Cave Creek watershed that produces a high volume of runoff to Cave Buttes Dam, the District would be able to discharge impounded floodwaters from the Cave Buttes Dam impoundment and direct the discharges to the Reach 11 reservoir. This alternative works if volume is available in the Reach 11 reservoir, little to no inflow is coming into the Reach 11 dam, and agreements are reached between the District, Bureau, and the CAP. Structural modifcations to the Cave Buttes Dam or dikes would require a permit from the Corps of Engineers.

Six nonstructural alternatives for below and above Cave Buttes Dam were developed, evaluated, and rankings assigned based on point values from a set of eight evaluation criteria. The preferred nonstructural alternative for both downstream and upstream is to develop a site specific emergency action plan. This alternative could be combined with limited purchase of properties and/or easements within floodprone areas for the full PMF. In this manner, the District would regulate development within the inundation limits for the full PMF both upstream and downstream.

Seven structural alternatives for Powerline, Vineyard Road, and Rittenhouse FRS were developed, evaluated, and rankings assigned based on point values from a set of eight evaluation criteria. The seven alternatives were applied to the three dams as a set since the dams are operationally and functionally linked. The preferred structural alternative is to upgrade the three dams to high hazard dams capable of safely passing the full PMF.

The second preferred alternative, construction of detention basins, was not preferred due to the high cost of land acquisition and construction costs.

Three nonstructural alternatives for the pool areas of Powerline, Vineyard Road, and Rittenhouse FRS were developed, evaluated, and rankings assigned based on point values from a set of eight evaluation criteria. The preferred nonstructural alternative for the pool area is to develop a site specific emergency action plan. This alternative could be combined with limited purchase of properties and/or easements within floodprone areas for the full PMF (in the event that the upgrade to high hazard dam is promulgated). In this manner, the District would regulate development within the inundation limits for the full PMF around the impoundment area.

The preferred structural and nonstructural flood control alternatives evaluated and examined as part of this study should assist the District in the management of their risk and liability associated with the dams under consideration. The goal of the alternatives study was to identify a set of flood control measures, both structural and nonstructural, that could potentially reduce risk and liability associated with dam ownership. The preferred alternatives, based on the assignment of point values and ranking, should meet this important District goal.

7.0 References

1. Kimley-Horn and Associates, Inc., "Policy and Program Report". Prepared for Flood Control District of Maricopa County. April 2000. Structures Assessment Program. FCD Contract No. FCD 98-41. PCN PLAN.01.00.

2. Kimley-Horn and Associates, Inc., "Individual Structures Assessment Report - Cave Buttes Dam, Powerline FRS, Vineyard Road FRS, and Rittenhouse FRS". Prepared for Flood Control District of Maricopa County. June 2000. Structures Assessment Program. FCD Contract No. FCD 98-41. PCN PLAN.01.00.

3. Federal Emergency Management Agency. Flood Insurance Manual. 1994 Edition. Revised May 2000.

4. United States Department of the Interior - Bureau of Reclamation - Arizona Projects Office. "Reach 11 Dikes and Detention Basins - Hayden/Rhodes Aqueduct" November 1989.

5. "East Mesa Area Drainage Master Plan". Dibble & Associates. Prepared for Flood Control District of Maricopa County. July 1998.

6. "White Tanks FRS #3 - Basin Concepts Design Issues Memorandum". Dames & Moore. Prepared for Flood Control District of Maricopa County. February 2000.

7. "East Maricopa Floodway Capacity Mitigation Study Report". Huitt-Zollars. Prepared for Flood Control District of Maricopa County. February 2000.

8. "Queen Creek/Sanoki Wash Hydraulic Master Plan Alternate Analysis Report" (Draft). Huitt-Zollars. Prepared for Flood Control District of Maricopa County. March 2000.

9. "Preliminary East Maricopa Floodway Capacity Mitigation and Multi-Use Corridor Study (Alternative I)". Collins/Pina Consulting Engineers. Prepared for Flood Control District of Maricopa County. May 2000.

10. U.S Bureau of Reclamation. "Central Arizona Project, Salt-Gila Aqueduct, Reach 3, Protective Works", Plan sheet 344-330-2538. February, 1979. (Sonoqui Detention Dike).

ALTERNATIVES ANALYSIS REPORT

Appendix A: Cave Buttes Structural Alternative No. 1:
Low Level Outlet Dike No. 2

Opinion of Probable Construction Cost

Alternative No. 1 - Low Level Outlet in Dike No. 2

Item No.	Description	Unit	Quantity	Unit Cost	Total
1	2-10x6 RCB (10-ft span by 6-ft rise)	LF	250	$ 700.00	$ 175,000.00
2	Inlet Structure w/gate	L Sum	1	$ 25,000.00	$ 25,000.00
3	Outlet Structure w/dissipator	L Sum	1	$ 8,000.00	$ 8,000.00
4	Floodway Channel concrete lined trapezoidal. Bottom width 12 ft. Depth 4-ft. 4-in thick concrete section. 1.5:1 H:V Length 13,360 LF	SY	39,200	$ 25.00	$ 980,000.00
				Construction	$ 1,188,000.00

Land Cost			$ 227,100.00
Engineering and Construction Mgt		10%	$ 118,800.00
Operation and Maintenance		10%	$ 118,800.00
	Subtotal		$ 1,652,700.00
Contingency		25%	$ 413,175.00
Total Costs			$ 2,065,875.00

CURRENT DATE: 06-19-2000
CURRENT TIME: 15:54:22

FILE DATE: 06-19-2000
FILE NAME: DIKE2

FHWA CULVERT ANALYSIS
HY-8, VERSION 6.0

SITE DATA				CULVERT SHAPE, MATERIAL, INLET				
V NO.	INLET ELEV. (ft)	OUTLET ELEV. (ft)	CULVERT LENGTH (ft)	BARRELS SHAPE MATERIAL	SPAN (ft)	RISE (ft)	MANNING n	INLET TYPE
1	1624.00	1622.00	250.01	1 RCB	10.00	6.00	.012	CONVENTIONAL
2								
3								
4								
5								
6								

SUMMARY OF CULVERT FLOWS (cfs) FILE: DIKE2 DATE: 06-19-2000

ELEV (ft)	TOTAL	1	2	3	4	5	6	ROADWAY	ITR
1624.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1625.91	75.0	75.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1627.03	150.0	150.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1627.90	225.0	225.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1628.71	300.0	300.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1629.48	375.0	375.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1630.26	450.0	450.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1631.05	525.0	525.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1631.88	600.0	600.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1632.77	675.0	675.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1633.73	750.0	750.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1679.10	2288.0	2288.0	0.0	0.0	0.0	0.0	0.0	OVERTOPPING	

SUMMARY OF ITERATIVE SOLUTION ERRORS FILE: DIKE2 DATE: 06-19-2000

HEAD ELEV (ft)	HEAD ERROR (ft)	TOTAL FLOW (cfs)	FLOW ERROR (cfs)	% FLOW ERROR
1624.00	0.000	0.00	0.00	0.00
1625.91	0.000	75.00	0.00	0.00
1627.03	0.000	150.00	0.00	0.00
1627.90	0.000	225.00	0.00	0.00
1628.71	0.000	300.00	0.00	0.00
1629.48	0.000	375.00	0.00	0.00
1630.26	0.000	450.00	0.00	0.00
1631.05	0.000	525.00	0.00	0.00
1631.88	0.000	600.00	0.00	0.00
1632.77	0.000	675.00	0.00	0.00
1633.73	0.000	750.00	0.00	0.00

<1> TOLERANCE (ft) = 0.010 <2> TOLERANCE (%) = 1.000

CURRENT DATE: 06-19-2000 FILE DATE: 06-19-2000
CURRENT TIME: 15:54:22 FILE NAME: DIKE2

PERFORMANCE CURVE FOR CULVERT 1 - 1(10.00 (ft) BY 6.00 (ft)) RCB

DIS-CHARGE FLOW (cfs)	HEAD-WATER ELEV. (ft)	INLET CONTROL DEPTH (ft)	OUTLET CONTROL DEPTH (ft)	FLOW TYPE <F4>	NORMAL DEPTH (ft)	CRIT. DEPTH (ft)	OUTLET DEPTH (ft)	TW DEPTH (ft)	OUTLET VEL. (fps)	TW VEL. (fps)
0.00	1624.00	0.00	-2.00	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
75.00	1625.91	1.91	1.64	1-S2n	0.82	1.21	0.84	0.85	8.96	6.66
150.00	1627.03	3.03	2.12	1-S2n	1.31	1.92	1.34	1.28	11.19	8.45
225.00	1627.90	3.90	2.62	1-S2n	1.71	2.51	1.77	1.61	12.71	9.66
300.00	1628.71	4.71	3.16	1-S2n	2.08	3.04	2.18	1.91	13.75	10.59
375.00	1629.48	5.48	3.77	1-S2n	2.43	3.53	2.57	2.17	14.60	11.36
450.00	1630.26	6.25	4.44	1-S2n	2.76	3.99	2.95	2.40	15.28	12.02
525.00	1631.05	7.05	5.17	1-S2n	3.08	4.42	3.30	2.62	15.93	12.60
600.00	1631.88	7.88	5.98	1-S2n	3.38	4.83	3.63	2.82	16.54	13.11
675.00	1632.77	8.77	6.86	5-S2n	3.68	5.22	3.98	3.01	16.95	13.57
750.00	1633.73	9.73	7.81	5-S2n	3.97	5.60	4.30	3.19	17.43	14.00

El. inlet face invert 1624.00 ft El. outlet invert 1622.00 ft
El. inlet throat invert 0.00 ft El. inlet crest 0.00 ft

***** SITE DATA ***** CULVERT INVERT **************

INLET STATION	0.00 ft
INLET ELEVATION	1624.00 ft
OUTLET STATION	250.00 ft
OUTLET ELEVATION	1622.00 ft
NUMBER OF BARRELS	1
SLOPE (V/H)	0.0080
CULVERT LENGTH ALONG SLOPE	250.01 ft

***** CULVERT DATA SUMMARY ************************

BARREL SHAPE	BOX
BARREL SPAN	10.00 ft
BARREL RISE	6.00 ft
BARREL MATERIAL	CONCRETE
BARREL MANNING'S n	0.012
INLET TYPE	CONVENTIONAL
INLET EDGE AND WALL	1:1 BEVEL
INLET DEPRESSION	NONE

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******* REGULAR CHANNEL CROSS SECTION ****************

BOTTOM WIDTH	12.00 ft
SIDE SLOPE H/V (X:1)	1.5
CHANNEL SLOPE V/H (ft/ft)	0.005
MANNING'S n (.01-0.1)	0.013
CHANNEL INVERT ELEVATION	1622.00 ft
CULVERT NO.1 OUTLET INVERT ELEVATION	1622.00 ft

******* UNIFORM FLOW RATING CURVE FOR DOWNSTREAM CHANNEL

FLOW (cfs)	W.S.E. (ft)	FROUDE NUMBER	DEPTH (ft)	VEL. (f/s)	SHEAR (psf)
0.00	1622.00	0.000	0.00	0.00	0.00
75.00	1622.85	1.274	0.85	6.66	0.26
150.00	1623.28	1.319	1.28	8.45	0.40
225.00	1623.61	1.339	1.61	9.66	0.50
300.00	1623.91	1.352	1.91	10.59	0.60
375.00	1624.17	1.361	2.17	11.36	0.68
450.00	1624.40	1.367	2.40	12.02	0.75
525.00	1624.62	1.372	2.62	12.60	0.82
600.00	1624.82	1.376	2.82	13.11	0.88
675.00	1625.01	1.379	3.01	13.57	0.94
750.00	1625.19	1.381	3.19	14.00	1.00

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ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ ROADWAY OVERTOPPING DATA ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
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ROADWAY SURFACE	GRAVEL
EMBANKMENT TOP WIDTH	14.00 ft
CREST LENGTH	2800.00 ft
OVERTOPPING CREST ELEVATION	1679.10 ft

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Dike No. 2 Outlet Channel (Dike No. 2 to Reach 11 Detention Basin)
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	Trapezoidal Channe
Flow Element	Trapezoidal Channe
Method	Manning's Formula
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.013	
Slope	0.005000	ft/ft
Left Side Slope	1.50	H : V
Right Side Slope	1.50	H : V
Bottom Width	12.00	ft
Discharge	750.00	cfs

Results		
Depth	3.19	ft
Flow Area	53.6	ft²
Wetted Perim	23.51	ft
Top Width	21.57	ft
Critical Depth	4.14	ft
Critical Slope	0.001927	ft/ft
Velocity	14.00	ft/s
Velocity Head	3.05	ft
Specific Energ	6.24	ft
Froude Numb	1.57	
Flow Type	Supercritical	

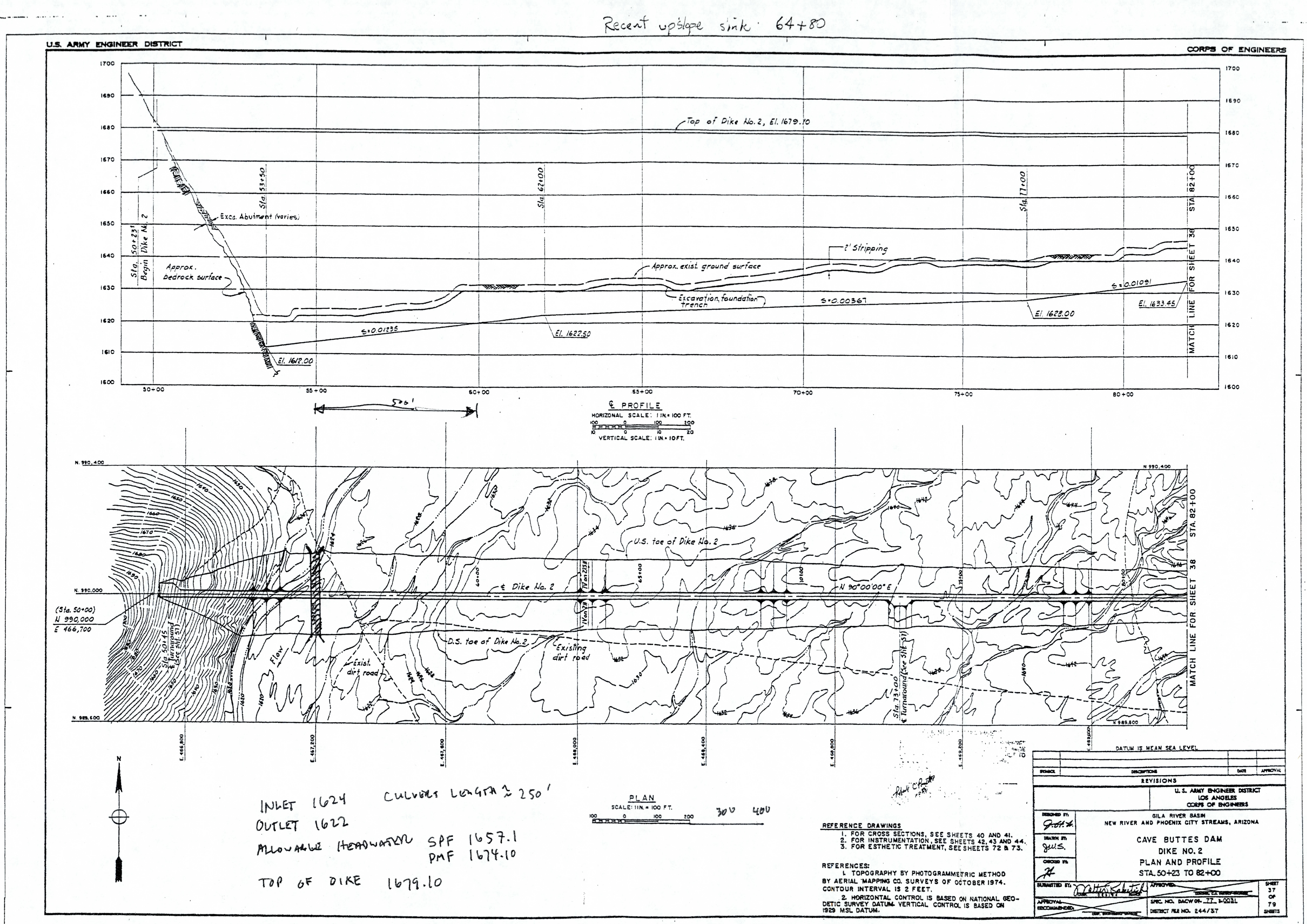
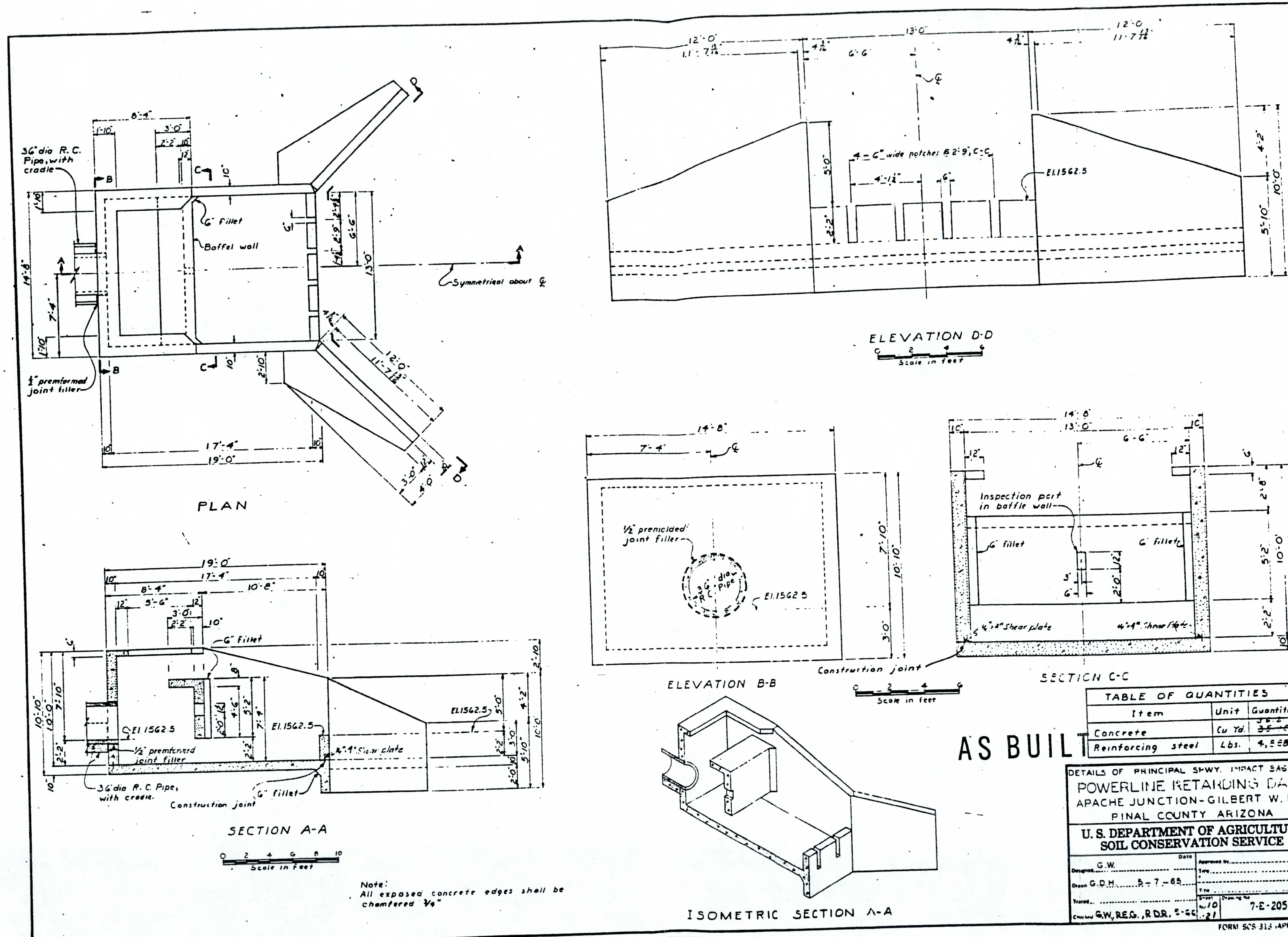


Figure 2-12 Embankment Plan, Profile, and Sections Dike No. 2 (Sheet 1)



Note:
All exposed concrete edges shall be chamfered ¾"

AS BUILT

TABLE OF QUANTITIES		
Item	Unit	Quantity
Concrete	Cu Yd.	~~35.0~~ 36.2
Reinforcing steel	Lbs.	4,558

DETAILS OF PRINCIPAL SPWY. IMPACT BASIN
POWERLINE RETARDING DAM
APACHE JUNCTION-GILBERT W. P. P.
PINAL COUNTY ARIZONA

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed G.W.	Approved by
Drawn G.D.H. 5-7-65	Title
Traced	Title
Checked G.W., R.E.G., R.D.R. 5-66	Sheet No. 10 of 21 — Drawing No. 7-E-20570

FORM SCS-313 (APRIL 1963)

ALT No 1 BID ITEM No 3

ALTERNATIVES ANALYSIS REPORT

Appendix B: Cave Buttes Structural Alternative No. 2: Floodway from Emergency Spillway to CAP Canal

Opinion of Probable Construction Cost

Alternative No. 2 - Emergency Spillway Floodway

Item No.	Description	Unit	Quantity	Unit Cost	Total
1	Concrete Line trapezoidal channel. Bottom width 24-ft Depth 6-ft. 1.5:1 H:V 4-in thick concrete Length 3000 LF	SY	12,800	$ 25.00	$ 320,000.00
2	Concrete Line trapezoidal channel. Bottom width 24-ft Depth 12-ft. 1.5:1 H:V 4-in thick concrete Length 3300 LF	SY	24,640	$ 25.00	$ 616,000.00
3	Confluence/Junctions Structure with canal gates (gates twin leaf 12-ft w by 12-high steel)	L Sum	1	$ 250,000.00	$ 250,000.00
				Construction	$ 1,186,000.00

Land Cost		$ 78,300.00
Engineering and Construction Mgt	10%	$ 118,600.00
Operation and Maintenance	10%	$ 118,600.00
Subtotal		$ 1,501,500.00
Contingency	25%	$ 375,375.00
Total Costs		$ 1,876,875.00

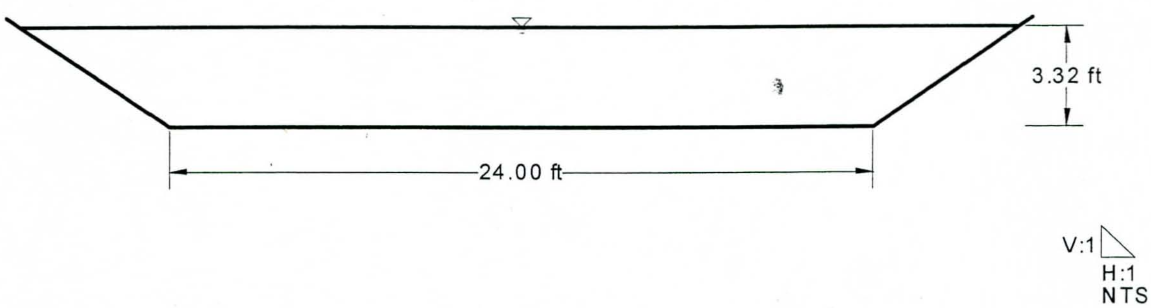
Cross Section
Cross Section for Trapezoidal Channel

Project Description	
Worksheet	Trapezoidal Channel Emergency Spillway
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Channel Depth

Section Data		
Mannings Coeffic	0.013	
Slope	0.020000	ft/ft
Depth	3.32	ft
Left Side Slope	1.50	H : V
Right Side Slope	1.50	H : V
Bottom Width	24.00	ft
Discharge	3,000.00	cfs

CHANNEL PARAMETERS

EL AT SPILLWAY ~ 1620 ft
EL AT CAP CANAL ~ 1494 ft (AS-BUILT PLAN GRANITE REEF AQUEDUCT)
LENGTH CHANNEL ~ 6300'
SLOPE = $\frac{126}{6300}$ = 0.02 ft/ft
CONCRETE LINED N=0.013
24 ft BOTTOM WIDTH (SAME AS CAP)
SIDE SLOPES 1.5:1



Low-Flow Channel from Emergency Spillway to CAP Canal
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	Trapezoidal Channel Emergency Spillway
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.013	
Slope	0.020000	ft/ft
Left Side Slope	1.50	H : V
Right Side Slope	1.50	H : V
Bottom Width	24.00	ft
Discharge	3,000.00	cfs

Results		
Depth	3.32	ft
Flow Area	96.3	ft²
Wetted Perim	35.98	ft
Top Width	33.97	ft
Critical Depth	6.78	ft
Critical Slope	0.001597	ft/ft
Velocity	31.16	ft/s
Velocity Head	15.09	ft
Specific Enerç	18.41	ft
Froude Numb	3.26	
Flow Type	Supercritical	

Low-Flow Channel from Emergency Spillway to CAP Canal
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	Trapezoidal Channel Emergency Spillway
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.013	
Slope	0.020000	ft/ft
Left Side Slope	1.50	H : V
Right Side Slope	1.50	H : V
Bottom Width	12.00	ft
Discharge	3,000.00	cfs

← 12 FT BOTTOM WIDTH

Results		
Depth	4.64	ft
Flow Area	88.0	ft²
Wetted Perim	28.73	ft
Top Width	25.92	ft
Critical Depth	8.77	ft
Critical Slope	0.001634	ft/ft
Velocity	34.09	ft/s
Velocity Head	18.06	ft
Specific Energ	22.70	ft
Froude Numb	3.26	
Flow Type	Supercritical	

Low-Flow Channel from Emergency Spillway to CAP Canal
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	Trapezoidal Channel Emergency Spillway
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.013	
Slope	0.000500	ft/ft
Left Side Slope	1.50	H : V
Right Side Slope	1.50	H : V
Bottom Width	24.00	ft
Discharge	3,000.00	cfs

FLATTER SLOPE

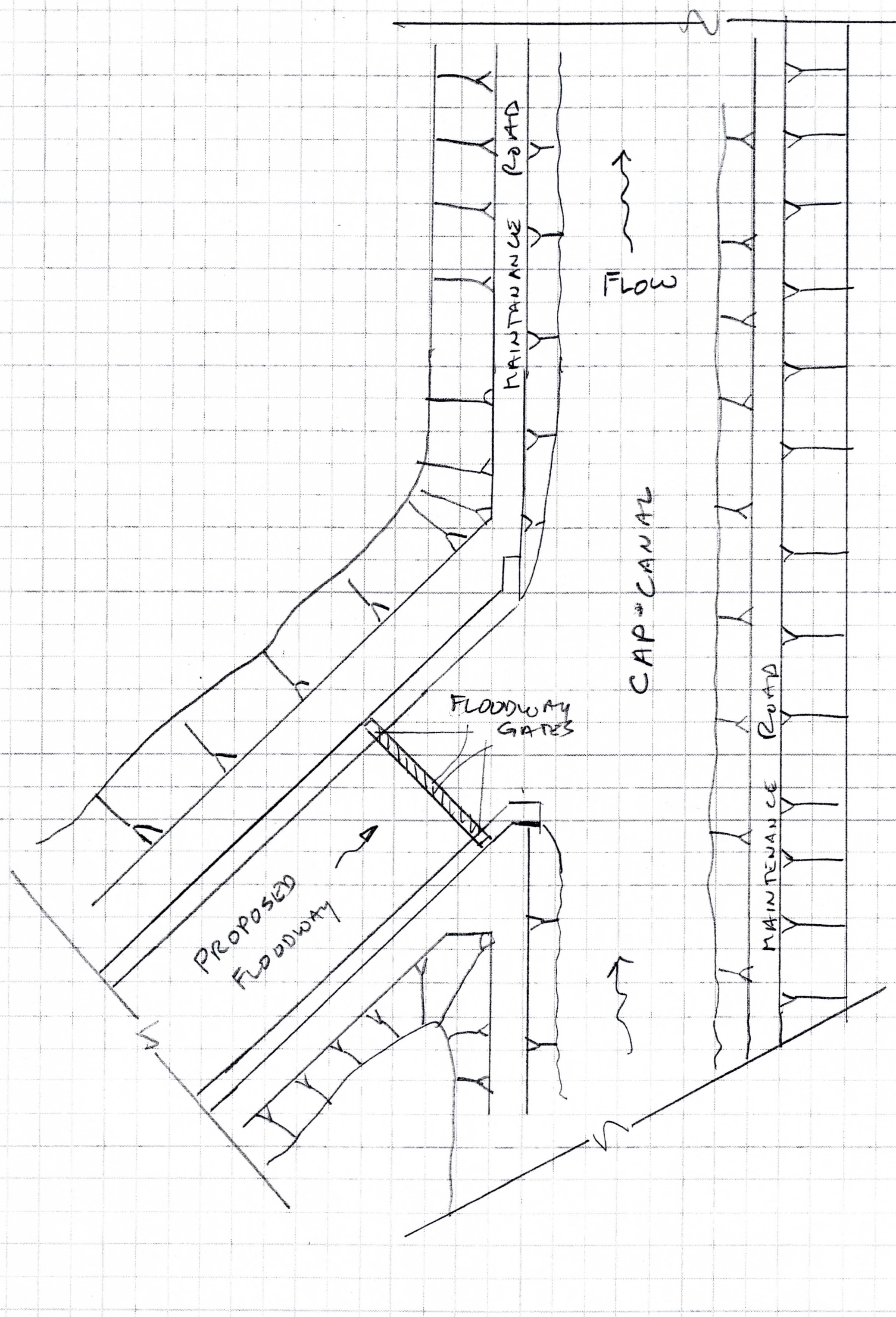
Results		
Depth	9.26	ft
Flow Area	351.0	ft²
Wetted Perim	57.40	ft
Top Width	51.79	ft
Critical Depth	6.78	ft
Critical Slope	0.001597	ft/ft
Velocity	8.55	ft/s
Velocity Head	1.14	ft
Specific Energ	10.40	ft
Froude Numb	0.58	
Flow Type	Subcritical	

Kimley-Horn and Associates, Inc.

Job STRUCTURES Subject ALTS ANALYSIS Sheet No. ____ of ____ Job No. 091131003

Designed by ____ Date ____ Checked by ____ Date ____

CAVE BUTTES DAM
FLOODWAY CONFLUENCE W/CAP CANAL
SCHEMATIC STRUCTURE PLAN



ALTERNATIVES ANALYSIS REPORT

Appendix C: Cave Buttes Structural Alternative No. 3: Low Level Outlet Dike No. 3

Opinion of Probable Construction Cost

Alternative No. 3 - Low Level Outlet in Dike No. 3

Item No.	Description	Unit	Quantity	Unit Cost	Total
1	2-8x4 RCB (8-ft span by 4-ft span)	LF	80	$ 500.00	$ 40,000.00
2	Inlet headwall w/ gates	L Sum	1	$ 18,000.00	$ 18,000.00
3	Outlet headwall w/wings and access barrier	L Sum	1	$ 5,500.00	$ 5,500.00
4	Earth-lined trapezoidal channel 10-ft bottom 3-ft deep 2:1 H:V Length 500 LF	CY	889	$ 10.00	$ 8,890.00
				Construction	$ 72,390.00

Land Cost		$ 18,400.00
Engineering and Construction Mgt	10%	$ 7,239.00
Operation and Maintenance	10%	$ 7,239.00
Subtotal		$ 105,268.00
Contingency	25%	$ 26,317.00
Total Costs		$ 131,585.00

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CURRENT DATE: 06-19-2000                                  FILE DATE: 06-19-2000
CURRENT TIME: 16:29:55                                    FILE NAME: DIKE3

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ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ     FHWA CULVERT ANALYSIS     ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ       HY-8, VERSION 6.0       ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
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³              SITE DATA             ³        CULVERT SHAPE, MATERIAL, INLET     ³
³ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ´
³ L ³  INLET    OUTLET   CULVERT ³ BARRELS                                       ³
³ V ³  ELEV.    ELEV.    LENGTH  ³ SHAPE          SPAN    RISE  MANNING   INLET  ³
³NO.³  (ft)     (ft)     (ft)    ³ MATERIAL       (ft)    (ft)     n       TYPE  ³
³ 1 ³1669.50   1668.50    80.01  ³ 2 RCB          8.00    4.00   .012  CONVENTIONAL³
³ 2 ³                            ³                                               ³
³ 3 ³                            ³                                               ³
³ 4 ³                            ³                                               ³
³ 5 ³                            ³                                               ³
³ 6 ³                            ³                                               ³
ÀÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÙ

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SUMMARY OF CULVERT FLOWS (cfs)          FILE: DIKE3            DATE: 06-19-2000

 ELEV (ft)   TOTAL      1        2        3        4        5        6   ROADWAY ITR
  1669.50      0.0      0.0     0.0      0.0      0.0      0.0      0.0     0.00  1
  1670.62     10.0     10.0     0.0      0.0      0.0      0.0      0.0     0.00  1
  1670.69     20.0     20.0     0.0      0.0      0.0      0.0      0.0     0.00  1
  1670.74     30.0     30.0     0.0      0.0      0.0      0.0      0.0     0.00  1
  1670.80     40.0     40.0     0.0      0.0      0.0      0.0      0.0     0.00  1
  1670.85     50.0     50.0     0.0      0.0      0.0      0.0      0.0     0.00  1
  1670.90     60.0     60.0     0.0      0.0      0.0      0.0      0.0     0.00  1
  1670.95     70.0     70.0     0.0      0.0      0.0      0.0      0.0     0.00  1
  1670.99     80.0     80.0     0.0      0.0      0.0      0.0      0.0     0.00  1
  1671.07     90.0     90.0     0.0      0.0      0.0      0.0      0.0     0.00  1
  1671.18    100.0    100.0     0.0      0.0      0.0      0.0      0.0     0.00  1
  1679.10    660.0    660.0     0.0      0.0      0.0      0.0      0.0 OVERTOPPING
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ

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  SUMMARY OF ITERATIVE SOLUTION ERRORS    FILE: DIKE3          DATE: 06-19-2000

        HEAD            HEAD           TOTAL          FLOW          % FLOW
      ELEV (ft)      ERROR (ft)      FLOW (cfs)     ERROR (cfs)      ERROR
       1669.50          0.000           0.00           0.00           0.00
       1670.62          0.000          10.00           0.00           0.00
       1670.69          0.000          20.00           0.00           0.00
       1670.74          0.000          30.00           0.00           0.00
       1670.80          0.000          40.00           0.00           0.00
       1670.85          0.000          50.00           0.00           0.00
       1670.90          0.000          60.00           0.00           0.00
       1670.95          0.000          70.00           0.00           0.00
       1670.99          0.000          80.00           0.00           0.00
       1671.07          0.000          90.00           0.00           0.00
       1671.18          0.000         100.00           0.00           0.00
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
 <1> TOLERANCE (ft) = 0.010                           <2> TOLERANCE (%) = 1.000
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
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CURRENT DATE: 06-19-2000
CURRENT TIME: 16:29:55

FILE DATE: 06-19-2000
FILE NAME: DIKE3

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PERFORMANCE CURVE FOR CULVERT 1 - 2(8.00 (ft) BY 4.00 (ft)) RCB

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DIS-CHARGE	HEAD-WATER ELEV. (ft)	INLET CONTROL DEPTH (ft)	OUTLET CONTROL DEPTH (ft)	FLOW TYPE <F4>	NORMAL DEPTH (ft)	CRIT. DEPTH (ft)	OUTLET DEPTH (ft)	TW DEPTH (ft)	OUTLET VEL. (fps)	TW VEL. (fps)
0.00	1669.50	0.00	-1.00	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
10.00	1670.62	0.40	1.12	1-S2n	0.09	0.23	0.09	0.53	7.07	2.13
20.00	1670.68	0.58	1.18	1-S2n	0.18	0.37	0.18	0.80	7.07	2.70
30.00	1670.74	0.76	1.24	1-S2n	0.27	0.48	0.28	1.02	6.71	3.09
40.00	1670.80	0.92	1.30	1-S2n	0.35	0.58	0.35	1.20	7.07	3.39
50.00	1670.85	1.06	1.35	1-S2n	0.42	0.67	0.43	1.37	7.24	3.64
60.00	1670.90	1.20	1.40	1-S2n	0.47	0.76	0.49	1.51	7.69	3.86
70.00	1670.95	1.33	1.45	1-S2n	0.51	0.84	0.51	1.65	8.58	4.04
80.00	1671.00	1.45	1.50	1-S2n	0.55	0.92	0.59	1.78	8.45	4.21
90.00	1671.07	1.57	1.54	1-S2n	0.60	1.00	0.65	1.90	8.72	4.36
100.00	1671.18	1.68	1.59	1-S2n	0.64	1.07	0.70	2.02	8.98	4.50

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El. inlet face invert 1669.50 ft El. outlet invert 1668.50 ft
El. inlet throat invert 0.00 ft El. inlet crest 0.00 ft

ÄÄ

***** SITE DATA ***** CULVERT INVERT **************

INLET STATION	0.00 ft
INLET ELEVATION	1669.50 ft
OUTLET STATION	80.00 ft
OUTLET ELEVATION	1668.50 ft
NUMBER OF BARRELS	2
SLOPE (V/H)	0.0125
CULVERT LENGTH ALONG SLOPE	80.01 ft

***** CULVERT DATA SUMMARY ************************

BARREL SHAPE	BOX
BARREL SPAN	8.00 ft
BARREL RISE	4.00 ft
BARREL MATERIAL	CONCRETE
BARREL MANNING'S n	0.012
INLET TYPE	CONVENTIONAL
INLET EDGE AND WALL	1:1 BEVEL
INLET DEPRESSION	NONE

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CURRENT DATE: 06-19-2000
CURRENT TIME: 16:29:55

FILE DATE: 06-19-2000
FILE NAME: DIKE3

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ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ TAILWATER ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
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******* REGULAR CHANNEL CROSS SECTION ****************

BOTTOM WIDTH	8.00 ft
SIDE SLOPE H/V (X:1)	1.5
CHANNEL SLOPE V/H (ft/ft)	0.005
MANNING'S n (.01-0.1)	0.030
CHANNEL INVERT ELEVATION	1668.50 ft
CULVERT NO.1 OUTLET INVERT ELEVATION	1668.50 ft

******* UNIFORM FLOW RATING CURVE FOR DOWNSTREAM CHANNEL

FLOW (cfs)	W.S.E. (ft)	FROUDE NUMBER	DEPTH (ft)	VEL. (f/s)	SHEAR (psf)
0.00	1668.50	0.000	0.00	0.00	0.00
10.00	1669.03	0.513	0.53	2.13	0.17
20.00	1669.30	0.532	0.80	2.70	0.25
30.00	1669.52	0.541	1.02	3.09	0.32
40.00	1669.70	0.546	1.20	3.39	0.38
50.00	1669.87	0.549	1.37	3.64	0.43
60.00	1670.01	0.552	1.51	3.86	0.47
70.00	1670.15	0.554	1.65	4.04	0.51
80.00	1670.28	0.556	1.78	4.21	0.56
90.00	1670.40	0.557	1.90	4.36	0.59
100.00	1670.52	0.558	2.02	4.50	0.63

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ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ ROADWAY OVERTOPPING DATA ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
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ROADWAY SURFACE	GRAVEL
EMBANKMENT TOP WIDTH	12.00 ft
CREST LENGTH	3245.00 ft
OVERTOPPING CREST ELEVATION	1679.10 ft

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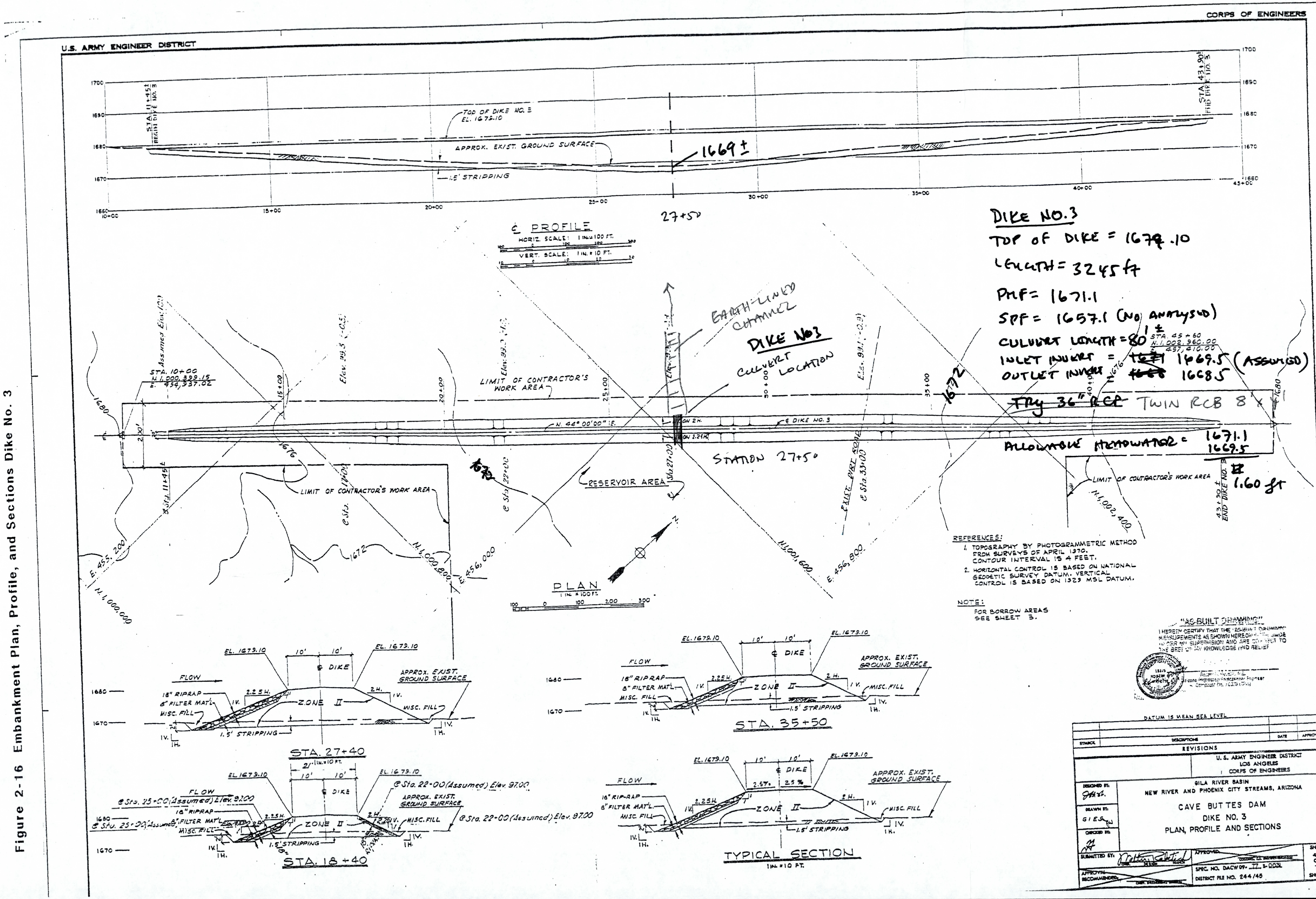
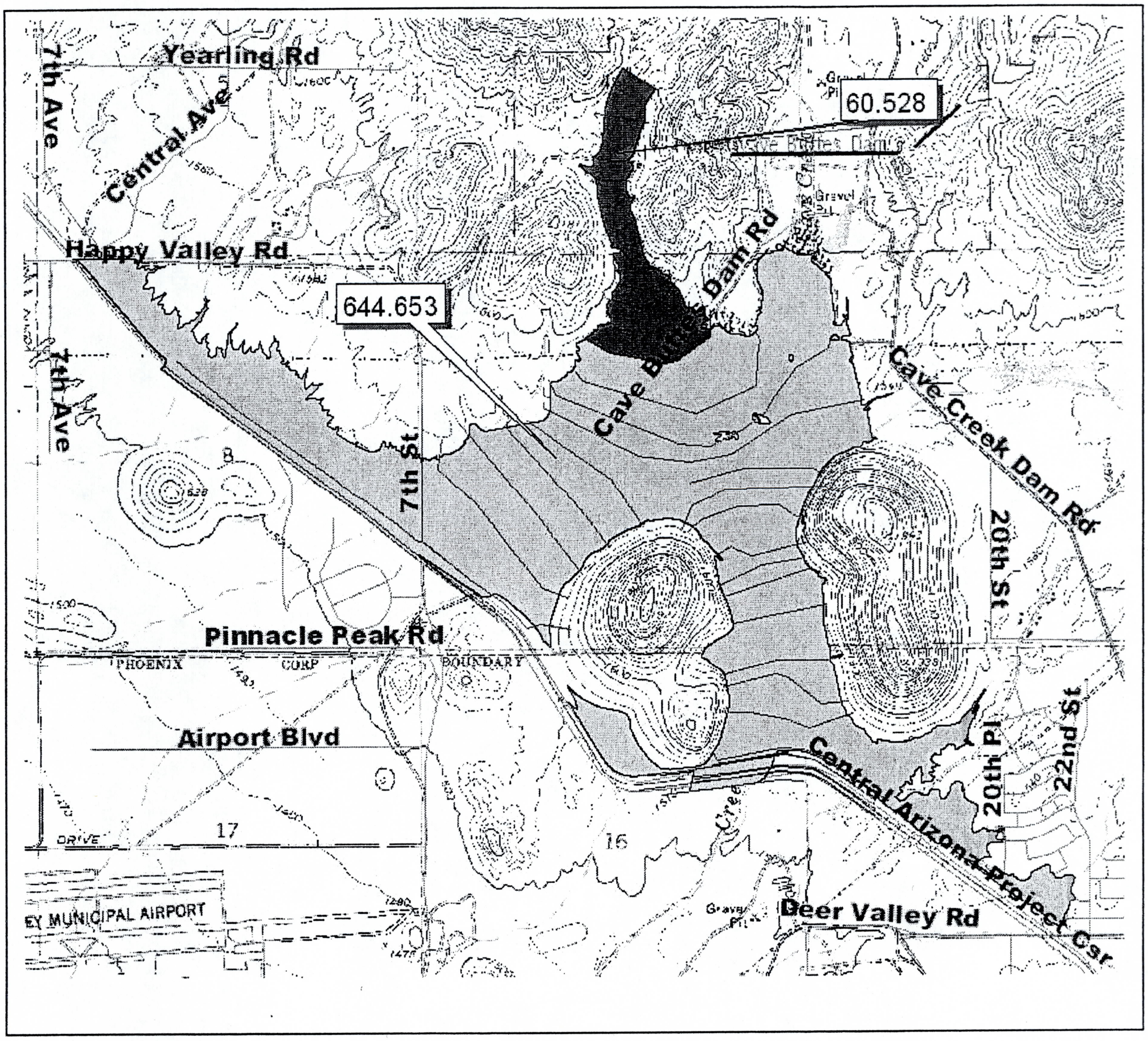
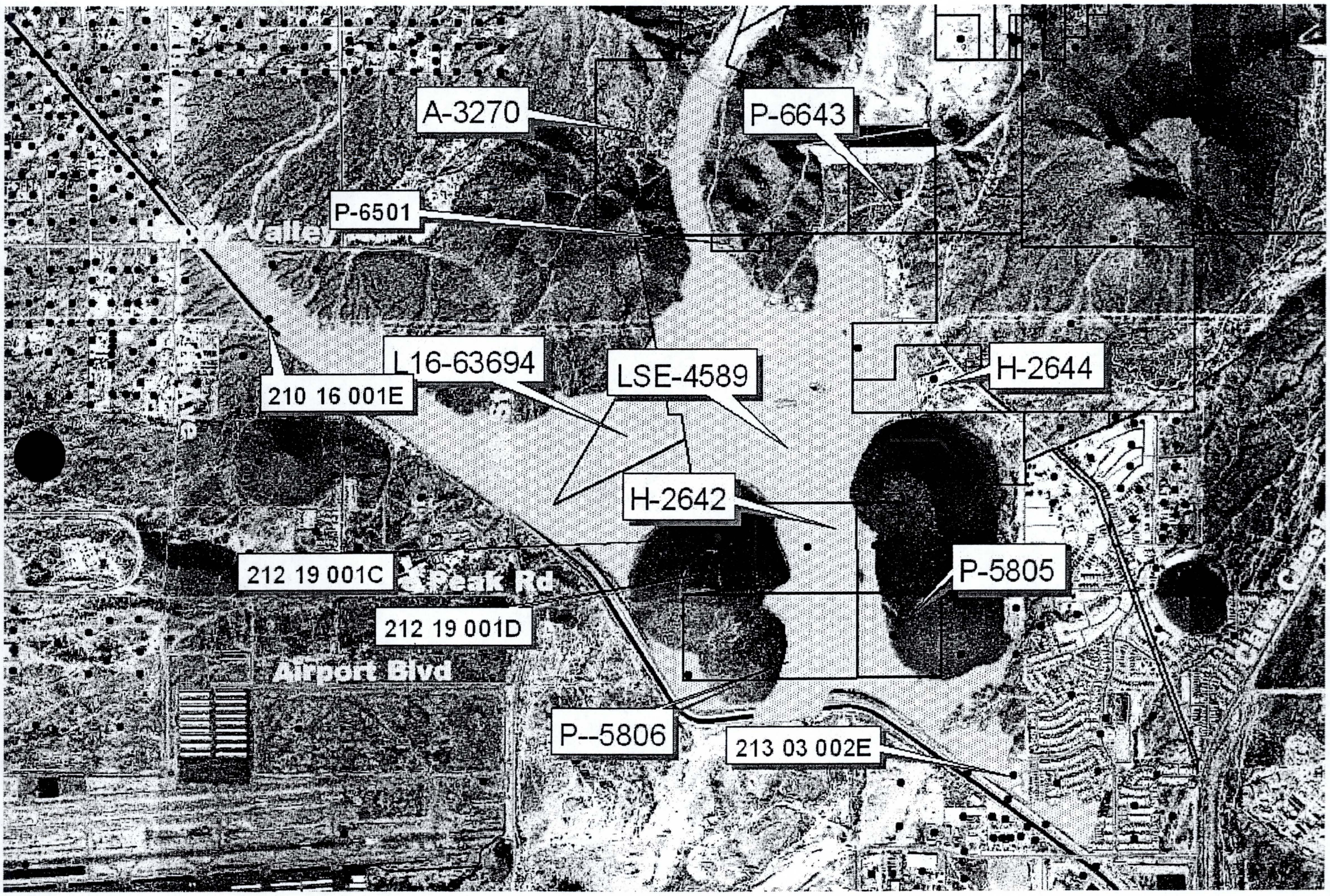


Figure 2-16 Embankment Plan, Profile, and Sections Dike No. 3

ALTERNATIVES ANALYSIS REPORT

Appendix D: Cave Buttes Dam Nonstructural Alternatives Back-Up Data


Yearling Rd
7th Ave
Central Ave
60.528
Cave Buttes Dam
Happy Valley Rd
644.653
Cave Buttes Dam Rd
Cave Creek Dam Rd
7th Ave
7th St
20th St
Pinnacle Peak Rd
PHOENIX CORP BOUNDARY
Airport Blvd
20th Pl
22nd St
Central Arizona Project Csr
DRIVE
17
16
MUNICIPAL AIRPORT
Deer Valley Rd


A-3270
P-6643
P-6501
Valley
L16-63694
LSE-4589
H-2644
210 16 001E
H-2642
212 19 001C
Peak Rd
P-5805
212 19 001D
Airport Blvd
P--5806
213 03 002E

Areas

Location/Name	Area sq. ft	Area acres
New Lands - 1	2907642.23	66.8
New Lands - 2	18282050.59	419.7
New Lands - 3	10114872.07	232.2
New Lands - Total	31304564.89	718.6539
New Lands Zone A - 1	4372752.84	100.4
New Lands Zone A - 2	3913708.26	89.8
New Lands Zone A - Total	8286461.1	190.231
New Lands Zone X - Total	23018103.79	528.4
PMF (red)	116518912.7	2674.9
SPF (blue)	72587831.31	1666.4
1660' (cyan)	78040207.44	1791.6
1680' (green)	130226869.6	2989.6

FLOOD CONTROL ADVISORY BOARD
Meeting of August 25, 1999
INFORMATION SHEET

AB
2

99A013

AGENDA ITEM NO. 2

AGENDA ITEM: IGA with Phoenix to discharge effluent into Cave Creek Dam – IGA FCD 99013

ACTION REQUIRED: To determine whether the Flood Control Advisory Board (FCAB) should approve and recommend that the Board of Directors adopt Resolution FCD IGA 99013 which allows the City of Phoenix to discharge treated effluent into Cave Creek Dam.

BACKGROUND: The City of Phoenix is constructing the Cave Creek Water Reclamation Plant and Discharge Pipeline with a maximum capacity of eight (8.0) million gallons per day (24.5 acre-feet per day). PHOENIX will expand the capacity of the Cave Creek Water Reclamation Plant as development expands within the Plant's service area. The Discharge Pipeline delivers treated effluent to users for turf irrigation and other non-potable uses. During the winter, the demand for treated effluent from the plant reduces and the Plant output remains relatively constant. Therefore, PHOENIX will discharge the surplus treated effluent into a tributary of Cave Creek Wash which conveys the treated effluent into the storage pool above Cave Creek Dam, owned and operated by the DISTRICT, as shown on Exhibit A of the IGA which is attached.

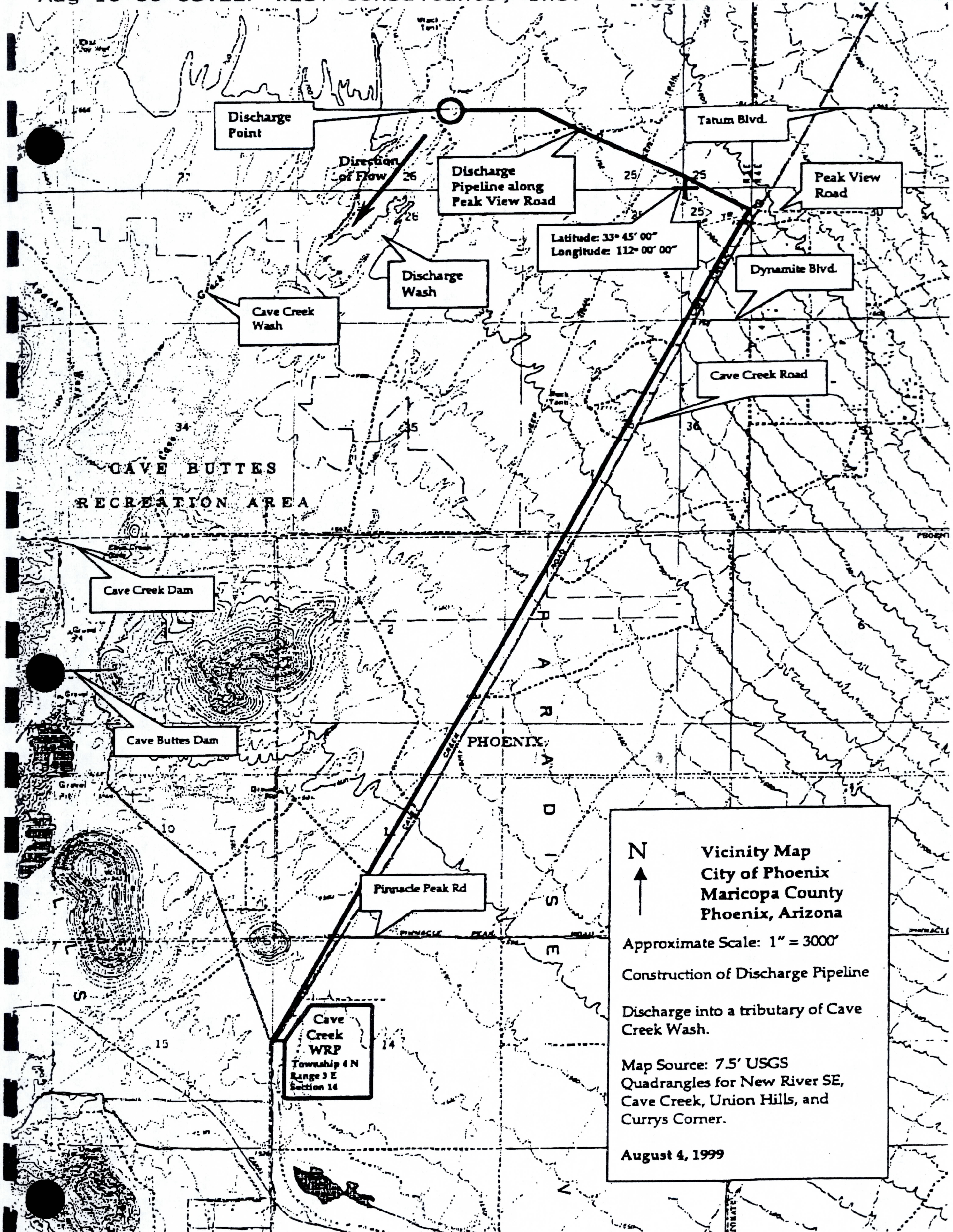
The District will excavate materials within the flood pool of the dam to replace the volume occupied by the treated effluent and sediment from the Treatment Plant. The District will retain ownership of the excavated material.

Phoenix will lease the land required to maintain the additional storage volume and reimburse all District costs, including staff time, for excavating the additional storage volume. Phoenix will pursue ground water recharge credits from the State of Arizona. Phoenix will be responsible for the quantity and quality of the water and for adverse impacts resulting from the discharge of treated effluent onto DISTRICT property. Phoenix will provide all City permits for the DISTRICT to complete it's obligations. Phoenix will complete planning and environmental studies for a multi-use park and recreational facilities and provide a permanent water supply for the development.

This project is within District 3.

STAFF RECOMMENDS THE FOLLOWING ACTION: It is moved that the Flood ~~Control~~ Advisory Board (FCAB) endorse and recommend that the Board of Directors adopt Resolution FCD 99R011 which directs and authorizes the Chief Engineer and General Manager to negotiate and acquire real property consisting of approximately 20 acres at the corner of Gilbert Road and Riggs Road, ~~subject~~ to the ratification and approval of the Board of Directors.

Enclosures: IGA FCD 99013



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Vicinity Map
City of Phoenix
Maricopa County
Phoenix, Arizona

Approximate Scale: 1" = 3000'

Construction of Discharge Pipeline

Discharge into a tributary of Cave Creek Wash.

Map Source: 7.5' USGS Quadrangles for New River SE, Cave Creek, Union Hills, and Currys Corner.

August 4, 1999

Kimley-Horn
and Associates, Inc.

Job STRUCTURES ASSESSMENT Subject ALTERNATIVES Sheet No. ____ of ____ Job No. 091131003
Designed by RAE Date 7/19/2000 Checked by ________ Date ________

CAVE BUTTES DAM MITIGATE THROUGH FLOOD INSURANCE

DOWNSTREAM (EMERGENCY SPILLWAY TO CAP CANAL)

NO. ACRES INUNDATED BY FULL PMF = 644 acres

NO. ACRES ZONE AE = 85 ACRES

NO. ACRES ZONE X = 644 - 85 = 559 ac

ASSUME ZONING IS 2 DWELLING UNITS PER ACRE CONSISTING OF SINGLE FAMILY RESIDENCES

ANNUAL PREMIUMS FOR $100,000 OF FLOOD INSURANCE COVERAGE FOR SINGLE FAMILY RESIDENCE

- POST FIRM ZONE AE 1 foot above BFE = $301.00/yr
- ZONE X PREFERRED RISK POLICY = $221/yr

ZONE	NO. ACRES	DWELLING UNITS	UNIT PREMIUM	ANNUAL PREMIUMS	30 YEARS
AE	85	170	$301	$51,170	$1,535,100.00
X	559	1118	$221	$247,078	$7,412,340.00

Federal Emergency Management Agency

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

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COVERAGE INFORMATION

- How Much Coverage Is Available?
- What Does It Cover?
- What Is Covered in My Basement?
- What is Increased Cost of Compliance (ICC) Coverage?
- When Will My Policy Go Into Effect?
- What Is A Flood?

Flood Insurance Coverage Available Limits Of Liability

Coverage Category	EmergencyProgram	Regular Program
BUILDING COVERAGE		
Single family dwelling	35,000	250,000
2-4 family dwelling	35,000	250,000
Other residential	100,000	250,000
Non-residential	100,000	500,000
CONTENTS COVERAGE		
Residential	10,000	100,000
Non-residential	100,000	500,000

What Does It Cover?

The Standard Flood Insurance Policy (SFIP) Forms contain complete definitions of the coverages they provide. Direct physical losses caused by "floods" are covered. Also covered are losses resulting from flood-related erosion caused by waves or currents of water activity exceeding anticipated cyclical levels, or caused by a severe storm, flash flood, abnormal tidal surge, or the like, which result in flooding, as defined. Damage caused by mudslides (i.e., mudflows), as specifically defined in the policy forms, is covered.

What Is Covered in My Basement?

The NFIP defines a basement as any area of a building with a floor that is below ground level on all sides. While flood insurance does not cover basement improvements, such as finished walls, floors or ceilings, or personal belongings that may be kept in a basement , such as furniture and other contents, it does cover structural elements, essential equipment and other basic items normally located in a basement. Many of these items are covered under building coverage, and some are covered under contents coverage. The NFIP encourages people to purchase both building and contents coverage for the broadest protection.

The following items are covered under building coverage, as long as they are connected to a power source and installed in their functioning location:

- Sump pumps.
- Well water tanks and pumps, cisterns and the water in them.
- Oil tanks and the oil in them, natural gas tanks and the gas in them.
- Pumps and/or tanks used in conjunction with solar energy.
- Furnaces, hot water heaters, air conditioners, and heat pumps.
- Electrical junction and circuit breaker boxes, and required utility connections.
- Foundation elements.
- Stairways, staircases, elevators and dumbwaiters.
- Unpainted drywall and sheet rock walls and ceilings, including fiberglass insulation.
- Cleanup.

The Following items are covered under contents coverage:

- Clothes washers.
- Clothes dryers.
- Food Freezers and the food in them.

What Is Increased Cost of Compliance (ICC) Coverage?

Increased Cost of Compliance (ICC) under the NFIP provides for the payment of a claim to help pay for the cost to comply with State or community floodplain management laws or ordinances from a flood event in which a building has been declared substantially damaged or repetitively damaged. When an insured building is damaged by a flood and the State or community declares the building to be substantially damaged or repetitively damaged, ICC will help pay for the cost to elevate, floodproof, demolish or relocate the building up to $20,000. This coverage is in addition to the building coverage for the repair of actual physical damages from flood under the Standard Flood Insurance Policy (SFIP).

When Will My Policy Go Into Effect?

There is a **30-day waiting period** before a flood insurance policy can become effective. In most instances, the insurance producer who writes your policy can provide you with the date that your policy should go into effect.

What Is A Flood?

Under the National Flood Insurance Program (NFIP) a flood is defined as **a general and temporary condition** of partial or complete inundation of normally dry land by:

- The overflow of inland or tidal waters.
- The unusual and rapid accumulation or runoff of surface waters from any source.
- Mudslides (i.e., mudflows) which are proximately caused by flooding, as defined above and are akin to a river of liquid and flowing mud on the surfaces of normally dry land areas, including your premises, as when earth is carried by a current of water and deposited along the path of the current.
- The collapse or subsidence of land along the shore of a lake or other body of water as a result of erosion or undermining caused by waves or currents of water exceeding the cyclical levels which result in flood as defined above.

To qualify as a **general and temporary condition**, the flood must affect either two or more adjacent properties or two or more acres of land and have a distinct beginning point and ending point.

Also, to qualify, the flood waters can only be surface water that covers land that is normally dry.

Updated: April 14, 2000

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Cost Information

A number of factors are considered in determining the premium for flood insurance coverage. They include:

- amount of coverage purchased
- location
- age of the building
- building occupancy
- design of the building
- for buildings in Special Flood Hazard Areas, elevation of the building.
- buildings eligible for special low-cost coverage at a pre-determined, reduced premium rate are single-family and 1-4 family dwellings located in zones B, C, & X. Ask your insurance agent if you're eligible for a Preferred Risk Policy.

- Average Cost & Coverage
- Cost Comparision
- Premium Examples for a $100,000 home
- Preferred Risk Policy Premiums

Updated: December 9,1998

Cost and Coverage Data as of May 1, 2000

Occupancy Type	Regular Program	
	Coverage	Premium*
Single family	$124,300	$570
Two to four family	$101,700	$524
Other residential	$85,900	$665
Non-residential	$218,600	$1,514

** Premium values are based on Pre-FIRM Special Flood Hazard Area rates and includes Federal Policy Fee & Expense Constant.*

Updated: July 6, 2000

Premium Examples For A $100,000 Single Family Home

If you own a home in a community that participates in the National Flood Insurance Program, you are eligible for flood insurance. More than 19,000 communities participate, so its likely that your community does participate.

There are many factors that affect the price you'll pay for flood insurance. The higher your flood risk, the higher the premium. If you purchase $100,000 in building coverage for your home, your annual premium will vary depending on the area in which you live.

- If the property is located near the ocean and therefore subject to storm surge and hurricane damage, your building is most likely in a V Zone. Premiums is V zones can be more than $1,000 annually because your home is in the highest risk area.
- If the property is located near a river, lake or stream, your building is probably in an A zone. Premiums in A zones can be about $595 annually because of the high potential for flooding.
- If the property is located in a low-risk area, referred to as B, C, X or A99 zones, your premium could be as low as $306 annually using standard rates. You may also be able to get the Preferred Risk Policy. Click here for premium rates for the PRP.

Below are annual premiums for $100,000 of flood insurance coverage for a residential single family home:

Pre or Post-FIRM	Zone	Other Rating Factors	Premium
Pre-FIRM***	Zone V1-30,VE	No Enclosure	$845.00****
		With Enclosure	$1,090.00
Post-FIRM***	Zone V1-30,VE	At BFE*	$ 850.00
	Built between 1975-1981	1 Foot below BFE	$ 2,180.00

Pre-FIRM	Zone A1-30, AE	No Basement	$ 595.00
		With Basement	$ 700.00
Post-FIRM	Zone A1-30, AE	At BFE	$ 431.00
		1 Foot above BFE	$ 301.00
		1 Foot below BFE	$ 1,251.00
Pre-FIRM	Zone AO, AH	With Certification**	$ 201.00
		Without Certification	$ 585.00
Pre/Post-FIRM	Zone B, C, X, A99	No Basement	$ 351.00
		With Basement	$ 441.00

**BFE-Base Flood Elevation found on Flood Insurance Rate Map*

***Certification is determined by an Elevation Certificate completed by a licensed engineer, surveyor or architect*

****Pre/Post FIRM is determined by the date of the initial Flood Insurance Rate Map*

*****Premium values are based on total written premium plus Expense Constant, Federal Policy Fee and Increased Cost of Compliance premium. Effective date: May 1, 2000*

Updated: July 6, 2000

Federal Emergency Management Agency

How to Purchase and How to Pay for Flood Insurance

To buy a flood insurance policy, call your insurance agent or contact one of the WriteYour Own companies, private insurance companies that write flood insurance under a special arrangement with the Federal government. If your agent does not write flood insurance or you don't have an agent, you may call the National Flood Insurance Program's (NFIP) toll free number to obtain the name of an agent in your area who does write flood insurance. The number is 1-888-CALL FLOOD, ext. 445. You can also check your local Yellow Pages directory.

It's a good idea to have the same agent who writes your homeowners or other insurance policies also write your flood insurance policy so in the event you need to file a claim, you only have to work with one insurance agency or company.

How can you pay for flood insurance?

In addition to paying the full annual premium by (cash, check or money order), you can now buy flood insurance with a credit card (Visa or MasterCard).

Another way flood insurance premiums can be paid is through an escrow account established by your mortgage lender. In fact, if your lender requires you to buy flood insurance and escrows for other types of insurance or taxes, the lender is required to also escrow flood insurance premium payments. Ask your insurance agent or lender for details.

Updated: October 6, 1998

Federal Emergency Management Agency

Why You Should Have a Preferred Risk Policy

- The Preferred Risk Policy can save you money!
- The Preferred Risk Policy ensures you financial protection against flood damage at a special low price for owners of homes not in high-risk flood areas.
- Flood damage is not covered under most homeowner's policies. To get coverage, you have to buy a separate policy.
- In the past 25 years, the NFIP has paid one-quarter of its claims to cover flood losses to those homes in moderate to minimal flood risk zones.
- The Preferred Risk Policy provides several coverage combinations for both the building and its contents that range from $20,000 building/$5,000 contents to $250,000 building/ $60,000 contents.
- People should consider this low-cost protection for their homes and contents because floods occur even in areas no one considers high-risk.
- When a flood occurs, there is no guarantee that it will be declared a Federal disaster and that you will qualify for Federal assistance.
- Disaster relief is often in the form of a low-interest loan that must be repaid. This adds to your total debt and may wipe out any equity that you have accumulated.
- As a condition for receiving disaster assistance, the homeowner must purchase and maintain a flood insurance policy for future protection.
- To be eligible for a Preferred Risk Policy, the building must be in a low-risk (B, C, or X) zone on the effective date of the current term.
- You can save about 30% of the standard application premium costs if you purchase a Preferred Risk Policy. Most people invest a major part of their income in a home. Protecting these assets from loss must be a concern.

"Life is not waterproof-Be flood alert."

For more information, call 1-888-CALL-FLOOD ext. 445,
TDD# 1-800-427-5593

F-437 (12/99)

Updated: June 30, 2000

Preferred Risk Policy Premiums

If your single family home is located in a low–risk area, which is a B, C, or X zone on the current flood insurance rate map for your area, you may be eligible for the Preferred Risk Policy. This policy covers both your home and contents with one premium, which can be as little as $106 a year.

Preferred Risk Premiums

Building with a Basement

Coverage Amount	Contents	Premium
$ 20,000	$ 5,000	$131
$ 30,000	$ 8,000	$156
$ 50,000	$12,000	$196
$ 75,000	$18,000	$221
$100,000	$25,000	$246
$125,000	$30,000	$261
$150,000	$38,000	$276
$200,000	$50,000	$306
$250,000	$60,000	$326

Building without a Basement

Coverage Amount	Contents	Premium
$ 20,000	$ 5,000	$106
$ 30,000	$ 8,000	$131
$ 50,000	$12,000	$171
$ 75,000	$18,000	$196
$100,000	$25,000	$221
$125,000	$30,000	$236
$150,000	$38,000	$251
$200,000	$50,000	$281
$250,000	$60,000	$301

Building deductible $500 and Contents deductible $500 applied separately
Premium includes Federal Policy Fee and Increased Cost of Compliance premium
Effective date: June 1, 1998

Preferred Risk Policies (PRP) are only available for owners of 1-4 family residential buildings. Additionally should any of the following conditions apply to your home, based on its flood history regardless of ownership, a PRP cannot be written: *

- 2 loss payments, each more than $1,000
- 3 or more loss payments, regardless of amount
- 2 Federal Disaster Relief payments, each more than $1,000
- 3 Federal Disaster Relief payments, regardless of amount
- 1 flood insurance claim payment and 1 flood disaster relief payment (including loans and grants), each more than $1,000

If your home is in a low-risk area, and one or more of the above conditions apply or you own a building other than a 1-4 family home that is located in a B, C, or X zone, you can still purchase flood insurance at the low-risk Standard Rates. For premium examples for $100,000 of coverage for a single-family home click here

*Contact your insurance agent for all the eligibility requirements for a PRP.

Updated: 10/6/1998

Federal Emergency Management Agency

ALTERNATIVES ANALYSIS REPORT

Appendix E: Powerline, Vineyard Road, and Rittenhouse FRS
Structural Alternative No. 1:
Segmentation

Opinion of Probable Construction Cost

FRS Structural Alternative No. 1 - Segmentation

Item	Description	Unit	Quantity	Unit Cost	Total
1	Embankment fill	CY	211,815	$ 7.00	$ 1,482,705.00
2	Culvert RCP 6 ft diam	LF	280	$ 175.00	$ 49,000.00
3	Headwall w/gates	L SUM	1	$ 14,000.00	$ 14,000.00
4	Outlet Headwall	L SUM	1	$ 4,400.00	$ 4,400.00
5	Guardrail	LF	12,180	$ 10.00	$ 121,800.00
				Construction	$ 1,671,905.00

Land Cost		$ -
Engineering and Construction Mgt	10%	$ 167,190.50
Operation and Maintenance	10%	$ 167,190.50
Subtotal		$ 2,006,286.00
Contingency	25%	$ 501,571.50
Total Costs		$ 2,507,857.50

Opinion of Probable Construction Cost

FRS Structural Alternative No. 1 - Segmentation

Item	Description	Unit	Quantity	Unit Cost	Total
1	Embankment fill	CY	39,926	$ 7.00	$ 279,482.00
2	Culvert RCP 6 ft diam	LF	128	$ 175.00	$ 22,400.00
3	Headwall w/gates	L SUM	1	$ 13,000.00	$ 13,000.00
4	Outlet Headwall	L SUM	1	$ 3,500.00	$ 3,500.00
5	Guardrail	LF	4,000	$ 10.00	$ 40,000.00
				Total	$ 358,382.00

Opinion of Probable Construction Cost

FRS Structural Alternative No. 1 - Segmentation

Item	Description	Unit	Quantity	Unit Cost	Total
1	Embankment fill	CY	80,656	$ 7.00	$ 564,592.00
2	Culvert RCP 6 ft diam	LF	220	$ 175.00	$ 38,500.00
3	Headwall w/gates	L SUM	1	$ 13,000.00	$ 13,000.00
4	Outlet Headwall	L SUM	1	$ 3,500.00	$ 3,500.00
5	Guardrail	LF	5,800	$ 10.00	$ 58,000.00
				Total	$ 677,592.00

Kimley-Horn and Associates, Inc.

Job STRUCTURES Subject ARTS ANALYSIS Sheet No. ____ of ____ Job No. 091131003

Designed by ____ Date ____ Checked by ____ Date ____

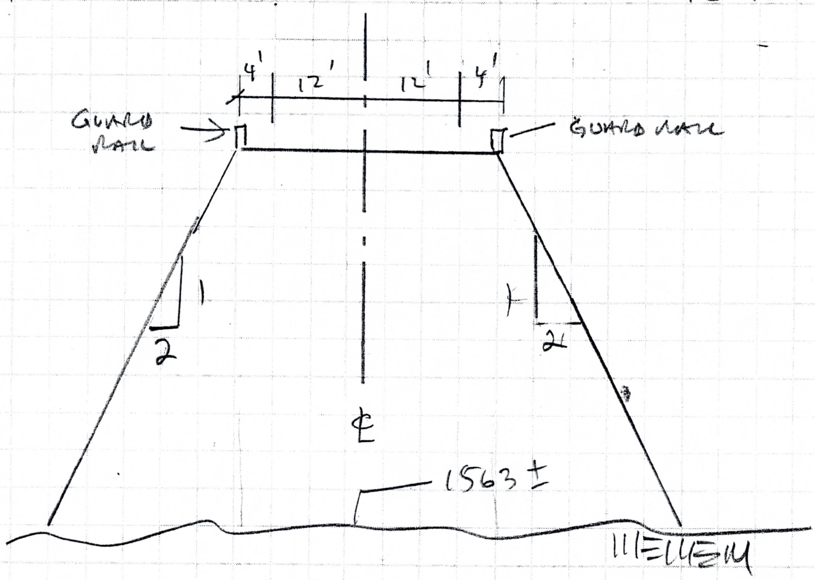
SEGMENTATION

POWERLINE FRS

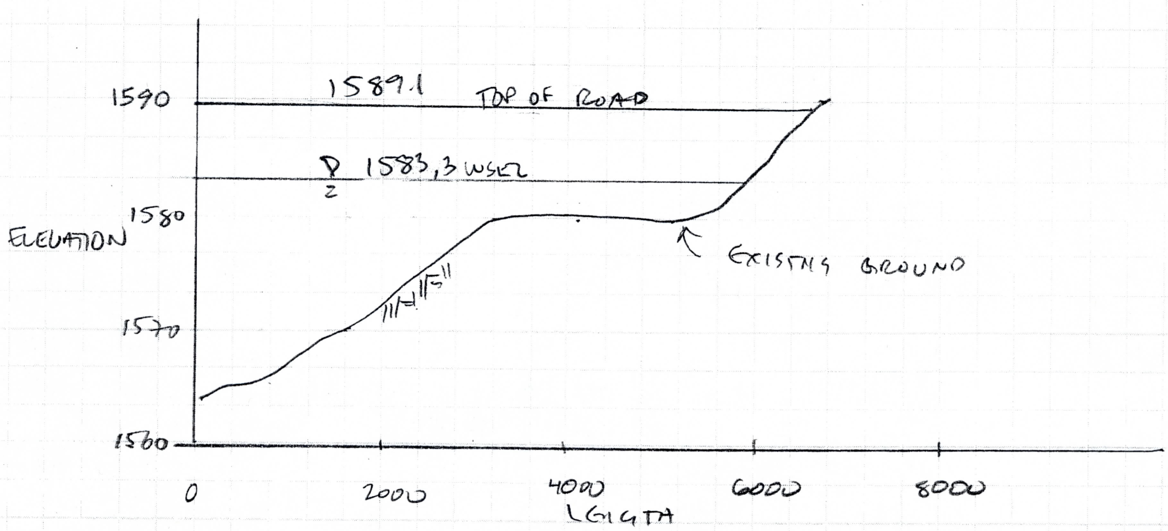
POOL/CREST ELEVATION = 1583.3 - ft
TOP OF DAM = 1589.1 - ft

Δ 5.8 - ft

ASSUME TOP OF ROAD ELEVATION = 1589.1 - ft
LENGTH OF ROAD SEGMENT = 6,600 - ft
TWO - LANE (ONE LANE EAST/WEST) 12 - ft LANE WIDTH
FOUR FOOT SHOULDERS 2:1 H:V
TOP WIDTH = 12+12+4+4 = 32 ft



EMBANKMENT SECTION



PROFILE

Kimley-Horn and Associates, Inc.

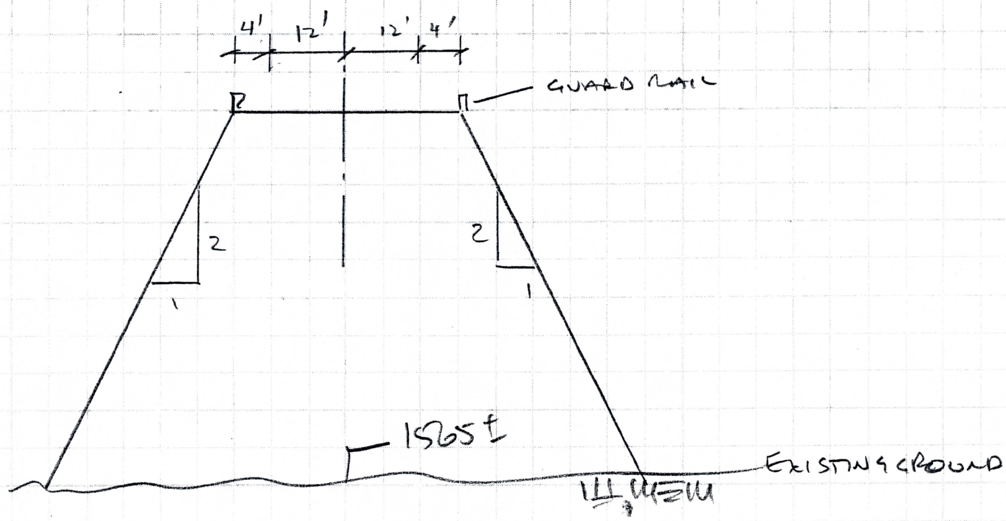
Job STRUCTURES Subject ALTS ANALYSIS Sheet No. ____ of ____ Job No. 091131003

Designed by ________ Date ________ Checked by ________ Date ________

VINEYARD ROAD FRS

POOL/CREST ELEVATION = 1574.8-FT
TOP OF DAM = 1579.5-FT
Δ = 4.7 FT

ASSUME TOP OF ROAD SEGMENT = 1579.5-FT
LENGTH OF ROAD SEGMENT = 2000-FT
ASSUME TWO LANE W/ 12-FT LANE WIDTHS FOUR FOOT SHOULDERS
TOP WIDTH = 32-FT



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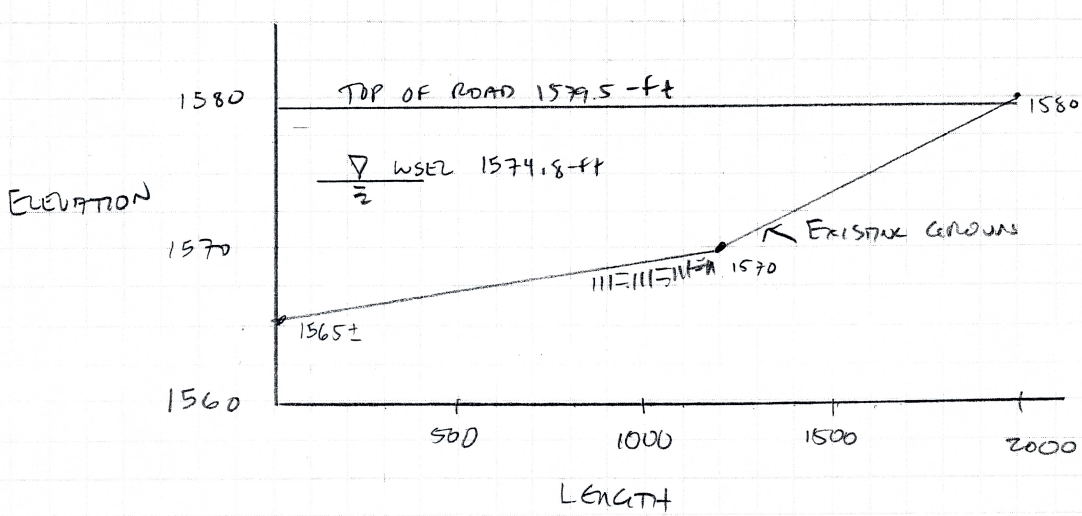

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Sheet No. ____ of ____

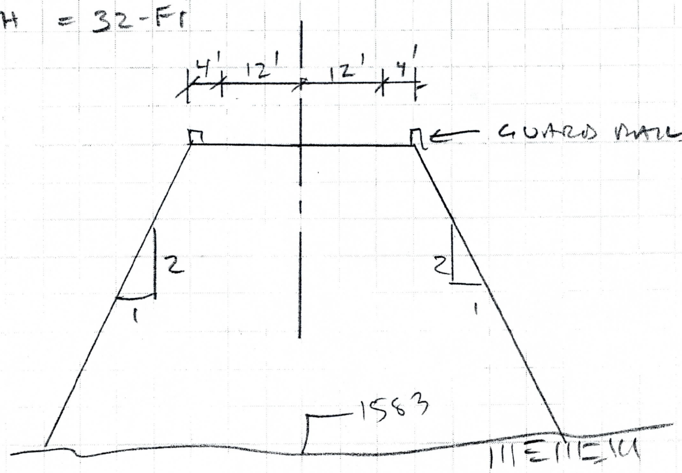
Job STRUCTURES Subject ALTS ANALYSIS Job No. 091131003

Designed by ____ Date ____ Checked by ____ Date ____

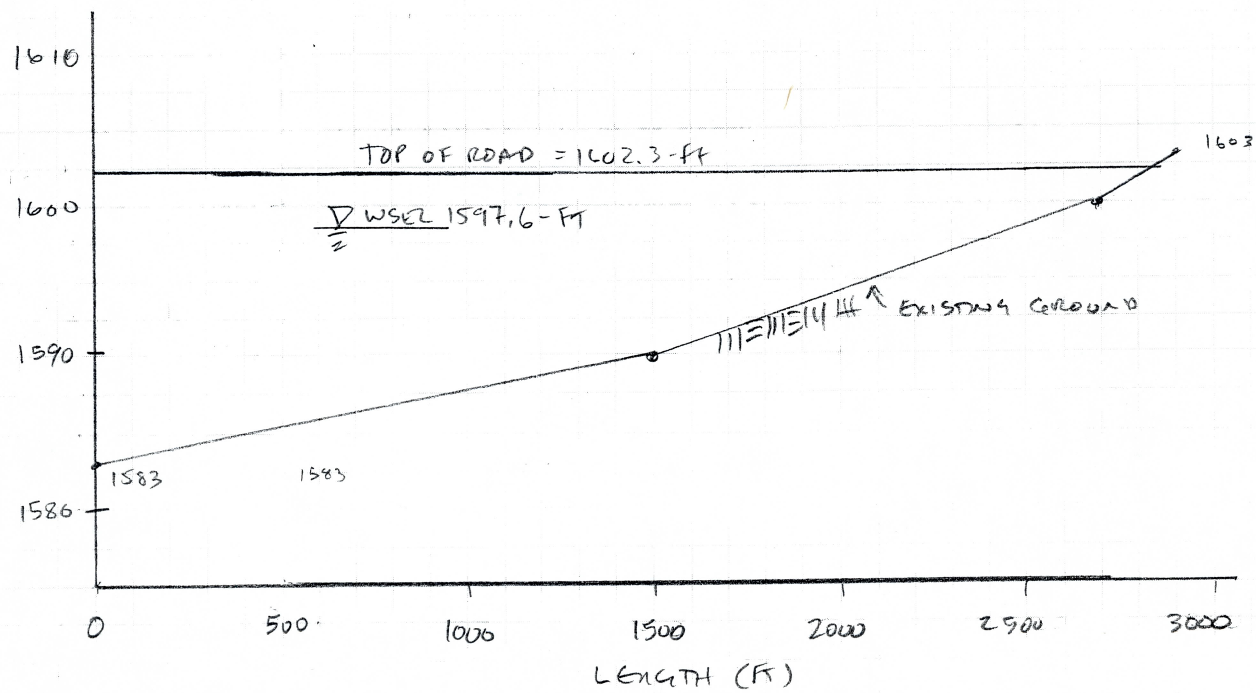
RITTENHOUSE FRS

POOL/CREST ELEVATION = 1597.6 -ft
TOP OF DAM = 1602.3 -ft
Δ = 4.7 -ft

ASSUME TOP OF ROAD SEGMENT = 1602.3 -ft
LENGTH OF ROAD SEGMENT = 2,900-Ft
ASSUME TWO LANE W/12-FT LANES 4-FT SHOULDERS 2:1 H:V
TOP WIDTH = 32-FT



EMBANKMENT SECTION



PROFILE

Kimley-Horn and Associates, Inc.

Sheet No. ______ of ______

Job ______ Subject ______ Job No. ______

Designed by ______ Date ______ Checked by ______ Date ______

Low Flow Culverts Powerline, Vineyard Road, Rittenhouse FRS

Powerline CULVERT SLOPE = 0.00016 ft/ft (match low-flow channel)

TOP OF ROAD = 1589.1 ft
LENGTH OF SEGMENT = 6,600 ft
CREST WIDTH = 32 ft paved

1589.1 − 1563 = 26.1 ft 26.1 ft × 2 =

CULVERT LENGTH = 2(26.1) × 2 + 32 = 136 ft ±

U/S invert 1563 + (0.00016)(68) = 1563.01
D/S invert 1563 − (0.00016)(68) = 1562.98

Q = 360 cfs

Vineyard Road Culvert slope = 0.00004 ft/ft (match low-flow channel)

TOP OF ROAD = 1579.5
LENGTH OF SEGMENT = 2000-ft
CREST WIDTH = 32 ft paved

1589.1 − 1565 = 24.1 ft

CULVERT LENGTH = 2(24.1) × 2 + 32 = 128 ft
U/S invert = 1565 + (0.00004)(64) = 1565.0026
D/S invert = 1565 − (0.00004)(64) = 1564.99
Q = 138 cfs

RITTENHOUSE CULVERT SLOPE = 0.00012 ft/ft (match low-flow channel)

TOP OF ROAD = 1602.3
LENGTH OF SEGMENT = 2,900-ft
CREST WIDTH = 32 ft

1602.3 − 1583 = 19.3 ft
CULVERT LENGTH = 2(19.3) × 2 + 32 = 109.2 use 110 ft

U/S invert = 1583 + (0.00012)(55) = 1583.0066
D/S invert = 158 − (0.00012)(55) = 1582.99

Q = 310 cfs

CURRENT DATE: 07-24-2000 FILE DATE: 07-24-2000
CURRENT TIME: 17:28:27 FILE NAME: P-LOW

FHWA CULVERT ANALYSIS
HY-8, VERSION 6.0

SITE DATA				CULVERT SHAPE, MATERIAL, INLET				
C V NO.	INLET ELEV. (ft)	OUTLET ELEV. (ft)	CULVERT LENGTH (ft)	BARRELS SHAPE MATERIAL	SPAN (ft)	RISE (ft)	MANNING n	INLET TYPE
1	1563.01	1562.98	136.00	2 RCP	6.00	6.00	.012	CONVENTIONAL
2								
3								
4								
5								
6								

SUMMARY OF CULVERT FLOWS (cfs) FILE: P-LOW DATE: 07-24-2000

ELEV (ft)	TOTAL	1	2	3	4	5	6	ROADWAY	ITR
1563.01	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1564.68	36.0	36.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1565.39	72.0	72.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1565.96	108.0	108.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1566.46	144.0	144.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1566.92	180.0	180.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1567.35	216.0	216.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1567.76	252.0	252.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1568.16	288.0	288.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1568.54	324.0	324.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1568.93	360.0	360.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1589.10	1357.4	1357.4	0.0	0.0	0.0	0.0	0.0	OVERTOPPING	

SUMMARY OF ITERATIVE SOLUTION ERRORS FILE: P-LOW DATE: 07-24-2000

HEAD ELEV (ft)	HEAD ERROR (ft)	TOTAL FLOW (cfs)	FLOW ERROR (cfs)	% FLOW ERROR
1563.01	0.000	0.00	0.00	0.00
1564.68	0.000	36.00	0.00	0.00
1565.39	0.000	72.00	0.00	0.00
1565.96	0.000	108.00	0.00	0.00
1566.46	0.000	144.00	0.00	0.00
1566.92	0.000	180.00	0.00	0.00
1567.35	0.000	216.00	0.00	0.00
1567.76	0.000	252.00	0.00	0.00
1568.16	0.000	288.00	0.00	0.00
1568.54	0.000	324.00	0.00	0.00
1568.93	0.000	360.00	0.00	0.00

<1> TOLERANCE (ft) = 0.010 <2> TOLERANCE (%) = 1.000

CURRENT DATE: 07-24-2000 FILE DATE: 07-24-2000
CURRENT TIME: 17:28:27 FILE NAME: P-LOW

PERFORMANCE CURVE FOR CULVERT 1 - 2(6.00 (ft) BY 6.00 (ft)) RCP

DIS-CHARGE	HEAD-WATER ELEV. (ft)	INLET CONTROL DEPTH (ft)	OUTLET CONTROL DEPTH (ft)	FLOW TYPE <F4>	NORMAL DEPTH (ft)	CRIT. DEPTH (ft)	OUTLET DEPTH (ft)	TW DEPTH (ft)	OUTLET VEL. (fps)	TW VEL. (fps)
0.00	1563.01	0.00	-0.03	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
36.00	1564.68	1.24	1.67	3-M2t	2.14	1.09	1.19	1.19	4.52	0.58
72.00	1565.39	1.98	2.38	3-M2t	3.19	1.56	1.80	1.80	5.05	0.76
108.00	1565.96	2.60	2.95	3-M2t	4.18	1.94	2.29	2.29	5.43	0.88
144.00	1566.46	3.13	3.45	3-M2t	6.00	2.25	2.72	2.72	5.77	0.98
180.00	1566.92	3.60	3.91	3-M2t	6.00	2.54	3.10	3.10	6.10	1.06
216.00	1567.35	4.01	4.34	3-M2t	6.00	2.79	3.46	3.46	6.40	1.13
252.00	1567.76	4.38	4.75	3-M2t	6.00	3.03	3.79	3.79	6.70	1.19
288.00	1568.16	4.73	5.15	3-M2t	6.00	3.24	4.10	4.10	7.00	1.25
324.00	1568.54	5.07	5.53	3-M2t	6.00	3.45	4.39	4.39	7.32	1.30
360.00	1568.93	5.40	5.92	3-M2t	6.00	3.65	4.68	4.68	7.63	1.35

El. inlet face invert 1563.01 ft El. outlet invert 1562.98 ft
El. inlet throat invert 0.00 ft El. inlet crest 0.00 ft

***** SITE DATA ***** CULVERT INVERT **************

INLET STATION	0.00 ft
INLET ELEVATION	1563.01 ft
OUTLET STATION	136.00 ft
OUTLET ELEVATION	1562.98 ft
NUMBER OF BARRELS	2
SLOPE (V/H)	0.0002
CULVERT LENGTH ALONG SLOPE	136.00 ft

***** CULVERT DATA SUMMARY ************************

BARREL SHAPE	CIRCULAR
BARREL DIAMETER	6.00 ft
BARREL MATERIAL	CONCRETE
BARREL MANNING'S n	0.012
INLET TYPE	CONVENTIONAL
INLET EDGE AND WALL	BEVELED EDGE (1:1)
INLET DEPRESSION	NONE

CURRENT DATE: 07-24-2000
CURRENT TIME: 17:28:27

FILE DATE: 07-24-2000
FILE NAME: P-LOW

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** REGULAR CHANNEL CROSS SECTION ****************

BOTTOM WIDTH	50.00 ft
SIDE SLOPE H/V (X:1)	1.5
CHANNEL SLOPE V/H (ft/ft)	0.000
MANNING'S n (.01-0.1)	0.035
CHANNEL INVERT ELEVATION	1562.98 ft
CULVERT NO.1 OUTLET INVERT ELEVATION	1562.98 ft

******* UNIFORM FLOW RATING CURVE FOR DOWNSTREAM CHANNEL

FLOW (cfs)	W.S.E. (ft)	FROUDE NUMBER	DEPTH (ft)	VEL. (f/s)	SHEAR (psf)
0.00	1562.98	0.000	0.00	0.00	0.00
36.00	1564.17	0.094	1.19	0.58	0.01
72.00	1564.78	0.100	1.80	0.76	0.02
108.00	1565.27	0.103	2.29	0.88	0.02
144.00	1565.70	0.105	2.72	0.98	0.03
180.00	1566.08	0.106	3.10	1.06	0.03
216.00	1566.44	0.107	3.46	1.13	0.04
252.00	1566.77	0.108	3.79	1.19	0.04
288.00	1567.08	0.109	4.10	1.25	0.04
324.00	1567.37	0.110	4.39	1.30	0.04
360.00	1567.66	0.110	4.68	1.35	0.05

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ROADWAY SURFACE	PAVED
EMBANKMENT TOP WIDTH	32.00 ft
CREST LENGTH	6600.00 ft
OVERTOPPING CREST ELEVATION	1589.10 ft

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CURRENT DATE: 07-25-2000 FILE DATE: 07-25-2000
CURRENT TIME: 07:47:15 FILE NAME: VINLOW

FHWA CULVERT ANALYSIS
HY-8, VERSION 6.0

L V NO.	SITE DATA: INLET ELEV. (ft)	OUTLET ELEV. (ft)	CULVERT LENGTH (ft)	CULVERT SHAPE, MATERIAL, INLET: BARRELS SHAPE MATERIAL	SPAN (ft)	RISE (ft)	MANNING n	INLET TYPE
1	1565.00	1564.99	128.00	1 RCP	6.00	6.00	.012	CONVENTIONAL
2								
3								
4								
5								
6								

SUMMARY OF CULVERT FLOWS (cfs) FILE: VINLOW DATE: 07-25-2000

ELEV (ft)	TOTAL	1	2	3	4	5	6	ROADWAY	ITR
1565.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1566.47	14.0	14.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1567.07	28.0	28.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1567.55	42.0	42.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1567.96	56.0	56.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1568.33	70.0	70.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1568.64	84.0	84.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1568.99	98.0	98.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1569.30	112.0	112.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1569.56	126.0	126.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1569.88	140.0	140.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1579.50	487.5	487.5	0.0	0.0	0.0	0.0	0.0	OVERTOPPING	

SUMMARY OF ITERATIVE SOLUTION ERRORS FILE: VINLOW DATE: 07-25-2000

HEAD ELEV (ft)	HEAD ERROR (ft)	TOTAL FLOW (cfs)	FLOW ERROR (cfs)	% FLOW ERROR
1565.00	0.000	0.00	0.00	0.00
1566.47	0.000	14.00	0.00	0.00
1567.07	0.000	28.00	0.00	0.00
1567.55	0.000	42.00	0.00	0.00
1567.96	0.000	56.00	0.00	0.00
1568.33	0.000	70.00	0.00	0.00
1568.64	0.000	84.00	0.00	0.00
1568.99	0.000	98.00	0.00	0.00
1569.30	0.000	112.00	0.00	0.00
1569.56	0.000	126.00	0.00	0.00
1569.88	0.000	140.00	0.00	0.00

<1> TOLERANCE (ft) = 0.010 <2> TOLERANCE (%) = 1.000

CURRENT DATE: 07-25-2000 FILE DATE: 07-25-2000
CURRENT TIME: 07:47:15 FILE NAME: VINLOW

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PERFORMANCE CURVE FOR CULVERT 1 - 1(6.00 (ft) BY 6.00 (ft)) RCP

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DIS- [illegible]RGE [illegible]W [illegible]s)	HEAD- WATER ELEV. (ft)	INLET CONTROL DEPTH (ft)	OUTLET CONTROL DEPTH (ft)	FLOW TYPE <F4>	NORMAL DEPTH (ft)	CRIT. DEPTH (ft)	OUTLET DEPTH (ft)	TW DEPTH (ft)	OUTLET VEL. (fps)	TW VEL. (fps)
0.00	1565.00	0.00	-0.01	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
14.00	1566.47	1.06	1.47	2-M2c	2.26	0.93	0.93	0.78	4.84	0.35
28.00	1567.07	1.67	2.07	2-M2c	3.38	1.37	1.37	1.18	5.72	0.46
42.00	1567.55	2.19	2.55	2-M2c	4.53	1.70	1.70	1.50	6.34	0.54
56.00	1567.96	2.66	2.96	2-M2c	6.00	1.97	1.97	1.78	6.89	0.60
70.00	1568.33	3.08	3.33	2-M2c	6.00	2.22	2.22	2.04	7.35	0.65
84.00	1568.64	3.45	3.64	2-M2c	6.00	2.45	2.45	2.27	7.73	0.69
98.00	1568.99	3.78	3.99	2-M2c	6.00	2.65	2.65	2.49	8.14	0.73
112.00	1569.30	4.09	4.30	2-M2c	6.00	2.84	2.84	2.69	8.48	0.77
126.00	1569.56	4.38	4.56	2-M2c	6.00	3.03	3.03	2.89	8.79	0.80
140.00	1569.88	4.66	4.88	2-M2c	6.00	3.20	3.20	3.07	9.14	0.83

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El. inlet face invert 1565.00 ft El. outlet invert 1564.99 ft
El. inlet throat invert 0.00 ft El. inlet crest 0.00 ft

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***** SITE DATA ***** CULVERT INVERT **************

INLET STATION	0.00 ft
INLET ELEVATION	1565.00 ft
OUTLET STATION	128.00 ft
OUTLET ELEVATION	1564.99 ft
NUMBER OF BARRELS	1
SLOPE (V/H)	0.0001
CULVERT LENGTH ALONG SLOPE	128.00 ft

***** CULVERT DATA SUMMARY ************************

BARREL SHAPE	CIRCULAR
BARREL DIAMETER	6.00 ft
BARREL MATERIAL	CONCRETE
BARREL MANNING'S n	0.012
INLET TYPE	CONVENTIONAL
INLET EDGE AND WALL	BEVELED EDGE (1:1)
INLET DEPRESSION	NONE

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CURRENT DATE: 07-25-2000
CURRENT TIME: 07:47:15

FILE DATE: 07-25-2000
FILE NAME: VINLOW

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******* REGULAR CHANNEL CROSS SECTION ****************

BOTTOM WIDTH	50.00 ft
SIDE SLOPE H/V (X:1)	1.5
CHANNEL SLOPE V/H (ft/ft)	0.000
MANNING'S n (.01-0.1)	0.035
CHANNEL INVERT ELEVATION	1564.99 ft
CULVERT NO.1 OUTLET INVERT ELEVATION	1564.99 ft

******* UNIFORM FLOW RATING CURVE FOR DOWNSTREAM CHANNEL

FLOW (cfs)	W.S.E. (ft)	FROUDE NUMBER	DEPTH (ft)	VEL. (f/s)	SHEAR (psf)
0.00	1564.99	0.000	0.00	0.00	0.00
14.00	1565.77	0.070	0.78	0.35	0.00
28.00	1566.17	0.075	1.18	0.46	0.01
42.00	1566.49	0.077	1.50	0.54	0.01
56.00	1566.77	0.079	1.78	0.60	0.01
70.00	1567.03	0.080	2.04	0.65	0.01
84.00	1567.26	0.081	2.27	0.69	0.01
98.00	1567.48	0.082	2.49	0.73	0.02
112.00	1567.68	0.083	2.69	0.77	0.02
126.00	1567.88	0.083	2.89	0.80	0.02
140.00	1568.06	0.084	3.07	0.83	0.02

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ROADWAY SURFACE	PAVED
EMBANKMENT TOP WIDTH	32.00 ft
CREST LENGTH	2000.00 ft
OVERTOPPING CREST ELEVATION	1579.50 ft

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CURRENT DATE: 07-25-2000　　FILE DATE: 07-25-2000
CURRENT TIME: 07:49:39　　FILE NAME: RITLOW

FHWA CULVERT ANALYSIS
HY-8, VERSION 6.0

SITE DATA				CULVERT SHAPE, MATERIAL, INLET				
C V NO.	INLET ELEV. (ft)	OUTLET ELEV. (ft)	CULVERT LENGTH (ft)	BARRELS SHAPE MATERIAL	SPAN (ft)	RISE (ft)	MANNING n	INLET TYPE
1	1583.01	1582.99	110.00	2 RCP	6.00	6.00	.012	CONVENTIONAL
2								
3								
4								
5								
6								

SUMMARY OF CULVERT FLOWS (cfs)　　FILE: RITLOW　　DATE: 07-25-2000

ELEV (ft)	TOTAL	1	2	3	4	5	6	ROADWAY	ITR
1583.01	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1584.56	31.0	31.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1585.23	62.0	62.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1585.77	93.0	93.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1586.26	124.0	124.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1586.69	155.0	155.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1587.10	186.0	186.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1587.49	217.0	217.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1587.86	248.0	248.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1588.22	279.0	279.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1588.58	310.0	310.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1602.30	1063.8	1063.8	0.0	0.0	0.0	0.0	0.0	OVERTOPPING	

SUMMARY OF ITERATIVE SOLUTION ERRORS　　FILE: RITLOW　　DATE: 07-25-2000

HEAD ELEV (ft)	HEAD ERROR (ft)	TOTAL FLOW (cfs)	FLOW ERROR (cfs)	% FLOW ERROR
1583.01	0.000	0.00	0.00	0.00
1584.56	0.000	31.00	0.00	0.00
1585.23	0.000	62.00	0.00	0.00
1585.77	0.000	93.00	0.00	0.00
1586.26	0.000	124.00	0.00	0.00
1586.69	0.000	155.00	0.00	0.00
1587.10	0.000	186.00	0.00	0.00
1587.49	0.000	217.00	0.00	0.00
1587.86	0.000	248.00	0.00	0.00
1588.22	0.000	279.00	0.00	0.00
1588.58	0.000	310.00	0.00	0.00

<1> TOLERANCE (ft) = 0.010　　<2> TOLERANCE (%) = 1.000

CURRENT DATE: 07-25-2000
CURRENT TIME: 07:49:39

FILE DATE: 07-25-2000
FILE NAME: RITLOW

PERFORMANCE CURVE FOR CULVERT 1 - 2(6.00 (ft) BY 6.00 (ft)) RCP

DIS- CHARGE	HEAD- WATER ELEV. (ft)	INLET CONTROL DEPTH (ft)	OUTLET CONTROL DEPTH (ft)	FLOW TYPE <F4>	NORMAL DEPTH (ft)	CRIT. DEPTH (ft)	OUTLET DEPTH (ft)	TW DEPTH (ft)	OUTLET VEL. (fps)	TW VEL. (fps)
0.00	1583.01	0.00	-0.02	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
31.00	1584.56	1.13	1.55	3-M2t	1.98	0.99	1.19	1.19	3.91	0.50
62.00	1585.23	1.78	2.22	3-M2t	2.91	1.44	1.79	1.79	4.36	0.66
93.00	1585.77	2.35	2.76	3-M2t	3.76	1.81	2.29	2.29	4.69	0.76
124.00	1586.26	2.84	3.25	3-M2t	4.70	2.08	2.71	2.71	4.99	0.85
155.00	1586.69	3.28	3.68	3-M2t	6.00	2.35	3.09	3.09	5.27	0.92
186.00	1587.10	3.67	4.09	3-M2t	6.00	2.58	3.45	3.45	5.53	0.98
217.00	1587.49	4.02	4.48	3-M2t	6.00	2.79	3.78	3.78	5.79	1.03
248.00	1587.86	4.34	4.85	3-M2t	6.00	3.01	4.09	4.09	6.05	1.08
279.00	1588.22	4.65	5.21	3-M2t	6.00	3.19	4.38	4.38	6.32	1.13
310.00	1588.58	4.94	5.57	3-M2t	6.00	3.37	4.66	4.66	6.59	1.17

El. inlet face invert 1583.01 ft El. outlet invert 1582.99 ft
El. inlet throat invert 0.00 ft El. inlet crest 0.00 ft

***** SITE DATA ***** CULVERT INVERT **************

INLET STATION	0.00 ft
INLET ELEVATION	1583.01 ft
OUTLET STATION	110.00 ft
OUTLET ELEVATION	1582.99 ft
NUMBER OF BARRELS	2
SLOPE (V/H)	0.0002
CULVERT LENGTH ALONG SLOPE	110.00 ft

***** CULVERT DATA SUMMARY *************************

BARREL SHAPE	CIRCULAR
BARREL DIAMETER	6.00 ft
BARREL MATERIAL	CONCRETE
BARREL MANNING'S n	0.012
INLET TYPE	CONVENTIONAL
INLET EDGE AND WALL	BEVELED EDGE (1:1)
INLET DEPRESSION	NONE

CURRENT DATE: 07-25-2000
CURRENT TIME: 07:49:39

FILE DATE: 07-25-2000
FILE NAME: RITLOW

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* REGULAR CHANNEL CROSS SECTION ****************

BOTTOM WIDTH	50.00 ft
SIDE SLOPE H/V (X:1)	1.5
CHANNEL SLOPE V/H (ft/ft)	0.000
MANNING'S n (.01-0.1)	0.035
CHANNEL INVERT ELEVATION	1582.99 ft
CULVERT NO.1 OUTLET INVERT ELEVATION	1582.99 ft

******* UNIFORM FLOW RATING CURVE FOR DOWNSTREAM CHANNEL

FLOW (cfs)	W.S.E. (ft)	FROUDE NUMBER	DEPTH (ft)	VEL. (f/s)	SHEAR (psf)
0.00	1582.99	0.000	0.00	0.00	0.00
31.00	1584.18	0.082	1.19	0.50	0.01
62.00	1584.78	0.086	1.79	0.66	0.01
93.00	1585.28	0.089	2.29	0.76	0.02
124.00	1585.70	0.091	2.71	0.85	0.02
155.00	1586.08	0.092	3.09	0.92	0.02
186.00	1586.44	0.093	3.45	0.98	0.03
217.00	1586.77	0.094	3.78	1.03	0.03
248.00	1587.08	0.094	4.09	1.08	0.03
279.00	1587.37	0.095	4.38	1.13	0.03
310.00	1587.65	0.095	4.66	1.17	0.04

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ROADWAY SURFACE	PAVED
EMBANKMENT TOP WIDTH	32.00 ft
CREST LENGTH	2900.00 ft
OVERTOPPING CREST ELEVATION	1602.30 ft

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Powerline FRS Low Flow Channel
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	Powerline Low-Flow C
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Discharge

Input Data		
Mannings Coeffic	0.035	
Slope	000160	ft/ft
Depth	4.00	ft
Left Side Slope	1.50	H : V
Right Side Slope	15.00	H : V
Bottom Width	50.00	ft

Results		
Discharge	356.65	cfs
Flow Area	332.0	ft²
Wetted Perim	117.34	ft
Top Width	116.00	ft
Critical Depth	1.09	ft
Critical Slope	0.018298	ft/ft
Velocity	1.07	ft/s
Velocity Head	0.02	ft
Specific Energ	4.02	ft
Froude Numb	0.11	
Flow Type	Subcritical	

Vineyard Road FRS Low-Flow Channel
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	Vineyard Road FRS Low-Flc
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Discharge

Input Data		
Mannings Coeffic	0.035	
Slope	000040	ft/ft
Depth	4.00	ft
Left Side Slope	1.50	H : V
Right Side Slope	1.50	H : V
Bottom Width	50.00	ft

Results		
Discharge	138.04	cfs
Flow Area	224.0	ft²
Wetted Perim	64.42	ft
Top Width	62.00	ft
Critical Depth	0.61	ft
Critical Slope	0.021320	ft/ft
Velocity	0.62	ft/s
Velocity Head	0.01	ft
Specific Energ	4.01	ft
Froude Numb	0.06	
Flow Type	Subcritical	

Rittenhouse FRS Low-Flow Channel
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	Rittenhouse FRS Low-flow
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Discharge

Input Data		
Mannings Coeffic	0.035	
Slope	000120	ft/ft
Depth	4.00	ft
Left Side Slope	1.50	H : V
Right Side Slope	15.00	H : V
Bottom Width	50.00	ft

Results		
Discharge	308.87	cfs
Flow Area	332.0	ft²
Wetted Perim	117.34	ft
Top Width	116.00	ft
Critical Depth	1.00	ft
Critical Slope	0.018786	ft/ft
Velocity	0.93	ft/s
Velocity Head	0.01	ft
Specific Energ	4.01	ft
Froude Numb	0.10	
Flow Type	Subcritical	

Kimley-Horn and Associates
37 Brookside Road Waterbury, CT 06708 USA (203) 755-1666

ALTERNATIVES ANALYSIS REPORT

Appendix F: Powerline, Vineyard Road, and Rittenhouse FRS
Structural Alternative No. 2:
Upgrade to High Hazard Dam

Opinion of Probable Construction Cost

FRS Structural Alternative No. 2 - High Hazard Dam

Item	Description	Unit	Quantity	Unit Cost	Total
1	Embankment Fill	CY	260,720	$ 7.00	$ 1,825,040.00
2	Spillway Grading	SY	34,623	$ 6.00	$ 207,738.00
3					$ -
4					$ -
	* item 2 includes widening approach channel and control section				$ -
				Total	$ 2,032,778.00

Land Cost		$ -
Engineering and Construction Mgt	10%	$ 203,277.80
Operation and Maintenance	10%	$ 203,277.80
Subtotal		$ 2,439,333.60
Contingency	25%	$ 609,833.40
Total Costs		$ 3,049,167.00

Opinion of Probable Construction Cost

FRS Structural Alternative No. 2 - High Hazard Dam

Item	Description	Unit	Quantity	Unit Cost	Total
1	Embankment Fill	CY	516,608	$ 7.00	$ 3,616,256.00
2	Spillway Grading	SY	35,784	$ 6.00	$ 214,704.00
3					$ -
4					$ -
	* item 2 includes widening approach channel and control section				$ -
				Total	$ 3,830,960.00

Land Cost		$ -
Engineering and Construction Mgt	10%	$ 383,096.00
Operation and Maintenance	10%	$ 383,096.00
Subtotal		$ 4,597,152.00
Contingency	25%	$ 1,149,288.00
Total Costs		$ 5,746,440.00

Opinion of Probable Construction Cost

FRS Structural Alternative No. 2 - High Hazard Dam

Item	Description	Unit	Quantity	Unit Cost	Total
1	Embankment Fill	CY	226,171	$ 7.00	$ 1,583,197.00
2	Spillway Grading	SY	34,623	$ 6.00	$ 207,738.00
3					$ -
4					$ -
	* item 2 includes widening approach channel and control section				$ -
				Construction	$ 1,790,935.00

Land Cost			$ -
Engineering and Construction Mgt		10%	$ 179,093.50
Operation and Maintenance		10%	$ 179,093.50
	Subtotal		$ 2,149,122.00
Contingency		25%	$ 537,280.50
Total Costs			$ 2,686,402.50

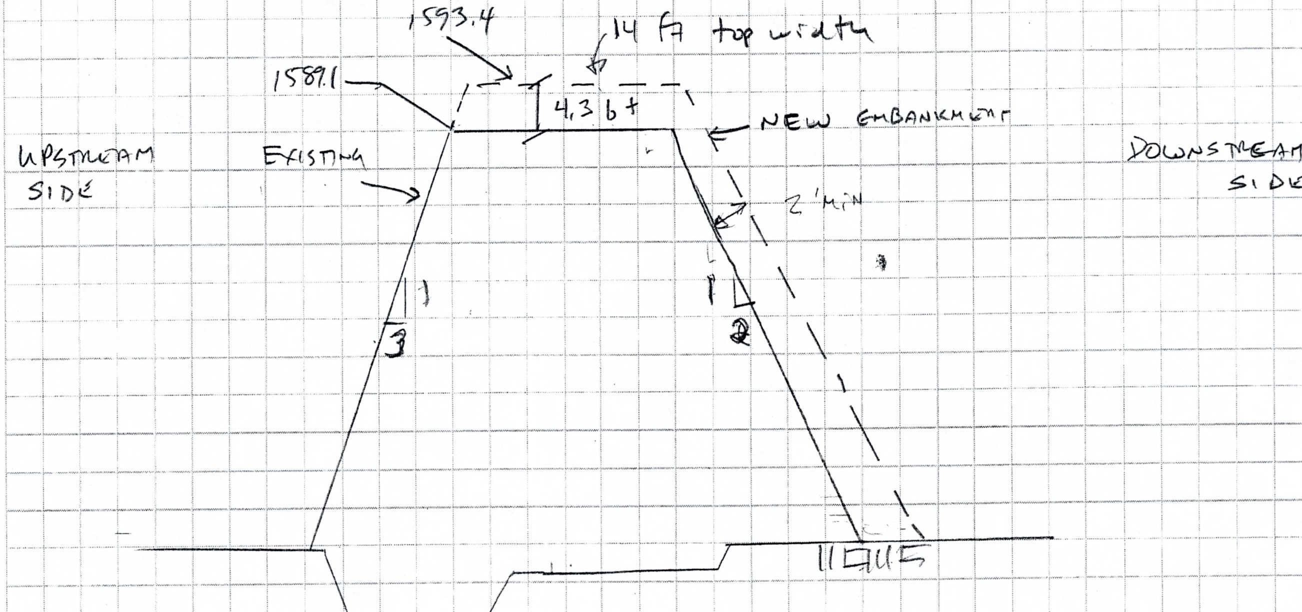
UpgradeHighHazardDam.xls/Rittenhouse

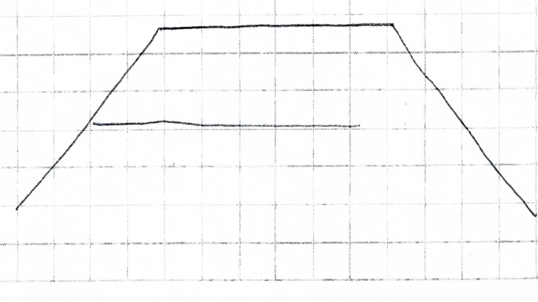
HIGH HAZARD DAM

POWERLINE FRS

	EXISTING	PROPOSED	Δ
TOP OF DAM	1589.1	1593.4	4.3 ft
LENGTH OF SPILLWAY (ft)	600 →	900.	300 ft
LENGTH OF DAM	2.54 miles or 13,411 feet		

TYPICAL EXISTING EMBANKMENT SECTION





Kimley-Horn and Associates, Inc.

Job STRUCTURES Subject POWERLINE
Designed by Date 8-30-08 Checked by Date
Sheet No. ___ of ___
Job No. 091131003

(A) $\frac{1}{2}(4.36 \times 13.08) = 28.5$ SF

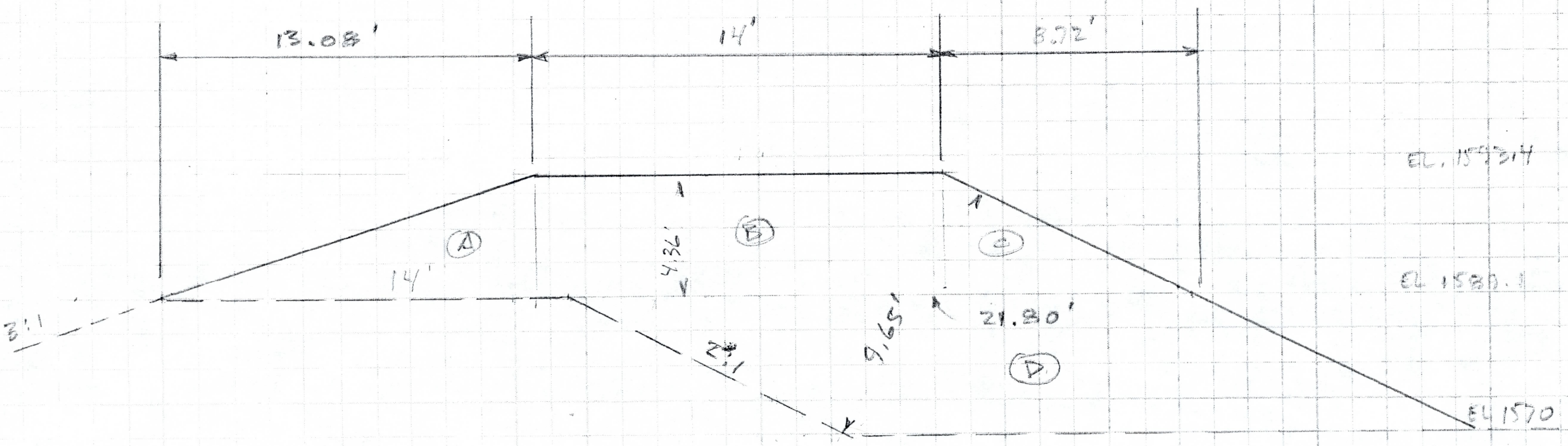
(B) $4.36 \times 14 = 61.0$

(C) $\frac{1}{2}(4.36 \times 8.72) = 19.0$

(D) $(89.1 - 70) \times 21.80 = 416.4$

524.90 SF/LF

$524.90 \times 13{,}411 = 7{,}039{,}434$ CF
$= 260{,}720$ CY



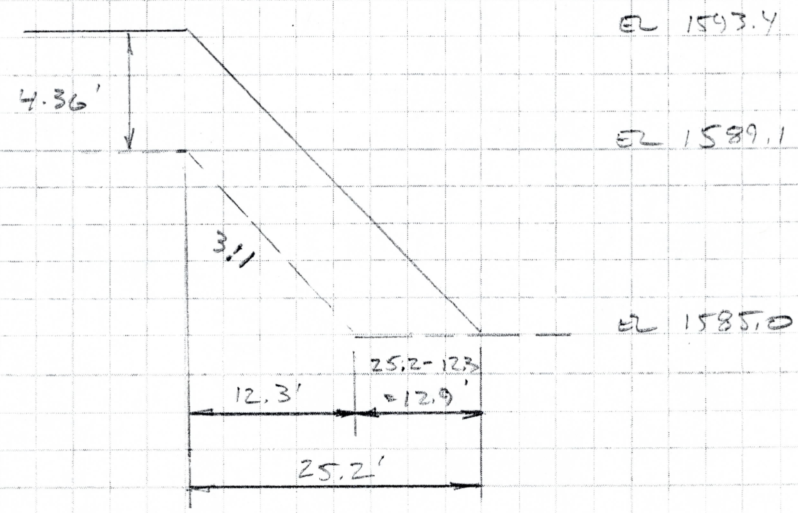
Kimley-Horn
and Associates, Inc.

Sheet No. ____ of ____

Job STRUCTURES Subject ____ Job No. 091131003

Designed by R B[illegible] Date 8.30.00 Checked by ____ Date ____

POWER LINE



WIDTH 900' LENGTH 12.9'

900 x 12.9 = 11,610 SF

WIDTH 300' LENGTH 1000'

300 x 1000 = 300,000 SF

311,610 SF

= 34,623 SY

Kimley-Horn and Associates, Inc.

Sheet No. ______ of ______

Job STRUCTURES Subject ALT3 ANALYSIS Job No. 091131007

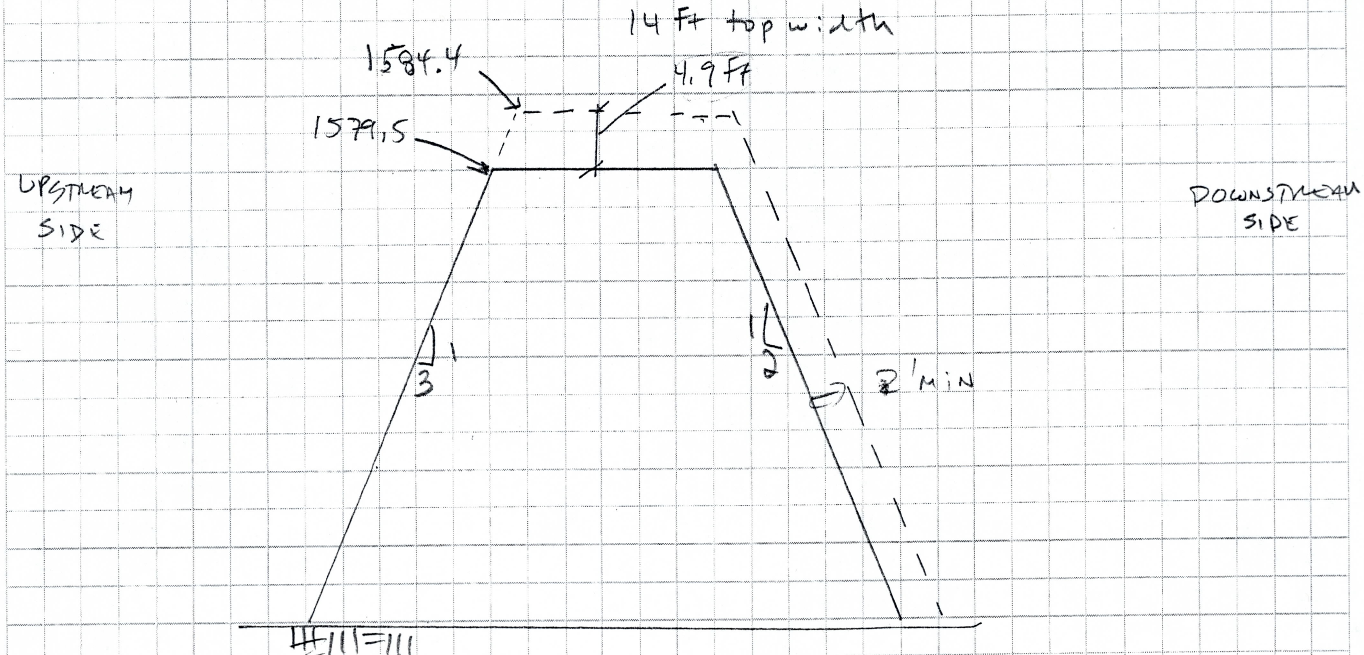
Designed by ________ Date ________ Checked by ________ Date ________

HIGH HAZARD DAM

VINEYARD FRS

	EXISTING	PROPOSED	Δ
TOP OF DAM (ft)	1579.5	1584.4	4.9 ft
LENGTH OF SPILLWAY (ft)	600	900	300
LENGTH OF DAM (5.46 mi)	28,828 ft		

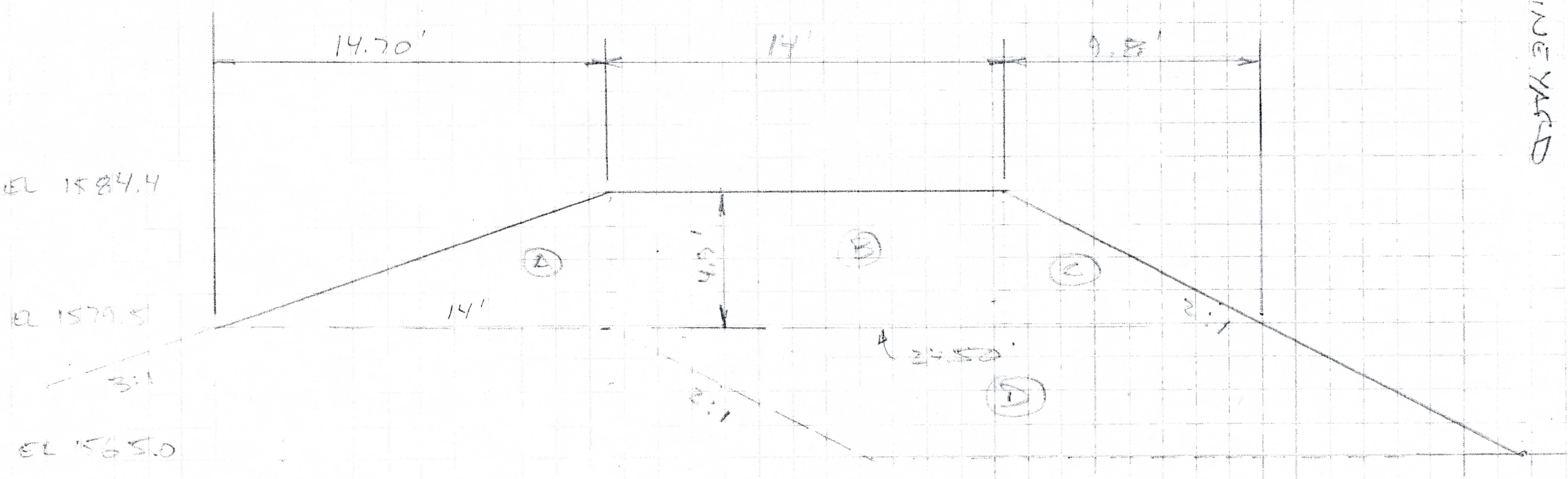
TYPICAL EXISTING EMBANKMENT SECTION



Kimley-Horn and Associates, Inc.

Job STRUCTURES Subject
Designed by R BUTTERWEG Date 8.30.00 Checked by
Sheet No. ___ of ___
Job No. 291210083
Date

VINEYARD



A ½ (4.9 × 14.70) = 36.0 SF

B 4.9 × 14 = 68.6

C ½ (4.9 × 9.8) = 24.0

D (79.5 − 65) × 24.50 = 355.3

483.85 SF/LF

483.85 × 28,828 = 13,948,428 CF

= 516,608 CY

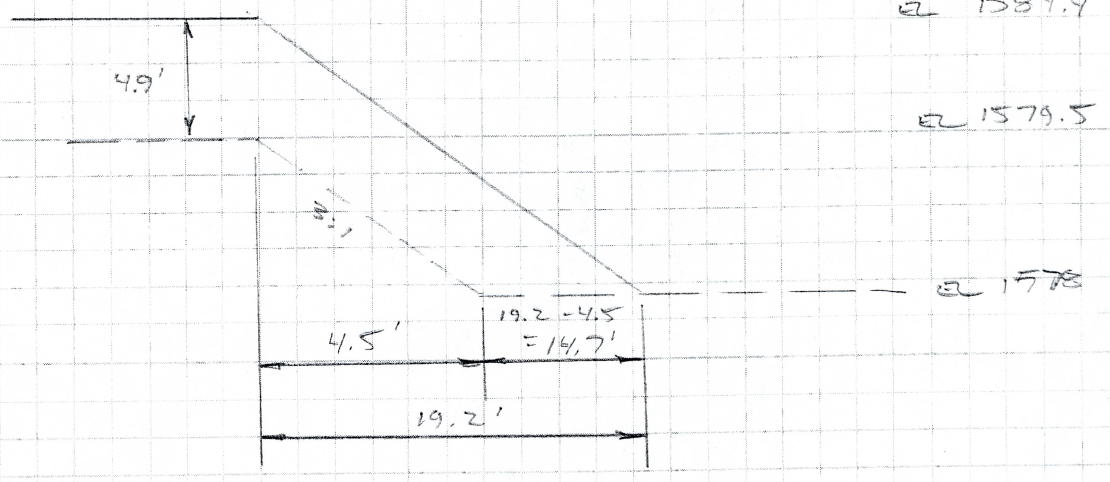
Kimley-Horn and Associates, Inc.

Sheet No. ______ of ______

Job STRUCTURES Subject ______ Job No. 091131003

Designed by RBLITEWEG Date 8.30.00 Checked by ______ Date ______

VINEYARD



WIDTH 750' LENGTH = 14.7'

750 × 14.7 = 11,025 SF

WIDTH 150' LENGTH = 1000'

150 × 1000 = 150,000 SF

161,025 SF / SPILLWAY

2 SPILLWAYS =

161,025 × 2 = 322,050 SF

= 35,784 SY

Kimley-Horn and Associates, Inc.

Sheet No. ______ of ______

Job STRUCTURES Subject ALTS ANALYSIS Job No. 091131003

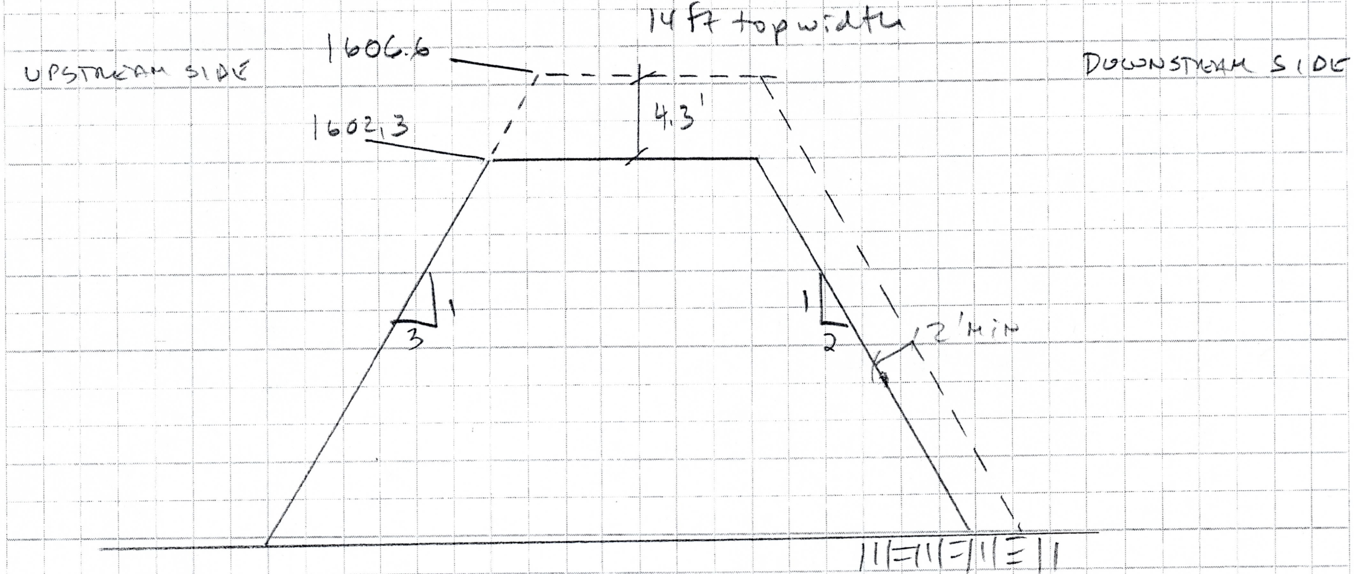
Designed by ______ Date ______ Checked by ______ Date ______

HIGH HAZARD DAM

RITTENHOUSE FRS

	EXISTING	PROPOSED	Δ
TOP OF DAM (FT)	1602.3	1606.6	4.3 (FT)
LENGTH OF SPILLWAY (FT)	600	900	300 FT
LENGTH OF DAM (FT) 3.6 mi	19,000 ft		

TYPICAL EMBANKMENT SECTION



Kimley-Horn and Associates, Inc.

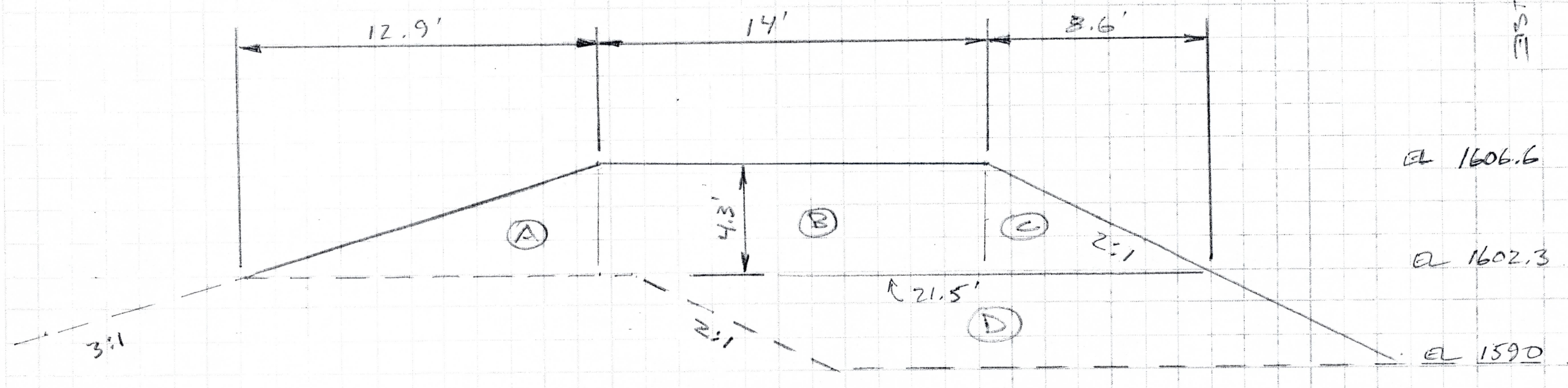
Job STRUCTURES Subject

Designed by R BUTTNER Date 8.30.08 Checked by

Sheet No. ___ of ___

Job No. 09131003 Date

RITTEN HOUSE



A	½ (4.3 x 12.9)	=	27.7
B	4.3 x 14	=	17.2
C	½ (4.3 x 8.6)	=	18.5
D	(1602 − 1590) x 21.5	=	258.0
			321.4 SF/LF

321.4 x 19,000 = 6,106,600 CF

= 226,171 CY

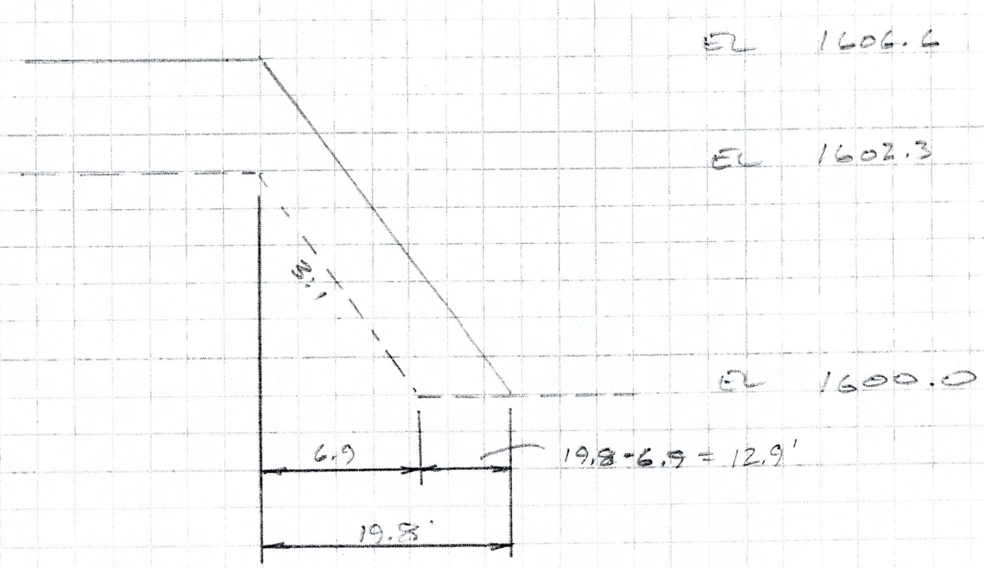
Kimley-Horn
and Associates, Inc.

Sheet No. ____ of ____

Job STRUCTURES Subject ____ Job No. 091131003

Designed by R RUTTENEG Date 8.30.00 Checked by ____ Date ____

RITTENHOUSE



WIDTH = 900'   LENGTH = 12.9'

900 x 12.9 = 11,610 SF

WIDTH = 300'   LENGTH = 1000'

300 x 1000 = 300,000 SF

311,610 SF

= 34,623 SY

ALTERNATIVES ANALYSIS REPORT

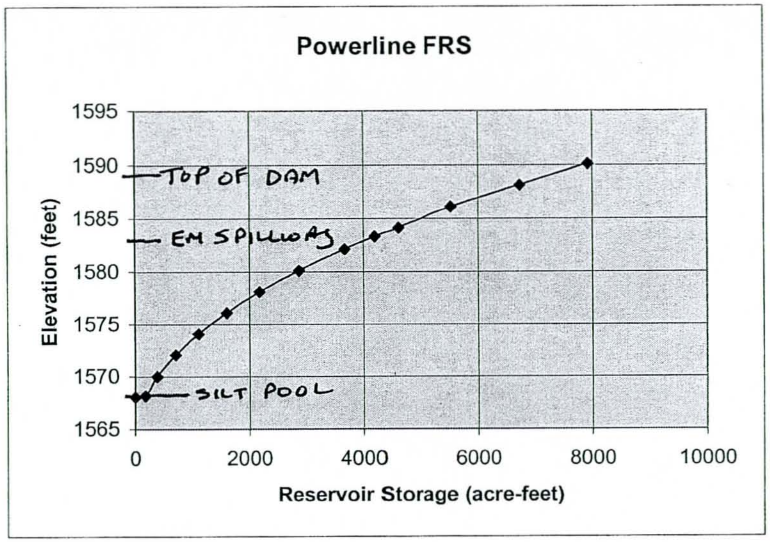
<u>HEC-1 PMP Powerline FRS</u>
<u>Upgrade Dam to High Hazard</u>

EMERGENCY SPILLWAY
1583.3 (ft)

TOP DAM = 1589.1 ft

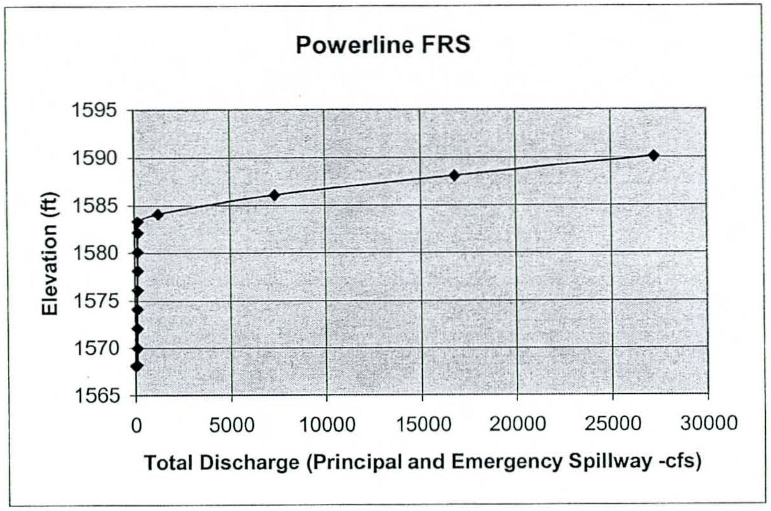
SILT POOL = 1568.2

Elevation	Storage	Discharge
1568.1	0	0
1568.2	175	75
1570	380	92
1572.1	700	106
1574.1	1100	119
1576.1	1600	130
1578.1	2175	141
1580.1	2875	150
1582.1	3675	159
1583.3	4200	165
1584.1	4600	1228
1586.1	5525	7360
1588.1	6725	16800
1590.1	7925	27280



EMERGENCY SPILLWAY RATING CURVE
Q = CLH^(EXP)
C = 3.0
EXP = 1.5

Elevation	H	L = 600	L = 900	L = 1000
1584.3	1	1,800	2,700	3,000
1585.3	2	5,091	7,637	8,485
1586.3	3	9,353	14,030	15,588
1587.3	4	14,400	21,600	24,000
1588.3	5	20,125	30,187	33,541
1589.3	6	26,454	39,682	44,091
1590.3	7	33,336	50,005	55,561
1591.3	8	40,729	61,094	67,882
1592.3	9	48,600	72,900	81,000
1593.3	10	56,921	85,381	94,868



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1****************************************                    ****************************************
 *                                      *                    *                                      *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *                    *   U.S. ARMY CORPS OF ENGINEERS       *
 *              JUN   1998              *                    *   HYDROLOGIC ENGINEERING CENTER      *
 *           VERSION 4.1                *                    *        609 SECOND STREET             *
 *                                      *                    *     DAVIS, CALIFORNIA 95616          *
 * RUN DATE   24AUG00  TIME  14:59:46   *                    *        (916) 756-1104                *
 *                                      *                    *                                      *
****************************************                    ****************************************


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


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

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

1                                         HEC-1 INPUT                                                    PAGE  1

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

     1          ID  WEEKES WASH - POWERLINE F.R.S ANALYSIS
     2          ID  CONVERSION OF MODEL FROM TR-20
     3          ID                FULL PMP: SCS EXCESS & HYDROGRAPH
     4          ID  DEVELOPMENT. MUSKINGUM ROUTING
     5          ID  CONDITIONS:   1: 6-HR PMP STORM IS CONTAINED WITHIN THE BREAKOUT
     6          ID                POINT OF WEEKES WASH AT JUNCTION ROAD.
     7          ID                2: EXISTING CONDITIONS PMP SAME AS FOR VINEYARD AND
     8          ID                   RITTENHOUSE FRS'S.  JULY 27, 989 RUN
     9          ID                3: SUPERSTITION FREEWAY IN PLACE.
    10          ID                4: ADJUSTED WATERSHED AREAS.
    11          ID                5: BREAKOUT CURVE FOR NORTH DIVERSION DAM.
    12          ID                6: NO MODIFICATIONS TO POWERLINE F.R.S.
    13          ID  TIME INCREMENT = 5 MINUTES
    14          ID  FILE=PALT.DAT   BASE MODEL=PADJPMP.DAT  MODIFIED TOP OF DAM AND EMERGENCY SPIL
    15          ID  TO PREVENT OVERTOPPING OF DAM AND OBTAIN THREE FEET OF FREEBOARD.
    16          IT       5       0       0     150
    17          IO       5

    18          KK  15 WATERSHED 15
    19          KM  HYDROGRAPH FOR WATERSHED 15
    20          PB    7.60
    21          IN      15
    22          PI     .05     .05     .05     .05     .15     .15     .15     .15     .30     .30
    23          PI     .30     .30    2.30    1.10     .60     .50     .18     .18     .18     .18
    24          PI     .10     .10     .10     .10
    25          BA    1.76
    26          LS       0      94
    27          UD    0.18

    28          KK  R15 ROUTE HYDROGRAPH FROM WS 15
    29          KM  ROUTE HYDROGRAPH FROM WATERSHED 15
    30          RM       1    0.11     0.3

    31          KK  14 WATERSHED 14
    32          KM  HYDROGRAPH FOR WATERSHED 14
    33          BA    1.11
    34          LS       0      93
    35          UD    0.15

    36          KK  16 WATERSHED 16
    37          KM  HYDROGRAPH FOR WATERSHED 16
    38          BA    2.16
    39          LS       0      95
    40          UD    0.35

    41          KK  114 CONCENTRATION PT. 114 (INCLUDES WATERSHEDS 14, 15, & 16)
    42          KM  COMBINE All THREE HYOROGRAPHS AT CP 114
    43          HC       3

    44          KK  R114 ROUTE CP 114
    45          KM  ROUTE HYDROGRAPH FROM CP114 TO CP 113
    46          RM       1    0.17     0.3
1                                         HEC-1 INPUT                                                    PAGE  2

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

    47          KK  13 WATERSHED 13
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 48          KM  HYDROGRAPH FOR WATERSHED 13
 49          BA    1.29
 50          LS       0       96
 51          UD   0.126

 52          KK  113 CONCENTRATION PT 113 FOR HYDROGRAPHS CP 114 AND WS 13
 53          KM  COMBINE HYDROGRAPHS CP 114 AND WS 13
 54          HC       2

 55          KK  R113 ROUTE CP 113 TO CP 112
 56          KM  ROUTE HYDROGRAPH FOR CP 113
 57          RM       1    0.19      0.3

 58          KK  12 WATERSHED 12
 59          KM  HYDROGRAPH FOR WATERSHED 12
 60          BA    1.32
 61          LS       0       97
 62          UD    0.27

 63          KK  112 CONCENTRATION PT. 112 FOR HYDROGRAPHS FROM CP 113 AND WS 12.
 64          KM  COMBINE HYDROGRAPH FROM CP 113 AND WS 12
 65          HC       2

 66          KK  R112 ROUTE CP 112 TO CP 111
 67          KM  ROUTE HYDROGRAPH CP 112 TO CP 111
 68          RM       1    0.07      0.3

 69          KK  11 WATERSHED 11
 70          KM  HYDROGRAPH FOR WATERSHED 11
 71          BA     0.7
 72          LS       0       95
 73          UD   0.186

 74          KK  11 WATERSHED 11
 75          KM  HYDROGRAPH FOR WATERSHED 17
 76          BA    0.37
 77          LS       0       93
 78          UD   0.138

 79          KK  10 WATERSHED 10
 80          KM  HYDROGRAPH FOR WATERSHED 10
 81          BA    0.54
 82          LS       0       97
 83          UD    0.14

 84          KK  111 CONCENTRATION PT. 111
 85          KM  COMBINE HYDROGRAPHS FROM CP 112. WS 11. AND WS 17
 86          HC       4
1                                  HEC-1 INPUT                                        PAGE  3

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


 87          KK  RD111 ROUTE CP 111 TO PT AT WHICH BREAKOUT WOULD OCCUR.
 88          KM  ROUTE HYDROGRAPH CP111 TO THE POINT AT WHICH THE BREAKOUT OCCURS.
 89          RM       5    0.64      0.3

 90          KK  2A WATERSHED 2A
 91          KM  NYOROGRAPN FOR WATERSHED 2A
 92          BA    1.09
 93          LS       0       90
 94          UD    0.22

 95          KK  CP2A CONCENTRATION POINT 2A
 96          KM  COMBINE ROUTED HYDROGRAPH FROW CP 111 AND WATERSHED 2A.
 97          HC       2

 98          KK  DIVERTPOINT OF DIVERSION FOR WEEKES WASH
 99          DT  FLOW
100          DI       0     6500     7000     9000    10000    15000
101          DQ       0        0      260     1300     1880     4600

102          KK  RR2A ROUTE REMAINING HYDROGRAPH TO SUPERSTITION FREEWAY
103          KM  ROUTE THE REMAINING HYDROGRAPH TO DETENTION NORTH OF SUPERSTITION FREEWAY
104          RM       2    0.42      0.3

105          KK  2BE WATERSHED 28 EAST
106          KM  HYDROGRAPH FOR WATERSHED 28 EAST
107          BA    1.22
108          LS       0       93
109          UD    0.48

110          KK  102B CONCENTRATION POINT NORTH OF FREEWAY
111          KM  CONCENTRATION POINT NORTH OF SUPERSTITION FREEWAY AT DETENTION AREA
112          HC       2

113          KK  DET2BE ROUTE THROUGH FREEWAY (WEEKES WASH DETENTION BASIN)
114          KM  ROUTE FLOW THROUGH WEEKES WASH DETENTION BASIN AND THEN FREEWAY
115          RS       1     ELEV     1636
116          SV       0        6       13       29       47       69       93      122      153      184
117          SV     217      239
118          SE    1636     1637     1638     1640     1642     1644     1646     1648     1650     1652
119          SE    1654     1655
120          SQ       0       22      194      584     1050     1604     2236     2900     3534     4408
121          SQ    4622     4860
122          ST    1650      320      2.2      1.5
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 123          KK  FRWAY
 124          KM  DIVERT ALL FLOW THAT GO UNDER THE ROAD
 125          DT  SPILL
 126          DI       0    3665    5000   10000   15000
 127          DQ       0       0    1335    6335   11335
1                                        HEC-1 INPUT                                        PAGE  4

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


 128          KK  CHAN ROUTE FLOWS THROUGH THE CHANNEL
 129          KM  ROUTE FLOW THROUGH THE IMPROVED CHANNEL
 130          RM       1     0.1     0.3

 131          KK  WASH ROUTE FLOWS THROUGH THE WASH
 132          KM  ROUTE FLOWS THROUGH THE NATURAL WASH SYSTEM
 133          RM       2     0.5     0.3

 134          KK  RSPILL
 135          KM  RETRIEVE FLOW DIVERTED FROM THE EMERGENCY SPILLWAY
 136          DR  SPILL

 137          KK  IDAHO
 138          KM  ROUTE THE RETRIEVED FLOW DOWN IDAHO ROAD.
 139          RM       1     0.1     0.3

 140          KK  WW
 141          KM  COMBINE ALL THE FLOW FROM THE FREEWAY
 142          HC       2

 143          KK  RWW
 144          KM  ROUTE THE FLOW TO POWERLINE F.R.S.
 145          RM       4    0.98     0.3

 146          KK  28WW WATERSHED 2B WEST, WEST
 147          KM  HYDRDGRAPH FOR WATERSHED 2B WEST, WEST
 148          BA    0.45
 149          LS       0      92
 150          UD     0.4

 151          KK  2BWE WATERSHED 2B WEST, EAST
 152          KM  RUNOFF FROM EAST OF IDAHO ROAD 2B WEST, EAST
 153          KO       1
 154          BA     0.7
 155          LS       0      92
 156          UD     0.3

 157          KK  DET2BW ROUTE THROUGH THE DETENTION NORTH OF THE FREEWAY (N. DIVERSION DAM)
 158          KM  ROUTE FLOWS THROUGH THE DETENTION NORTH OF THE SUPERSTITION FREEWAY
 159          RS       1    ELEV  1623.5
 160          SV       0     0.6     8.3    17.5    28.2      36    48.6
 161          SE  1623.5    1624    1626    1628    1630    1632    1634
 162          SQ       0       0      31      93     135     169     208
 163          ST    1630     110     2.2     1.5

 164          KK  C004 COMBINE FLOWS JUST NORTH OF FREEWAY
 165          KM  COMBINE THE FLOWS JUST NORTH OF THE FREEWAY
 166          HC       2
1                                        HEC-1 INPUT                                        PAGE  5

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


 167          KK  NDIV
 168          DT  EXIT
 169          DI       0    1218    5000   10000
 170          DQ       0       0    3782    8782

 171          KK  NDROUT ROUTE THROUGH THE FREEWAY
 172          KM  ROUTE COMBINED FLOW THROUGH 3 BBL. 6 X 8 BOX CULVERTS
 173          RS       1    ELEV  1622.6
 174          SV       0    0.02    0.16    1.12    4.17   25.42
 175          SE  1622.6    1623    1624    1626    1628    1630
 176          SQ       0       0     101     372     738    1111
 177          ST    1630    2875     2.2     1.5

 178          KK  R2BW ROUTE TO CP102
 179          KM  ROUTE THE FLOWS FROM DETENTION POND TO CP102 (POWERLINE F.R.S.)
 180          RM       2     0.4     0.3

 181          KK  2BS WATERSHED 2B SOUTH OF SUPERTITION FREEWAY
 182          KM  HYDROGRAPH FOR WATERSHED 2B SOUTH
 183          BA    1.91
 184          LS       0      91
 185          UD    0.32

 186          KK  102 CONCENTRATION PT. 102, WEEKS WASH WATERSHED. AT POWERLINE FRS.
 187          KM  COMBINE HYDROGRAPHS FROM CP D111 AND WS 2 (WE ARE NOW AT POWERLINE DAM)
 188          HC       3

 189          KK  5 WATERSHED 5 ( BEGINNING OF SIPHON DRAW WATERSHED)
 190          KM  HYDROGRAPH FOR WS 5 (BEGINNIAG OF THE WATERSHED FOR SIPHON DRAW TO THE DAM)
 191          BA    5.65
 192          LS       0      95
 193          UD    0.71
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KK  95 ROUTE HYDROGRAPH FROM WS 5 TO CP 104
KM  ROUTE HYDROGRAPH FOR WS 5 TO CP 104
RM       1    0.07      0.3

KK  4 WATERSHED 4
KM  HYOROGRAPH FOR WATERSHED 4
BA   11.85
LS       0       95
UD     1.3

KK  104 CONCENTRATION POINT 104
KM  COMBINE HYDROGRAPHS AT CP 104
HC       2

KK  R104
KM  ROUTE CP 104 TO 106
RM       2      0.5      0.3
1                                    HEC-1 INPUT                                          PAGE  6

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

KK  7 WATERSHED 7
KM  HYDROGRAPH FOR WATERSHED 7
BA    0.61
LS       0       93
UD     0.3

KK  6 WATERSHEAD 6
KM  HYDROGRAPH FOR WATERSHEAD 6
BA    7.86
LS       0       93
UD    1.08

KK  106 CONCENTRATION PT. 106. HYDROGRAPHS FROM R5, WS 4, & WS 6
KM  COMBINE HYDROGRAPHS FROM R5, WS 4. AND WS 6
HC       3

KK  R106 ROUTE THE HYDROGRAPH FROM CP 106 TO WHERE NEXT WASH ENTERS.
KM  ROUTE HYDROGRAPH FROM CP 104 TO HERE OTHER WASH ENTERS
RM       1     0.27      0.3

KK  3N WATERSHED 3N
KM  HYDROGRAPH FOR WATERSHED 3N
BA    2.89
LS       0       95
UD    0.41

KK  CULV ROUTE FLOWS THROUGH CULVERTS
KM  ROUTE FLOWS THROUGH THE CULVERTS
RS       1     ELEV     1665
SV       0     0.15      0.6      1.5      3.1      5.6      9.1     13.7    19.45    26.35
SE    1665   1665.5     1666   1666.5     1667   1667.5     1668   1668.5     1669   1669.5
SQ       0      119      375      688     1063     1462     1938     2375     2875     3375

KK  RCULV ROUTE FLOWS TO W103
KM  ROUTE THE FLOWS TO CONCENTRATION POINT W103
RM       2      0.5      0.3

KK  3S WATERSHED 3 SOUTH
KM  HYDRDGRAPH FOR WATERSHED 3 SOUTH
BA    2.39
LS       0       90
UD   0.436

KK     103
HC       2

KK  3A WATERSHED 3A
KM  HYDROGRAPH FOR WATERSHED 3A
BA     1.2
LS       0       91
UD     0.4
1                                    HEC-1 INPUT                                          PAGE  7

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

KK  CP103
KM  CONCENTRATION POINT FOR WEST FORK OF SIPHON DRAW
HC       2

KK  W103 CONCENTRATION PT. W103 WHERE UNNAMED WASH ENTERS.
KM  COMBINE HYDROGRAPHS FROM CP 104 AND WS 3 - SIPHON DRAW WATERSHED
HC       2

KK  RW103 ROUTE RESULTING HYDROGRAPH TO POWERLINE DAM STRUCTURE
KM  ROUTE THE HYDROGRAPH TO POWERLINE DAM
RM       1     0.16      0.3

KK  PLD CONCENTRATION PT. AT POWERLINE DAM
KM  COMBINE HYDROGRAPHS FROM WEEKES WASH SUB-BASIN AND SIPHON DRAW SUB-BASIN
KO       3
HC       2
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         263           KK  POWERLINE FRS RESERVOIR ROUTING THROUGH THE STRUCTURE
         264           KM  ************************************************************************
         265           KM  RESERVOIR RATING CURVE
         266           KM   CHANGE RS RECORD FIELD 3 TO "1583.3" THE ELEVATION OF EMERGENCY SPILLWAY
         267           KM   REPORT ASSUMED THAT PMP ROUTING STARTING WITH FULL RESERVOIR
         268           KM   ITERATED ON TOP OF DAM ELEVATION TO PREVENT OVERTOPPING AND
         269           KM   OBTAIN THREE FEET OF FREEBOARD.  TOP OF DAM CHANGED FROM 1589.1-FT TO 1593.4-
         270           KM  ************************************************************************
         271           KM   INCREASED EMERGENCY SPILLWAY WIDTH TO 900-FT FROM 600-FT.  REFLECTED IN SQ RE
         272           RS       1    ELEV  1583.3
         273           SV       0     175     380     700    1100    1600    2175    2875    3675    4200
         274           SV    4600    5525    6725    7925
         275           SQ       0      75      92     106     119     130     141     150     159     165
         276           SQ    2700   14030   30187   50000
         277           SE  1568.1  1568.2    1570  1572.1  1574.1  1576.1  1578.1  1580.1  1582.1  1583.3
         278           SE  1584.1  1586.1  1588.1  1590.1
         279           SS  1583.3       0       0       0
         280           ST  1593.4   13358     2.2     1.5
         281           ZZ
1*****************************************                                          ****************************************
 *                                       *                                          *                                      *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                                          *   U.S. ARMY CORPS OF ENGINEERS       *
 *             JUN   1998                *                                          *   HYDROLOGIC ENGINEERING CENTER      *
 *           VERSION 4.1                 *                                          *        609 SECOND STREET             *
 *                                       *                                          *     DAVIS, CALIFORNIA 95616          *
 *  RUN DATE   24AUG00  TIME  14:59:46   *                                          *         (916) 756-1104               *
 *                                       *                                          *                                      *
*****************************************                                          ****************************************


                        WEEKES WASH - POWERLINE F.R.S ANALYSIS
                        CONVERSION OF MODEL FROM TR-20
                                      FULL PMP: SCS EXCESS & HYDROGRAPH
                        DEVELOPMENT. MUSKINGUM ROUTING
                        CONDITIONS:   1: 6-HR PMP STORM IS CONTAINED WITHIN THE BREAKOUT
                                      POINT OF WEEKES WASH AT JUNCTION ROAD.
                                      2: EXISTING CONDITIONS PMP SAME AS FOR VINEYARD AND
                                         RITTENHOUSE FRS'S.  JULY 27, 989 RUN
                                      3: SUPERSTITION FREEWAY IN PLACE.
                                      4: ADJUSTED WATERSHED AREAS.
                                      5: BREAKOUT CURVE FOR NORTH DIVERSION DAM.
                                      6: NO MODIFICATIONS TO POWERLINE F.R.S.
                        TIME INCREMENT = 5 MINUTES
                        FILE=PALT.DAT   BASE MODEL=PADJPMP.DAT  MODIFIED TOP OF DAM AND EMERGENCY SPIL
                        TO PREVENT OVERTOPPING OF DAM AND OBTAIN THREE FEET OF FREEBOARD.

  17 IO          OUTPUT CONTROL VARIABLES
                       IPRNT           5  PRINT CONTROL
                       IPLOT           0  PLOT CONTROL
                       QSCAL          0.  HYDROGRAPH PLOT SCALE

    IT           HYDROGRAPH TIME DATA
                        NMIN           5  MINUTES IN COMPUTATION INTERVAL
                       IDATE      1    0  STARTING DATE
                       ITIME        0000  STARTING TIME
                          NQ         150  NUMBER OF HYDROGRAPH ORDINATES
                      NDDATE      1    0  ENDING DATE
                      NDTIME        1225  ENDING TIME
                       ICENT          19  CENTURY MARK

                 COMPUTATION INTERVAL     .08 HOURS
                      TOTAL TIME BASE   12.42 HOURS

        ENGLISH UNITS
             DRAINAGE AREA          SQUARE MILES
             PRECIPITATION DEPTH    INCHES
             LENGTH, ELEVATION      FEET
             FLOW                   CUBIC FEET PER SECOND
             STORAGE VOLUME         ACRE-FEET
             SURFACE AREA           ACRES
             TEMPERATURE            DEGREES FAHRENHEIT
***** WARNING *****  POSSIBLE INSTABILITIES IN THE MUSKINGUM ROUTING FOR REACH  R114 R.
      ADJUST NSTPS AND/OR COMPUTATION INTERVAL TO MEET CRITERIA IN USER MANUAL).
***** WARNING *****  POSSIBLE INSTABILITIES IN THE MUSKINGUM ROUTING FOR REACH  R113 R.
      ADJUST NSTPS AND/OR COMPUTATION INTERVAL TO MEET CRITERIA IN USER MANUAL).
***** WARNING *****  POSSIBLE INSTABILITIES IN THE MUSKINGUM ROUTING FOR REACH  RR2A R.
      ADJUST NSTPS AND/OR COMPUTATION INTERVAL TO MEET CRITERIA IN USER MANUAL).
***** WARNING *****  POSSIBLE INSTABILITIES IN THE MUSKINGUM ROUTING FOR REACH  WASH R.
      ADJUST NSTPS AND/OR COMPUTATION INTERVAL TO MEET CRITERIA IN USER MANUAL).
***** WARNING *****  POSSIBLE INSTABILITIES IN THE MUSKINGUM ROUTING FOR REACH  RWW   .
      ADJUST NSTPS AND/OR COMPUTATION INTERVAL TO MEET CRITERIA IN USER MANUAL).


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

          **************
          *            *
 151 KK   *   2BWE W   *     ATERSHED 2B WEST, EAST
          *            *
          **************
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153 KO          OUTPUT CONTROL VARIABLES
                      IPRNT          1  PRINT CONTROL
                      IPLOT          0  PLOT CONTROL
                      QSCAL         0.  HYDROGRAPH PLOT SCALE

              SUBBASIN RUNOFF DATA

154 BA          SUBBASIN CHARACTERISTICS
                      TAREA        .70  SUBBASIN AREA

                PRECIPITATION DATA

 20 PB                STORM       7.60  BASIN TOTAL PRECIPITATION

 22 PI            INCREMENTAL PRECIPITATION PATTERN
```

.02	.02	.02	.02	.02	.02	.02	.02	.02	.02
.02	.02	.05	.05	.05	.05	.05	.05	.05	.05
.05	.05	.05	.05	.10	.10	.10	.10	.10	.10
.10	.10	.10	.10	.10	.10	.77	.77	.77	.37
.37	.37	.20	.20	.20	.17	.17	.17	.06	.06
.06	.06	.06	.06	.06	.06	.06	.06	.06	.06
.03	.03	.03	.03	.03	.03	.03	.03	.03	.03
.03	.03								

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155 LS          SCS LOSS RATE
                      STRTL        .17  INITIAL ABSTRACTION
                     CRVNBR      92.00  CURVE NUMBER
                      RTIMP        .00  PERCENT IMPERVIOUS AREA

156 UD          SCS DIMENSIONLESS UNITGRAPH
                       TLAG        .30  LAG
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UNIT HYDROGRAPH
20 END-OF-PERIOD ORDINATES

138.	447.	848.	990.	909.	711.	451.	302.	207.	138.
94.	63.	42.	28.	19.	13.	9.	6.	4.	1.

****

HYDROGRAPH AT STATION 2BWE W

****

DA MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q	*	DA MON	HRMN	ORD	RAIN	LOSS	EXCESS	COMP Q
1	0000	1	.00	.00	.00	0.	*	1	0615	76	.00	.00	.00	132.
1	0005	2	.02	.02	.00	0.	*	1	0620	77	.00	.00	.00	99.
1	0010	3	.02	.02	.00	0.	*	1	0625	78	.00	.00	.00	69.
1	0015	4	.02	.02	.00	0.	*	1	0630	79	.00	.00	.00	45.
1	0020	5	.02	.02	.00	0.	*	1	0635	80	.00	.00	.00	30.
1	0025	6	.02	.02	.00	0.	*	1	0640	81	.00	.00	.00	21.
1	0030	7	.02	.02	.00	0.	*	1	0645	82	.00	.00	.00	14.
1	0035	8	.02	.02	.00	0.	*	1	0650	83	.00	.00	.00	9.
1	0040	9	.02	.02	.00	0.	*	1	0655	84	.00	.00	.00	6.
1	0045	10	.02	.02	.00	0.	*	1	0700	85	.00	.00	.00	4.
1	0050	11	.02	.02	.00	0.	*	1	0705	86	.00	.00	.00	3.
1	0055	12	.02	.02	.00	0.	*	1	0710	87	.00	.00	.00	2.
1	0100	13	.02	.02	.00	0.	*	1	0715	88	.00	.00	.00	1.
1	0105	14	.05	.04	.01	1.	*	1	0720	89	.00	.00	.00	1.
1	0110	15	.05	.04	.01	4.	*	1	0725	90	.00	.00	.00	0.
1	0115	16	.05	.04	.01	11.	*	1	0730	91	.00	.00	.00	0.
1	0120	17	.05	.03	.02	23.	*	1	0735	92	.00	.00	.00	0.
1	0125	18	.05	.03	.02	37.	*	1	0740	93	.00	.00	.00	0.
1	0130	19	.05	.03	.02	53.	*	1	0745	94	.00	.00	.00	0.
1	0135	20	.05	.03	.02	69.	*	1	0750	95	.00	.00	.00	0.
1	0140	21	.05	.02	.03	84.	*	1	0755	96	.00	.00	.00	0.
1	0145	22	.05	.02	.03	99.	*	1	0800	97	.00	.00	.00	0.
1	0150	23	.05	.02	.03	112.	*	1	0805	98	.00	.00	.00	0.
1	0155	24	.05	.02	.03	125.	*	1	0810	99	.00	.00	.00	0.
1	0200	25	.05	.02	.03	136.	*	1	0815	100	.00	.00	.00	0.
1	0205	26	.10	.03	.07	151.	*	1	0820	101	.00	.00	.00	0.
1	0210	27	.10	.03	.07	175.	*	1	0825	102	.00	.00	.00	0.
1	0215	28	.10	.02	.07	214.	*	1	0830	103	.00	.00	.00	0.
1	0220	29	.10	.02	.08	259.	*	1	0835	104	.00	.00	.00	0.
1	0225	30	.10	.02	.08	303.	*	1	0840	105	.00	.00	.00	0.
1	0230	31	.10	.02	.08	341.	*	1	0845	106	.00	.00	.00	0.
1	0235	32	.10	.02	.08	369.	*	1	0850	107	.00	.00	.00	0.
1	0240	33	.10	.02	.08	392.	*	1	0855	108	.00	.00	.00	0.
1	0245	34	.10	.01	.09	411.	*	1	0900	109	.00	.00	.00	0.
1	0250	35	.10	.01	.09	426.	*	1	0905	110	.00	.00	.00	0.
1	0255	36	.10	.01	.09	438.	*	1	0910	111	.00	.00	.00	0.
1	0300	37	.10	.01	.09	449.	*	1	0915	112	.00	.00	.00	0.
1	0305	38	.76	.06	.70	543.	*	1	0920	113	.00	.00	.00	0.
1	0310	39	.76	.04	.72	827.	*	1	0925	114	.00	.00	.00	0.
1	0315	40	.76	.03	.74	1365.	*	1	0930	115	.00	.00	.00	0.
1	0320	41	.37	.01	.36	1948.	*	1	0935	116	.00	.00	.00	0.
1	0325	42	.37	.01	.36	2370.	*	1	0940	117	.00	.00	.00	0.
1	0330	43	.37	.01	.36	2518.	*	1	0945	118	.00	.00	.00	0.
1	0335	44	.20	.00	.20	2424.	*	1	0950	119	.00	.00	.00	0.
1	0340	45	.20	.00	.20	2212.	*	1	0955	120	.00	.00	.00	0.
1	0345	46	.20	.00	.20	1945.	*	1	1000	121	.00	.00	.00	0.
1	0350	47	.17	.00	.16	1703.	*	1	1005	122	.00	.00	.00	0.
1	0355	48	.17	.00	.16	1491.	*	1	1010	123	.00	.00	.00	0.
1	0400	49	.17	.00	.16	1313.	*	1	1015	124	.00	.00	.00	0.

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          1     0405   50     .06      .00      .06       1170.         *          1      1020  125       .00       .00       .00           0.
          1     0410   51     .06      .00      .06       1028.         *          1      1025  126       .00       .00       .00           0.
          1     0415   52     .06      .00      .06        872.         *          1      1030  127       .00       .00       .00           0.
          1     0420   53     .06      .00      .06        725.         *          1      1035  128       .00       .00       .00           0.
          1     0425   54     .06      .00      .06        600.         *          1      1040  129       .00       .00       .00           0.
          1     0430   55     .06      .00      .06        506.         *          1      1045  130       .00       .00       .00           0.
          1     0435   56     .06      .00      .06        445.         *          1      1050  131       .00       .00       .00           0.
          1     0440   57     .06      .00      .06        404.         *          1      1055  132       .00       .00       .00           0.
          1     0445   58     .06      .00      .06        375.         *          1      1100  133       .00       .00       .00           0.
          1     0450   59     .06      .00      .06        356.         *          1      1105  134       .00       .00       .00           0.
          1     0455   60     .06      .00      .06        343.         *          1      1110  135       .00       .00       .00           0.
          1     0500   61     .06      .00      .06        335.         *          1      1115  136       .00       .00       .00           0.
          1     0505   62     .03      .00      .03        326.         *          1      1120  137       .00       .00       .00           0.
          1     0510   63     .03      .00      .03        311.         *          1      1125  138       .00       .00       .00           0.
          1     0515   64     .03      .00      .03        286.         *          1      1130  139       .00       .00       .00           0.
          1     0520   65     .03      .00      .03        259.         *          1      1135  140       .00       .00       .00           0.
          1     0525   66     .03      .00      .03        234.         *          1      1140  141       .00       .00       .00           0.
          1     0530   67     .03      .00      .03        215.         *          1      1145  142       .00       .00       .00           0.
          1     0535   68     .03      .00      .03        203.         *          1      1150  143       .00       .00       .00           0.
          1     0540   69     .03      .00      .03        194.         *          1      1155  144       .00       .00       .00           0.
          1     0545   70     .03      .00      .03        189.         *          1      1200  145       .00       .00       .00           0.
          1     0550   71     .03      .00      .03        185.         *          1      1205  146       .00       .00       .00           0.
          1     0555   72     .03      .00      .03        183.         *          1      1210  147       .00       .00       .00           0.
          1     0600   73     .03      .00      .03        181.         *          1      1215  148       .00       .00       .00           0.
          1     0605   74     .00      .00      .00        176.         *          1      1220  149       .00       .00       .00           0.
          1     0610   75     .00      .00      .00        160.         *          1      1225  150       .00       .00       .00           0.
                                                                        *
*****************************************************************************************************************************************

    TOTAL RAINFALL =     7.60, TOTAL LOSS =      .95, TOTAL EXCESS =     6.65

 PEAK FLOW       TIME                            MAXIMUM AVERAGE FLOW
                                         6-HR         24-HR        72-HR       12.42-HR
+   (CFS)        (HR)
                              (CFS)
+    2518.       3.50                     500.         242.         242.          242.
                           (INCHES)      6.647        6.648        6.648         6.648
                            (AC-FT)       248.         248.         248.          248.

                             CUMULATIVE AREA =      .70 SQ MI

 ***** WARNING *****   POSSIBLE INSTABILITIES IN THE MUSKINGUM ROUTING FOR REACH   R2BW R.
       ADJUST NSTPS AND/OR COMPUTATION INTERVAL TO MEET CRITERIA IN USER MANUAL).
 ***** WARNING *****   POSSIBLE INSTABILITIES IN THE MUSKINGUM ROUTING FOR REACH   R104   .
       ADJUST NSTPS AND/OR COMPUTATION INTERVAL TO MEET CRITERIA IN USER MANUAL).
 ***** WARNING *****   POSSIBLE INSTABILITIES IN THE MUSKINGUM ROUTING FOR REACH   R106 R.
       ADJUST NSTPS AND/OR COMPUTATION INTERVAL TO MEET CRITERIA IN USER MANUAL).

 WARNING --- ROUTED OUTFLOW (    3550.) IS GREATER THAN MAXIMUM OUTFLOW (    3375.)  IN STORAGE-OUTFLOW TABLE

 WARNING --- ROUTED OUTFLOW (    4688.) IS GREATER THAN MAXIMUM OUTFLOW (    3375.)  IN STORAGE-OUTFLOW TABLE

 WARNING --- ROUTED OUTFLOW (    5975.) IS GREATER THAN MAXIMUM OUTFLOW (    3375.)  IN STORAGE-OUTFLOW TABLE

 WARNING --- ROUTED OUTFLOW (    7149.) IS GREATER THAN MAXIMUM OUTFLOW (    3375.)  IN STORAGE-OUTFLOW TABLE

 WARNING --- ROUTED OUTFLOW (    8015.) IS GREATER THAN MAXIMUM OUTFLOW (    3375.)  IN STORAGE-OUTFLOW TABLE

 WARNING --- ROUTED OUTFLOW (    8468.) IS GREATER THAN MAXIMUM OUTFLOW (    3375.)  IN STORAGE-OUTFLOW TABLE

 WARNING --- ROUTED OUTFLOW (    8503.) IS GREATER THAN MAXIMUM OUTFLOW (    3375.)  IN STORAGE-OUTFLOW TABLE

 WARNING --- ROUTED OUTFLOW (    8211.) IS GREATER THAN MAXIMUM OUTFLOW (    3375.)  IN STORAGE-OUTFLOW TABLE

 WARNING --- ROUTED OUTFLOW (    7724.) IS GREATER THAN MAXIMUM OUTFLOW (    3375.)  IN STORAGE-OUTFLOW TABLE

 WARNING --- ROUTED OUTFLOW (    7145.) IS GREATER THAN MAXIMUM OUTFLOW (    3375.)  IN STORAGE-OUTFLOW TABLE

 WARNING --- ROUTED OUTFLOW (    6532.) IS GREATER THAN MAXIMUM OUTFLOW (    3375.)  IN STORAGE-OUTFLOW TABLE

 WARNING --- ROUTED OUTFLOW (    5911.) IS GREATER THAN MAXIMUM OUTFLOW (    3375.)  IN STORAGE-OUTFLOW TABLE

 WARNING --- ROUTED OUTFLOW (    5295.) IS GREATER THAN MAXIMUM OUTFLOW (    3375.)  IN STORAGE-OUTFLOW TABLE

 WARNING --- ROUTED OUTFLOW (    4689.) IS GREATER THAN MAXIMUM OUTFLOW (    3375.)  IN STORAGE-OUTFLOW TABLE

 WARNING --- ROUTED OUTFLOW (    4110.) IS GREATER THAN MAXIMUM OUTFLOW (    3375.)  IN STORAGE-OUTFLOW TABLE

 WARNING --- ROUTED OUTFLOW (    3578.) IS GREATER THAN MAXIMUM OUTFLOW (    3375.)  IN STORAGE-OUTFLOW TABLE
 ***** WARNING *****   POSSIBLE INSTABILITIES IN THE MUSKINGUM ROUTING FOR REACH   RCULV  .
       ADJUST NSTPS AND/OR COMPUTATION INTERVAL TO MEET CRITERIA IN USER MANUAL).
 ***** WARNING *****   POSSIBLE INSTABILITIES IN THE MUSKINGUM ROUTING FOR REACH   RW103  .
       ADJUST NSTPS AND/OR COMPUTATION INTERVAL TO MEET CRITERIA IN USER MANUAL).


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

              **************
              *            *
 259 KK       *   PLD CO   *     NCENTRATION PT. AT POWERLINE DAM
              *            *
              **************

 261 KO          OUTPUT CONTROL VARIABLES
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                    IPRNT          3  PRINT CONTROL
                    IPLOT          0  PLOT CONTROL
                    QSCAL          0.  HYDROGRAPH PLOT SCALE

     HC          HYDROGRAPH COMBINATION
                    ICOMP          2  NUMBER OF HYDROGRAPHS TO COMBINE

                                                ***

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

                    HYDROGRAPH AT STATION    PLD CO

  PEAK FLOW      TIME                   MAXIMUM AVERAGE FLOW
                                     6-HR       24-HR       72-HR      12.42-HR
 +   (CFS)       (HR)
                           (CFS)
 +   62037.      5.08               31329.      15734.      15734.      15734.
                         (INCHES)    6.188       6.432       6.432       6.432
                          (AC-FT)   15535.      16146.      16146.      16146.

                      CUMULATIVE AREA =    47.07 SQ MI

1
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RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
+	HYDROGRAPH AT	15 WAT	7811.	3.33	1303.	630.	630.	1.76		
+	ROUTED TO	R15 RO	7263.	3.50	1303.	630.	630.	1.76		
+	HYDROGRAPH AT	14 WAT	5157.	3.33	808.	390.	390.	1.11		
+	HYDROGRAPH AT	16 WAT	7509.	3.58	1626.	786.	786.	2.16		
+	3 COMBINED AT	114 CO	18407.	3.50	3736.	1806.	1806.	5.03		
+	ROUTED TO	R114 R	17324.	3.67	3736.	1806.	1806.	5.03		
+	HYDROGRAPH AT	13 WAT	6288.	3.33	988.	477.	477.	1.29		
+	2 COMBINED AT	113 CO	20640.	3.58	4721.	2284.	2284.	6.32		
+	ROUTED TO	R113 R	19453.	3.75	4720.	2284.	2284.	6.32		
+	HYDROGRAPH AT	12 WAT	5102.	3.50	1027.	497.	497.	1.32		
+	2 COMBINED AT	112 CO	23064.	3.75	5745.	2780.	2780.	7.64		
+	ROUTED TO	R112 R	22898.	3.83	5744.	2780.	2780.	7.64		
+	HYDROGRAPH AT	11 WAT	3091.	3.33	527.	255.	255.	.70		
+	HYDROGRAPH AT	11 WAT	1745.	3.33	269.	130.	130.	.37		
+	HYDROGRAPH AT	10 WAT	2623.	3.33	420.	203.	203.	.54		
+	4 COMBINED AT	111 CO	26008.	3.75	6958.	3369.	3369.	9.25		
+	ROUTED TO	RD111	24384.	4.42	6953.	3369.	3369.	9.25		
+	HYDROGRAPH AT	2A WAT	4274.	3.42	752.	363.	363.	1.09		
+	2 COMBINED AT	CP2A C	25069.	4.42	7700.	3732.	3732.	10.34		
+	DIVERSION TO	FLOW	10078.	4.42	1931.	933.	933.	10.34		
+	HYDROGRAPH AT	DIVERT	14992.	4.42	5768.	2799.	2799.	10.34		
	ROUTED TO									

+		RR2A R	14420.	4.75	5761.	2799.	2799.	10.34		
+	HYDROGRAPH AT	2BE WA	3612.	3.75	887.	429.	429.	1.22		
+	2 COMBINED AT	102B C	15418.	4.75	6635.	3228.	3228.	11.56		
+ +	ROUTED TO	DET2BE	15297.	4.83	6584.	3226.	3226.	11.56	1655.95	4.83
+	DIVERSION TO	SPILL	11632.	3.50	3598.	1739.	1739.	11.56		
+	HYDROGRAPH AT	FRWAY	3665.	3.50	2986.	1488.	1488.	11.56		
+	ROUTED TO	CHAN R	3665.	4.42	2985.	1487.	1487.	11.56		
+	ROUTED TO	WASH R	3674.	7.08	2973.	1487.	1487.	11.56		
+	HYDROGRAPH AT	RSPILL	11632.	4.83	3598.	1739.	1739.	.00		
+	ROUTED TO	IDAHO	11575.	4.92	3598.	1739.	1739.	.00		
+	2 COMBINED AT	WW	15234.	4.92	6571.	3226.	3226.	11.56		
+	ROUTED TO	RWW	14308.	5.83	6550.	3224.	3224.	11.56		
+	HYDROGRAPH AT	28WW W	1428.	3.67	322.	155.	155.	.45		
+	HYDROGRAPH AT	2BWE W	2518.	3.50	500.	242.	242.	.70		
+ +	ROUTED TO	DET2BW	2324.	3.58	473.	239.	239.	.70	1634.24	3.58
+	2 COMBINED AT	C004 C	3748.	3.67	787.	394.	394.	1.15		
+	DIVERSION TO	EXIT	2530.	3.25	279.	135.	135.	1.15		
+	HYDROGRAPH AT	NDIV	1218.	3.25	508.	259.	259.	1.15		
+ +	ROUTED TO	NDROUT	1202.	4.42	507.	259.	259.	1.15	1630.05	4.42
+	ROUTED TO	R2BW R	1140.	4.75	507.	259.	259.	1.15		
+	HYDROGRAPH AT	2BS WA	6567.	3.50	1341.	648.	648.	1.91		
+	3 COMBINED AT	102 CO	15458.	5.83	8007.	4131.	4131.	14.62		
+	HYDROGRAPH AT	5 WATE	14021.	4.00	4235.	2056.	2056.	5.65		
+	ROUTED TO	95 ROU	13965.	4.08	4235.	2056.	2056.	5.65		
+	HYDROGRAPH AT	4 WATE	20674.	4.58	8726.	4313.	4313.	11.85		
+	2 COMBINED AT	104 CO	31941.	4.33	12916.	6369.	6369.	17.50		
+	ROUTED TO	R104	30612.	4.83	12896.	6369.	6369.	17.50		
+	HYDROGRAPH AT	7 WATE	2219.	3.50	444.	215.	215.	.61		
+	HYDROGRAPH AT	6 WATE	15051.	4.42	5650.	2764.	2764.	7.86		
+	3 COMBINED AT	106 CO	44513.	4.67	18941.	9348.	9348.	25.97		
+	ROUTED TO	R106 R	43483.	4.92	18924.	9348.	9348.	25.97		
	HYDROGRAPH AT									

```
+                    3N WAT     9372.    3.67       2174.      1052.      1052.      2.89

          ROUTED TO
+                    CULV R     8503.    3.83       2174.      1052.      1052.      2.89
+                                                                                          1674.63       3.83

          ROUTED TO
+                    RCULV      7534.    4.33       2172.      1052.      1052.      2.89

          HYDROGRAPH AT
+                    3S WAT     7134.    3.67       1647.       796.       796.      2.39

          2 COMBINED AT
+                      103     11756.    4.08       3818.      1848.      1848.      5.28

          HYDROGRAPH AT
+                    3A WAT     3763.    3.67        842.       407.       407.      1.20

          2 COMBINED AT
+                    CP103     14234.    4.00       4660.      2256.      2256.      6.48

          2 COMBINED AT
+                    W103 C    50553.    4.83      23444.     11603.     11603.     32.45

          ROUTED TO
+                    RW103     50251.    5.00      23436.     11603.     11603.     32.45

          2 COMBINED AT
+                    PLD CO    62037.    5.08      31329.     15734.     15734.     47.07

          ROUTED TO
+                    POWERL    52945.    5.83      30070.     15559.     15559.     47.07
+                                                                                          1590.40       5.83
1                             SUMMARY OF DAM OVERTOPPING/BREACH ANALYSIS FOR STATION    DET2BE
                          (PEAKS SHOWN ARE FOR INTERNAL TIME STEP USED  DURING BREACH FORMATION)

     PLAN  1 ..............               INITIAL VALUE     SPILLWAY CREST     TOP OF DAM
                               ELEVATION     1636.00            1650.00          1650.00
                               STORAGE          0.               153.             153.
                               OUTFLOW          0.              3534.            3534.

                  RATIO    MAXIMUM      MAXIMUM     MAXIMUM     MAXIMUM    DURATION    TIME OF     TIME OF
                   OF     RESERVOIR      DEPTH      STORAGE     OUTFLOW    OVER TOP   MAX OUTFLOW  FAILURE
                  PMF      W.S.ELEV    OVER DAM      AC-FT        CFS       HOURS       HOURS       HOURS

                  1.00     1655.95       5.95         260.      15297.      3.42        4.83         .00
1                             SUMMARY OF DAM OVERTOPPING/BREACH ANALYSIS FOR STATION    DET2BW
                          (PEAKS SHOWN ARE FOR INTERNAL TIME STEP USED  DURING BREACH FORMATION)

     PLAN  1 ..............               INITIAL VALUE     SPILLWAY CREST     TOP OF DAM
                               ELEVATION     1623.50            1630.00          1630.00
                               STORAGE          0.                28.              28.
                               OUTFLOW          0.               135.             135.

                  RATIO    MAXIMUM      MAXIMUM     MAXIMUM     MAXIMUM    DURATION    TIME OF     TIME OF
                   OF     RESERVOIR      DEPTH      STORAGE     OUTFLOW    OVER TOP   MAX OUTFLOW  FAILURE
                  PMF      W.S.ELEV    OVER DAM      AC-FT        CFS       HOURS       HOURS       HOURS

                  1.00     1634.24       4.24          50.       2324.      3.33        3.58         .00
1                             SUMMARY OF DAM OVERTOPPING/BREACH ANALYSIS FOR STATION    NDROUT
                          (PEAKS SHOWN ARE FOR INTERNAL TIME STEP USED  DURING BREACH FORMATION)

     PLAN  1 ..............               INITIAL VALUE     SPILLWAY CREST     TOP OF DAM
                               ELEVATION     1622.60            1630.00          1630.00
                               STORAGE          0.                25.              25.
                               OUTFLOW          0.              1111.            1111.

                  RATIO    MAXIMUM      MAXIMUM     MAXIMUM     MAXIMUM    DURATION    TIME OF     TIME OF
                   OF     RESERVOIR      DEPTH      STORAGE     OUTFLOW    OVER TOP   MAX OUTFLOW  FAILURE
                  PMF      W.S.ELEV    OVER DAM      AC-FT        CFS       HOURS       HOURS       HOURS

                  1.00     1630.05        .05          26.       1202.       .25        4.42         .00
1                             SUMMARY OF DAM OVERTOPPING/BREACH ANALYSIS FOR STATION    POWERL
                          (PEAKS SHOWN ARE FOR INTERNAL TIME STEP USED  DURING BREACH FORMATION)

     PLAN  1 ..............               INITIAL VALUE     SPILLWAY CREST     TOP OF DAM
                               ELEVATION     1583.30            1583.30          1593.40
                               STORAGE       4200.              4200.            9905.
                               OUTFLOW        165.               165.           82692.

                  RATIO    MAXIMUM      MAXIMUM     MAXIMUM     MAXIMUM    DURATION    TIME OF     TIME OF
                   OF     RESERVOIR      DEPTH      STORAGE     OUTFLOW    OVER TOP   MAX OUTFLOW  FAILURE
                  PMF      W.S.ELEV    OVER DAM      AC-FT        CFS       HOURS       HOURS       HOURS

                  1.00     1590.40        .00        8103.      52945.       .00        5.83         .00
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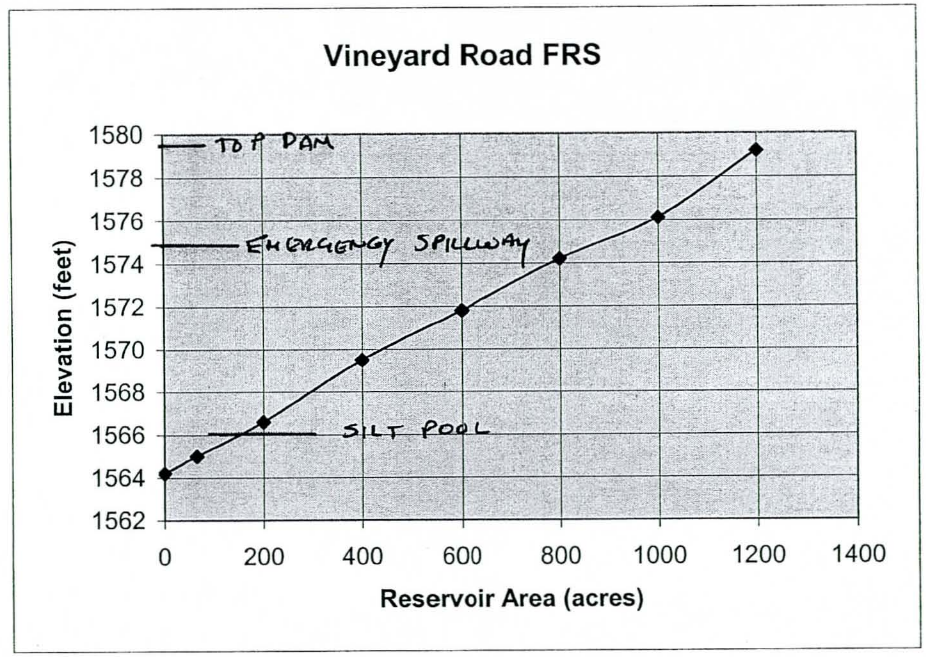
*** NORMAL END OF HEC-1 ***

ALTERNATIVES ANALYSIS REPORT

<u>HEC-1 PMP Vineyard Road and Rittenhouse FRS</u>
<u>Upgrade Dam to High Hazard</u>

Area	Elevation
0	1564.2
65	1565
200	1566.6
400	1569.5
600	1571.8
800	1574.2
1000	1576.1
1200	1579.2

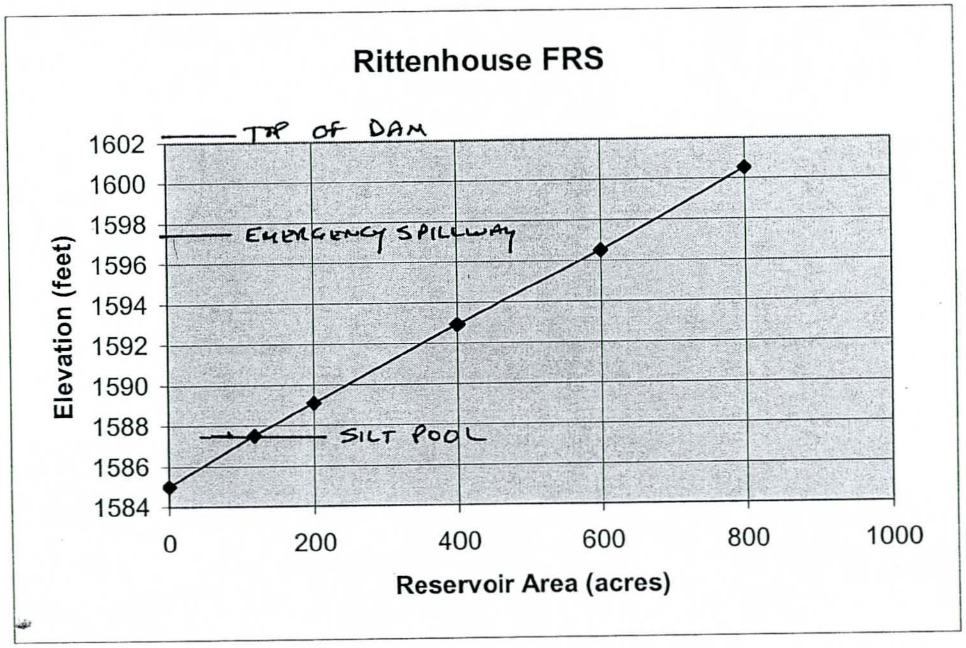


EMERGENCY SPILLWAY CREST = 1574.8 ft
TOP OF DAM (EXISTING) = 1579.5 ft
SILT POOL = 1566.2 ft
54" RCP PRINC SPILLWAY

KHA 091131003

Area	Elevation
0	1585
118	1587.5
200	1589.1
400	1592.9
600	1596.5
800	1600.5

EMERGENCY SPILLWAY CREST = 1597.6 FT
TOP OF DAM (EXISTING) = 1602.3 FT
SILT POOL = 1587.5 (FT)



KHA 071131003

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1*****************************************                  ***************************************
 *                                       *                  *                                     *
 *  FLOOD HYDROGRAPH PACKAGE  (HEC-1)    *                  *  U.S. ARMY CORPS OF ENGINEERS       *
 *            JUN  1998                  *                  *  HYDROLOGIC ENGINEERING CENTER      *
 *          VERSION 4.1                  *                  *        609 SECOND STREET            *
 *                                       *                  *     DAVIS, CALIFORNIA 95616         *
 *      DATE   24AUG00  TIME  15:22:12   *                  *        (916) 756-1104               *
 *                                       *                  *                                     *
 *****************************************                  ***************************************


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


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

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

                                                                                                PAGE  1
1                                      HEC-1 INPUT

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

      1         ID  VINEYARD AND RITTENHOUSE F.R.S. ANALYSIS
      2         ID   6-HR PMP STORM : SCS EXCESS & HYDROGRAPH DEVELOPMENT.
      3         ID  MUSKINGUM ROUTING
      4         ID  TIME INCREMENT= 5 MINUTES
      5         ID  FILENAME = VRALT.DAT   BASE MODEL=VRADJPMP.DAT MODIFIED TOP OF DAM AND EMERGEN
      6         ID  TO PREVENT OVERTOPPING OF DAM AND OBTAIN THREE FEET OF FREEBOARD.  START ITERA
      7         ID  THEN VINEYARD FRS
                *DIAGRAM
      8         IT       5       0       0     150
      9         IO       5
     10         IN      15

     11         KK  SUBR1
     12         KM  HYDROGRAPH FOR SUBBASIN R1
     13         BA    1.56
     14         PB    7.60
     15         PI     .05     .05     .05     .05     .15     .15     .15     .15     .30     .30
     16         PI     .30     .30    2.30    1.10     .60     .50     .18     .18     .18     .18
     17         PI     .10     .10     .10     .10
     18         LS       0      94
     19         UD    0.72

     20         KK  RR2a
     21         KM  ROUTE HYDROGRAPH FROM SUBBASIN R1 TO A2 ACROSS HWY 60
     22         RS       1    STOR       0
     23         SA       0       1      26     115     224     326
     24         SE    1800  1810.6    1820    1830    1840    1850
     25         SQ       0     290     692    1204    2198    6601   17223   34714
     26         SE    1811    1812    1813    1814  1815.9  1818.5  1819.5  1820.5

     27         KK  RR2-1
     28         KM  ROUTE HYDROGRAPH FROM SUBBASIN R1 THRU REACH R2-1
     29         RM       9    1.06     0.3

     30         KK  RR2-2
     31         KM  ROUTE HYDROGRAPH FROM SUBBASIN R1 THRU REACH R2-2
     32         RM       9    1.06     0.3

     33         KK  RR2-3
     34         KM  ROUTE HYDROGRAPH FROM SUBBASIN R1 THRU REACH R2-3
     35         RM       9    1.06     0.3

     36         KK  SUBR2
     37         KM  HYDROGRAPH FOR SUBBASIN R2
     38         BA    8.43
     39         LS       0      93
     40         UD     1.9

     41         KK  HCR2
     42         KM  COMBINE HYDROGRAPHS FOR SUBBASINS R1 & R2
     43         HC       2
                                                                                                PAGE  2
1                                      HEC-1 INPUT

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

     44         KK  SUB R3
     45         KM  HYDROGRAPH FOR SUBBASIN R3
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 46          BA     1.1
 47          LS       0       95
 48          UD    0.69

 49          KK  RR4a
 50          KM  ROUTE HYDROGRAPH FROM SUBBASIN R3 TO R4 ACROSS HWY 60
 51          RS       1    STOR       0
 52          SA       0       1      26     115     205
 53          SE  1823.4  1827.8    1840    1850    1860
 54          SQ       0     412    1127    1832    9597   23670   46850
 55          SE  1827.8    1830    1832  1833.6    1837    1838    1839

 56          KK  RR4-1
 57          KM  ROUTE HYDROGRAPH FROM SUBBASIN R3 THRU REACH R4-1
 58          RM       7    0.85     0.3

 59          KK  RR4-2
 60          KM  ROUTE HYDROGRAPH FROM SUBBASIN R3 THRU REACH R4-2
 61          RM       7    0.85     0.3

 62          KK  SUBR4
 63          KM  HYDROGRAPH FOR SUBBASIN R4
 64          BA    2.67
 65          LS       0       93
 66          UD    1.01

 67          KK  HCR4
 68          KM  COMBINE HYDROGRAPHS FOR SUBBASINS R3 & R4
 69          HC       2

 70          KK  SUBR5
 71          KM  HYDROGRAPH FOR SUBBASIN R5
 72          BA    2.71
 73          LS       0       98
 74          UD    0.39

 75          KK  RR6
 76          KM  ROUTE HYDROGRAPH FROM SUBBASIN R5
 77          RM       5    0.51     0.3

 78          KK  SUBR6
 79          KM  HYDROGRAPH FOR SUBBASIN R6
 80          BA    4.19
 81          LS       0       97
 82          UD     0.7

 83          KK  HCR6
 84          KM  COMBINE HYDROGRAPHS FOR SUBBASINS R5 & R6
 85          HC       2
                                         HEC-1 INPUT                                       PAGE  3
1

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


 86          KK  RR7
 87          KM  ROUTE HYDROGRAPH FROM SUBBASINS R5 & R6
 88          RM       6    0.68     0.3

 89          KK  SUBR7
 90          KM  HYDROGRAPH FOR SUBBASIN R7
 91          BA    2.67
 92          LS       0       97
 93          UD    0.41

 94          KK  HCR7
 95          KM  COMBINE HYDROGRAPHS FOR SUBBASINS R5 THRU R7
 96          HC       2

 97          KK  RRB-1
 98          KM  ROUTE HYDROGRAPH FROM SUBBASINS R5 THRU R1 THRU REACH R8-1
 99          RM       6     0.7     0.3

100          KK  RR8-2
101          KM  ROUTE HYDROGRAPH FROM SUBBASINS R5 THRU R7 THRU REACH R8-2
102          RM       6     0.7     0.3

103          KK  SUBR8
104          KM  HYDROGRAPH FOR SUBBASIN R8
105          BA       2
106          LS       0       93
107          UD    0.83

108          KK  HCR8
109          KM  COMBINE HYDROGRAPHS FOR SUBBASINS R5 THRU R8
110          HC       2

111          KK  RR9a
112          KM  ROUTE HYDROGRAPH FROM SUBBASINS R5 THRU R8 TO R9 ACROSS HWY 60
113          RS       1    STOR       0
114          SA       0       1      26      77     141
115          SE  1813.9  1829.5    1840    1850    1860
116          SQ       0     563    2236    4601    5986   27109
117          SE  1829.5    1831    1833    1835    1836    1840

118          KK  RR9
119          KM  ROUTE HYDROGRAPH FROM SUBBASINS RS THRU R8
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      120          RM       4     0.44      0.3

      121          KK  SUBR9
      122          KM  KM HYDROGRAPH FOR SUBBASIN R9
      123          BA    0.26
      124          LS       0        90
      125          UD    0.27
1                                           HEC-1 INPUT                                      PAGE  4

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


      126          KK  HCR9
      127          KM  COMBINE HYDROGRAPHS FOR SUBBASINS R5 THRU R9
      128          HC       2

      129          KK  SUBR10
      130          KM  HYDROGRAPH FOR SUBBASIN R10
      131          BA    3.35
      132          LS       0        96
      133          UD    0.92

      134          KK  SUBR11
      135          KM  HYDROGRAPH FOR SUBBASIN R11
      136          BA     3.9
      137          LS       0        96
      138          UD    0.78

      139          KK  HCR11
      140          KM  COMBINE HYDROGRAPHS FOR SUBBASINS R10 & R11
      141          HC       2

      142          KK  RR12
      143          KM  ROUTE HYDROGRAPH FROM SUBBASINS R10 & R11
      144          RM       5     0.55      0.3

      145          KK  SUBR12
      146          KM  HYDROGRAPH FOR SUBBASIN R12
      147          BA    4.46
      148          LS       0        95
      149          UD    0.76

      150          KK  HCR12
      151          KM  COMBINE HYDROGRAPHS FOR SU8BASINS R10 THRU R12
      152          HC       2

      153          KK  RR13a
      154          KM  ROUTE HYDROGRAPH FROM SUBBASINS R10 THRU R12 TO R13 ACROSS HWY 60
      155          RS       1     STOR        0
      156          SA       0        1       83      186      358
      157          SE  1793.6   1819.4     1840     1850     1860
      158          SQ       0     2071     5240     9357    14240    18616    62142
      159          SE  1819.4     1822     1824     1826     1828   1829.6   1834.5

      160          KK  RR13
      161          KM  ROUTE HYDROGRAPH FROM SUBBASINS R10 THRU R13
      162          RM       4     0.41      0.3

      163          KK  SUBR13
      164          KM  HYDROGRAPH FOR SUBBASIN R13
      165          BA    1.27
      166          LS       0        95
      167          UD    0.85
1                                           HEC-1 INPUT                                      PAGE  5

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


      168          KK  HCR13a
      169          KM  COMBINE HYDROGRAPHS FOR SUBBASINS R10 THRU R13
      170          HC       2

      171          KK  HCR13b
      172          KM  COMBINE HYDROGRAPHS FOR SUBBASINS R5 THRU R13
      173          HC       2

      174          KK  RR14
      175          KM  ROUTE HYDROGRAPH FROM SUBBASINS R5 THRU R13
      176          RM       9      1.1      0.3

      177          KK  SUBR14
      178          KM  HVDROGRAPH FOR SUBBASIN R14
      179          BA    2.82
      180          LS       0        92
      181          UD    0.76

      182          KK  HCR14a
      183          KM  COMBINE HYDROGRAPHS FOR SUBBASINS RS THRU R14
      184          HC       2

      185          KK  HCR14b
      186          KM  COMBINE HYDROGRAPHS FOR SUBBASINS R3 THRU R14
      187          HC       2

      188          KK  RR15-1
      189          KM  ROUTE HYDROGRAPH FROM SUBBASINS R3 THRU R14 THRU REACH R15-1
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 190          RM        5    0.63      0.3

 191          KK  RR15-2
 192          KM  ROUTE HYDROGRAPH FROM SUBBASINS R3 THRU R14 THRU REACH R15-2
 193          RM        5    0.63      0.3

 194          KK  SUBR15
 195          KM  HYDROGRAPH FOR SUBBASIN R15
 196          BA     6.3
 197          LS        0       93
 198          UD    1.47

 199          KK  HCR15a
 200          KM  COMBINE HYDROGRAPHS FOR SUBBASINS R3 THRU R15
 201          HC        2

 202          KK  HCR15b
 203          KM  COMBINE HYDROGRAPHS FOR SUBBASINS R1 THRU R15
 204          HC        2

 205          KK  HDR16
 206          KM  DIVERT FLOW FROM RITTENHOUSE (TO VINEYARD FRS)
 207          DT  DVR16
 208          DI        0      100      500     1000  100000
 209          DQ        0       50      105      105     105
1                                        HEC-1 INPUT                                       PAGE  6

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


 210          KK  RITTENHOUSE
 211          KM  ***************************************************************************
 212          KM  RESERVOIR ROUTING THRU RITTENHOUSE F.R.S.
 213          KM  CHANGED RS RECORD TO "ELEV" AND STARTING CONDITIONS AT ELEV = 1597.6
 214          KM  WHICH IS THE ELEVATION OF THE EMERGENCY SPILLWAY CREST
 215          KM  CHANGE SL RECORD FIELD FOUR TO EXPONENT = 0.5 (ORIFICE EQN)
 216          KM  CHANGE SS RECORD SPILLWAY CREST TO 1597.6
 217          KM  NOTE SILT POOL ELEVATION = 1587.5
 218          KM  ***************************************************************************
 219          KM    CHANGED TOP OF DAM TO 1606.6 AND INCREASE EMERGENCY SPILLWAY WIDTH TO 900-FT
 220          KM  ***************************************************************************
 221          KO        1        2
 222          RS        1     ELEV  1597.6
 223          SA        0      118      200      400      600      800
 224          SE     1585   1587.5   1589.1   1592.9   1596.5   1600.5
 225          SL   1577.7     5.94     0.62      0.5
 226          SS   1597.6      900        3      1.5
 227          ST   1606.6    19000      3.3      1.5

 228          KK  SUBVI
 229          KM  HYDROGRAPH FOR SUBBASIN V1
 230          BA     3.03
 231          LS        0       97
 232          UD     0.53

 233          KK  SUBV2
 234          KM  HYDROGRAPH FOR SUBBASIN V2
 235          BA     3.34
 236          LS        0       93
 237          UD     0.47

 238          KK  RV3
 239          KM  ROUTE HYDROGRAPH FROM SUBBASIN V2
 240          RM        4     0.38      0.3

 241          KK  SUBV3
 242          KM  HYDROGRAPH FOR SUBBASIN V3
 243          BA      0.4
 244          LS        0       95
 245          UD      0.3

 246          KK  HCV3
 247          KM  COMBINE HYDROGRAPHS FOR SUBBASINS V1 THRU V3
 248          HC        3

 249          KK  RV4
 250          KM  ROUTE HYDROGRAPH FROM SUBBASINS V1 THRU V3
 251          RM        4     0.43      0.3
1                                        HEC-1 INPUT                                       PAGE  7

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


 252          KK  SUBV4
 253          KM  HYDROGRAPH FOR SUBBASIN V4
 254          BA     1.85
 255          LS        0       95       15
 256          UD     0.52

 257          KK  HCV4
 258          KM  COMBINE HYDROGRAPHS FOR SUBBASINS V1 THRU V4
 259          HC        2

 260          KK  RV5
 261          KM  ROUTE HYDROGRAPH FROM SUBBASINS V1 THRU V4
 262          RM        8     0.66      0.3
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 263        KK  SUBV5
 264        KM  HYDROGRAPH FOR SUBBASIN V5
 265        BA    1.66
 266        LS       0       95
 267        UD    0.45

 268        KK  HCV5
 269        KM  COMBINE HYDROGRAPHS FOR SUBBASINS V1 THRU V5
 270        HC       2

 271        KK  RV6a
 272        KM  ROUTE HYDROGRAPH FROM SUBBASINS V1 THRU V5 TO V6 ACROSS HWY 60
 273        RS       1     STOR       0
 274        SA       0        1       45      147      205
 275        SE  1719.9   1739.7     1750     1760     1770
 276        SQ       0      387     2348     6889    19775    36999
 277        SE  1739.7   1740.7   1742.7   1745.6     1749     1750

 278        KK  RV6-1
 279        KM  ROUTE HYDROGRAPH FROM SUBBASINS V1 THRU V5 THRU REACH RV6-1
 280        RM       9     0.83      0.3

 281        KK  RV6-2
 282        KM  ROUTE HYDROGRAPH FROM SUBBASINS V1 THRU V5 THRU REACH RV6-2
 283        RM       9     0.83      0.3

 284        KK  SUBV6
 285        KM  HYDROGRAPH FOR SUBBASIN V6
 286        BA    3.59
 287        LS       0       87
 288        UD    0.99

 289        KK  HCV6
 290        KM  COMBINE HYDROGRAPHS FOR SUBBASINS V1 THRU V6
 291        HC       2
1                                    HEC-1 INPUT                                            PAGE  8

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


 292        KK  SUBV7
 293        KM  HYDROGRAPH FOR SUBBASIN V7
 294        BA    1.69
 295        LS       0       97
 296        UD    0.74

 297        KK  SUBV8
 298        KM  HYDROGRAPH FOR SUBBASIN V8
 299        BA    0.73
 300        LS       0       95       50
 301        UD    0.33

 302        KK  HCV8
 303        KM  COMBINE HYDROGRAPHS FOR SUBBASINS V7 & V8
 304        HC       2

 305        KK  RV9
 306        KM  ROUTE HYDROGRAPH FROM SUBBASINS V7 & V8
 307        RM       3     0.23      0.3

 308        KK  SUBV9
 309        KM  HYDROGRAPH FOR SUBBASIN V9
 310        BA    0.95
 311        LS       0       96
 312        UD    0.41

 313        KK  HCV9
 314        KM  COMBINE HYDROGRAPHS FOR SUBBASINS V7 THRU V9
 315        HC       2

 316        KK  RV10a
 317        KM  ROUTE HYDROGRAPH FROM SUBBASINS V7 THRU V9 TO V10 ACROSS HWY 60
 318        RS       1     STOR       0
 319        SA       0       19       77      179      294
 320        SE    1745     1770     1780     1790     1800
 321        SQ       0     1211     3230     6108     9716    14201    63126
 322        SE    1773     1775     1777     1779     1781   1783.3   1788.5

 323        KK  RV10-1
 324        KM  ROUTE HYDROGRAPH FROM SUBBASINS V7 THRU V9 THRU REACH V10-1
 325        RM       9     0.86      0.3

 326        KK  RV10-2
 327        KM  ROUTE HYDROGRAPH FROM SUBBASINS V7 THRU V9 THRU REACH V10-2
 328        RM       9     0.86      0.3

 329        KK  RV10-3
 330        KM  ROUTE HYDROGRAPH FROM SUBBASINS V7 THRU V9 THRU REACH V10-3
 331        RM       9     0.86      0.3
1                                    HEC-1 INPUT                                            PAGE  9

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


 332        KK  SUBV10
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  333          KM  HYDROGRAPH FOR SUBBASIN V10
  334          BA    5.48
  335          LS       0       95
  336          UD    1.54

  337          KK  HCV10
  338          KM  COMBINE HYDROGRAPHS FOR SUBBASINS V7 THRU V10
  339          HC       2

  340          KK  SUBV11
  341          KM  HYDROGRAPH FOR SUBBASIN V11
  342          BA    0.44
  343          LS       0       97
  344          UD    0.93

  345          KK  RV12a
  346          KM  ROUTE HYDROGRAPH FROM SUBBASIN V11 TO V12 ACROSS HWY 60
  347          RS       1     STOR       0
  348          SA       0       32      64      96
  349          SE  1773.7     1790    1800    1810
  350          SQ       0       24     237     363    5249   12106   38068
  351          SE  1773.7   1774.7  1778.7  1780.7  1784.5  1785.5  1787.5

  352          KK  RV12-1
  353          KM  ROUTE-HYDROGRAPH FROM SUBBASIN V11 THRU REACH V12-1
  354          RM       9     1.15     0.3

  355          KK  RV12-2
  356          KM  ROUTE HYDROGRAPH FROM SUBBASIN V11 THRU REACH V12-2
  357          RM       9     1.15     0.3

  358          KK  SUBV12
  359          KM  HYDROGRAPH FOR SUBBASIN V12
  360          BA    3.32
  361          LS       0       95
  362          UD    1.38

  363          KK  HCV12
  364          KM  COMBINE HYDROGRAPHS FOR SUBBASINS V11 & V12
  365          HC       2

  366          KK  SUBV13
  367          KM  HYDROGRAPH FOR SUBBASIN V13
  368          BA    2.35
  369          LS       0       97
  370          UD    0.99

  371          KK  KK RV14a
  372          KM  ROUTE HYDROGRAPH FROM SUBBASIN V13 TO V14 ACROSS HWY 60
  373          RS       1     STOR       0
  374          SA       0        1      38      96     154
  375          SE  1780.4   1786.9    1800    1810    1820
  376          SQ       0      120     372    1191    1925   16426   62329
1                                          HEC-1 INPUT                                  PAGE 10

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

  377          SE  1786.9     1788    1789  1791.5    1794    1796    1798

  378          KK  RV14-1
  379          KM  ROUTE HYDROGRAPH FROM SUBBASIN V13 THRU REACH V14-1
  380          RM       9     0.96     0.3

  381          KK  RV14-2
  382          KM  ROUTE HYDROGRAPH FROM SUBBASIN V13 THRU REACH V14-2
  383          RM       9     0.96     0.3

  384          KK  RY14-3
  385          KM  ROUTE HYDROGRAPH FROM SUBBASIN V13 THRU REACH V14-3
  386          RM       9     0.96     0.3

  387          KK  SUBV14
  388          KM  HYDROGRAPH FOR SUBBASIN V14
  389          BA    4.09
  390          LS       0       95
  391          UD    1.73

  392          KK  HCV14
  393          KM  COMBINE HYDROGRAPHS FOR SUBBASINS V13 & V14
  394          HC       2

  395          KK  SUBV15
  396          KM  HYDROGRAPH FOR SUBBASIN V15
  397          BA    1.64
  398          LS       0       97
  399          UD    0.62

  400          KK  RV16a
  401          KM  ROUTE HYDROGRAPH FROM SUBBASIN V15 TO V16 ACROSS HWY 60
  402          RS       1     STOR       0
  403          SA       0        1      58     141     160
  404          SE  1779.4     1786    1800    1810    1820
  405          SQ       0      192     572    1061    1866    9058   26139
  406          SE    1786   1787.5    1789  1790.5  1792.6    1796    1797

  407          KK  RV16
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 408         KM  ROUTE HYDROGRAPH FROM SUBBASIN V15
 409         RM       4    0.48     0.3

 410         KK  SUBV16
 411         KM  HYDROGRAPH FOR SUBBASIN V16
 412         BA    0.76
 413         LS       0       97
 414         UD    0.29

 415         KK  HCV16
 416         KM  COMBINE HYDROGRAPHS FOR SUBBASINS V15 & V16
 417         HC       2
1                                        HEC-1 INPUT                                          PAGE 11

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


 418         KK  SUBV17
 419         KM  HYDROGRAPH FOR SUBBASIN V17
 420         BA    4.06
 421         LS       0       97
 422         UD    0.69

 423         KK  RV18
 424         KM  ROUTE HYDROGRAPH FROM SUBBASIN V17
 425         RM       9    1.14     0.3

 426         KK  SUBV18
 427         KM  HYDROGRAPH FOR SUBBASIN V18
 428         BA    5.06
 429         LS       0       97
 430         UD    0.93

 431         KK  HCV18
 432         KM  COMBINE HYDROGRAPHS FOR SUBBASINS V17 & V18
 433         HC       2

 434         KK  RV19
 435         KM  ROUTE HYDROGRAPH FROM SUBBASINS V17 & V18
 436         RM       6    0.38     0.3

 437         KK  SUBV19
 438         KM  HYDROGRAPH FOR SUBBASIN V19
 439         BA    2.77
 440         LS       0       96
 441         UD    0.59

 442         KK  HCV19
 443         KM  COMBINE HYDROGRAPHS FOR SUBBASINS V17 THOU V19
 444         HC       2

 445         KK  RV20a
 446         KM  ROUTE HYDROGRAPH FROM SUBBASINS V17 THOU V19 TO V20 ACROSS HWY 60
 447         RS       1    STOR       0
 448         SA       0       1      13      51     154     294
 449         SE    1768  1796.4    1800    1810    1820    1830
 450         SQ       0    1226    4652    9549   16620   41739
 451         SE  1796.4    1798    1800    1802  1804.3  1808.5

 452         KK  RV20
 453         KM  ROUTE HYDROGRAPH FROM SUBBASINS V17 THRU V19
 454         RM       6    0.72     0.3

 455         KK  SUBV20
 456         KM  HYDROGRAPH FOR SUBBASIN V20
 457         BA    0.48
 458         LS       0       95
 459         UD    0.43
1                                        HEC-1 INPUT                                          PAGE 12

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


 460         KK  HCV20a
 461         KM  COMBINE HYDROGRAPHS FOR SUBBASINS V17 THOU V20
 462         HC       2

 463         KK  HCV20b
 464         KM  COMBINE HYDROGRAPHS FOR SUBBASINS V15 THRU V20
 465         HC       2

 466         KK  RV21
 467         KM  ROUTE HYDROGRAPH FROM SUBBASINS V15 THRU V20
 468         RM       9    1.21     0.3

 469         KK  SUBV21
 470         KM  HYDROGRAPH FOR SUBBASIN V21
 471         BA    4.43
 472         LS       0       94
 473         UD    1.83

 474         KK  HCV21a
 475         KM  COMBINE HYDROGRAPHS FOR SUBBASINS V15 THRU V21
 476         HC       2

 477         KK  HCV21b
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 478          KM  COMBINE HYDROGRAPHS FOR SUBBASINS V1 THRU V21
 479          HC        5

 480          KK  HCV22a
 481          KM  DIVERSION FROM RITTENHOUSE FRS
 482          DR  DVR16

 483          KK  HCV22b
 484          KM  COMBINING DIVERSION WITH UPSTREAM HYDROGRAPH
 485          HC        2

 486          KK  VINEYARD
 487          KM  ******************************************************************************
 488          KM  RESERVOIR ROUTING THRU VINEYARD F.R.S.
 489          KM  CHANGE RS RECORD FIELD 2 TO "ELEV" AND FIELD 3 TO "1574.8"
 490          KM  TO MODEL RESERVOIR STARTING CONDITIONS AT THE EMERGENCY SPILLWAY CREST AS
 491          KM  STATED IN JMM REPORT BUT NOT DONE IN JMM MODEL
 492          KM  CHANGE SL RECORD FIELD 4 TO "0.5" ORIFICE EQN
 493          KM  ******************************************************************************
 494          KM   CHANGE TOP OF DAM TO 1584.4 AND INCREASE SPILLWAY TO 900-FT
 495          KM  ******************************************************************************
 496          KO        1        2
 497          RS        1     ELEV  1574.8
 498          SA        0       65     200     400     600     800    1000    1200
 499          SE  1564.2     1565  1566.6  1569.5  1571.8  1574.2  1576.1  1579.2
 500          SL  1561.5     15.9     0.7     0.5
 501          SS  1574.8      900       3     1.5
 502          ST  1584.4    28829     3.3     1.5
 503          ZZ
1
                 SCHEMATIC DIAGRAM OF STREAM NETWORK
 INPUT
  LINE       (V) ROUTING           (--->) DIVERSION OR PUMP FLOW

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

  11         SUBR1
                 V
                 V
  20         RR2a
                 V
                 V
  27         RR2-1
                 V
                 V
  30         RR2-2
                 V
                 V
  33         RR2-3
                 .
                 .
  36             .           SUBR2
                 .               .
                 .               .
  41         HCR2 ............
                 .
                 .
  44             .           SUB R3
                 .               V
                 .               V
  49             .           RR4a
                 .               V
                 .               V
  56             .           RR4-1
                 .               V
                 .               V
  59             .           RR4-2
                 .               .
                 .               .
  62             .               .           SUBR4
                 .               .               .
                 .               .               .
  67             .           HCR4 ............
                 .               .
                 .               .
  70             .               .           SUBR5
                 .               .               V
                 .               .               V
  75             .               .           RR6
                 .               .               .
                 .               .               .
  78             .               .               .           SUBR6
                 .               .               .               .
                 .               .               .               .
  83             .               .           HCR6 ............
                 .               .               V
                 .               .               V
  86             .               .           RR7
                 .               .               .
                 .               .               .
  89             .               .               .           SUBR7
                 .               .               .               .
                 .               .               .               .
  94             .               .           HCR7 ............
                 .               .               V
                 .               .               V
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   97        .          .     RRB-1
             .          .         V
             .          .         V
  100        .          .     RR8-2
             .          .          .
             .          .          .
             .          .          .     SUBR8
             .          .          .          .
             .          .          .          .
  108        .          .     HCR8 ............
             .          .         V
             .          .         V
  111        .          .     RR9a
             .          .         V
             .          .         V
  118        .          .     RR9
             .          .          .
             .          .          .
  121        .          .          .     SUBR9
             .          .          .          .
             .          .          .          .
  126        .          .     HCR9 ............
             .          .          .
             .          .          .
  129        .          .          .     SUBR10
             .          .          .          .
             .          .          .          .
  134        .          .          .          .     SUBR11
             .          .          .          .          .
             .          .          .          .          .
  139        .          .          .     HCR11 ............
             .          .          .         V
             .          .          .         V
  142        .          .          .     RR12
             .          .          .          .
             .          .          .          .
  145        .          .          .          .     SUBR12
             .          .          .          .          .
             .          .          .          .          .
  150        .          .          .     HCR12 ............
             .          .          .         V
             .          .          .         V
  153        .          .          .     RR13a
             .          .          .         V
             .          .          .         V
             .          .          .     RR13
             .          .          .          .
             .          .          .          .
  163        .          .          .          .     SUBR13
             .          .          .          .          .
             .          .          .          .          .
  168        .          .          .     HCR13a............
             .          .          .          .
             .          .          .          .
  171        .          .     HCR13b............
             .          .         V
             .          .         V
  174        .          .     RR14 .
             .          .          .
             .          .          .
  177        .          .          .     SUBR14
             .          .          .          .
             .          .          .          .
  182        .          .     HCR14a............
             .          .          .
             .          .          .
  185        .     HCR14b............
             .         V
             .         V
  188        .     RR15-1
             .         V
             .         V
  191        .     RR15-2
             .          .
             .          .
  194        .          .     SUBR15
             .          .          .
             .          .          .
  199        .     HCR15a............
             .          .
             .          .
  202   HCR15b............
             .
             .
  207        .------->  DVR16
  205   HDR16
            V
            V
        RITTEN
             .
             .
  228        .     SUBVI
             .          .
             .          .
  233        .          .     SUBV2
             .          .         V
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              .           .           V
 238          .           .      RV3
              .           .           .
              .           .           .
              .           .           .      SUBV3
              .           .           .           .
              .           .           .           .
 246          .      HCV3 .........................
              .           V
              .           V
 249          .      RV4
              .           .
              .           .
 252          .           .      SUBV4
              .           .           .
              .           .           .
 257          .      HCV4 .............
              .           V
              .           V
 260          .      RV5
              .           .
              .           .
 263          .           .      SUBV5
              .           .           .
              .           .           .
 268          .      HCV5 .............
              .           V
              .           V
 271          .      RV6a
              .           V
              .           V
 278          .      RV6-1
              .           V
              .           V
 281          .      RV6-2
              .           .
              .           .
 284          .           .      SUBV6
              .           .           .
              .           .           .
 289          .      HCV6 .............
              .           .
              .           .
 292          .           .      SUBV7
              .           .           .
              .           .           .
              .           .           .      SUBV8
              .           .           .           .
              .           .           .           .
 302          .           .      HCV8 .............
              .           .           V
              .           .           V
 305          .           .      RV9
              .           .           .
              .           .           .
 308          .           .           .      SUBV9
              .           .           .           .
              .           .           .           .
 313          .           .      HCV9 .............
              .           .           V
              .           .           V
 316          .           .      RV10a
              .           .           V
              .           .           V
 323          .           .      RV10-1
              .           .           V
              .           .           V
 326          .           .      RV10-2
              .           .           V
              .           .           V
 329          .           .      RV10-3
              .           .           .
              .           .           .
 332          .           .           .      SUBV10
              .           .           .           .
              .           .           .           .
 337          .           .      HCV10 ............
              .           .           .
              .           .           .
 340          .           .           .      SUBV11
              .           .           .           V
              .           .           .           V
 345          .           .           .      RV12a
              .           .           .           V
              .           .           .           V
 352          .           .           .      RV12-1
              .           .           .           V
              .           .           .           V
 355          .           .           .      RV12-2
              .           .           .           .
              .           .           .           .
 358          .           .           .           .      SUBV12
              .           .           .           .           .
              .           .           .           .           .
 363          .           .           .      HCV12 .............
              .           .           .           .
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               .           .           .           .
 366           .           .           .           .      SUBV13
               .           .           .           .           V
               .           .           .           .           V
  71           .           .           .           .      KK RV1
               .           .           .           .           V
               .           .           .           .           V
 378           .           .           .           .      RV14-1
               .           .           .           .           V
               .           .           .           .           V
 381           .           .           .           .      RV14-2
               .           .           .           .           V
               .           .           .           .           V
 384           .           .           .           .      RY14-3
               .           .           .           .           .
               .           .           .           .           .
 387           .           .           .           .           .      SUBV14
               .           .           .           .           .           .
               .           .           .           .           .           .
 392           .           .           .           .      HCV14 ............
               .           .           .           .           .
               .           .           .           .           .
 395           .           .           .           .           .      SUBV15
               .           .           .           .           .           V
               .           .           .           .           .           V
 400           .           .           .           .           .      RV16a
               .           .           .           .           .           V
               .           .           .           .           .           V
 407           .           .           .           .           .      RV16
               .           .           .           .           .           .
               .           .           .           .           .           .
 410           .           .           .           .           .           .      SUBV16
               .           .           .           .           .           .           .
               .           .           .           .           .           .           .
 415           .           .           .           .           .      HCV16 ............
               .           .           .           .           .           .
               .           .           .           .           .           .
 418           .           .           .           .           .           .      SUBV17
               .           .           .           .           .           .           V
               .           .           .           .           .           .           V
 423           .           .           .           .           .           .      RV18
               .           .           .           .           .           .           .
               .           .           .           .           .           .           .
 426           .           .           .           .           .           .           .      SUBV18
               .           .           .           .           .           .           .           .
               .           .           .           .           .           .           .           .
   1           .           .           .           .           .           .      HCV18 ............
               .           .           .           .           .           .           V
               .           .           .           .           .           .           V
 434           .           .           .           .           .           .      RV19
               .           .           .           .           .           .           .
               .           .           .           .           .           .           .
 437           .           .           .           .           .           .           .      SUBV19
               .           .           .           .           .           .           .           .
               .           .           .           .           .           .           .           .
 442           .           .           .           .           .           .      HCV19 ............
               .           .           .           .           .           .           V
               .           .           .           .           .           .           V
 445           .           .           .           .           .           .      RV20a
               .           .           .           .           .           .           V
               .           .           .           .           .           .           V
 452           .           .           .           .           .           .      RV20
               .           .           .           .           .           .           .
               .           .           .           .           .           .           .
 455           .           .           .           .           .           .           .      SUBV20
               .           .           .           .           .           .           .           .
               .           .           .           .           .           .           .           .
 460           .           .           .           .           .           .     HCV20a............
               .           .           .           .           .           .           .
               .           .           .           .           .           .           .
 463           .           .           .           .           .     HCV20b............
               .           .           .           .           .           V
               .           .           .           .           .           V
 466           .           .           .           .           .      RV21
               .           .           .           .           .           .
               .           .           .           .           .           .
 469           .           .           .           .           .           .      SUBV21
               .           .           .           .           .           .           .
               .           .           .           .           .           .           .
 474           .           .           .           .           .     HCV21a............
               .           .           .           .           .           .
               .           .           .           .           .           .
 477           .     HCV21b................................................
               .           .
               .           .
 482           .           .           .<-------   DVR16
  80           .           .     HCV22a
               .           .           .
               .           .           .
 483           .     HCV22b............
               .           V
               .           V
 486           .     VINEYA

  (***) RUNOFF ALSO COMPUTED AT THIS LOCATION
1*********************************************                                                 *****************************************
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*                                          *          *                                       *
*  FLOOD HYDROGRAPH PACKAGE  (HEC-1)       *          *   U.S. ARMY CORPS OF ENGINEERS        *
*          JUN   1998                      *          *   HYDROLOGIC ENGINEERING CENTER       *
*        VERSION 4.1                       *          *        609 SECOND STREET              *
*                                          *          *    DAVIS, CALIFORNIA 95616            *
*  N DATE   24AUG00   TIME  15:22:12       *          *        (916) 756-1104                 *
*                                          *          *                                       *
********************************************          *****************************************



                    VINEYARD AND RITTENHOUSE F.R.S. ANALYSIS
                     6-HR PMP STORM : SCS EXCESS & HYDROGRAPH DEVELOPMENT.
                    MUSKINGUM ROUTING
                    TIME INCREMENT= 5 MINUTES
                    FILENAME = VRALT.DAT   BASE MODEL=VRADJPMP.DAT MODIFIED TOP OF DAM AND EMERGEN
                    TO PREVENT OVERTOPPING OF DAM AND OBTAIN THREE FEET OF FREEBOARD.  START ITERA
                    THEN VINEYARD FRS

  9 IO         OUTPUT CONTROL VARIABLES
                     IPRNT           5  PRINT CONTROL
                     IPLOT           0  PLOT CONTROL
                     QSCAL          0.  HYDROGRAPH PLOT SCALE

    IT         HYDROGRAPH TIME DATA
                      NMIN           5  MINUTES IN COMPUTATION INTERVAL
                     IDATE      1    0  STARTING DATE
                     ITIME        0000  STARTING TIME
                        NQ         150  NUMBER OF HYDROGRAPH ORDINATES
                    NDDATE      1    0  ENDING DATE
                    NDTIME        1225  ENDING TIME
                     ICENT          19  CENTURY MARK

                 COMPUTATION INTERVAL     .08 HOURS
                      TOTAL TIME BASE   12.42 HOURS

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


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


          **************
          *            *
210 KK    *   RITTEN   *    HOUSE
          *            *
          **************

221 KO         OUTPUT CONTROL VARIABLES
                     IPRNT           1  PRINT CONTROL
                     IPLOT           2  PLOT CONTROL
                     QSCAL          0.  HYDROGRAPH PLOT SCALE

             HYDROGRAPH ROUTING DATA

222 RS         STORAGE ROUTING
                     NSTPS           1  NUMBER OF SUBREACHES
                      ITYP        ELEV  TYPE OF INITIAL CONDITION
                    RSVRIC     1597.60  INITIAL CONDITION
                         X         .00  WORKING R AND D COEFFICIENT

223 SA               AREA           .0      118.0      200.0      400.0      600.0      800.0

224 SE         ELEVATION     1585.00    1587.50    1589.10    1592.90    1596.50    1600.50

225 SL         LOW-LEVEL OUTLET
                     ELEVL     1577.70  ELEVATION AT CENTER OF OUTLET
                     CAREA        5.94  CROSS-SECTIONAL AREA
                      COQL         .62  COEFFICIENT
                      EXPL         .50  EXPONENT OF HEAD

226 SS         SPILLWAY
                      CREL     1597.60  SPILLWAY CREST ELEVATION
                     SPWID      900.00  SPILLWAY WIDTH
                      COQW        3.00  WEIR COEFFICIENT
                      EXPW        1.50  EXPONENT OF HEAD

    ST         TOP OF DAM
                     TOPEL     1606.60  ELEVATION AT TOP OF DAM
                    DAMWID    19000.00  DAM WIDTH
                      COQD        3.30  WEIR COEFFICIENT
                      EXPD        1.50  EXPONENT OF HEAD

                                                 ***

                                   COMPUTED STORAGE-ELEVATION DATA
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STORAGE	.00	98.33	349.86	1468.14	3256.01	6046.44
ELEVATION	1585.00	1587.50	1589.10	1592.90	1596.50	1600.50

COMPUTED OUTFLOW-ELEVATION DATA

(EXCLUDING FLOW OVER DAM)

OUTFLOW	.00	83.46	87.47	91.88	96.76	102.19	108.26	115.11	122.87	131.76
ELEVATION	1585.00	1585.68	1586.47	1587.38	1588.43	1589.67	1591.14	1592.89	1595.01	1597.60
OUTFLOW	145.39	239.44	493.88	988.66	1803.33	3018.12	4713.00	6967.24	9861.32	13475.21
ELEVATION	1597.63	1597.72	1597.86	1598.06	1598.33	1598.64	1599.02	1599.46	1599.95	1600.50

COMPUTED STORAGE-OUTFLOW-ELEVATION DATA

(INCLUDING FLOW OVER DAM)

STORAGE	.00	2.02	19.97	84.49	98.33	228.80	349.86	471.13	853.97	1463.12
OUTFLOW	79.80	83.46	87.47	91.88	92.46	96.76	99.73	102.19	108.26	115.11
ELEVATION	1585.00	1585.68	1586.47	1587.38	1587.50	1588.43	1589.10	1589.67	1591.14	1592.89
STORAGE	1468.14	2425.94	3256.01	3944.47	3963.60	4020.74	4116.73	4252.79	4430.58	4652.48
OUTFLOW	115.15	122.87	128.07	131.76	145.39	239.44	493.88	988.66	1803.33	3018.12
ELEVATION	1592.90	1595.01	1596.50	1597.60	1597.63	1597.72	1597.86	1598.06	1598.33	1598.64
STORAGE	4921.32	5240.41	5613.89	6046.44						
OUTFLOW	4713.00	6967.24	9861.32	13475.21						
ELEVATION	1599.02	1599.46	1599.95	1600.50						

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HYDROGRAPH AT STATION RITTEN

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DA	MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE	DA	MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE	DA	MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE
1		0000	1	132.	3944.5	1597.6	1		0410	51	2839.	4621.9	1598.6	1		0820	101	38976.	8708.7	1603.5
1		0005	2	132.	3943.6	1597.6	1		0415	52	3396.	4715.6	1598.7	1		0825	102	38698.	8681.2	1603.5
1		0010	3	132.	3942.7	1597.6	1		0420	53	4025.	4816.2	1598.9	1		0830	103	38371.	8649.0	1603.5
1		0015	4	132.	3941.8	1597.6	1		0425	54	4725.	4923.2	1599.0	1		0835	104	38002.	8612.6	1603.4
1		0020	5	132.	3941.0	1597.6	1		0430	55	5498.	5036.3	1599.2	1		0840	105	37596.	8572.6	1603.4
1		0025	6	132.	3940.1	1597.6	1		0435	56	6342.	5155.1	1599.3	1		0845	106	37155.	8529.0	1603.3
		0030	7	132.	3939.2	1597.6	1		0440	57	7256.	5279.1	1599.5	1		0850	107	36680.	8482.0	1603.3
		0035	8	132.	3938.3	1597.6	1		0445	58	8236.	5408.0	1599.7	1		0855	108	36171.	8431.6	1603.2
		0040	9	132.	3937.5	1597.6	1		0450	59	9276.	5540.7	1599.9	1		0900	109	35619.	8376.8	1603.2
1		0045	10	132.	3936.6	1597.6	1		0455	60	10367.	5676.3	1600.0	1		0905	110	35031.	8318.3	1603.1
1		0050	11	132.	3935.7	1597.6	1		0500	61	11499.	5813.6	1600.2	1		0910	111	34405.	8256.0	1603.0
1		0055	12	132.	3934.8	1597.6	1		0505	62	12660.	5951.3	1600.4	1		0915	112	33740.	8189.7	1603.0
1		0100	13	132.	3934.0	1597.6	1		0510	63	13836.	6088.1	1600.6	1		0920	113	33037.	8119.3	1602.9
1		0105	14	132.	3933.1	1597.6	1		0515	64	15011.	6222.5	1600.7	1		0925	114	32295.	8044.9	1602.8
1		0110	15	132.	3932.2	1597.6	1		0520	65	16171.	6353.2	1600.9	1		0930	115	31514.	7966.4	1602.7
1		0115	16	132.	3931.3	1597.6	1		0525	66	17303.	6478.9	1601.0	1		0935	116	30698.	7884.1	1602.6
1		0120	17	132.	3930.5	1597.6	1		0530	67	18393.	6598.7	1601.2	1		0940	117	29848.	7798.1	1602.5
1		0125	18	132.	3929.6	1597.6	1		0535	68	19435.	6711.9	1601.3	1		0945	118	28970.	7708.9	1602.4
1		0130	19	132.	3928.8	1597.6	1		0540	69	20425.	6818.5	1601.4	1		0950	119	28070.	7617.2	1602.3
1		0135	20	132.	3928.0	1597.6	1		0545	70	21361.	6918.5	1601.6	1		0955	120	27152.	7523.1	1602.2
1		0140	21	132.	3927.3	1597.6	1		0550	71	22245.	7012.3	1601.7	1		1000	121	26222.	7427.4	1602.1
1		0145	22	132.	3926.7	1597.6	1		0555	72	23071.	7099.4	1601.8	1		1005	122	25286.	7330.6	1602.0
1		0150	23	132.	3926.3	1597.6	1		0600	73	23858.	7181.9	1601.9	1		1010	123	24349.	7233.2	1601.9
1		0155	24	132.	3926.2	1597.6	1		0605	74	24614.	7260.7	1601.9	1		1015	124	23417.	7135.7	1601.8
1		0200	25	132.	3926.3	1597.6	1		0610	75	25348.	7337.1	1602.0	1		1020	125	22494.	7038.6	1601.7
1		0205	26	132.	3926.8	1597.6	1		0615	76	26073.	7412.1	1602.1	1		1025	126	21585.	6942.3	1601.6
1		0210	27	132.	3927.8	1597.6	1		0620	77	26799.	7486.8	1602.2	1		1030	127	20693.	6847.2	1601.5
1		0215	28	132.	3929.2	1597.6	1		0625	78	27535.	7562.4	1602.3	1		1035	128	19820.	6753.4	1601.4
1		0220	29	132.	3931.3	1597.6	1		0630	79	28289.	7639.5	1602.4	1		1040	129	18969.	6661.3	1601.2
1		0225	30	132.	3934.4	1597.6	1		0635	80	29067.	7718.8	1602.5	1		1045	130	18142.	6571.2	1601.1
1		0230	31	132.	3938.3	1597.6	1		0640	81	29870.	7800.3	1602.5	1		1050	131	17341.	6483.2	1601.0
1		0235	32	132.	3943.4	1597.6	1		0645	82	30696.	7883.9	1602.6	1		1055	132	16567.	6397.4	1600.9
1		0240	33	134.	3949.8	1597.6	1		0650	83	31540.	7969.0	1602.7	1		1100	133	15821.	6313.9	1600.8
1		0245	34	140.	3957.6	1597.6	1		0655	84	32394.	8054.9	1602.8	1		1105	134	15102.	6232.8	1600.7
1		0250	35	149.	3966.9	1597.6	1		0700	85	33249.	8140.6	1602.9	1		1110	135	14411.	6154.1	1600.6
1		0255	36	163.	3978.0	1597.7	1		0705	86	34092.	8224.8	1603.0	1		1115	136	13747.	6077.9	1600.5
1		0300	37	183.	3991.0	1597.7	1		0710	87	34910.	8306.3	1603.1	1		1120	137	13111.	6004.1	1600.4
1		0305	38	210.	4006.1	1597.7	1		0715	88	35692.	8384.1	1603.2	1		1125	138	12502.	5932.7	1600.4
1		0310	39	246.	4023.8	1597.7	1		0720	89	36423.	8456.6	1603.3	1		1130	139	11919.	5863.8	1600.3
1		0315	40	293.	4044.5	1597.8	1		0725	90	37093.	8522.9	1603.3	1		1135	140	11362.	5797.1	1600.2
1		0320	41	355.	4068.8	1597.8	1		0730	91	37692.	8582.1	1603.4	1		1140	141	10829.	5732.7	1600.1
1		0325	42	435.	4097.4	1597.8	1		0735	92	38212.	8633.3	1603.4	1		1145	142	10320.	5670.5	1600.0
1		0330	43	538.	4130.6	1597.9	1		0740	93	38645.	8676.0	1603.5	1		1150	143	9833.	5610.4	1599.9
1		0335	44	668.	4169.1	1597.9	1		0745	94	38987.	8709.7	1603.5	1		1155	144	9369.	5552.4	1599.9
1		0340	45	832.	4213.2	1598.0	1		0750	95	39238.	8734.4	1603.5	1		1200	145	8926.	5496.5	1599.8
1		0345	46	1033.	4263.6	1598.1	1		0755	96	39397.	8750.0	1603.6	1		1205	146	8504.	5442.6	1599.7
1		0350	47	1280.	4320.6	1598.2	1		0800	97	39467.	8756.9	1603.6	1		1210	147	8101.	5390.5	1599.7
		0355	48	1578.	4384.8	1598.3	1		0805	98	39454.	8755.6	1603.6	1		1215	148	7718.	5340.4	1599.6
		0400	49	1934.	4456.3	1598.4	1		0810	99	39362.	8746.6	1603.6	1		1220	149	7353.	5292.1	1599.5
		0405	50	2353.	4535.3	1598.5	1		0815	100	39200.	8730.7	1603.5	1		1225	150	7006.	5245.7	1599.5

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PEAK OUTFLOW IS 39467. AT TIME 8.00 HOURS

PEAK FLOW TIME MAXIMUM AVERAGE FLOW

```
                                              6-HR          24-HR          72-HR       12.42-HR
+    (CFS)          (HR)
                                 (CFS)
+    39467.         8.00                     29063.         15816.         15816.         15816.
                               (INCHES)       5.666          6.381          6.381          6.381
                                (AC-FT)      14412.         16230.         16230.         16230.

 PEAK STORAGE       TIME                              MAXIMUM AVERAGE STORAGE
                                              6-HR          24-HR          72-HR       12.42-HR
+   (AC-FT)         (HR)
      8757.         8.00                      7703.          6067.          6067.          6067.

   PEAK STAGE       TIME                              MAXIMUM AVERAGE STAGE
                                              6-HR          24-HR          72-HR       12.42-HR
+    (FEET)         (HR)
    1603.56         8.00                    1602.42        1600.36        1600.36        1600.36

                              CUMULATIVE AREA =    47.69 SQ MI
```

```
1                                                                STATION    RITTEN

                          (I) INFLOW,  (O) OUTFLOW
          0.    10000.    20000.    30000.    40000.    50000.        0.        0.        0.        0.        0.        0.        0.
                                                                                  (S) STORAGE
          0.        0.        0.        0.        0.        0.     2000.     4000.     6000.     8000.    10000.        0.        0.
 DAHRMN PER
  10000   1I---------.---------.---------.---------.---------.---------.---------S---------.---------.---------.---------.---------.
  10005   2I         .         .         .         .         .         .         S         .         .         .         .         .
  10010   3I         .         .         .         .         .         .         S         .         .         .         .         .
  10015   4I         .         .         .         .         .         .         S         .         .         .         .         .
  10020   5I         .         .         .         .         .         .         S         .         .         .         .         .
  10025   6I         .         .         .         .         .         .         S         .         .         .         .         .
  10030   7I         .         .         .         .         .         .         S         .         .         .         .         .
  10035   8I         .         .         .         .         .         .         S         .         .         .         .         .
  10040   9I         .         .         .         .         .         .         S         .         .         .         .         .
  10045  10I         .         .         .         .         .         .         S         .         .         .         .         .
  10050  11I . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . S . . . . . . . . . . . . . . . . . . . . . . . . .
  10055  12I         .         .         .         .         .         .         S         .         .         .         .         .
  10100  13I         .         .         .         .         .         .         S         .         .         .         .         .
  10105  14I         .         .         .         .         .         .         S         .         .         .         .         .
  10110  15I         .         .         .         .         .         .         S         .         .         .         .         .
  10115  16I         .         .         .         .         .         .         S         .         .         .         .         .
  10120  17I         .         .         .         .         .         .         S         .         .         .         .         .
  10125  18I         .         .         .         .         .         .         S         .         .         .         .         .
  10130  19I         .         .         .         .         .         .         S         .         .         .         .         .
  10135  20I         .         .         .         .         .         .         S         .         .         .         .         .
  10140  21I . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . S . . . . . . . . . . . . . . . . . . . . . . . . .
  10145  22I         .         .         .         .         .         .         S         .         .         .         .         .
  10150  23I         .         .         .         .         .         .         S         .         .         .         .         .
  10155  24I         .         .         .         .         .         .         S         .         .         .         .         .
  10200  25I         .         .         .         .         .         .         S         .         .         .         .         .
  10205  26I         .         .         .         .         .         .         S         .         .         .         .         .
  10210  27I         .         .         .         .         .         .         S         .         .         .         .         .
  10215  28I         .         .         .         .         .         .         S         .         .         .         .         .
  10220  29I         .         .         .         .         .         .         S         .         .         .         .         .
  10225  300I        .         .         .         .         .         .         S         .         .         .         .         .
  10230  310I. . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . S . . . . . . . . . . . . . . . . . . . . . . . . .
  10235  320I        .         .         .         .         .         .         S         .         .         .         .         .
  10240  330I        .         .         .         .         .         .         S         .         .         .         .         .
  10245  340I        .         .         .         .         .         .         S         .         .         .         .         .
  10250  350 I       .         .         .         .         .         .         S         .         .         .         .         .
  10255  360 I       .         .         .         .         .         .         S         .         .         .         .         .
  10300  370 I       .         .         .         .         .         .         S         .         .         .         .         .
  10305  380  I      .         .         .         .         .         .         S         .         .         .         .         .
  10310  390  I      .         .         .         .         .         .         S         .         .         .         .         .
  10315  400   I     .         .         .         .         .         .         S         .         .         .         .         .
  10320  410 . I . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . S . . . . . . . . . . . . . . . . . . . . . . . . .
  10325  420    I    .         .         .         .         .         .         S         .         .         .         .         .
  10330  43.O    I   .         .         .         .         .         .         .S        .         .         .         .         .
  10335  44.O     I  .         .         .         .         .         .         .S        .         .         .         .         .
  10340  45.O      I .         .         .         .         .         .         .S        .         .         .         .         .
  10345  46.O       I.         .         .         .         .         .         .S        .         .         .         .         .
  10350  47.O        I         .         .         .         .         .         . S       .         .         .         .         .
  10355  48.  O      .I        .         .         .         .         .         . S       .         .         .         .         .
  10400  49.  O      .  I      .         .         .         .         .         . S       .         .         .         .         .
  10405  50.  O      .   I     .         .         .         .         .         .  S      .         .         .         .         .
  10410  51. .O. . . . . . I . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .S. . . . . . . . . . . . . . . . . . . . . . .
  10415  52.  O      .       I .         .         .         .         .         .   S     .         .         .         .         .
  10420  53.   O     .        I.         .         .         .         .         .   S     .         .         .         .         .
  10425  54.    O    .         .I        .         .         .         .         .    S    .         .         .         .         .
  10430  55.    O    .         . I       .         .         .         .         .    S    .         .         .         .         .
  10435  56.     O   .         .   I     .         .         .         .         .     S   .         .         .         .         .
  10440  57.      O  .         .     I   .         .         .         .         .     S   .         .         .         .         .
  10445  58.       O .         .      I  .         .         .         .         .      S  .         .         .         .         .
  10450  59.        O.         .        I.         .         .         .         .       S .         .         .         .         .
  10455  60.         O         .         I         .         .         .         .       S .         .         .         .         .
  10500  61. . . . . O. . . . . . . . . . I . . . . . . . . . . . . . . . . . . . . . . . S. . . . . . . . . . . . . . . . . . . . .
  10505  62.         .  O      .         .  I      .         .         .         .         .S        .         .         .         .
  10510  63.         .   O     .         .   I     .         .         .         .         .S        .         .         .         .
  10515  64.         .    O    .         .   I     .         .         .         .         . S       .         .         .         .
  10520  65.         .     O   .         .     I   .         .         .         .         .  S      .         .         .         .
  10525  66.         .      O  .         .     I   .         .         .         .         .  S      .         .         .         .
  10530  67.         .       O .         .     I   .         .         .         .         .   S     .         .         .         .
  10535  68.         .        O.         .     I   .         .         .         .         .    S    .         .         .         .
  10540  69.         .         O         .     I   .         .         .         .         .    S    .         .         .         .
  10545  70.         .         .O        .     I   .         .         .         .         .     S   .         .         .         .
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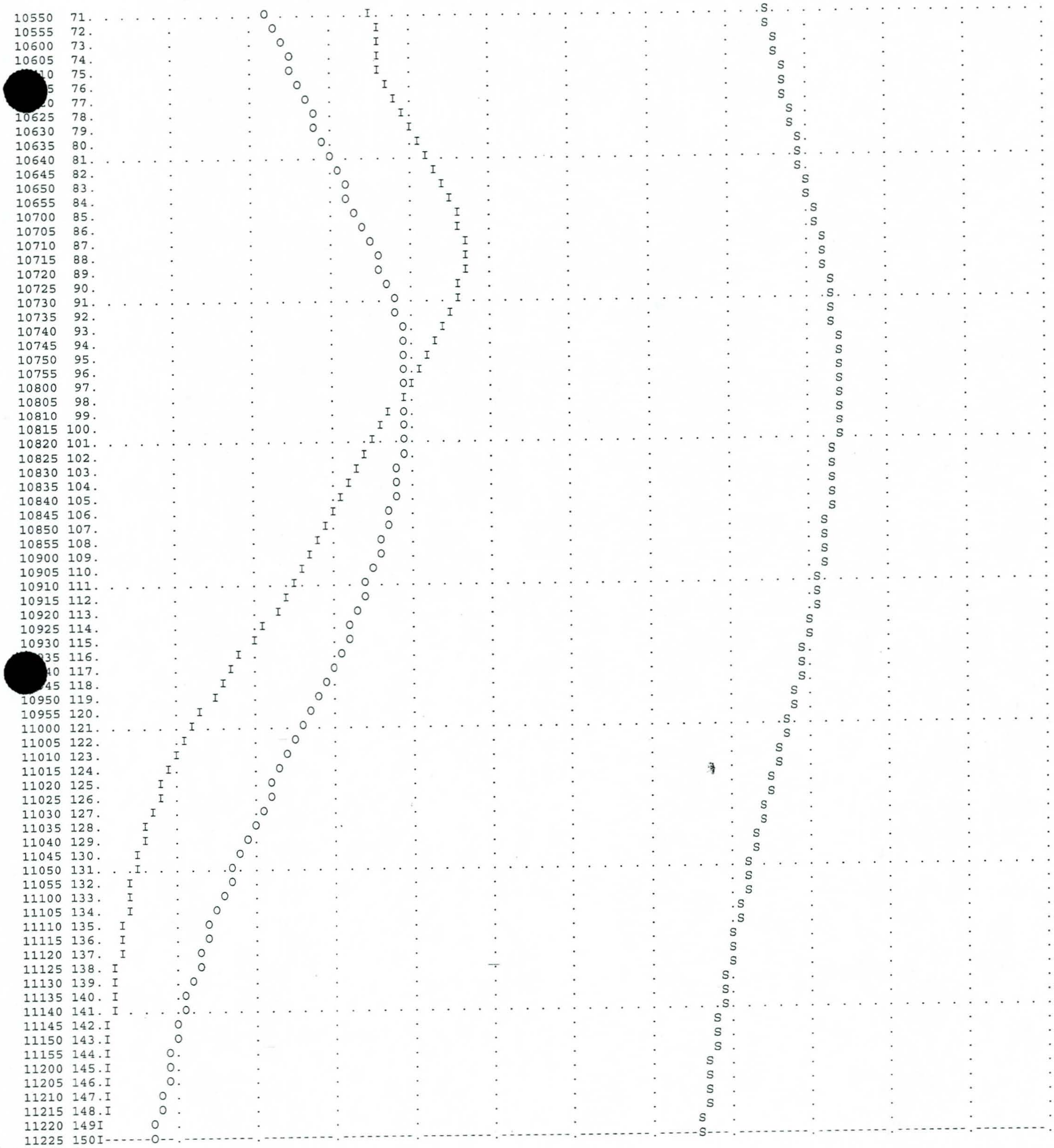

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              **************
              *            *
 486 KK       *   VINEYA   *      RD
              *            *
              **************

 496 KO           OUTPUT CONTROL VARIABLES
                         IPRNT          1  PRINT CONTROL
                         IPLOT          2  PLOT CONTROL
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QSCAL 0. HYDROGRAPH PLOT SCALE

HYDROGRAPH ROUTING DATA

RS STORAGE ROUTING

	NSTPS	1	NUMBER OF SUBREACHES
	ITYP	ELEV	TYPE OF INITIAL CONDITION
	RSVRIC	1574.80	INITIAL CONDITION
	X	.00	WORKING R AND D COEFFICIENT

498 SA	AREA	.0	65.0	200.0	400.0	600.0	800.0	1000.0	1200.0
499 SE	ELEVATION	1564.20	1565.00	1566.60	1569.50	1571.80	1574.20	1576.10	1579.20

500 SL LOW-LEVEL OUTLET

	ELEVL	1561.50	ELEVATION AT CENTER OF OUTLET
	CAREA	15.90	CROSS-SECTIONAL AREA
	COQL	.70	COEFFICIENT
	EXPL	.50	EXPONENT OF HEAD

501 SS SPILLWAY

	CREL	1574.80	SPILLWAY CREST ELEVATION
	SPWID	900.00	SPILLWAY WIDTH
	COQW	3.00	WEIR COEFFICIENT
	EXPW	1.50	EXPONENT OF HEAD

502 ST TOP OF DAM

	TOPEL	1584.40	ELEVATION AT TOP OF DAM
	DAMWID	28829.00	DAM WIDTH
	COQD	3.30	WEIR COEFFICIENT
	EXPD	1.50	EXPONENT OF HEAD

***

COMPUTED STORAGE-ELEVATION DATA

STORAGE	.00	17.33	219.47	1072.90	2215.18	3889.36	5595.86	9001.12
ELEVATION	1564.20	1565.00	1566.60	1569.50	1571.80	1574.20	1576.10	1579.20

COMPUTED OUTFLOW-ELEVATION DATA

(EXCLUDING FLOW OVER DAM)

OUTFLOW	.00	156.21	167.07	179.56	194.06	211.11	231.45	256.13	286.69	325.53
ELEVATION	1564.20	1564.56	1565.00	1565.55	1566.23	1567.09	1568.22	1569.73	1571.82	1574.80
OUTFLOW	352.00	530.43	1010.05	1939.25	3467.11	5742.55	8913.70	13130.80	18541.17	25294.41
ELEVATION	1574.85	1574.98	1575.20	1575.51	1575.90	1576.39	1576.96	1577.62	1578.37	1579.20

COMPUTED STORAGE-OUTFLOW-ELEVATION DATA

(INCLUDING FLOW OVER DAM)

STORAGE	.00	1.61	17.33	62.78	152.01	219.47	325.33	625.22	1072.90	1168.30
OUTFLOW	146.67	156.21	167.00	179.56	194.06	201.58	211.11	231.45	252.47	256.13
ELEVATION	1564.20	1564.56	1565.00	1565.55	1566.23	1566.60	1567.09	1568.22	1569.50	1569.73
STORAGE	2215.18	2224.34	3889.36	4387.56	4426.54	4542.40	4738.99	5022.28	5401.22	5595.86
OUTFLOW	286.48	286.69	318.11	325.53	352.00	530.43	1010.05	1939.25	3467.11	4342.75
ELEVATION	1571.80	1571.82	1574.20	1574.80	1574.85	1574.98	1575.20	1575.51	1575.90	1576.10
STORAGE	5885.51	6477.40	7185.98	8022.64	9001.12					
OUTFLOW	5742.55	8913.70	13130.80	18541.17	25294.41					
ELEVATION	1576.39	1576.96	1577.62	1578.37	1579.20					

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HYDROGRAPH AT STATION VINEYA

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DA MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE	*	DA MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE	*	DA MON	HRMN	ORD	OUTFLOW	STORAGE	STAGE
1	0000	1	326.	4387.6	1574.8	*	1	0410	51	4902.	5714.5	1576.2	*	1	0820	101	40121.	11015.1	1580.8
1	0005	2	326.	4385.4	1574.8	*	1	0415	52	5758.	5888.5	1576.4	*	1	0825	102	39189.	10891.9	1580.7
1	0010	3	325.	4383.1	1574.8	*	1	0420	53	6702.	6072.3	1576.6	*	1	0830	103	38225.	10764.3	1580.6
1	0015	4	325.	4380.9	1574.8	*	1	0425	54	7729.	6264.2	1576.8	*	1	0835	104	37239.	10633.2	1580.5
1	0020	5	325.	4378.7	1574.8	*	1	0430	55	8828.	6462.2	1576.9	*	1	0840	105	36237.	10499.6	1580.4
1	0025	6	325.	4376.5	1574.8	*	1	0435	56	9987.	6664.4	1577.1	*	1	0845	106	35225.	10364.0	1580.3
1	0030	7	325.	4374.3	1574.8	*	1	0440	57	11191.	6868.1	1577.3	*	1	0850	107	34208.	10227.4	1580.2
1	0035	8	325.	4372.1	1574.8	*	1	0445	58	12425.	7071.7	1577.5	*	1	0855	108	33192.	10090.2	1580.1
1	0040	9	325.	4369.9	1574.8	*	1	0450	59	13677.	7273.5	1577.7	*	1	0900	109	32178.	9952.8	1580.0
1	0045	10	325.	4367.7	1574.8	*	1	0455	60	14941.	7472.9	1577.9	*	1	0905	110	31171.	9815.7	1579.9
1	0050	11	325.	4365.5	1574.8	*	1	0500	61	16209.	7669.5	1578.1	*	1	0910	111	30172.	9679.1	1579.8
1	0055	12	325.	4363.3	1574.8	*	1	0505	62	17480.	7863.2	1578.2	*	1	0915	112	29186.	9543.6	1579.6
	0100	13	325.	4361.1	1574.8	*	1	0510	63	18754.	8054.4	1578.4	*	1	0920	113	28213.	9409.1	1579.5
	0105	14	325.	4358.9	1574.8	*	1	0515	64	20025.	8242.7	1578.6	*	1	0925	114	27256.	9276.1	1579.4
	0110	15	325.	4356.7	1574.8	*	1	0520	65	21294.	8428.3	1578.7	*	1	0930	115	26315.	9144.6	1579.3
1	0115	16	325.	4354.6	1574.8	*	1	0525	66	22554.	8610.7	1578.9	*	1	0935	116	25392.	9014.9	1579.2
1	0120	17	325.	4352.7	1574.8	*	1	0530	67	23803.	8789.6	1579.0	*	1	0940	117	24489.	8887.2	1579.1
1	0125	18	325.	4350.8	1574.8	*	1	0535	68	25037.	8964.8	1579.2	*	1	0945	118	23606.	8761.5	1579.0
1	0130	19	325.	4349.1	1574.8	*	1	0540	69	26256.	9136.3	1579.3	*	1	0950	119	22744.	8638.0	1578.9
1	0135	20	325.	4347.7	1574.8	*	1	0545	70	27458.	9304.3	1579.5	*	1	0955	120	21904.	8516.9	1578.8
1	0140	21	325.	4346.8	1574.8	*	1	0550	71	28633.	9467.3	1579.6	*	1	1000	121	21086.	8398.1	1578.7
1	0145	22	325.	4346.4	1574.8	*	1	0555	72	29796.	9627.5	1579.7	*	1	1005	122	20292.	8281.9	1578.6

1	0150	23	325.	4346.5	1574.8	*	1	0600	73	30950.	9785.6	1579.8	*	1	1010	123	19520.	8168.2	1578.5
1	0155	24	325.	4347.3	1574.8	*	1	0605	74	32098.	9941.9	1580.0	*	1	1015	124	18773.	8057.2	1578.4
1	0200	25	325.	4349.0	1574.8	*	1	0610	75	33242.	10097.0	1580.1	*	1	1020	125	18050.	7949.0	1578.3
1	0205	26	325.	4351.6	1574.8	*	1	0615	76	34384.	10251.1	1580.2	*	1	1025	126	17351.	7843.7	1578.2
	0210	27	325.	4355.4	1574.8	*	1	0620	77	35524.	10404.1	1580.3	*	1	1030	127	16678.	7741.2	1578.1
	0215	28	325.	4360.5	1574.8	*	1	0625	78	36657.	10555.5	1580.5	*	1	1035	128	16028.	7641.6	1578.0
	0220	29	325.	4367.2	1574.8	*	1	0630	79	37776.	10704.7	1580.6	*	1	1040	129	15403.	7544.9	1577.9
1	0225	30	325.	4375.7	1574.8	*	1	0635	80	38873.	10850.1	1580.7	*	1	1045	130	14801.	7451.1	1577.9
1	0230	31	326.	4386.1	1574.8	*	1	0640	81	39935.	10990.5	1580.8	*	1	1050	131	14223.	7360.1	1577.8
1	0235	32	330.	4398.6	1574.8	*	1	0645	82	40946.	11123.8	1580.9	*	1	1055	132	13669.	7272.2	1577.7
1	0240	33	340.	4413.5	1574.8	*	1	0650	83	41894.	11248.4	1581.0	*	1	1100	133	13137.	7187.1	1577.6
1	0245	34	356.	4430.9	1574.9	*	1	0655	84	42762.	11362.2	1581.1	*	1	1105	134	12628.	7104.7	1577.5
1	0250	35	380.	4451.0	1574.9	*	1	0700	85	43535.	11463.3	1581.1	*	1	1110	135	12140.	7025.1	1577.5
1	0255	36	412.	4473.9	1574.9	*	1	0705	86	44201.	11550.3	1581.2	*	1	1115	136	11673.	6948.2	1577.4
1	0300	37	453.	4499.9	1574.9	*	1	0710	87	44749.	11621.9	1581.3	*	1	1120	137	11225.	6873.9	1577.3
1	0305	38	506.	4529.4	1575.0	*	1	0715	88	45173.	11677.0	1581.3	*	1	1125	138	10798.	6802.3	1577.3
1	0310	39	573.	4563.2	1575.0	*	1	0720	89	45466.	11715.2	1581.3	*	1	1130	139	10389.	6733.0	1577.2
1	0315	40	658.	4602.6	1575.0	*	1	0725	90	45625.	11735.9	1581.3	*	1	1135	140	9998.	6666.2	1577.1
1	0320	41	769.	4648.9	1575.1	*	1	0730	91	45653.	11739.5	1581.3	*	1	1140	141	9624.	6601.7	1577.1
1	0325	42	910.	4703.2	1575.2	*	1	0735	92	45553.	11726.5	1581.3	*	1	1145	142	9266.	6539.4	1577.0
1	0330	43	1091.	4766.8	1575.2	*	1	0740	93	45330.	11697.6	1581.3	*	1	1150	143	8924.	6479.2	1577.0
1	0335	44	1318.	4841.1	1575.3	*	1	0745	94	44992.	11653.5	1581.3	*	1	1155	144	8597.	6421.2	1576.9
1	0340	45	1602.	4927.1	1575.4	*	1	0750	95	44547.	11595.5	1581.2	*	1	1200	145	8284.	6365.1	1576.9
1	0345	46	1951.	5025.6	1575.5	*	1	0755	96	44004.	11524.7	1581.2	*	1	1205	146	7985.	6311.0	1576.8
1	0350	47	2374.	5137.2	1575.6	*	1	0800	97	43364.	11441.0	1581.1	*	1	1210	147	7700.	6258.8	1576.7
1	0355	48	2877.	5262.3	1575.8	*	1	0805	98	42645.	11347.0	1581.1	*	1	1215	148	7426.	6208.4	1576.7
1	0400	49	3464.	5400.5	1575.9	*	1	0810	99	41859.	11243.9	1581.0	*	1	1220	149	7165.	6159.7	1576.7
1	0405	50	4138.	5551.4	1576.1	*	1	0815	100	41015.	11132.8	1580.9	*	1	1225	150	6915.	6112.7	1576.6
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PEAK OUTFLOW IS    45653. AT TIME    7.50 HOURS

 PEAK FLOW      TIME                                  MAXIMUM AVERAGE FLOW
                                           6-HR          24-HR          72-HR        12.42-HR
+   (CFS)       (HR)
                             (CFS)
+   45653.       7.50                     31294.        17135.         17135.         17135.
                           (INCHES)        5.582         6.326          6.326          6.326
                            (AC-FT)       15518.        17584.         17584.         17584.

PEAK STORAGE    TIME                                 MAXIMUM AVERAGE STORAGE
                                           6-HR          24-HR          72-HR        12.42-HR
+  (AC-FT)      (HR)
   [illegible]740.     7.50                      9796.         7452.          7452.          7452.

[illegible] STAGE      TIME                                  MAXIMUM AVERAGE STAGE
                                           6-HR          24-HR          72-HR        12.42-HR
+   (FEET)      (HR)
   1581.35       7.50                    1579.82       1577.69        1577.69        1577.69

                         CUMULATIVE AREA =    52.12 SQ MI

1                                                             STATION    VINEYA

                             (I) INFLOW,   (O) OUTFLOW
          0.     10000.     20000.     30000..    40000.     50000.     60000.        0.        0.        0.        0.        0.        0.
                                                                                      (S) STORAGE
          0.         0.         0.         0.         0.         0.      4000.     6000.     8000.    10000.    12000.        0.        0.
DAHRMN PER
 10000    1I---------.---------.---------.---------.---------.---------.-S-------.---------.---------.---------.---------.---------.
 10005    2I         .         .         .         .         .         . S       .         .         .         .         .         .
 10010    3I         .         .         .         .         .         . S       .         .         .         .         .         .
 10015    4I         .         .         .         .         .         . S       .         .         .         .         .         .
 10020    5I         .         .         .         .         .         . S       .         .         .         .         .         .
 10025    6I         .         .         .         .         .         . S       .         .         .         .         .         .
 10030    7I         .         .         .         .         .         . S       .         .         .         .         .         .
 10035    8I         .         .         .         .         .         . S       .         .         .         .         .         .
 10040    9I         .         .         .         .         .         . S       .         .         .         .         .         .
 10045   10I         .         .         .         .         .         . S       .         .         .         .         .         .
 10050   11I . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . S . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
 10055   12I         .         .         .         .         .         . S       .         .         .         .         .         .
 10100   13I         .         .         .         .         .         . S       .         .         .         .         .         .
 10105   14I         .         .         .         .         .         . S       .         .         .         .         .         .
 10110   15I         .         .         .         .         .         . S       .         .         .         .         .         .
 10115   16I         .         .         .         .         .         . S       .         .         .         .         .         .
 10120   17I         .         .         .         .         .         . S       .         .         .         .         .         .
 10125   18I         .         .         .         .         .         . S       .         .         .         .         .         .
 10130   19I         .         .         .         .         .         . S       .         .         .         .         .         .
 10135   20I         .         .         .         .         .         . S       .         .         .         .         .         .
 10140   21I . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . S . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
 10145   22I         .         .         .         .         .         . S       .         .         .         .         .         .
 10150   23I         .         .         .         .         .         . S       .         .         .         .         .         .
 10155   24I         .         .         .         .         .         . S       .         .         .         .         .         .
 10200   25OI        .         .         .         .         .         . S       .         .         .         .         .         .
 [illegible]05   26OI        .         .         .         .         .         . S       .         .         .         .         .         .
 [illegible]0   27OI        .         .         .         .         .         . S       .         .         .         .         .         .
 [illegible]15   28OI        .         .         .         .         .         . S       .         .         .         .         .         .
 10220   29OI        .         .         .         .         .         . S       .         .         .         .         .         .
 10225   30O I       .         .         .         .         .         . S       .         .         .         .         .         .
 10230   31O I . . . . . . . . . . . . . . . . . . . . . . . . . . . . S . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
 10235   32O I       .         .         .         .         .         . S       .         .         .         .         .         .
 10240   33O  I      .         .         .         .         .         . S       .         .         .         .         .         .
 10245   34O  I      .         .         .         .         .         . S       .         .         .         .         .         .
 10250   35O   I     .         .         .         .         .         . S       .         .         .         .         .         .
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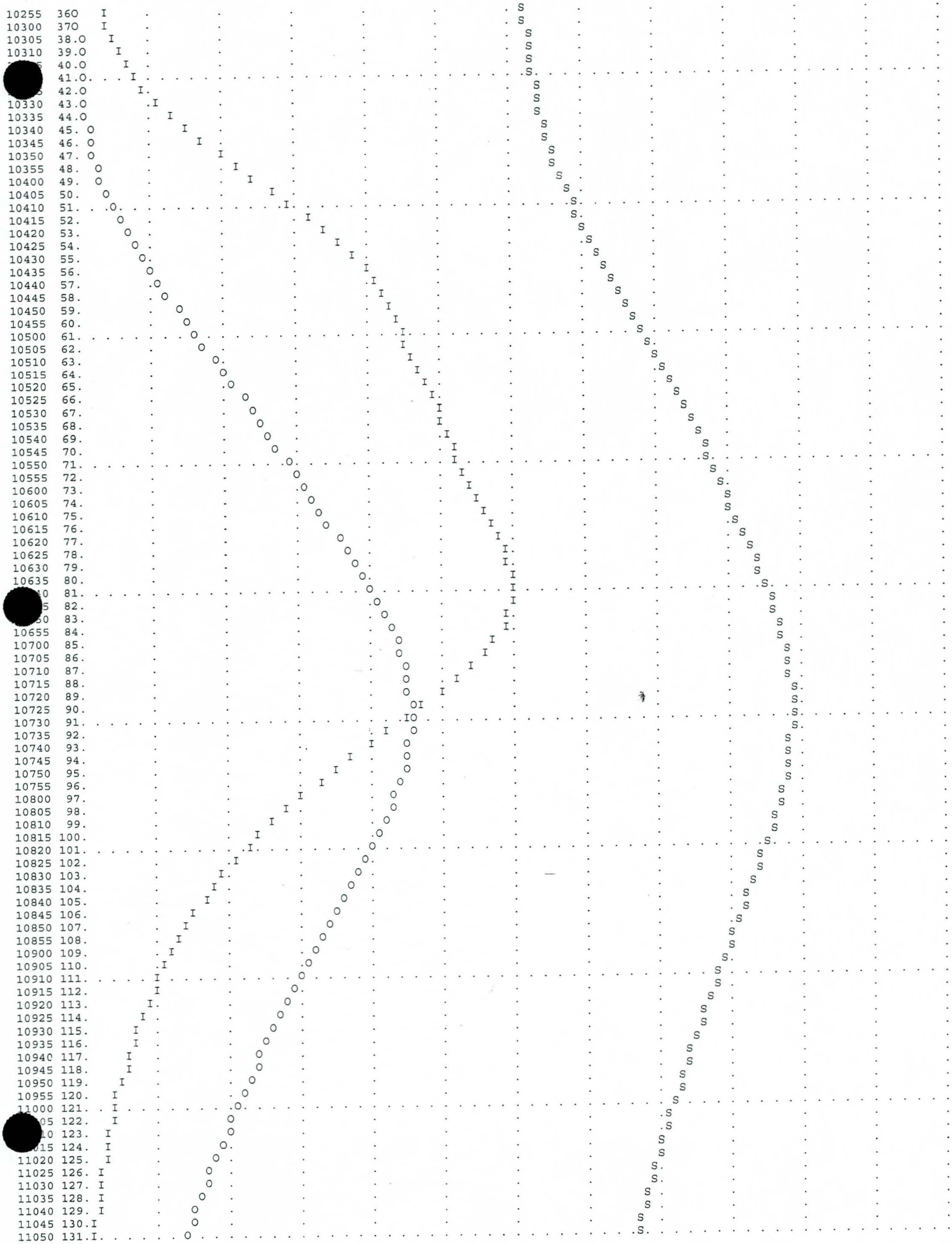

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                                                                                      S  .        .        .        .        .
 11055 132.I       .  O       .        .        .        .        .        .     S  .        .        .        .        .
 11100 133.I       . O        .        .        .        .        .        .     S  .        .        .        .        .
 11105 134.I       . O        .        .        .        .        .        .     S  .        .        .        .        .
 11110 135.I       . O        .        .        .        .        .        .    S   .        .        .        .        .
 11115 136.I       . O        .        .        .        .        .        .    S   .        .        .        .        .
 11120 137.I       .O         .        .        .        .        .        .   S    .        .        .        .        .
 11125 138.I       .O         .        .        .        .        .        .   S    .        .        .        .        .
 11130 139.I       O          .        .        .        .        .        .   S    .        .        .        .        .
 11135 140I        O          .        .        .        .        .        .  S     .        .        .        .        .
 11140 141I . . . .O . . . . . . . . . . . . . . . . . . . . . . . . . . . . .S. . . . . . . . . . . . . . . . . . . .
 11145 142I       O.          .        .        .        .        .        .  S     .        .        .        .        .
 11150 143I       O.          .        .        .        .        .        . S      .        .        .        .        .
 11155 144I       O.          .        .        .        .        .        . S      .        .        .        .        .
 11200 145I      O .          .        .        .        .        .        . S      .        .        .        .        .
 11205 146I      O .          .        .        .        .        .        . S      .        .        .        .        .
 11210 147I      O .          .        .        .        .        .        .S       .        .        .        .        .
 11215 148I     O  .          .        .        .        .        .        .S       .        .        .        .        .
 11220 149I     O  .          .        .        .        .        .        .S       .        .        .        .        .
 11225 150I------O--.---------.---------.---------.---------.---------.---------.S--------.---------.---------.---------.---------.
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RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
HYDROGRAPH AT	SUBR1	3807.	4.00	1150.	558.	558.	1.56		
ROUTED TO	RR2a	3719.	4.08	1145.	554.	554.	1.56	1816.80	4.08
ROUTED TO	RR2-1	3425.	5.17	1144.	554.	554.	1.56		
ROUTED TO	RR2-2	3211.	6.25	1142.	554.	554.	1.56		
ROUTED TO	RR2-3	3047.	7.33	1141.	554.	554.	1.56		
HYDROGRAPH AT	SUBR2	11120.	5.25	5825.	2961.	2961.	8.43		
2 COMBINED AT	HCR2	11305.	5.33	6879.	3515.	3515.	9.99		
HYDROGRAPH AT	SUB R3	2765.	4.00	825.	400.	400.	1.10		
ROUTED TO	RR4a	2738.	4.00	823.	399.	399.	1.10	1834.00	4.00
ROUTED TO	RR4-1	2524.	4.92	822.	399.	399.	1.10		
ROUTED TO	RR4-2	2364.	5.83	821.	399.	399.	1.10		
HYDROGRAPH AT	SUBR4	5325.	4.33	1923.	939.	939.	2.67		
2 COMBINED AT	HCR4	5688.	4.33	2717.	1338.	1338.	3.77		
HYDROGRAPH AT	SUBR5	9129.	3.58	2139.	1037.	1037.	2.71		
ROUTED TO	RR6	8376.	4.17	2138.	1037.	1037.	2.71		
HYDROGRAPH AT	SUBR6	10625.	4.00	3243.	1577.	1577.	4.19		
2 COMBINED AT	HCR6	18745.	4.08	5381.	2614.	2614.	6.90		
ROUTED TO	RR7	17496.	4.75	5376.	2614.	2614.	6.90		
HYDROGRAPH AT	SUBR7	8777.	3.67	2075.	1005.	1005.	2.67		
2 COMBINED AT	HCR7	19355.	4.75	7400.	3618.	3618.	9.57		
ROUTED TO	RRB-1	18569.	5.42	7392.	3618.	3618.	9.57		
ROUTED TO	RR8-2	17946.	6.08	7385.	3618.	3618.	9.57		

+	HYDROGRAPH AT	SUBR8	4460.	4.17	1447.	703.	703.	2.00		
+	2 COMBINED AT	HCR8	18806.	6.08	8756.	4322.	4322.	11.57		
+	ROUTED TO	RR9a	18749.	6.08	8755.	4317.	4317.	11.57	1838.42	6.08
+										
+	ROUTED TO	RR9	18464.	6.58	8749.	4317.	4317.	11.57		
+	HYDROGRAPH AT	SUBR9	940.	3.50	179.	87.	87.	.26		
+	2 COMBINED AT	HCR9	18472.	6.58	8890.	4403.	4403.	11.83		
+	HYDROGRAPH AT	SUBR10	7264.	4.25	2540.	1240.	1240.	3.35		
+	HYDROGRAPH AT	SUBR11	9282.	4.08	2967.	1444.	1444.	3.90		
+	2 COMBINED AT	HCR11	16406.	4.17	5506.	2684.	2684.	7.25		
+	ROUTED TO	RR12	15918.	4.67	5502.	2684.	2684.	7.25		
+	HYDROGRAPH AT	SUBR12	10652.	4.08	3340.	1623.	1623.	4.46		
+	2 COMBINED AT	HCR12	23340.	4.50	8820.	4307.	4307.	11.71		
+	ROUTED TO	RR13a	23270.	4.50	8819.	4298.	4298.	11.71	1830.12	4.50
+										
+	ROUTED TO	RR13	22920.	4.92	8815.	4298.	4298.	11.71		
+	HYDROGRAPH AT	SUBR13	2857.	4.17	949.	462.	462.	1.27		
+	2 COMBINED AT	HCR13a	24549.	4.92	9751.	4761.	4761.	12.98		
+	2 COMBINED AT	HCR13b	34428.	5.17	18485.	9164.	9164.	24.81		
+	ROUTED TO	RR14	33681.	6.25	18450.	9164.	9164.	24.81		
+	HYDROGRAPH AT	SUBR14	6528.	4.08	2008.	974.	974.	2.82		
+	2 COMBINED AT	HCR14a	34646.	6.25	20069.	10138.	10138.	27.63		
+	2 COMBINED AT	HCR14b	38050.	6.17	22653.	11476.	11476.	31.40		
+	ROUTED TO	RR15-1	37604.	6.75	22607.	11473.	11473.	31.40		
+	ROUTED TO	RR15-2	37177.	7.42	22558.	11461.	11461.	31.40		
+	HYDROGRAPH AT	SUBR15	9906.	4.83	4459.	2215.	2215.	6.30		
+	2 COMBINED AT	HCR15a	39265.	7.33	26039.	13676.	13676.	37.70		
+	2 COMBINED AT	HCR15b	46895.	7.25	32649.	17192.	17192.	47.69		
+	DIVERSION TO	DVR16	105.	7.25	105.	90.	90.	47.69		
+	HYDROGRAPH AT	HDR16	46790.	7.25	32544.	17102.	17102.	47.69		
+	ROUTED TO	RITTEN	39467.	8.00	29063.	15816.	15816.	47.69	1603.56	8.00
+										
+	HYDROGRAPH AT	SUBVI	8828.	3.75	2351.	1140.	1140.	3.03		
+	HYDROGRAPH AT	SUBV2	9959.	3.75	2428.	1174.	1174.	3.34		

+	ROUTED TO	RV3	9504.	4.08	2427.	1174.	1174.	3.34		
+	HYDROGRAPH AT	SUBV3	1484.	3.50	301.	146.	146.	.40		
+	3 COMBINED AT	HCV3	17764.	4.00	5073.	2460.	2460.	6.77		
+	ROUTED TO	RV4	17150.	4.42	5071.	2460.	2460.	6.77		
+	HYDROGRAPH AT	SUBV4	5387.	3.75	1405.	682.	682.	1.85		
+	2 COMBINED AT	HCV4	20165.	4.33	6467.	3142.	3142.	8.62		
+	ROUTED TO	RV5	19535.	5.00	6463.	3142.	3142.	8.62		
+	HYDROGRAPH AT	SUBV5	5158.	3.67	1248.	604.	604.	1.66		
+	2 COMBINED AT	HCV5	20522.	5.00	7675.	3747.	3747.	10.28		
+ +	ROUTED TO	RV6a	20577.	5.00	7674.	3740.	3740.	10.28	1749.05	5.00
+	ROUTED TO	RV6-1	19679.	5.83	7667.	3740.	3740.	10.28		
+	ROUTED TO	RV6-2	19121.	6.67	7659.	3740.	3740.	10.28		
+	HYDROGRAPH AT	SUBV6	6650.	4.33	2325.	1131.	1131.	3.59		
+	2 COMBINED AT	HCV6	20261.	6.67	9827.	4871.	4871.	13.87		
+	HYDROGRAPH AT	SUBV7	4164.	4.00	1307.	636.	636.	1.69		
+	HYDROGRAPH AT	SUBV8	2613.	3.50	570.	277.	277.	.73		
+	2 COMBINED AT	HCV8	5831.	3.83	1872.	913.	913.	2.42		
+	ROUTED TO	RV9	5785.	4.08	1872.	913.	913.	2.42		
+	HYDROGRAPH AT	SUBV9	3104.	3.67	727.	352.	352.	.95		
+	2 COMBINED AT	HCV9	8132.	3.83	2596.	1265.	1265.	3.37		
+ +	ROUTED TO	RV10a	6003.	4.42	2137.	1036.	1036.	3.37	1778.93	4.42
+	ROUTED TO	RV10-1	5822.	5.33	2134.	1036.	1036.	3.37		
+	ROUTED TO	RV10-2	5646.	6.25	2131.	1036.	1036.	3.37		
+	ROUTED TO	RV10-3	5493.	7.08	2129.	1034.	1034.	3.37		
+	HYDROGRAPH AT	SUBV10	8566.	4.83	3989.	1994.	1994.	5.48		
+	2 COMBINED AT	HCV10	8566.	4.83	5838.	3028.	3028.	8.85		
+	HYDROGRAPH AT	SUBV11	956.	4.25	339.	166.	166.	.44		
+ +	ROUTED TO	RV12a	947.	4.33	338.	166.	166.	.44	1781.15	4.33
+	ROUTED TO	RV12-1	877.	5.50	337.	166.	166.	.44		
+	ROUTED TO	RV12-2	821.	6.67	337.	166.	166.	.44		
+	HYDROGRAPH AT	SUBV12	5576.	4.67	2436.	1208.	1208.	3.32		

+	2 COMBINED AT	HCV12	5654.	4.75	2731.	1374.	1374.	3.76		
+	HYDROGRAPH AT	SUBV13	4925.	4.25	1806.	884.	884.	2.35		
+ +	ROUTED TO	KK RV1	4925.	4.33	1804.	882.	882.	2.35	1794.41	4.33
+	ROUTED TO	RV14-1	4714.	5.33	1801.	882.	882.	2.35		
+	ROUTED TO	RV14-2	4522.	6.25	1799.	882.	882.	2.35		
+	ROUTED TO	RY14-3	4354.	7.25	1796.	882.	882.	2.35		
+	HYDROGRAPH AT	SUBV14	5908.	5.08	2946.	1488.	1488.	4.09		
+	2 COMBINED AT	HCV14	6513.	7.08	4566.	2370.	2370.	6.44		
+	HYDROGRAPH AT	SUBV15	4420.	3.92	1271.	617.	617.	1.64		
+ +	ROUTED TO	RV16a	4215.	4.00	1269.	615.	615.	1.64	1793.71	4.00
+	ROUTED TO	RV16	4010.	4.50	1268.	615.	615.	1.64		
+	HYDROGRAPH AT	SUBV16	2892.	3.50	591.	286.	286.	.76		
+	2 COMBINED AT	HCV16	4550.	4.50	1843.	901.	901.	2.40		
+	HYDROGRAPH AT	SUBV17	10358.	4.00	3143.	1528.	1528.	4.06		
	ROUTED TO	RV18	9411.	5.08	3138.	1528.	1528.	4.06		
+	HYDROGRAPH AT	SUBV18	10998.	4.25	3895.	1904.	1904.	5.06		
+	2 COMBINED AT	HCV18	16009.	4.83	6997.	3432.	3432.	9.12		
+	ROUTED TO	RV19	15921.	5.25	6995.	3432.	3432.	9.12		
+	HYDROGRAPH AT	SUBV19	7618.	3.83	2114.	1025.	1025.	2.77		
+	2 COMBINED AT	HCV19	17770.	5.08	9048.	4458.	4458.	11.89		
+ +	ROUTED TO	RV20a	17753.	5.17	9047.	4448.	4448.	11.89	1804.49	5.17
+	ROUTED TO	RV20	17683.	5.75	9037.	4448.	4448.	11.89		
+	HYDROGRAPH AT	SUBV20	1526.	3.67	361.	175.	175.	.48		
+	2 COMBINED AT	HCV20a	17830.	5.75	9360.	4623.	4623.	12.37		
+	2 COMBINED AT	HCV20b	20089.	5.17	11122.	5524.	5524.	14.77		
+	ROUTED TO	RV21	19790.	6.50	11095.	5524.	5524.	14.77		
+	HYDROGRAPH AT	SUBV21	6082.	5.17	3123.	1584.	1584.	4.43		
	2 COMBINED AT	HCV21a	23838.	6.25	14139.	7108.	7108.	19.20		
+	5 COMBINED AT	HCV21b	59994.	6.67	37000.	18751.	18751.	52.12		
+	HYDROGRAPH AT	HCV22a	105.	2.33	105.	90.	90.	.00		
+	2 COMBINED AT	HCV22b	60099.	6.67	37105.	18840.	18840.	52.12		

```
          ROUTED TO
+                         VINEYA     45653.    7.50       31294.      17135.      17135.      52.12
+                                                                                                        1581.35      7.50
1                              SUMMARY OF DAM OVERTOPPING/BREACH ANALYSIS FOR STATION   RITTEN
                            (PEAKS SHOWN ARE FOR INTERNAL TIME STEP USED  DURING BREACH FORMATION)

      PLAN  1 ...............                INITIAL VALUE    SPILLWAY CREST     TOP OF DAM
                                 ELEVATION      1597.60           1597.60          1606.60
                                 STORAGE           3944.             3944.           11991.
                                 OUTFLOW            132.              132.           73059.

                RATIO      MAXIMUM      MAXIMUM     MAXIMUM     MAXIMUM    DURATION     TIME OF     TIME OF
                  OF      RESERVOIR      DEPTH      STORAGE     OUTFLOW    OVER TOP   MAX OUTFLOW   FAILURE
                 PMF       W.S.ELEV    OVER DAM      AC-FT        CFS       HOURS       HOURS        HOURS

                1.00      1603.56         .00        8757.      39467.       .00         8.00         .00
1                              SUMMARY OF DAM OVERTOPPING/BREACH ANALYSIS FOR STATION   VINEYA
                            (PEAKS SHOWN ARE FOR INTERNAL TIME STEP USED  DURING BREACH FORMATION)

      PLAN  1 ...............                INITIAL VALUE    SPILLWAY CREST     TOP OF DAM
                                 ELEVATION      1574.80           1574.80          1584.40
                                 STORAGE           4388.             4388.           16198.
                                 OUTFLOW            326.              326.           80737.

                RATIO      MAXIMUM      MAXIMUM     MAXIMUM     MAXIMUM    DURATION     TIME OF     TIME OF
                  OF      RESERVOIR      DEPTH      STORAGE     OUTFLOW    OVER TOP   MAX OUTFLOW   FAILURE
                 PMF       W.S.ELEV    OVER DAM      AC-FT        CFS       HOURS       HOURS        HOURS

                1.00      1581.35         .00       11740.      45653.       .00         7.50         .00


 *** NORMAL END OF HEC-1 ***
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ALTERNATIVES ANALYSIS REPORT

Appendix G: Powerline, Vineyard Road, and Rittenhouse FRS
Structural Alternative No. 3:
Modifications to Improve Performance

Opinion of Probable Construction Cost

FRS Structural Alternative No. 3
Modifications to Improve Performance

Sill Structure

Item	Description	Unit	Quantity	Unit Cost	Total
1	Structure Excavation	CY	1,068	$ 15.00	$ 16,020.00
2	Concrete sill	CY	400	$ 250.00	$ 100,000.00
3	Steel Reinforcing	lbs	13,018	$ 0.40	$ 5,207.20
				Construction	$ 121,227.20

Abutment Slope Protection

Item	Description	Unit	Quantity	Unit Cost	Total
1	Toe Down Excavation	CY	167	$ 15.00	$ 2,505.00
2	RipRap (D50 = 1.6ft)	CY	330	$ 50.00	$ 16,500.00
				Construction	$ 19,005.00

Subtotal Construction		$ 140,232.20
Land Cost		$ -
Engineering and Construction Mgt	10%	$ 14,023.22
Operation and Maintenance	10%	$ 14,023.22
Subtotal		$ 168,278.64
Contingency	25%	$ 42,069.66
Total Costs		$ 210,348.30

Opinion of Probable Construction Cost

FRS Structural Alternative No. 3
Modifications to Improve Performance

Sill Structure

Item	Description	Unit	Quantity	Unit Cost	Total
1	Structure Excavation	CY	1,085	$ 15.00	$ 16,275.00
2	Concrete sill	CY	400	$ 250.00	$ 100,000.00
3	Steel Reinforcing	lbs	13,018	$ 0.40	$ 5,207.20
				Construction	$ 121,482.20

Abutment Slope Protection

Item	Description	Unit	Quantity	Unit Cost	Total
1	Toe Down Excavation	CY	334	$ 15.00	$ 5,010.00
2	RipRap (D50 = 1.0ft)	CY	660	$ 50.00	$ 33,000.00
				Construction	$ 38,010.00

Subtotal Construction		$ 159,492.20
Land Cost		$ -
Engineering and Construction Mgt	10%	$ 15,949.22
Operation and Maintenance	10%	$ 15,949.22
Subtotal		$ 191,390.64
Contingency	25%	$ 47,847.66
Total Costs		$ 239,238.30

Opinion of Probable Construction Cost

FRS Structural Alternative No. 3
Modifications to Improve Performance

Sill Structure

Item	Description	Unit	Quantity	Unit Cost	Total
1	Structure Excavation	CY	1,068	$ 15.00	$ 16,020.00
2	Concrete sill	CY	400	$ 250.00	$ 100,000.00
3	Steel Reinforcing	lbs	13,018	$ 0.40	$ 5,207.20
				Construction	$ 121,227.20

Abutment Slope Protection

Item	Description	Unit	Quantity	Unit Cost	Total
1	Toe Down Excavation	CY	167	$ 15.00	$ 2,505.00
2	RipRap (D50 = 1.4ft)	CY	330	$ 50.00	$ 16,500.00
				Construction	$ 19,005.00

	Subtotal Construction		$ 140,232.20
Land Cost			$ -
Engineering and Construction Mgt		10%	$ 14,023.22
Operation and Maintenance		10%	$ 14,023.22
	Subtotal		$ 168,278.64
Contingency		25%	$ 42,069.66
Total Costs			$ 210,348.30

Control Sill Depth
Calculations
Powerline, Vineyard Road,
and Rittenhouse FRS

	Powerline FRS	Vineyard Road FRS North Spillway	Vineyard Road FRS South Spillway	Rittenhouse FRS
Flowrate (cfs)	16600	6400	6400	12800
b- Channel Bottom Width (ft)	600	300	300	600
q- Unit Discharge (cfs/ft)	27.7	21.3	21.3	21.3
h - Drop Height (ft)	1	1	1	1
Downstream Depth of Flow (ft) (use hydraulic depth, ft) RM 10.803	3.22	3.18	3.04	2.69
h/Y < .99?	0.3 yes	0.3 yes	0.3 yes	0.4 yes
Eqn 6.14 Scour Depth (ft)	3.4	2.9	3.0	3.1
Add 30%	1.0	0.9	0.9	0.9
Total Sill depth (ft)	4.5	3.8	3.9	4.1
Design depth (ft)	4.5	4.0	4.0	4.5

Powerline FRS Spillway
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	PowerlineSpilllw
Flow Element	Trapezoidal Cha
Method	Manning's Formi
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.030	
Slope	ı.006300	ft/ft
Left Side Slope	2.00	H : V
Right Side Slope	2.00	H : V
Bottom Width	600.00	ft
Discharge	5,600.00	cfs

Results		
Depth	3.22	ft
Flow Area	1,953.1	ft²
Wetted Perim	614.40	ft
Top Width	612.88	ft
Critical Depth	2.87	ft
Critical Slope	0.009288	ft/ft
Velocity	8.50	ft/s
Velocity Head	1.12	ft
Specific Energ	4.34	ft
Froude Numb	0.84	
Flow Type	Subcritical	

Vineyard Road North Spiillway
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	Vineyard Road North S|
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.030	
Slope	003900	ft/ft
Left Side Slope	2.00	H : V
Right Side Slope	2.00	H : V
Bottom Width	300.00	ft
Discharge	,400.00	cfs

Results		
Depth	3.18	ft
Flow Area	973.5	ft²
Wetted Perim	314.21	ft
Top Width	312.71	ft
Critical Depth	2.41	ft
Critical Slope	0.009887	ft/ft
Velocity	6.57	ft/s
Velocity Head	0.67	ft
Specific Energ	3.85	ft
Froude Numb	0.66	
Flow Type	Subcritical	

Vineyard Road South Spiillway
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	Vineyard Road North S|
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.030	
Slope	004500	ft/ft
Left Side Slope	2.00	H : V
Right Side Slope	2.00	H : V
Bottom Width	300.00	ft
Discharge	,400.00	cfs

Results		
Depth	3.04	ft
Flow Area	931.9	ft²
Wetted Perim	313.62	ft
Top Width	312.18	ft
Critical Depth	2.41	ft
Critical Slope	0.009887	ft/ft
Velocity	6.87	ft/s
Velocity Head	0.73	ft
Specific Enerç	3.78	ft
Froude Numb	0.70	
Flow Type	Subcritical	

Rittenhouse FRS Spillway
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	Rittenhouse Spill
Flow Element	Trapezoidal Char
Method	Manning's Formu
Solve For	Channel Depth

Input Data		
Mannings Coeffic	0.030	
Slope	).006800	ft/ft
Left Side Slope	2.00	H : V
Right Side Slope	2.00	H : V
Bottom Width	600.00	ft
Discharge	2,800.00	cfs

Results		
Depth	2.69	ft
Flow Area	1,630.7	ft²
Wetted Perim	612.05	ft
Top Width	610.77	ft
Critical Depth	2.41	ft
Critical Slope	0.009830	ft/ft
Velocity	7.85	ft/s
Velocity Head	0.96	ft
Specific Enerç	3.65	ft
Froude Numb	0.85	
Flow Type	Subcritical	

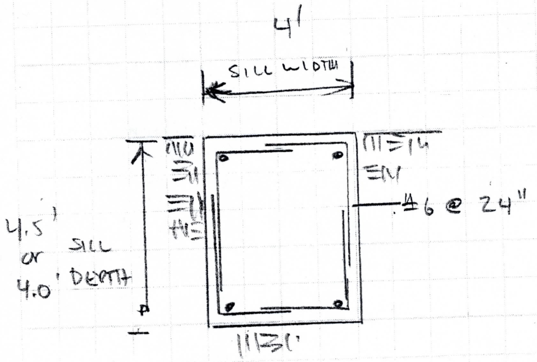
Kimley-Horn and Associates, Inc.

Sheet No. ____ of ____
Job No. ____
Date ____
Job ____ Subject ____
Designed by ____ Date ____ Checked by ____

CONTROL SILL QUANTITIES + COST ESTIMATES

FRS	SILL DEPTH (FT)	SILL LENGTH (FT)	SILL WIDTH (FT)	SILL VOLUME (FT³)	CONCRETE SILL VOLUME (CY)
POWERLINE/ RITTENHOUSE	4.5	610	4	10,980	407 x2 = 814
VINEYARD (2 spillways)	4.0	310	4	4,960	184 x2 = 368

1,182 CY CONCRETE



SILL SECTION

P longitudinal
610 (4) = 2440 ft
bents (4)
(2.5' + 2.5') x 4 = 20' @ 305 = 6,100 FT
TOTAL STEEL #6 = 8540 FT #6

R = P = 8540 FT of #6

V longitudinal
310 (4)(2) = 2440 FT
bents 4 (by 2 spillways)

(2.5 + 2.5) x 4 = 20 x 155 x 2 = 6,200 FT
TOTAL STEEL #6 = 8,640 FT #6

TOTAL STEEL #6 ≈ 26,000 L.F
TOTAL CONCRETE ≈ 1,200 CY

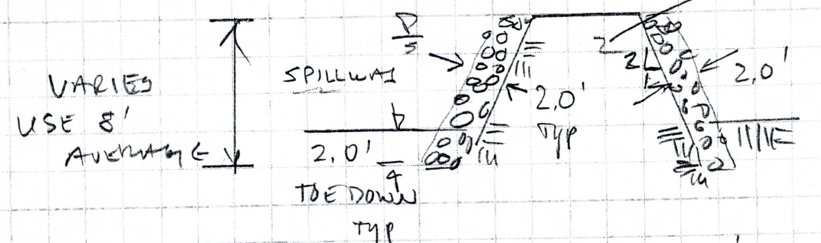
NEED UNIT COST STEEL #6 L.F, OR LBS
UNIT COST CONCRE CY

PER DAM

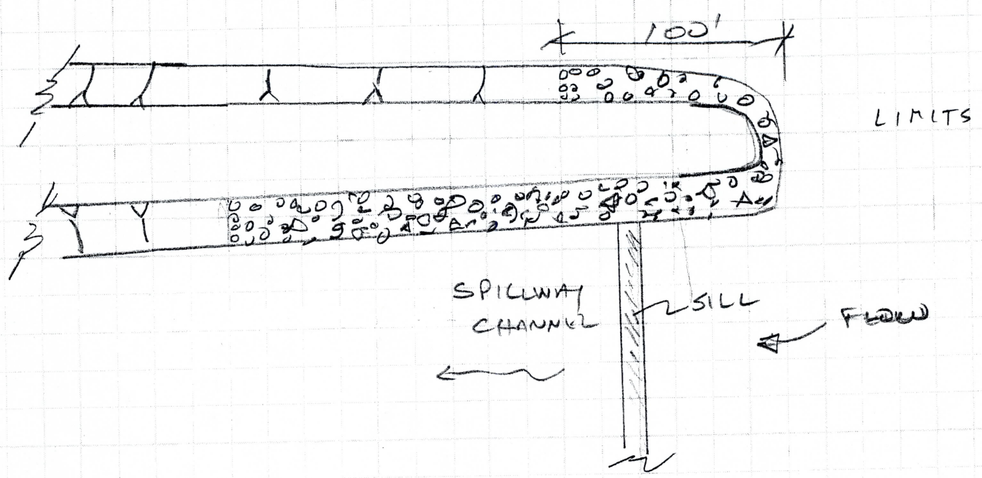
	STEEL	CONC
P	8,667	400
V	8667	400
R	8667	400

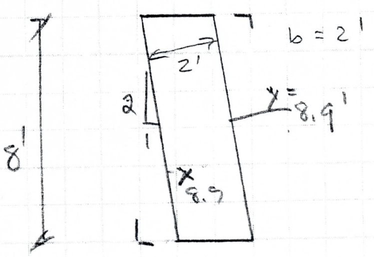
STEEL WT. #6 BAR 1.502 LBS/LF

RIP RAP QUANTITIES



LENGTH OF RIP-RAP BLANKET ASSUME = 500'





$\text{AREA} = \frac{1}{2}(b)(x+y)$

$= \frac{1}{2}(2)(8.9+8.9) = 17.8\ ft^2$

$\text{VOLUME} = 17.8(500) = 8{,}900\ ft^2 = 330\ CY$

ASSUME $145\ lb/ft^3$

NO TONS RIP RAP $= (8900\ ft^3)(145\ lb/ft^3)\left(\frac{1\ ton}{2000\ lb}\right)$

$=$ 645 tons per spillway abutment

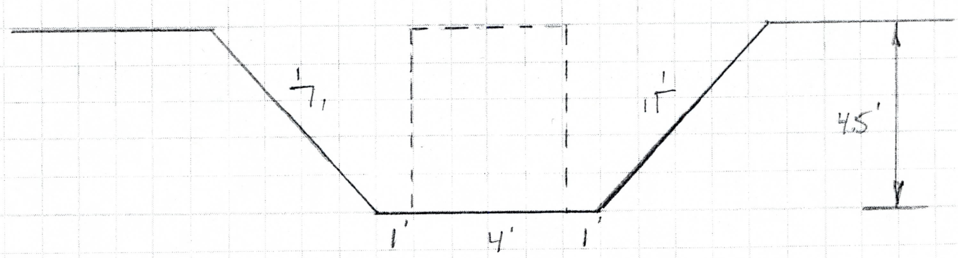
POWERLINE	645 tons	$D_{50} = 1.6$ ft
VINEYARD	1,290 ton	$D_{50} = 1.0$ ft
RITTENHOUSE	645 ton	$D_{50} = 1.4$ ft

Kimley-Horn and Associates, Inc.

Job STRUCTURES Subject ALTS ANALYSIS Sheet No. ___ of ___ Job No. 071131003

Designed by ___ Date ___ Checked by ___ Date ___

SILL EXCAVATION



POWERLINE / RITTENHOUSE

$6 \times 4.5 = 27$

$2 \cdot \frac{1}{2}(4.5 \times 4.5) = 20.25$

47.25 SF/LF

SILL LENGTH = 610'

$610 \times 47.25 = 28,823$ CF

= 1068 CY

VINEYARD

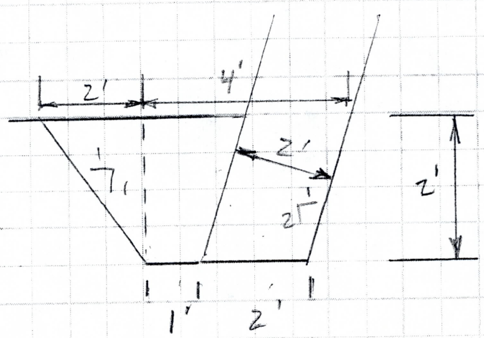
SILL LENGTH = 310'

(2 SPILLWAYS) = 620'

$620 \times 47.25 = 29,295$ CF

= 1085 CY

TOE DOWN EXCAVATION



$$2 \times \left(\frac{3+4}{2}\right) = 7.0$$

$$\tfrac{1}{2}(2 \times 2) = \underline{2.0}$$

9.0 SF/LF

LENGTH = 500'

9.0 x 500 = 4500 CF

= 167 CY

ALTERNATIVES ANALYSIS REPORT

Appendix H: Powerline, Vineyard Road, and Rittenhouse FRS
Structural Alternative No. 4:
Replace Dams with Detention Basins

Opinion of Probable Construction Cost

FRS Structural Alternative No. 4
Replace Dams with Detention Basins

Drainage Basin Excavation Costs

Item	Description	Unit	Quantity	Unit Cost	Total
1	Powerline FRS	CY	23,725,926	$ 2.00	$ 47,451,851.85
2	Vineyard FRS	CY	9,981,111	$ 2.00	$ 19,962,222.22
3	Rittenhouse FRS	CY	18,534,444	$ 2.00	$ 37,068,888.89
				Total	$ 104,482,962.96

Land Cost		$ -
Engineering and Construction Mgt	6%	$ 6,268,977.78
Operation and Maintenance	5%	$ 5,224,148.15
Subtotal		$ 115,976,088.89
Contingency	10%	$ 11,597,608.89
Total Costs		$ 127,573,697.78

FRS	100-year Capacity (af)	100-year Capacity (cf)	Max Depth (ft)	Ave Length (ft)	Ave Width (ft)	Invert
Powerline	4,019	175,067,640	5	8,000	4,377	1,563
Vineyard Road	4,112	179,118,720	5	26,000	1,378	1,563
Rittenhouse	3,875	168,795,000	5	14,000	2,411	1,578

Dention Basin Excavation

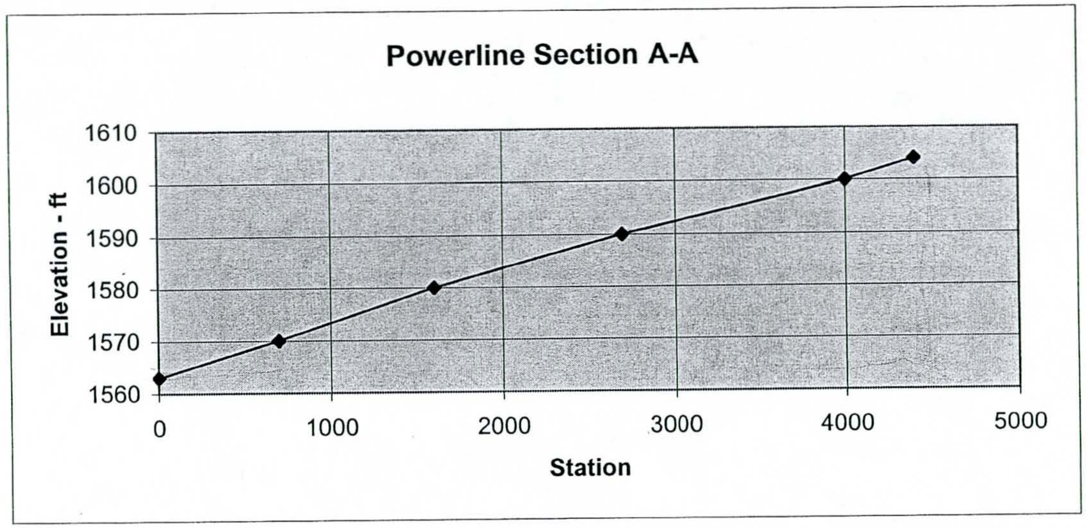
Dam	Section		Area			
				Length	8,000	
Powerline FRS	A-A		94,650	Volume	640,600,000	cf
	B-B		65,500		23,725,926	cy
		Ave	80,075			
Vineyard	A-A		15,340	Length	26,000	
	B-B		5,390			
		Ave	10,365	Volume	269,490,000	cf
					9,981,111	cy
Rittenhouse	A-A		31,840	Length	14,000	
	B-B		39,650			
		Ave	35,745	Volume	500,430,000	cf
					18,534,444	cy

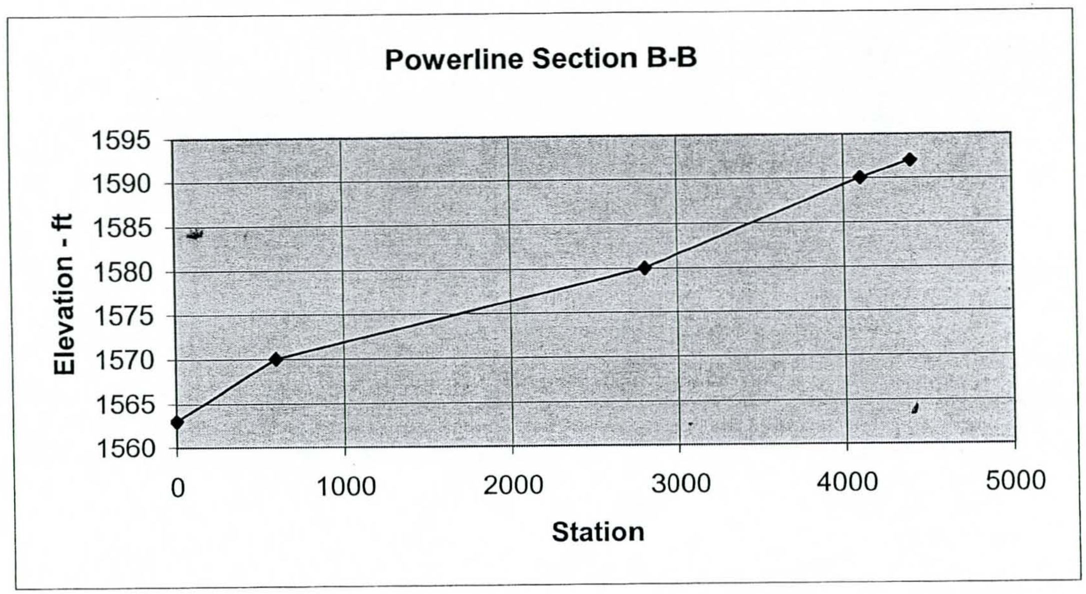
Drainage Basin Excavation Costs

FRS	Quantity	Unit cost ($)	Total ($)
Powerline FRS	23,725,926	$ 2.00	$ 47,451,852
Vineyard	9,981,111	$ 2.00	$ 19,962,222
Rittenhouse	18,534,444	$ 2.00	$ 37,068,889
			$ 104,482,963

Station	Elevation
0	1563
700	1570
1600	1580
2700	1590
4000	1600
4400	1604

AREA = 94,650 ft^2





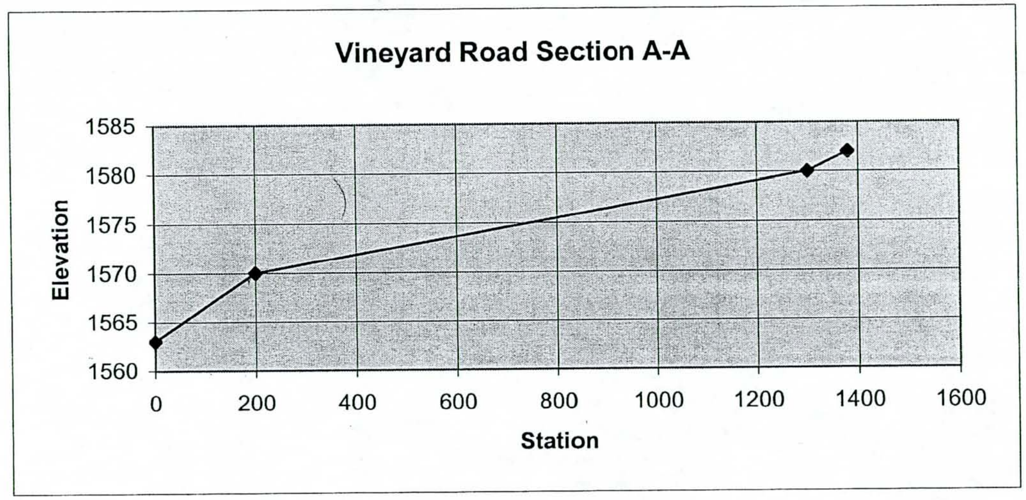
Station	Elevation
0	1563
600	1570
2800	1580
4100	1590
4400	1592

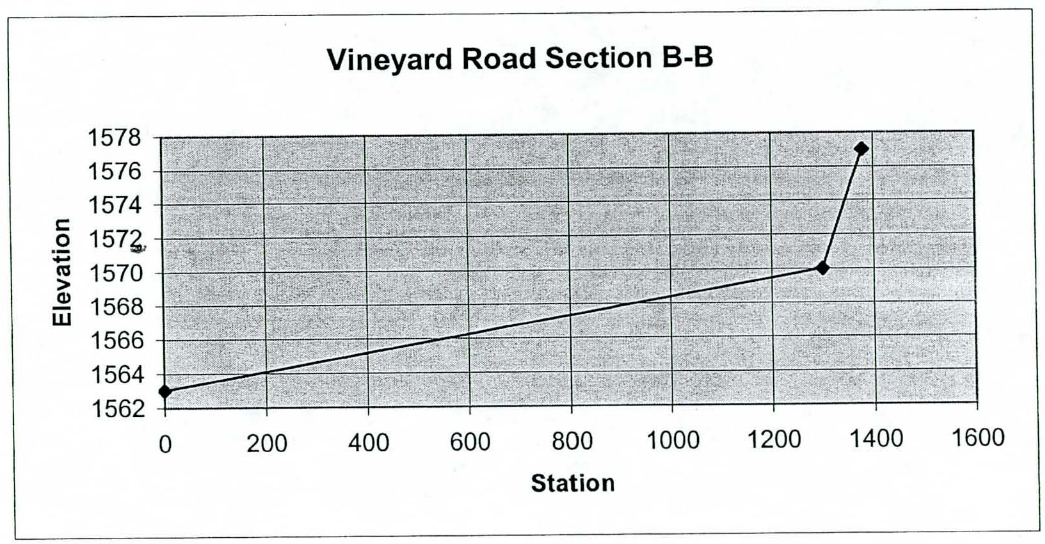
65500.00

DetentionXsections.xls/Powerline

Station	Elevation
0	1563
200	1570
1300	1580
1380	1582

15340.00



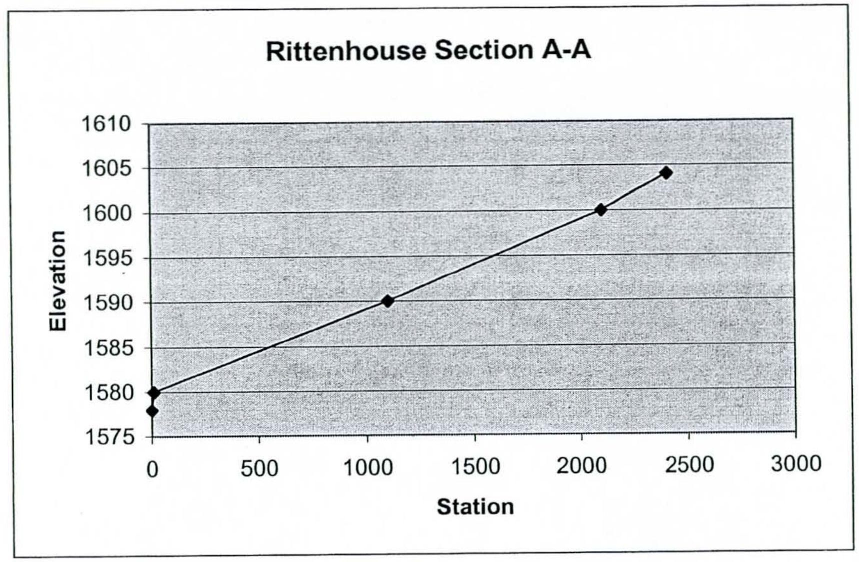


Station	Elevation
0	1563
1300	1570
1380	1577

5390.00

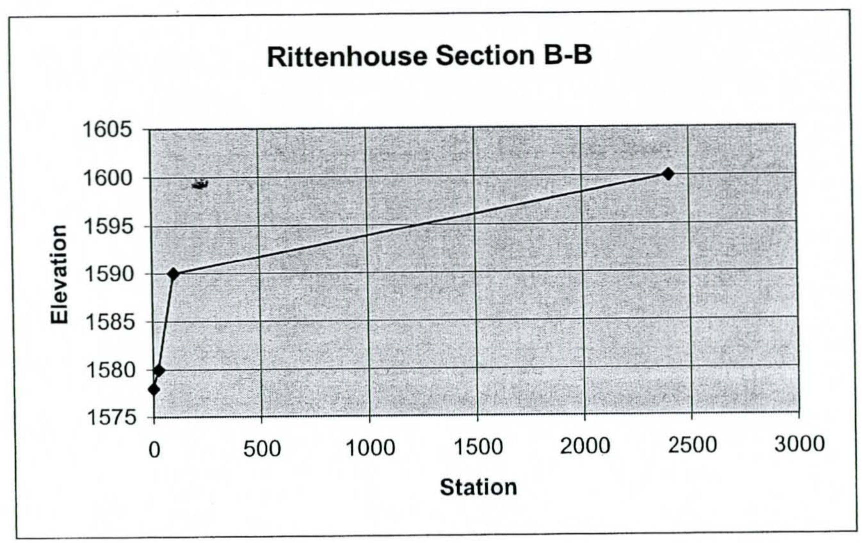
Station	Elevation
0	1578
10	1580
1100	1590
2100	1600
2400	1604

31840



Station	Elevation
0	1578
25	1580
100	1590
2400	1600

39650.00



ALTERNATIVES ANALYSIS REPORT

Appendix I: Powerline, Vineyard Road, and Rittenhouse FRS
Structural Alternative No. 5:
Levee/Floodway System

Opinion of Probable Construction Cost

FRS Structural Alternative No. 5 - Levee/Floodway

Item	Description	Unit	Quantity	Unit Cost	Total
1	Abutment Removal	CY	154,442	$ 5.00	$ 772,210.00
2	Remove Rittenhouse Dam	CY	650,885	$ 5.00	$ 3,254,425.00
3	Link Dams (Powerline to Vineyard)	CY	93,260	$ 7.00	$ 652,820.00
4	Construct Rittenhouse Levee	CY	358,374	$ 7.00	$ 2,508,618.00
5	Construct Floodway - Segment A	CY	1,704,889	$ 3.50	$ 5,967,111.50
6	Construct Floodway - Segment B	CY	7,240,148	$ 3.50	$ 25,340,518.00
7	Construct Floodway - Segment C	CY	5,810,489	$ 3.50	$ 20,336,711.50
8					
9					
				Total	$ 58,832,414.00

Land Cost (see below)			$ -
Engineering and Construction Mgt		10%	$ 5,883,241.40
Operation and Maintenance		10%	$ 5,883,241.40
	Subtotal		$ 70,598,896.80
Contingency		25%	$ 17,649,724.20
Total Costs			$ 88,248,621.00

* Land costs: If the District is required to purchase easements for the Sonoqui Detention structure (not already in place by the Bureau) the land costs are could be as much as for nine sections of land which is approximately 5,760 acres of land at $45,000/acre for $260 million. The assumption is easements that follow the geometic easement layout as for Powerline, Vineyard, and Rittenhouse

LEVEE / FLOODWAY

REMOVE ABUTMENTS

1. Powerline Left Abutment
2. Vineyard Road Left + Right Abutment

Remove Dam

③ Rittenhouse FRS cy's

Link Dams

④ Powerline to Vineyard

CONSTRUCT RITTENHOUSE LEVEE

⑤ Rittenhouse Levee

CONSTRUCT FLOODWAY

⑥ SEGMENT A Trap channel earth lined 540 ft bottom
POWERLINE $Q = 12,000$ cfs 8 ft =

⑦ SEGMENT B Trapezoidal earth-lined 770 ft bottom
VINEYARD $Q = 24,800$ cfs 10'

⑧ SEGMENT C Trapezoidal earth-lined 800 ft bottom
RITTENHOUSE
$Q = 34,800$ 12'

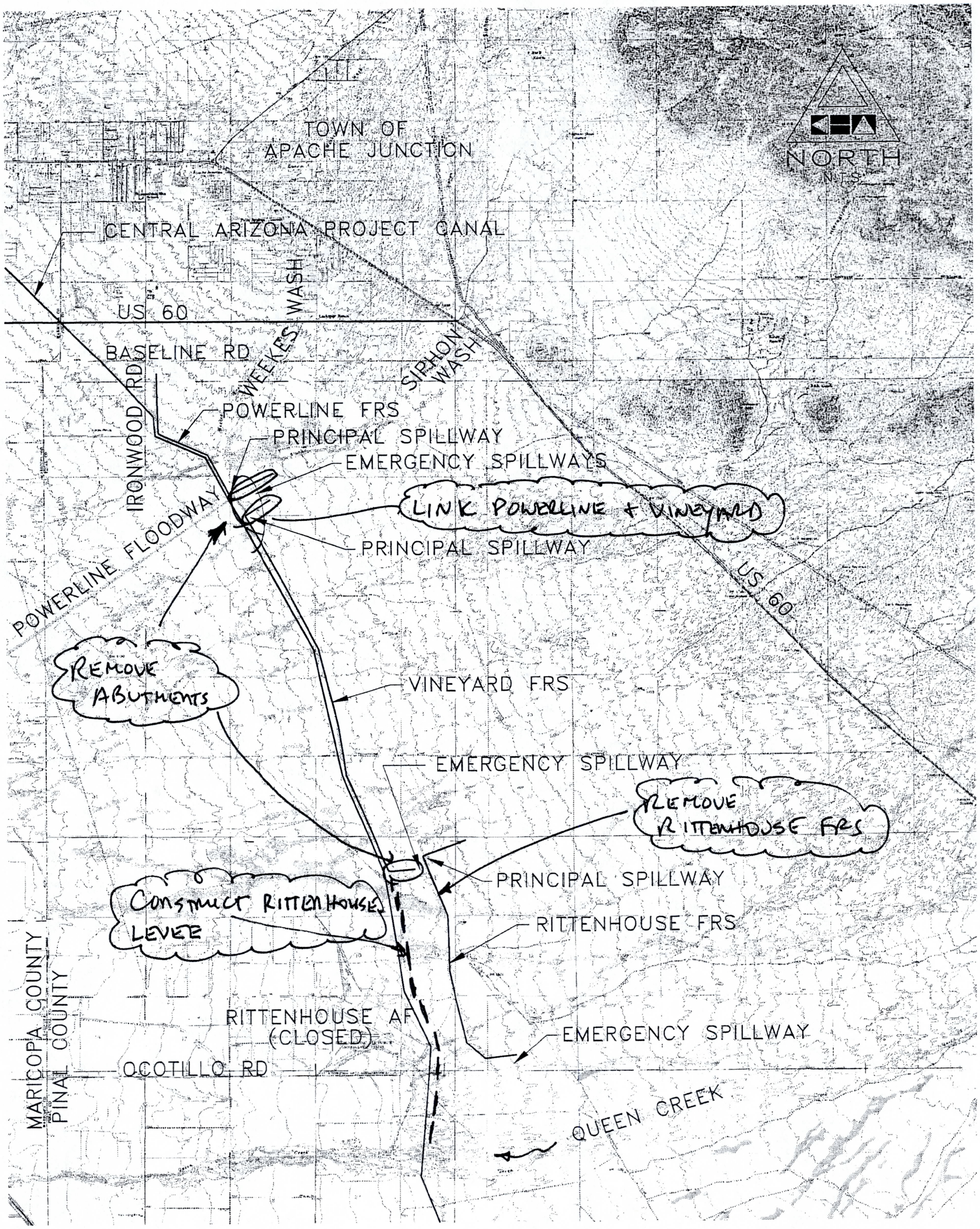


Figure 4-1. Powerline, Vineyard Road, and Rittenhouse FRS Location Map.

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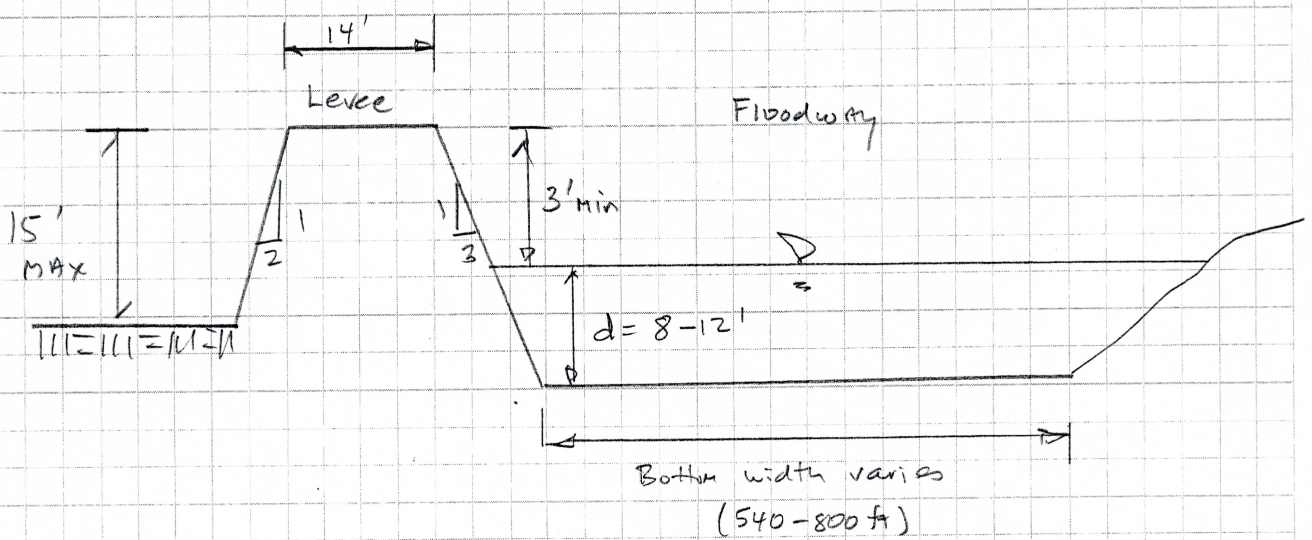
Sheet No. ______ of ______

Job STRUCTURES Subject ALTS ANALYSIS Job No 091131003

Designed by RAE Date ________ Checked by ________ Date ________

LEVEE / FLOODWAY

CONCEPT TYPICAL SECTION



Powerline Segement S = 0.0002
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	Powerline Levee/Floodway
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Bottom Width

Input Data		
Mannings Coeffic	0.030	
Slope	·.000200	ft/ft
Depth	8.00	ft
Left Side Slope	1.00	H : V
Right Side Slope	1.00	H : V
Discharge	2,000.00	cfs

Results		
Bottom Width	536.89	ft
Flow Area	4,359.2	ft²
Wetted Perim	559.52	ft
Top Width	552.89	ft
Critical Depth	2.49	ft
Critical Slope	0.009739	ft/ft
Velocity	2.75	ft/s
Velocity Head	0.12	ft
Specific Energ	8.12	ft
Froude Numb	0.17	
Flow Type	Subcritical	

Vineyard Segement S = 0.0002
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	Powerline Levee/Floodway
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Bottom Width

Input Data		
Mannings Coeffic	0.030	
Slope	ı.000200	ft/ft
Depth	10.00	ft
Left Side Slope	1.00	H : V
Right Side Slope	1.00	H : V
Discharge	ł,800.00	cfs

Results		
Bottom Width	764.73	ft
Flow Area	7,747.3	ft²
Wetted Perim	793.01	ft
Top Width	784.73	ft
Critical Depth	3.19	ft
Critical Slope	0.008959	ft/ft
Velocity	3.20	ft/s
Velocity Head	0.16	ft
Specific Energ	10.16	ft
Froude Numb	0.18	
Flow Type	Subcritical	

Rittenhouse Segment S = 0.0002
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	Rittenhouse Levee/Floodway
Flow Element	Trapezoidal Channel
Method	Manning's Formula
Solve For	Bottom Width

Input Data		
Mannings Coeffic	0.030	
Slope	'.000200	ft/ft
Depth	12.00	ft
Left Side Slope	1.00	H : V
Right Side Slope	1.00	H : V
Discharge	‡,800.00	cfs

Results		
Bottom Width	792.17	ft
Flow Area	9,650.1	ft²
Wetted Perim	826.11	ft
Top Width	816.17	ft
Critical Depth	3.91	ft
Critical Slope	0.008385	ft/ft
Velocity	3.61	ft/s
Velocity Head	0.20	ft
Specific Energ	12.20	ft
Froude Numb	0.18	
Flow Type	Subcritical	

Kimley-Horn and Associates, Inc.

Sheet No. ______ of ______

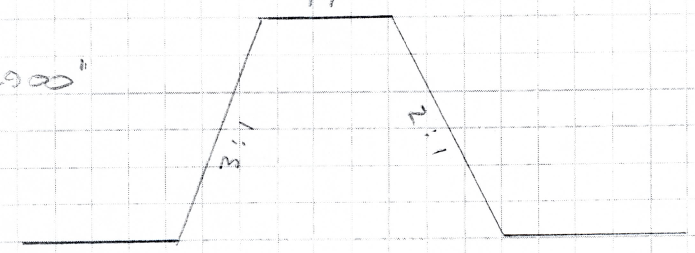
Job STRUCTURES Subject ALTS ANALYSIS WA I Job No. 091131003

Designed by R BUTTENEL Date 8-31-00 Checked by ________ Date ________

ABUTMENT REMOVAL

POWERLINE

L = 2900'



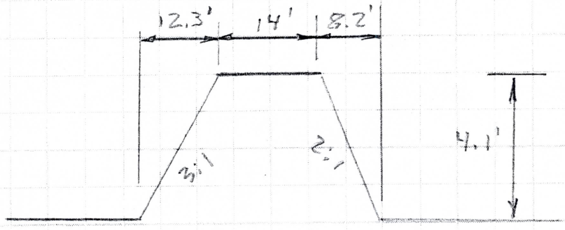
TOP EL 1589.1

E. END EL 1585.0

W. END EL 1565.0

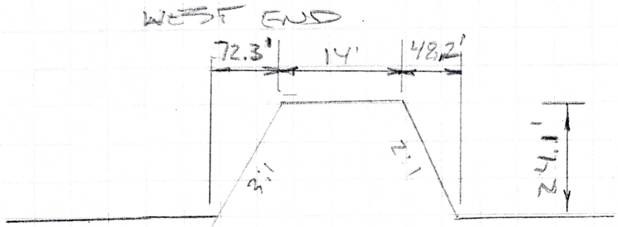
TYPICAL SECTION

EAST END



$$\left(\frac{14 + 34.3}{2}\right) \times 4.1 = 99.0 \text{ SF}$$

WEST END



$$\left(\frac{14 + 134.5}{2}\right) \times 24.1 = 1{,}789.4 \text{ SF}$$

VOLUME

$$\left(\frac{99.0 + 1{,}789.4}{2}\right) \times 2900 = 2{,}738{,}180 \text{ CF}$$

$$= \underline{101{,}414 \text{ CY}}$$

Kimley-Horn and Associates, Inc.

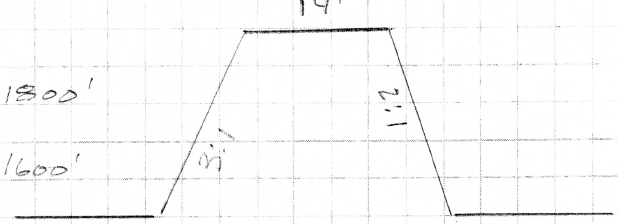
Sheet No. ______ of ______

Job STRUCTURES Subject ______ Job No. 093113003

Designed by R BUTENER Date 8.31.00 Checked by ______ Date ______

ABUTMENT REMOVAL

VINEYARD



TOP EL = 1579.5

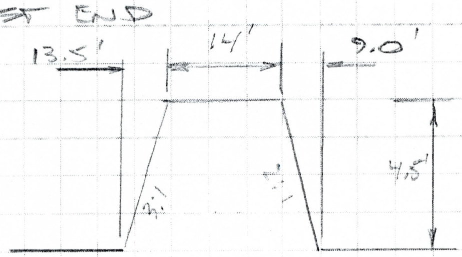
$L_{RIGHT} = 1800'$

$L_{LEFT} = 1600'$

E. END EL = 1575.0

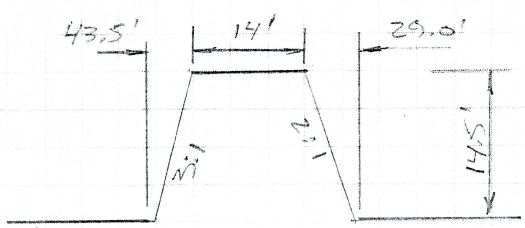
W. END EL = 1565.0

EAST END



$$\left(\frac{14 + 36.5}{2}\right) \times 4.5 = 113.6 \text{ SF}$$

WEST END



$$\left(\frac{14 + 86.5}{2}\right) \times 14.5 = 728.6 \text{ SF}$$

VOLUME

RIGHT ABUTMENT

$$\left(\frac{113.6 + 728.6}{2}\right) \times 1800 = 757,980 \text{ CF}$$

$= 28,074 \text{ CY}$

LEFT ABUTMENT

$$\left(\frac{113.6 + 728.6}{2}\right) \times 1600 = 673,760 \text{ CF}$$

$= 24,954 \text{ CY}$

53,028 CY

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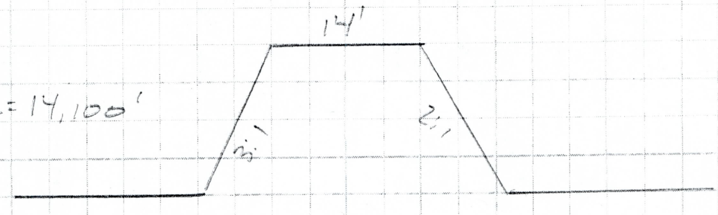
Sheet No. ______ of ______

Job [illegible] Subject ______ Job No. 091131003

Designed by R BUTTEWEG Date 8.31.00 Checked by ______ Date ______

REMOVE RITTENHOUSE DAM

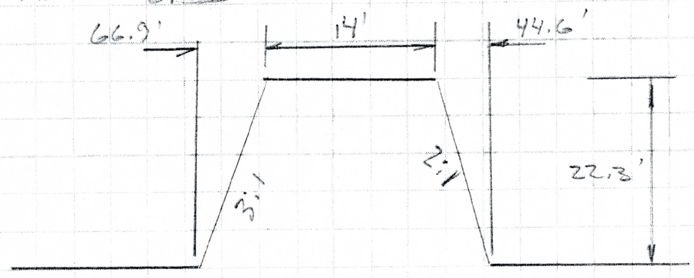
TOP EL 1602.3



N. END EL 1580
S. END EL: 1590

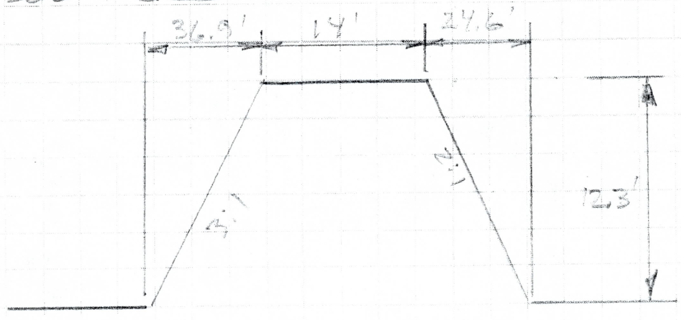
TYPICAL SECTION

NORTH END



$$\left(\frac{14 + 125.5}{2}\right) \times 22.3 = 1555.4 \text{ SF}$$

SOUTH END



$$\left(\frac{14 + 75.5}{2}\right) \times 12.3 = 550.4 \text{ SF}$$

Removal of Rittenhouse Dam

Location	End Area (SF)	Ave. End Area (SF)	Length (LF)	Volume (CF)	Volume (CY)
End of Right Abutment	0				
		777.7	2,800	2,177,560	80,651
North End Rittenhouse	1555.4				
		1052.9	14,100	14,845,890	549,848
South End Rittenhouse	550.4				
		275.2	2,000	550,400	20,386
End of Left Abutment	0				
				Total	650,885

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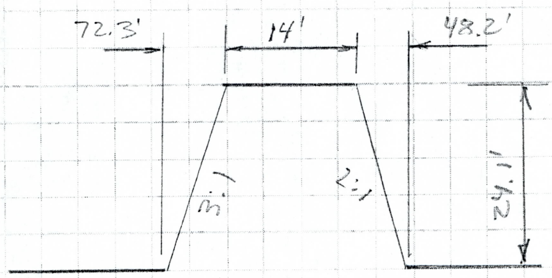
Sheet No. ______ of ______

Job ______ Subject ______ Job No. 091131003

Designed by R BUTTENEG Date 8.31.00 Checked by ______ Date ______

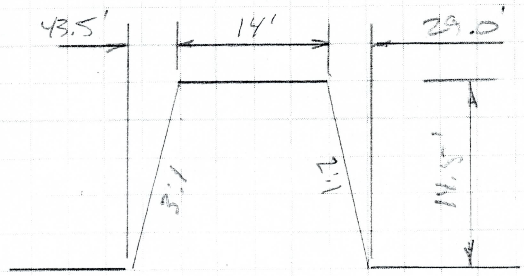
LINK DAMS (POWERLINE - VINEYARD) (L=2000')

SOUTH END OF POWERLINE



$$\left(\frac{14+134.5}{2}\right) \times 24.1 = 1,789.4 \text{ SF}$$

NORTH END OF VINEYARD



$$\left(\frac{14+86.5}{2}\right) \times 14.5 = 728.6 \text{ SF}$$

VOLUME

$$\left(\frac{1789.4+728.6}{2}\right) \times 2000 = 2,518,000 \text{ CF}$$

$$= \underline{93,260 \text{ CY}}$$

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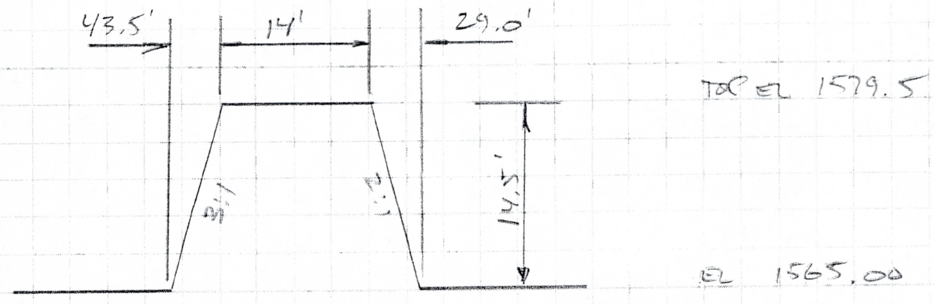
Sheet No. ______ of ______

Job STRUCTURES Subject ______ Job No. 091131003

Designed by R BUTLER Date 8-31-00 Checked by ______ Date ______

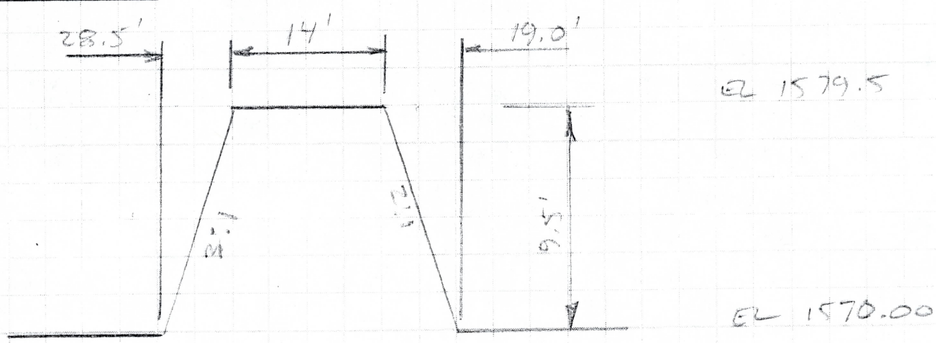
CONSTRUCT RITTENHOUSE LEVEE (L = 17,800')

NORTH END



$\left(\frac{14 + 86.5}{2}\right) \times 14.5 = 728.6$ SF

SOUTH END



$\left(\frac{14 + 61.5}{2}\right) \times 9.5 = 358.6$ SF

VOLUME

$\left(\frac{728.6 + 358.6}{2}\right) \times 17,800 = 9,676,080$ CF

$= \underline{358,374 \text{ CY}}$

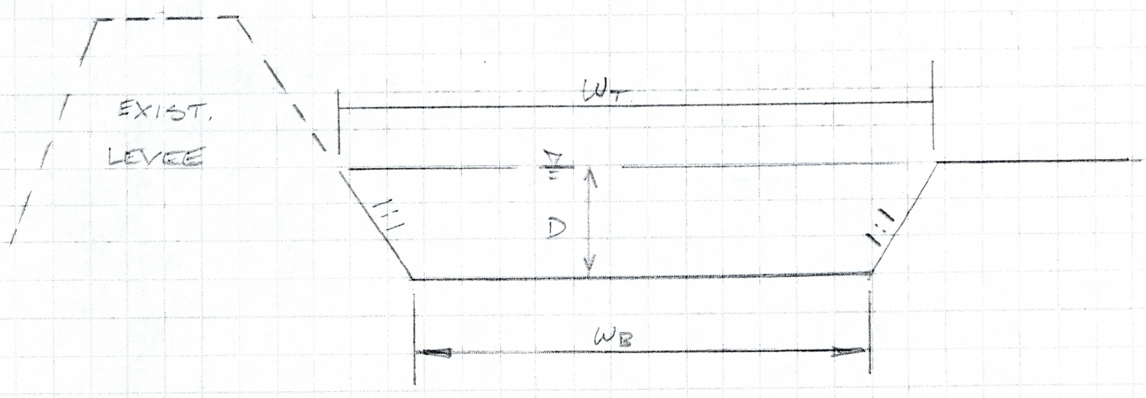
Kimley-Horn and Associates, Inc.

Sheet No. ______ of ______

Job STRUCTURES Subject ______ Job No. 091131003

Designed by R BUTTERS Date 8.30.00 Checked by ______ Date ______

FLOODWAY



TYPICAL SECTION

POWERLINE

$W_B = 540'$

$W_T = 556'$

$D = 8'$

$L = 10,500$ LF

END AREA

$$\frac{(540+556)}{2} \times 8 = 4384 \text{ SF}$$

VOLUME

$4384 \times 10,500 = 46,032,000$ CF

$= \underline{1,704,889 \text{ CY}}$ *

* ASSUMES ALL EXCAVATION TO DEPTH OF FLOW

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Sheet No. ______ of ______

Job STRUCTURES Subject ______ Job No. 091131003

Designed by R BUTTONE Date 8.30.00 Checked by ______ Date ______

FLOODWAY
VINEYARD

W_B = 770'
W_T = 790'
D = 10'
L = 23,500'

END AREA

$$\frac{(770+790)}{2} \times 10 = 7,800 \text{ SF}$$

VOLUME

$$7800 \times 23500 = 183,300,000 \text{ CF}$$

$$= \underline{\underline{6,788,889 \text{ CY}}}$$

RITTENHOUSE

W_B = 800'
W_T = 824'
D = 12'
L = 14,300'

END AREA

$$\left(\frac{800+824}{2}\right) \times 12 = 9,744 \text{ SF}$$

VOLUME

$$9,744 \times 14,300 = 139,339,200 \text{ CF}$$

$$= \underline{\underline{5,160,711 \text{ CY}}} \quad *$$

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Sheet No. ______ of ______

Job STRUCTURES Subject ______ Job No. 09131003

Designed by R BUTTNER Date 8-30-00 Checked by ______ Date ______

FLOODWAY
TRANSITIONS

POWERLINE - VINEYARD

END AREA
POWERLINE 4384 SF
VINEYARD 7800 SF

TRANSITION LENGTH 2000'

$$\left(\frac{4384 + 7800}{2}\right) \times 2000 = 12,184,000 \text{ CF}$$

$$= \underline{\underline{451,259 \text{ CY}}}$$

VINEYARD - RITTENHOUSE

END AREA
VINEYARD 7800 SF
RITTENHOUSE 9744 SF

TRANSITION LENGTH 2000'

$$\left(\frac{7800 + 9744}{2}\right) \times 2000 = 17,544,000 \text{ CF}$$

$$= \underline{\underline{649,778 \text{ CY}}} \quad *$$

Kimley-Horn and Associates, Inc.

Sheet No. ______ of ______

Job STRUCTURES Subject ______ Job No. 091131003

Designed by R BURGESS Date 9.30.00 Checked by ______ Date ______

FLOODWAY

SEGMENT 'A'

POWERLINE -	1,704,889 CY

SEGMENT 'B'

TRANS. POWERLINE - VINEYARD -	451,259 CY
VINEYARD -	6,788,889 CY
	7,240,148 CY

SEGMENT 'C'

TRANS. VINEYARD - RITTENHOUSE -	649,778 CY
RITTENHOUSE	5,160,711 CY
	5,810,489 CY

Routing through Sonoqui Dike

Routing 100-Year, 6-Hour Gen. Storms

Peak Q = 31,756 ft^3/s Area above Whitlow Dam = 142.9 square miles
Volume = 13,721 AF

Routed through Whitlow Ranch Dam
Peak Outflow Q = 819 ft^3/s

This flow was then routed downstream and combined with flood out of lower Queen Creek watershed (113.8 square miles)

Peak Inflow = 16,236 ft^3/s with flow from Whitlow = 17,000 ft^3/s
Peak Outflow = 1,113 ft^3/s
WS Elevation = ± 1580
Storage = 8424 AF

Routing ~~MPF~~ PMF

Peak Q = 68,714 ft^3/s Area above Whitlow Dam = 142.9 square miles
Volume = 27,562 AF

As in 100-year routing, routed through Whitlow Dam and down to Queen Creek. Combining flood hydrographs.

Peak Outflow Q = 943 ft^3/s

Lower Queen Creek:

Peak Inflow = 37,016 ft^3/s
Peak Outflow = 2,296 ft^3/s
WS Elevation = ± 1585
Storage = 13,095 AF

COPY

INPUT DATA FOR FLOOD ROUTING
SALT-GILA AQUEDUCT REACH 3

SONOQUI

STATION	STRUCT. TYPE	SIZE	INVERT ELEV.	CREST ELEV.	STORAGE CAPACITY DATA ELEV.	AC.FT	ELEV.	AC.FT.
161+82	Pipe OC	4-72"	1561.32	1573.5	1572	0	1574	23
470+00	Pipe OC	36"	1560.50	1585.0	1576	79	1578	1831
					1580	4964	1582	8718
					1584	12957	1586	17789
500+00	Pipe OC	2-42"	1555.40	1565.1	1559.6	0	1561.6	12
					1563.6	45		
1602+60	Pipe OC	42"	1557.81	1564.1				
1662+00	Wash Si	20’8"	1552.01	1558.7	1552	0	1554	1
					1556	5	1558	20
					1560	113		
1884+30	Pipe OC	54"	1554.43	1562.9				
1887+06	Pipe OC	54"	1559.50	1568.0				
915+45	Culvert	2-66"	1536.68	1555.5	*			
948+60	Culvert	54"	1525.83	1555.3	*			
970+40	Culvert	2-72"	1526.94	1554.8	1526	0	1528	0.6
					1530	2.8	1532	7.2
					1534	14.1	1536	22.4
					1538	39.2	1540	61.0
986+00	Culvert	48"	1531.52	1554.7				
991+00	Culvert	48"	1529.75	1555.1	*			
1009+00	Culvert	36"	1535.66	1554.9				
1012+00	Culvert	66"	1532.28	1554.9				
1036+00	Culvert	30"	1537.45	1556.4	1540	0	1542	2
					1544	8	1546	15
					1548	25		
1052+35	Culvert	42"	1534.43	1555.1	*			
1059+50	Culvert	42"	1534.53	1554.7				

* Original routing data not available.

COPY

ALTERNATIVES ANALYSIS REPORT

Appendix J: Powerline, Vineyard Road, and Rittenhouse FRS
Structural Alternative No. 6:
Utilization of Central Arizona Project Canal

Opinion of Probable Construction Cost

FRS Structural Alternative No. 6
Utilization of Central Arizona Project Canal

Item	Description	Unit	Quantity	Unit Cost	Total
1	Culvert 7-ft dia	LF	420	$ 225.00	$ 94,500.00
2	Inlet Structure w/gates	L Sum	1	$ 16,500.00	$ 16,500.00
3	Outlet Structure	L Sum	1	$ 5,000.00	$ 5,000.00
				Construction	$ 116,000.00

Land Cost		$ -
Engineering and Construction Mgt	10%	$ 11,600.00
Operation and Maintenance	10%	$ 11,600.00
Subtotal		$ 139,200.00
Contingency	25%	$ 34,800.00
Total Costs		$ 174,000.00

Opinion of Probable Construction Cost

FRS Structural Alternative No. 6
Utilization of Central Arizona Project Canal

Item	Description	Unit	Quantity	Unit Cost	Total
1	Culvert 7-ft dia	LF	700	$ 225.00	$ 157,500.00
2	Inlet Structure w/gates	L Sum	1	$ 16,500.00	$ 16,500.00
3	Outlet Structure	L Sum	1	$ 5,000.00	$ 5,000.00
				Construction	$ 179,000.00

Land Cost		$ -
Engineering and Construction Mgt	10%	$ 17,900.00
Operation and Maintenance	10%	$ 17,900.00
Subtotal		$ 214,800.00
Contingency	25%	$ 53,700.00
Total Costs		$ 268,500.00

Opinion of Probable Construction Cost

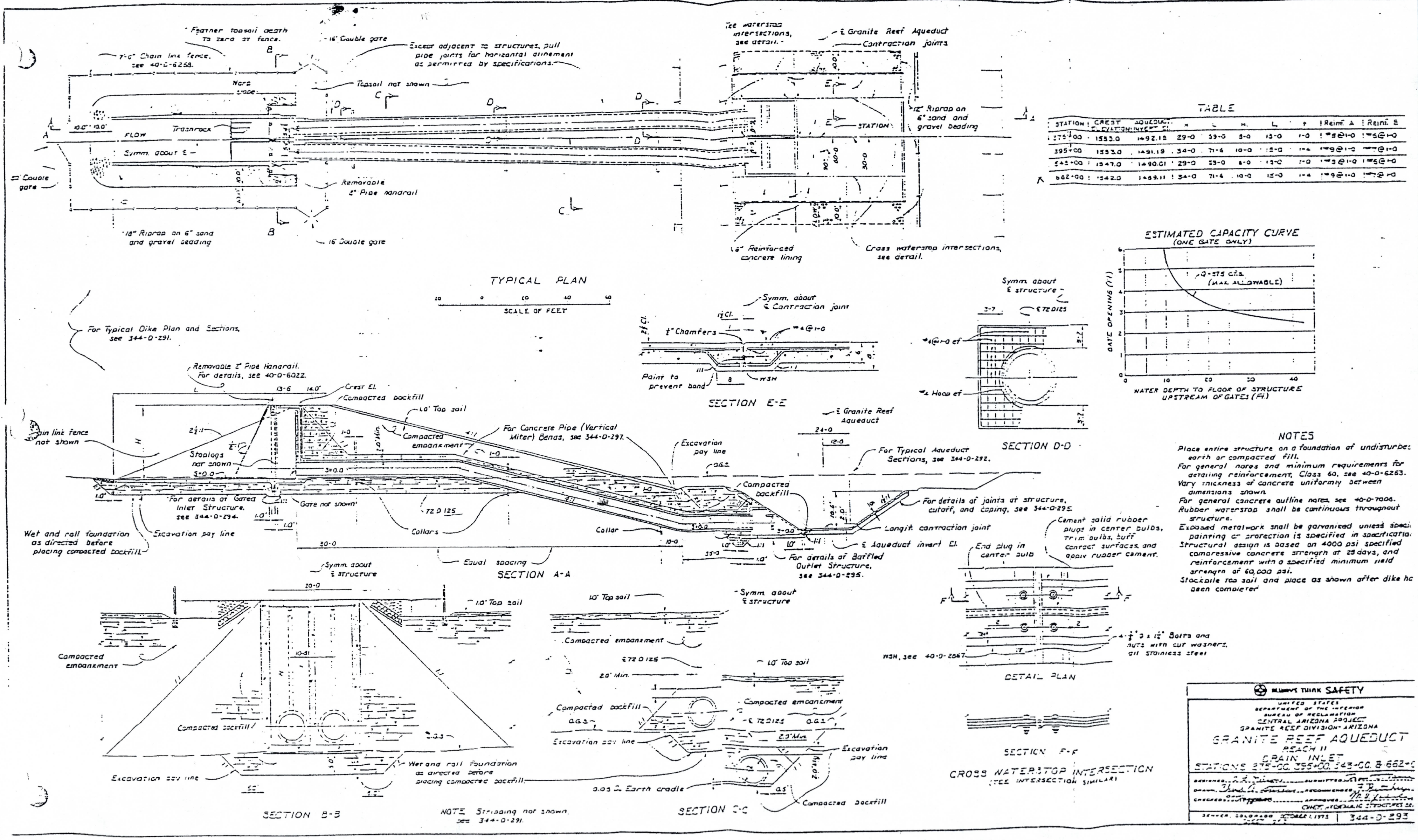
FRS Structural Alternative No. 6
Utilization of Central Arizona Project Canal

Item	Description	Unit	Quantity	Unit Cost	Total
1	Culvert 7-ft dia	LF	420	$ 225.00	$ 94,500.00
2	Inlet Structure w/gates	L Sum	1	$ 16,500.00	$ 16,500.00
3	Outlet Structure/energy dissipator	L Sum	1	$ 6,000.00	$ 6,000.00
4	Concrete trapezoidal Floodway 20-ft bottom 3.5-ft deep Length 310 LF	SY	1123	$ 25.00	$ 28,075.00
5	Sidespill weir 30-ft wide	L Sum	1	$ 5,000.00	$ 5,000.00
				Construction	$ 150,075.00

Land Cost		$ -
Engineering and Construction Mgt	10%	$ 15,007.50
Operation and Maintenance	10%	$ 15,007.50
Subtotal		$ 180,090.00
Contingency	25%	$ 45,022.50
Total Costs		$ 225,112.50

UtilizationCAPcanal.xls/Rittenhouse

FIGURE 3



TABLE

STATION	CREST ELEVATION	AQUEDUCT INVERT EL.	H	L	H₁	L₁	t	Reinf. A	Reinf. B
375+00	1553.0	1492.12	29-0	59-0	5-0	13-0	1-0	5@1-0	5@1-0
395+00	1553.0	1491.19	34-0	71-6	10-0	15-0	1-4	9@1-0	7@1-0
543+00	1547.0	1490.01	29-0	59-0	8-0	13-0	1-0	9@1-0	5@1-0
662+00	1542.0	1469.11	34-0	71-6	10-0	15-0	1-4	9@1-0	7@1-0

Kimley-Horn and Associates, Inc.

Sheet No. ____ of ____

Job STRUCTURES Subject ALTS ANALYSIS Job No. 091131003

Designed by ______ Date ______ Checked by ______ Date ______

CAPACITY OF CAP CANAL REACH 2 = 2750 cfs

CAPACITY OF NEW OUTLET FOR EACH DAM = 2750/3 = 916 cfs
use 900 cfs/outlet

POWERLINE PONDING ELEV = 1583.3 MAX HEAD = 20'
PRIN SPILLWAY INVERT = 1563.0
ASSUME CANAL INVERT = 1563 - 15 = 1548 ft @ POWERLINE

VINEYARD ROAD PONDING ELEV = 1574.8 MAX HEAD = 11.8
PRIN SPILLWAY INVERT = 1563
ASSUME CANAL INVERT = 1563 - 15 = 1548 ft @ VINEYARD

RITTENHOUSE PONDING ELEV = 1597.6 MAX HEAD = 19.4 Ft
PRIN SPILLWAY INVERT = 1578.2
ASSUME CANAL INVERT = 1578.2 - 15 = 1563.20 @ RITTENHOUSE

EACH OUTLET WILL CONSIST OF TWIN RCP WITH CAPACITY OF 450 cfs/each

FRS	LENGTH OF CULVERT TO CAP
POWERLINE	210'
VINEYARD	350'
RITTENHOUSE	100' + Floodway to side spill into canal (Floodway sized @ 900 cfs)

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CURRENT DATE: 07-20-2000                                FILE DATE: 07-20-2000
CURRENT TIME: 17:22:35                                  FILE NAME: POWER

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ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ   FHWA CULVERT ANALYSIS    ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ     HY-8, VERSION 6.0      ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
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³            SITE DATA             ³        CULVERT SHAPE, MATERIAL, INLET      ³
ÃÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ´
³ L ³  INLET   OUTLET  CULVERT ³ BARRELS                                        ³
³ V ³  ELEV.    ELEV.   LENGTH ³ SHAPE        SPAN   RISE  MANNING    INLET    ³
³NO.³  (ft)     (ft)     (ft)  ³ MATERIAL     (ft)   (ft)     n       TYPE     ³
³ 1 ³1563.00  1548.00   210.54 ³ 2 RCP        7.00   7.00   .012   CONVENTIONAL³
³ 2 ³                          ³                                                ³
³ 3 ³                          ³                                                ³
³ 4 ³                          ³                                                ³
³ 5 ³                          ³                                                ³
³ 6 ³                          ³                                                ³
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SUMMARY OF CULVERT FLOWS (cfs)        FILE: POWER              DATE: 07-20-2000

 ELEV (ft)   TOTAL       1       2       3       4       5       6  ROADWAY ITR
  1563.00      0.0     0.0     0.0     0.0     0.0     0.0     0.0     0.00  1
  1564.81     90.0    90.0     0.0     0.0     0.0     0.0     0.0     0.00  1
  1566.04    180.0   180.0     0.0     0.0     0.0     0.0     0.0     0.00  1
  1567.00    270.0   270.0     0.0     0.0     0.0     0.0     0.0     0.00  1
  1567.78    360.0   360.0     0.0     0.0     0.0     0.0     0.0     0.00  1
  1568.47    450.0   450.0     0.0     0.0     0.0     0.0     0.0     0.00  1
  1569.13    540.0   540.0     0.0     0.0     0.0     0.0     0.0     0.00  1
  1569.82    630.0   630.0     0.0     0.0     0.0     0.0     0.0     0.00  1
  1570.56    720.0   720.0     0.0     0.0     0.0     0.0     0.0     0.00  1
  1571.38    810.0   810.0     0.0     0.0     0.0     0.0     0.0     0.00  1
  1572.30    900.0   900.0     0.0     0.0     0.0     0.0     0.0     0.00  1
  1589.00   1933.8  1933.8     0.0     0.0     0.0     0.0     0.0 OVERTOPPING
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ

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  SUMMARY OF ITERATIVE SOLUTION ERRORS   FILE: POWER           DATE: 07-20-2000

        HEAD           HEAD            TOTAL          FLOW          % FLOW
      ELEV (ft)     ERROR (ft)       FLOW (cfs)    ERROR (cfs)       ERROR
       1563.00         0.000            0.00          0.00           0.00
       1564.81         0.000           90.00          0.00           0.00
       1566.04         0.000          180.00          0.00           0.00
       1567.00         0.000          270.00          0.00           0.00
       1567.78         0.000          360.00          0.00           0.00
       1568.47         0.000          450.00          0.00           0.00
       1569.13         0.000          540.00          0.00           0.00
       1569.82         0.000          630.00          0.00           0.00
       1570.56         0.000          720.00          0.00           0.00
       1571.38         0.000          810.00          0.00           0.00
       1572.30         0.000          900.00          0.00           0.00
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 <1> TOLERANCE (ft) = 0.010                      <2> TOLERANCE (%) = 1.000
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CURRENT DATE: 07-20-2000
CURRENT TIME: 17:22:35

FILE DATE: 07-20-2000
FILE NAME: POWER

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PERFORMANCE CURVE FOR CULVERT 1 - 2(7.00 (ft) BY 7.00 (ft)) RCP

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DIS- GE W s)	HEAD- WATER ELEV. (ft)	INLET CONTROL DEPTH (ft)	OUTLET CONTROL DEPTH (ft)	FLOW TYPE <F4>	NORMAL DEPTH (ft)	CRIT. DEPTH (ft)	OUTLET DEPTH (ft)	TW DEPTH (ft)	OUTLET VEL. (fps)	TW VEL. (fps)
0.00	1563.00	0.00	-15.00	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
90.00	1564.81	1.81	-10.63	1-S2n	0.74	1.67	0.67	1.23	22.51	2.78
180.00	1566.04	3.04	-10.16	1-S2n	0.99	2.41	1.07	1.87	23.46	3.58
270.00	1567.00	4.00	-9.70	1-S2n	1.25	2.99	1.35	2.38	25.86	4.14
360.00	1567.78	4.78	-9.21	1-S2n	1.46	3.49	1.59	2.83	27.20	4.57
450.00	1568.47	5.47	-8.69	1-S2n	1.62	3.91	1.81	3.23	28.32	4.93
540.00	1569.13	6.13	-8.11	1-S2n	1.78	4.31	2.01	3.60	29.54	5.24
630.00	1569.82	6.82	-7.49	1-S2n	1.93	4.66	2.23	3.95	29.81	5.51
720.00	1570.56	7.56	-6.81	1-S2n	2.09	4.99	2.39	4.28	30.90	5.75
810.00	1571.38	8.38	-6.08	1-S2n	2.21	5.28	2.58	4.58	31.36	5.97
900.00	1572.30	9.30	-5.28	1-S2n	2.33	5.57	2.77	4.88	31.75	6.18

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El. inlet face invert 1563.00 ft El. outlet invert 1548.00 ft
El. inlet throat invert 0.00 ft El. inlet crest 0.00 ft

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***** SITE DATA ***** CULVERT INVERT **************

INLET STATION	0.00 ft
INLET ELEVATION	1563.00 ft
OUTLET STATION	210.00 ft
OUTLET ELEVATION	1548.00 ft
NUMBER OF BARRELS	2
SLOPE (V/H)	0.0714
CULVERT LENGTH ALONG SLOPE	210.54 ft

***** CULVERT DATA SUMMARY ************************

BARREL SHAPE	CIRCULAR
BARREL DIAMETER	7.00 ft
BARREL MATERIAL	CONCRETE
BARREL MANNING'S n	0.012
INLET TYPE	CONVENTIONAL
INLET EDGE AND WALL	BEVELED EDGE (1:1)
INLET DEPRESSION	NONE

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CURRENT DATE: 07-20-2000
CURRENT TIME: 17:22:35

FILE DATE: 07-20-2000
FILE NAME: POWER

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** REGULAR CHANNEL CROSS SECTION ****************

BOTTOM WIDTH	25.00 ft
SIDE SLOPE H/V (X:1)	1.0
CHANNEL SLOPE V/H (ft/ft)	0.001
MANNING'S n (.01-0.1)	0.013
CHANNEL INVERT ELEVATION	1548.00 ft
CULVERT NO.1 OUTLET INVERT ELEVATION	1548.00 ft

******* UNIFORM FLOW RATING CURVE FOR DOWNSTREAM CHANNEL

FLOW (cfs)	W.S.E. (ft)	FROUDE NUMBER	DEPTH (ft)	VEL. (f/s)	SHEAR (psf)
0.00	1548.00	0.000	0.00	0.00	0.00
90.00	1549.23	0.442	1.23	2.78	0.04
180.00	1549.87	0.462	1.87	3.58	0.06
270.00	1550.38	0.472	2.38	4.14	0.07
360.00	1550.83	0.478	2.83	4.57	0.09
450.00	1551.23	0.483	3.23	4.93	0.10
540.00	1551.60	0.486	3.60	5.24	0.11
630.00	1551.95	0.488	3.95	5.51	0.12
720.00	1552.28	0.490	4.28	5.75	0.13
810.00	1552.58	0.492	4.58	5.97	0.14
900.00	1552.88	0.493	4.88	6.18	0.15

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ROADWAY SURFACE	PAVED
EMBANKMENT TOP WIDTH	14.00 ft
CREST LENGTH	8000.00 ft
OVERTOPPING CREST ELEVATION	1589.00 ft

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CURRENT DATE: 07-21-2000
CURRENT TIME: 09:48:07

FILE DATE: 07-21-2000
FILE NAME: VINEYARD

FHWA CULVERT ANALYSIS
HY-8, VERSION 6.0

	SITE DATA			CULVERT SHAPE, MATERIAL, INLET				
L V NO.	INLET ELEV. (ft)	OUTLET ELEV. (ft)	CULVERT LENGTH (ft)	BARRELS SHAPE MATERIAL	SPAN (ft)	RISE (ft)	MANNING n	INLET TYPE
1	1563.00	1548.00	350.32	2 RCP	7.00	7.00	.012	CONVENTIONAL
2								
3								
4								
5								
6								

SUMMARY OF CULVERT FLOWS (cfs) FILE: VINEYARD DATE: 07-21-2000

ELEV (ft)	TOTAL	1	2	3	4	5	6	ROADWAY	ITR
1563.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1564.91	90.0	90.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1566.14	180.0	180.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1567.10	270.0	270.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1567.88	360.0	360.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1568.57	450.0	450.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1569.23	540.0	540.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1569.92	630.0	630.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1570.66	720.0	720.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1571.48	810.0	810.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1572.40	900.0	900.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1578.00	1322.7	1322.7	0.0	0.0	0.0	0.0	0.0	OVERTOPPING	

SUMMARY OF ITERATIVE SOLUTION ERRORS FILE: VINEYARD DATE: 07-21-2000

HEAD ELEV (ft)	HEAD ERROR (ft)	TOTAL FLOW (cfs)	FLOW ERROR (cfs)	% FLOW ERROR
1563.00	0.000	0.00	0.00	0.00
1564.91	0.000	90.00	0.00	0.00
1566.14	0.000	180.00	0.00	0.00
1567.10	0.000	270.00	0.00	0.00
1567.88	0.000	360.00	0.00	0.00
1568.57	0.000	450.00	0.00	0.00
1569.23	0.000	540.00	0.00	0.00
1569.92	0.000	630.00	0.00	0.00
1570.66	0.000	720.00	0.00	0.00
1571.48	0.000	810.00	0.00	0.00
1572.40	0.000	900.00	0.00	0.00

<1> TOLERANCE (ft) = 0.010 <2> TOLERANCE (%) = 1.000

CURRENT DATE: 07-21-2000 FILE DATE: 07-21-2000
CURRENT TIME: 09:48:07 FILE NAME: VINEYARD

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PERFORMANCE CURVE FOR CULVERT 1 - 2(7.00 (ft) BY 7.00 (ft)) RCP

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DIS-CHARGE FLOW (cfs)	HEAD-WATER ELEV. (ft)	INLET CONTROL DEPTH (ft)	OUTLET CONTROL DEPTH (ft)	FLOW TYPE <F4>	NORMAL DEPTH (ft)	CRIT. DEPTH (ft)	OUTLET DEPTH (ft)	TW DEPTH (ft)	OUTLET VEL. (fps)	TW VEL. (fps)
0.00	1563.00	0.00	-15.00	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
90.00	1564.91	1.91	-10.63	1-S2n	0.81	1.67	0.77	1.00	19.18	3.46
180.00	1566.14	3.14	-10.13	1-S2n	1.14	2.41	1.11	1.52	22.31	4.47
270.00	1567.10	4.10	-9.64	1-S2n	1.44	2.99	1.39	1.94	24.91	5.17
360.00	1567.88	4.88	-9.11	1-S2n	1.64	3.49	1.72	2.30	24.31	5.73
450.00	1568.57	5.57	-8.54	1-S2n	1.85	3.91	1.96	2.63	25.42	6.19
540.00	1569.23	6.23	-7.90	1-S2n	2.05	4.31	2.18	2.93	26.43	6.59
630.00	1569.92	6.92	-7.20	1-S2n	2.22	4.66	2.36	3.22	27.55	6.94
720.00	1570.66	7.66	-6.43	1-S2n	2.37	4.99	2.57	3.48	28.01	7.26
810.00	1571.48	8.48	-5.60	1-S2n	2.53	5.28	2.77	3.73	28.56	7.55
900.00	1572.40	9.40	-4.69	1-S2n	2.68	5.57	2.95	3.97	29.19	7.82

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El. inlet face invert 1563.00 ft El. outlet invert 1548.00 ft
El. inlet throat invert 0.00 ft El. inlet crest 0.00 ft

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***** SITE DATA ***** CULVERT INVERT **************

INLET STATION	0.00 ft
INLET ELEVATION	1563.00 ft
OUTLET STATION	350.00 ft
OUTLET ELEVATION	1548.00 ft
NUMBER OF BARRELS	2
SLOPE (V/H)	0.0429
CULVERT LENGTH ALONG SLOPE	350.32 ft

***** CULVERT DATA SUMMARY ************************

BARREL SHAPE	CIRCULAR
BARREL DIAMETER	7.00 ft
BARREL MATERIAL	CONCRETE
BARREL MANNING'S n	0.012
INLET TYPE	CONVENTIONAL
INLET EDGE AND WALL	BEVELED EDGE (1:1)
INLET DEPRESSION	NONE

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CURRENT DATE: 07-21-2000
CURRENT TIME: 09:48:07

FILE DATE: 07-21-2000
FILE NAME: VINEYARD

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* REGULAR CHANNEL CROSS SECTION ****************

BOTTOM WIDTH	25.00 ft
SIDE SLOPE H/V (X:1)	1.0
CHANNEL SLOPE V/H (ft/ft)	0.001
MANNING'S n (.01-0.1)	0.013
CHANNEL INVERT ELEVATION	1548.00 ft
CULVERT NO.1 OUTLET INVERT ELEVATION	1548.00 ft

******* UNIFORM FLOW RATING CURVE FOR DOWNSTREAM CHANNEL

FLOW (cfs)	W.S.E. (ft)	FROUDE NUMBER	DEPTH (ft)	VEL. (f/s)	SHEAR (psf)
0.00	1548.00	0.000	0.00	0.00	0.00
90.00	1549.00	0.609	1.00	3.46	0.06
180.00	1549.52	0.639	1.52	4.47	0.09
270.00	1549.94	0.655	1.94	5.17	0.12
360.00	1550.30	0.665	2.30	5.73	0.14
450.00	1550.63	0.673	2.63	6.19	0.16
540.00	1550.93	0.678	2.93	6.59	0.18
630.00	1551.22	0.682	3.22	6.94	0.20
720.00	1551.48	0.686	3.48	7.26	0.22
810.00	1551.73	0.689	3.73	7.55	0.23
900.00	1551.97	0.691	3.97	7.82	0.25

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ROADWAY SURFACE	PAVED
EMBANKMENT TOP WIDTH	14.00 ft
CREST LENGTH	24000.00 ft
OVERTOPPING CREST ELEVATION	1578.00 ft

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CURRENT DATE: 07-21-2000
CURRENT TIME: 09:52:58

FILE DATE: 07-21-2000
FILE NAME: RITTEN

FHWA CULVERT ANALYSIS
HY-8, VERSION 6.0

C U L V NO.	SITE DATA INLET ELEV. (ft)	OUTLET ELEV. (ft)	CULVERT LENGTH (ft)	CULVERT SHAPE, MATERIAL, INLET BARRELS SHAPE MATERIAL	SPAN (ft)	RISE (ft)	MANNING n	INLET TYPE
1	1578.00	1576.00	100.02	2 RCP	7.00	7.00	.012	CONVENTIONAL
2								
3								
4								
5								
6								

SUMMARY OF CULVERT FLOWS (cfs) FILE: RITTEN DATE: 07-21-2000

ELEV (ft)	TOTAL	1	2	3	4	5	6	ROADWAY	ITR
1578.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1580.36	90.0	90.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1581.22	180.0	180.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1582.18	270.0	270.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1582.96	360.0	360.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1583.65	450.0	450.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1584.31	540.0	540.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1585.00	630.0	630.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1585.74	720.0	720.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1586.56	810.0	810.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1587.48	900.0	900.0	0.0	0.0	0.0	0.0	0.0	0.00	1
1600.00	1726.2	1726.2	0.0	0.0	0.0	0.0	0.0	OVERTOPPING	

SUMMARY OF ITERATIVE SOLUTION ERRORS FILE: RITTEN DATE: 07-21-2000

HEAD ELEV (ft)	HEAD ERROR (ft)	TOTAL FLOW (cfs)	FLOW ERROR (cfs)	% FLOW ERROR
1578.00	0.000	0.00	0.00	0.00
1580.36	0.000	90.00	0.00	0.00
1581.22	0.000	180.00	0.00	0.00
1582.18	0.000	270.00	0.00	0.00
1582.96	0.000	360.00	0.00	0.00
1583.65	0.000	450.00	0.00	0.00
1584.31	0.000	540.00	0.00	0.00
1585.00	0.000	630.00	0.00	0.00
1585.74	0.000	720.00	0.00	0.00
1586.56	0.000	810.00	0.00	0.00
1587.48	0.000	900.00	0.00	0.00

<1> TOLERANCE (ft) = 0.010 <2> TOLERANCE (%) = 1.000

CURRENT DATE: 07-21-2000 FILE DATE: 07-21-2000
CURRENT TIME: 09:52:58 FILE NAME: RITTEN

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PERFORMANCE CURVE FOR CULVERT 1 - 2(7.00 (ft) BY 7.00 (ft)) RCP

ÄÄÄ

DIS-C[illegible] (c[illegible])	HEAD-WATER ELEV. (ft)	INLET CONTROL DEPTH (ft)	OUTLET CONTROL DEPTH (ft)	FLOW TYPE <F4>	NORMAL DEPTH (ft)	CRIT. DEPTH (ft)	OUTLET DEPTH (ft)	TW DEPTH (ft)	OUTLET VEL. (fps)	TW VEL. (fps)
0.00	1578.00	0.00	-2.00	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
90.00	1580.36	1.99	2.36	1-S2n	0.96	1.67	1.06	0.71	11.84	6.16
180.00	1581.22	3.22	2.82	1-S2n	1.43	2.41	1.60	1.07	13.47	7.97
270.00	1582.18	4.18	3.26	1-S2n	1.72	2.99	2.03	1.37	14.55	9.25
360.00	1582.96	4.96	3.72	1-S2n	2.02	3.49	2.39	1.62	15.47	10.25
450.00	1583.65	5.65	4.20	1-S2n	2.27	3.91	2.75	1.86	16.04	11.09
540.00	1584.31	6.31	4.72	1-S2n	2.49	4.31	3.06	2.07	16.71	11.82
630.00	1585.00	7.00	5.28	1-S2n	2.72	4.66	3.36	2.27	17.25	12.46
720.00	1585.74	7.74	5.90	1-S2n	2.93	4.99	3.64	2.46	17.80	13.04
810.00	1586.56	8.56	6.54	1-S2n	3.13	5.28	3.88	2.64	18.50	13.57
900.00	1587.48	9.48	7.25	5-S2n	3.32	5.57	4.17	2.81	18.83	14.05

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El. inlet face invert 1578.00 ft El. outlet invert 1576.00 ft
El. inlet throat invert 0.00 ft El. inlet crest 0.00 ft

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***** SITE DATA ***** CULVERT INVERT **************

INLET STATION	0.00 ft
INLET ELEVATION	1578.00 ft
OUTLET STATION	100.00 ft
OUTLET ELEVATION	1576.00 ft
NUMBER OF BARRELS	2
SLOPE (V/H)	0.0200
CULVERT LENGTH ALONG SLOPE	100.02 ft

***** CULVERT DATA SUMMARY ************************

BARREL SHAPE	CIRCULAR
BARREL DIAMETER	7.00 ft
BARREL MATERIAL	CONCRETE
BARREL MANNING'S n	0.012
INLET TYPE	CONVENTIONAL
INLET EDGE AND WALL	BEVELED EDGE (1:1)
INLET DEPRESSION	NONE

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CURRENT DATE: 07-21-2000
CURRENT TIME: 09:52:58

FILE DATE: 07-21-2000
FILE NAME: RITTEN

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ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ TAILWATER ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
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** REGULAR CHANNEL CROSS SECTION ****************

BOTTOM WIDTH	20.00 ft
SIDE SLOPE H/V (X:1)	1.0
CHANNEL SLOPE V/H (ft/ft)	0.005
MANNING'S n (.01-0.1)	0.013
CHANNEL INVERT ELEVATION	1576.00 ft
CULVERT NO.1 OUTLET INVERT ELEVATION	1576.00 ft

******* UNIFORM FLOW RATING CURVE FOR DOWNSTREAM CHANNEL

FLOW (cfs)	W.S.E. (ft)	FROUDE NUMBER	DEPTH (ft)	VEL. (f/s)	SHEAR (psf)
0.00	1576.00	0.000	0.00	0.00	0.00
90.00	1576.71	1.291	0.71	6.16	0.22
180.00	1577.07	1.358	1.07	7.97	0.33
270.00	1577.37	1.394	1.37	9.25	0.43
360.00	1577.62	1.417	1.62	10.25	0.51
450.00	1577.86	1.434	1.86	11.09	0.58
540.00	1578.07	1.447	2.07	11.82	0.65
630.00	1578.27	1.457	2.27	12.46	0.71
720.00	1578.46	1.466	2.46	13.04	0.77
810.00	1578.64	1.472	2.64	13.57	0.82
900.00	1578.81	1.478	2.81	14.05	0.88

ÄÄÄ
ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ ROADWAY OVERTOPPING DATA ÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄÄ
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ROADWAY SURFACE	PAVED
EMBANKMENT TOP WIDTH	14.00 ft
CREST LENGTH	14000.00 ft
OVERTOPPING CREST ELEVATION	1600.00 ft

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Kimley-Horn and Associates, Inc.

Job STRUCTURES Subject ALTS ANALYSIS Sheet No. ___ of ___ Job No. 091131003

Designed by ___ Date ___ Checked by ___ Date ___

QUANTITIES

POWERLINE 7-ft diameter RCP 2(210) = 410 LF
1- INLET STRUCTURE w/ GATE
1- OUTLET INTO CAP CANAL

VINEYARD 7-ft diameter RCP 2(350) = 700 LF
1- INLET STRUCTURE w/ GATE
1- OUTLET STRUCTURE TO CAP CANAL

RITTENHOUSE 7-ft diameter RCP 2(100) = 200 LF
1- Inlet structure w/ GATE
1- OUTLET STRUCTURE TO NEW FLOODWAY
1- FLOODWAY TRAP CHANNEL 20' BOTTOM (1:1.55)
3.5 ft deep Length = 310-ft concrete lined

A) NEED UNIT COST 7-ft diameter pipe (total length 1310 ft)

B) CONCRETE INLET STRUCTURE w/ GATE (3)

C) OUTLET INTO CANAL (2)

D) FLOODWAY: CONCRETE LINED

E) SIDE SPILL INLET INTO CANAL

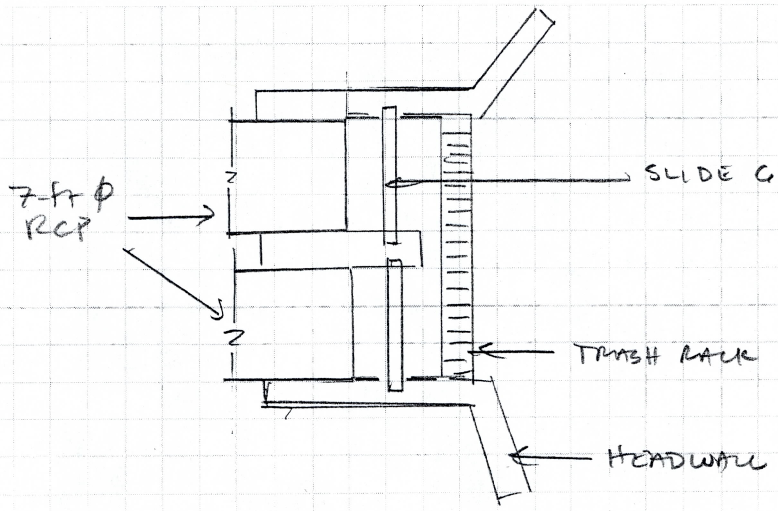
Kimley-Horn
and Associates, Inc.

Sheet No. ______ of ______

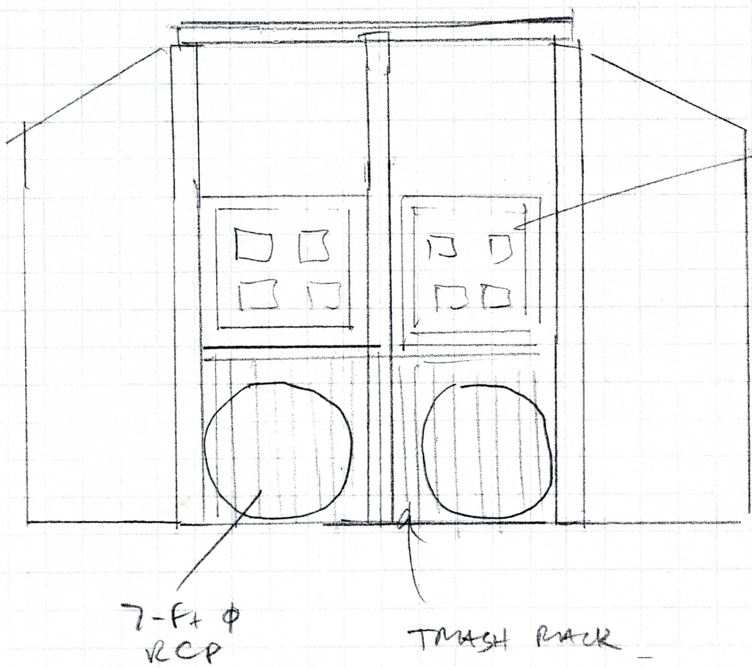
Job STRUCTURES Subject ALTB ANALYSIS Job No. 091131003

Designed by ________ Date ________ Checked by ________ Date ________

CONCEPTUAL
INLET STRUCTURE



PLAN VIEW



FRONT VIEW

Rittenhouse Floodway
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	Rittenhousefloo‹
Flow Element	Trapezoidal Cha
Method	Manning's Form‹
Solve For	Channel Depth

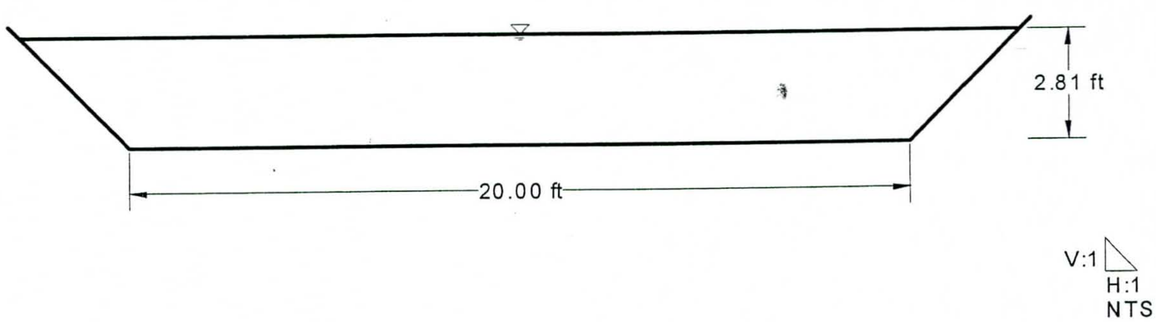
Input Data		
Mannings Coeffic	0.013	
Slope	005000	ft/ft
Left Side Slope	1.00	H : V
Right Side Slope	1.00	H : V
Bottom Width	20.00	ft
Discharge	900.00	cfs

Results		
Depth	2.81	ft
Flow Area	64.1	ft²
Wetted Perim‹	27.94	ft
Top Width	25.62	ft
Critical Depth	3.73	ft
Critical Slope	0.001922	ft/ft
Velocity	14.05	ft/s
Velocity Head	3.07	ft
Specific Ener‹	5.88	ft
Froude Numb‹	1.57	
Flow Type	Supercritical	

Cross Section
Cross Section for Trapezoidal Channel

Project Description	
Worksheet	Rittenhousefloo(
Flow Element	Trapezoidal Cha
Method	Manning's Formı
Solve For	Channel Depth

Section Data		
Mannings Coeffic	0.013	
Slope	005000	ft/ft
Depth	2.81	ft
Left Side Slope	1.00	H : V
Right Side Slope	1.00	H : V
Bottom Width	20.00	ft
Discharge	900.00	cfs



ALTERNATIVES ANALYSIS REPORT

Appendix K: Powerline, Vineyard Road, and Rittenhouse FRS
Structural Alternative No. 7:
Increase Capacity of Powerline Floodway

Opinion of Probable Construction Cost

FRS Structural Alternative No. 7
Increase Capacity of Powerline Floodway

Item	Description	Unit	Quantity	Unit Cost	Total
1	Concrete-lined rectangular channel. Bottom width = 52ft 5-ft depth. 4-in thick concrete Length 48,000 LF	SY	352,750	$ 25.00	$ 8,818,750.00
				Construction	$ 8,818,750.00

Land Cost		
Engineering and Construction Mgt	10%	$ 881,875.00
Operation and Maintenance	10%	$ 881,875.00
Subtotal		$ 10,582,500.00
Contingency	25%	$ 2,645,625.00
Total Costs		$ 13,228,125.00

Powerline Floodway
Worksheet for Trapezoidal Channel

Project Description	
Worksheet	Powerline Flood'
Flow Element	Trapezoidal Cha
Method	Manning's Form
Solve For	Discharge

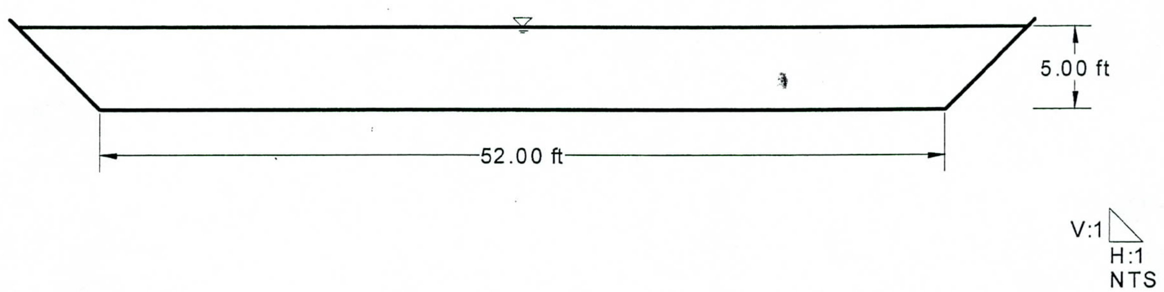
Input Data		
Mannings Coeffic	0.013	
Slope	005200	ft/ft
Depth	5.00	ft
Left Side Slope	1.00	H : V
Right Side Slope	1.00	H : V
Bottom Width	52.00	ft

Results		
Discharge	6,220.27	cfs
Flow Area	285.0	ft²
Wetted Perim	66.14	ft
Top Width	62.00	ft
Critical Depth	7.27	ft
Critical Slope	0.001483	ft/ft
Velocity	21.83	ft/s
Velocity Head	7.40	ft
Specific Energ	12.40	ft
Froude Numb	1.79	
Flow Type	Supercritical	

Cross Section
Cross Section for Trapezoidal Channel

Project Description	
Worksheet	Powerline Flood'
Flow Element	Trapezoidal Cha
Method	Manning's Formı
Solve For	Discharge

Section Data		
Mannings Coeffic	0.013	
Slope	005200	ft/ft
Depth	5.00	ft
Left Side Slope	1.00	H : V
Right Side Slope	1.00	H : V
Bottom Width	52.00	ft
Discharge	,220.27	cfs



Powerline Floodway #2
Worksheet for Rectangular Channel

Project Description	
Worksheet	Powerline Floodway Flatte
Flow Element	Rectangular Channel
Method	Manning's Formula
Solve For	Discharge

Input Data		
Mannings Coeffic	0.013	
Slope	003000	ft/ft
Depth	5.00	ft
Bottom Width	52.00	ft

Results		
Discharge	4,232.92	cfs
Flow Area	260.0	ft²
Wetted Perim	62.00	ft
Top Width	52.00	ft
Critical Depth	5.91	ft
Critical Slope	0.001790	ft/ft
Velocity	16.28	ft/s
Velocity Head	4.12	ft
Specific Energ	9.12	ft
Froude Numb	1.28	
Flow Type	Supercritical	

ALTERNATIVES ANALYSIS REPORT

Appendix L: Powerline, Vineyard Road, and Rittenhouse FRS Nonstructural Alternatives

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COVERAGE INFORMATION

Flood Insurance Coverage Available Limits Of Liability

Coverage Category	EmergencyProgram	Regular Program
BUILDING COVERAGE		
Single family dwelling	35,000	250,000
2-4 family dwelling	35,000	250,000
Other residential	100,000	250,000
Non-residential	100,000	500,000
CONTENTS COVERAGE		
Residential	10,000	100,000
Non-residential	100,000	500,000

What Does It Cover?

The Standard Flood Insurance Policy (SFIP) Forms contain complete definitions of the coverages they provide. Direct physical losses caused by "floods" are covered. Also covered are losses resulting from flood-related erosion caused by waves or currents of water activity exceeding anticipated cyclical levels, or caused by a severe storm, flash flood, abnormal tidal surge, or the like, which result in flooding, as defined. Damage caused by mudslides (i.e., mudflows), as specifically defined in the policy forms, is covered.

What Is Covered in My Basement?

The NFIP defines a basement as any area of a building with a floor that is below ground level on all sides. While flood insurance does not cover basement improvements, such as finished walls, floors or ceilings, or personal belongings that may be kept in a basement , such as furniture and other contents, it does cover structural elements, essential equipment and other basic items normally located in a basement. Many of these items are covered under building coverage, and some are covered under contents coverage. The NFIP encourages people to purchase both building and contents coverage for the broadest protection.

The following items are covered under building coverage, as long as they are connected to a power source and installed in their functioning location:

- Sump pumps.
- Well water tanks and pumps, cisterns and the water in them.
- Oil tanks and the oil in them, natural gas tanks and the gas in them.
- Pumps and/or tanks used in conjunction with solar energy.
- Furnaces, hot water heaters, air conditioners, and heat pumps.
- Electrical junction and circuit breaker boxes, and required utility connections.
- Foundation elements.
- Stairways, staircases, elevators and dumbwaiters.
- Unpainted drywall and sheet rock walls and ceilings, including fiberglass insulation.
- Cleanup.

The Following items are covered under contents coverage:

- Clothes washers.
- Clothes dryers.
- Food Freezers and the food in them.

What Is Increased Cost of Compliance (ICC) Coverage?

Increased Cost of Compliance (ICC) under the NFIP provides for the payment of a claim to help pay for the cost to comply with State or community floodplain management laws or ordinances from a flood event in which a building has been declared substantially damaged or repetitively damaged. When an insured building is damaged by a flood and the State or community declares the building to be substantially damaged or repetitively damaged, ICC will help pay for the cost to elevate, floodproof, demolish or relocate the building up to $20,000. This coverage is in addition to the building coverage for the repair of actual physical damages from flood under the Standard Flood Insurance Policy (SFIP).

When Will My Policy Go Into Effect?

There is a **30-day waiting period** before a flood insurance policy can become effective. In most instances, the insurance producer who writes your policy can provide you with the date that your policy should go into effect.

What Is A Flood?

Under the National Flood Insurance Program (NFIP) a flood is defined as **a general and temporary condition** of partial or complete inundation of normally dry land by:

- The overflow of inland or tidal waters.
- The unusual and rapid accumulation or runoff of surface waters from any source.
- Mudslides (i.e., mudflows) which are proximately caused by flooding, as defined above and are akin to a river of liquid and flowing mud on the surfaces of normally dry land areas, including your premises, as when earth is carried by a current of water and deposited along the path of the current.
- The collapse or subsidence of land along the shore of a lake or other body of water as a result of erosion or undermining caused by waves or currents of water exceeding the cyclical levels which result in flood as defined above.

To qualify as a **general and temporary condition**, the flood must affect either two or more adjacent properties or two or more acres of land and have a distinct beginning point and ending point.

Also, to qualify, the flood waters can only be surface water that covers land that is normally dry.

Updated: April 14, 2000

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Cost Information

A number of factors are considered in determining the premium for flood insurance coverage. They include:

- amount of coverage purchased
- location
- age of the building
- building occupancy
- design of the building
- for buildings in Special Flood Hazard Areas, elevation of the building.
- buildings eligible for special low-cost coverage at a pre-determined, reduced premium rate are single-family and 1-4 family dwellings located in zones B, C, & X. Ask your insurance agent if you're eligible for a Preferred Risk Policy.

- Average Cost & Coverage
- Cost Comparision
- Premium Examples for a $100,000 home
- Preferred Risk Policy Premiums

Updated: December 9,1998

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Cost and Coverage Data as of May 1, 2000

Occupancy Type	Regular Program	
	Coverage	Premium*
Single family	$124,300	$570
Two to four family	$101,700	$524
Other residential	$85,900	$665
Non-residential	$218,600	$1,514

** Premium values are based on Pre-FIRM Special Flood Hazard Area rates and includes Federal Policy Fee & Expense Constant.*

Updated: July 6, 2000

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Premium Examples For A $100,000 Single Family Home

If you own a home in a community that participates in the National Flood Insurance Program, you are eligible for flood insurance. More than 19,000 communities participate, so its likely that your community does participate.

There are many factors that affect the price you'll pay for flood insurance. The higher your flood risk, the higher the premium. If you purchase $100,000 in building coverage for your home, your annual premium will vary depending on the area in which you live.

- If the property is located near the ocean and therefore subject to storm surge and hurricane damage, your building is most likely in a V Zone. Premiums is V zones can be more than $1,000 annually because your home is in the highest risk area.
- If the property is located near a river, lake or stream, your building is probably in an A zone. Premiums in A zones can be about $595 annually because of the high potential for flooding.
- If the property is located in a low-risk area, referred to as B, C, X or A99 zones, your premium could be as low as $306 annually using standard rates. You may also be able to get the Preferred Risk Policy. Click here for premium rates for the PRP.

Below are annual premiums for $100,000 of flood insurance coverage for a residential single family home:

Pre or Post-FIRM	Zone	Other Rating Factors	Premium
Pre-FIRM***	Zone V1-30,VE	No Enclosure	$845.00****
		With Enclosure	$1,090.00
Post-FIRM***	Zone V1-30,VE	At BFE*	$ 850.00
	Built between 1975-1981	1 Foot below BFE	$ 2,180.00

Pre-FIRM	Zone A1-30, AE	No Basement	$ 595.00
		With Basement	$ 700.00
Post-FIRM	Zone A1-30, AE	At BFE	$ 431.00
		1 Foot above BFE	$ 301.00
		1 Foot below BFE	$ 1,251.00
Pre-FIRM	Zone AO, AH	With Certification**	$ 201.00
		Without Certification	$ 585.00
Pre/Post-FIRM	Zone B, C, X, A99	No Basement	$ 351.00
		With Basement	$ 441.00

**BFE-Base Flood Elevation found on Flood Insurance Rate Map*

***Certification is determined by an Elevation Certificate completed by a licensed engineer, surveyor or architect*

****Pre/Post FIRM is determined by the date of the initial Flood Insurance Rate Map*

*****Premium values are based on total written premium plus Expense Constant, Federal Policy Fee and Increased Cost of Compliance premium. Effective date: May 1, 2000*

Updated: July 6, 2000

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How to Purchase and How to Pay for Flood Insurance

To buy a flood insurance policy, call your insurance agent or contact one of the WriteYour Own companies, private insurance companies that write flood insurance under a special arrangement with the Federal government. If your agent does not write flood insurance or you don't have an agent, you may call the National Flood Insurance Program's (NFIP) toll free number to obtain the name of an agent in your area who does write flood insurance. The number is 1-888-CALL FLOOD, ext. 445. You can also check your local Yellow Pages directory.

It's a good idea to have the same agent who writes your homeowners or other insurance policies also write your flood insurance policy so in the event you need to file a claim, you only have to work with one insurance agency or company.

How can you pay for flood insurance?

In addition to paying the full annual premium by (cash, check or money order), you can now buy flood insurance with a credit card (Visa or MasterCard).

Another way flood insurance premiums can be paid is through an escrow account established by your mortgage lender. In fact, if your lender requires you to buy flood insurance and escrows for other types of insurance or taxes, the lender is required to also escrow flood insurance premium payments. Ask your insurance agent or lender for details.

Updated: October 6, 1998

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Why You Should Have a Preferred Risk Policy

- The Preferred Risk Policy can save you money!
- The Preferred Risk Policy ensures you financial protection against flood damage at a special low price for owners of homes not in high-risk flood areas.
- Flood damage is not covered under most homeowner's policies. To get coverage, you have to buy a separate policy.
- In the past 25 years, the NFIP has paid one-quarter of its claims to cover flood losses to those homes in moderate to minimal flood risk zones.
- The Preferred Risk Policy provides several coverage combinations for both the building and its contents that range from $20,000 building/$5,000 contents to $250,000 building/ $60,000 contents.
- People should consider this low-cost protection for their homes and contents because floods occur even in areas no one considers high-risk.
- When a flood occurs, there is no guarantee that it will be declared a Federal disaster and that you will qualify for Federal assistance.
- Disaster relief is often in the form of a low-interest loan that must be repaid. This adds to your total debt and may wipe out any equity that you have accumulated.
- As a condition for receiving disaster assistance, the homeowner must purchase and maintain a flood insurance policy for future protection.
- To be eligible for a Preferred Risk Policy, the building must be in a low-risk (B, C, or X) zone on the effective date of the current term.
- You can save about 30% of the standard application premium costs if you purchase a Preferred Risk Policy. Most people invest a major part of their income in a home. Protecting these assets from loss must be a concern.

"Life is not waterproof-Be flood alert."

For more information, call 1-888-CALL-FLOOD ext. 445,
TDD# 1-800-427-5593

F-437 (12/99)

Updated: June 30, 2000

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Preferred Risk Policy Premiums

If your single family home is located in a low–risk area, which is a B, C, or X zone on the current flood insurance rate map for your area, you may be eligible for the Preferred Risk Policy. This policy covers both your home and contents with one premium, which can be as little as $106 a year.

Preferred Risk Premiums

Building with a Basement

Coverage Amount	Contents	Premium
$ 20,000	$ 5,000	$131
$ 30,000	$ 8,000	$156
$ 50,000	$12,000	$196
$ 75,000	$18,000	$221
$100,000	$25,000	$246
$125,000	$30,000	$261
$150,000	$38,000	$276
$200,000	$50,000	$306
$250,000	$60,000	$326

Building without a Basement

Coverage Amount	Contents	Premium
$ 20,000	$ 5,000	$106
$ 30,000	$ 8,000	$131
$ 50,000	$12,000	$171
$ 75,000	$18,000	$196
$100,000	$25,000	$221
$125,000	$30,000	$236
$150,000	$38,000	$251
$200,000	$50,000	$281
$250,000	$60,000	$301

Building deductible $500 and Contents deductible $500 applied separately
Premium includes Federal Policy Fee and Increased Cost of Compliance premium
Effective date: June 1, 1998

Preferred Risk Policies (PRP) are only available for owners of 1-4 family residential buildings. Additionally should any of the following conditions apply to your home, based on its flood history regardless of ownership, a PRP cannot be written: *

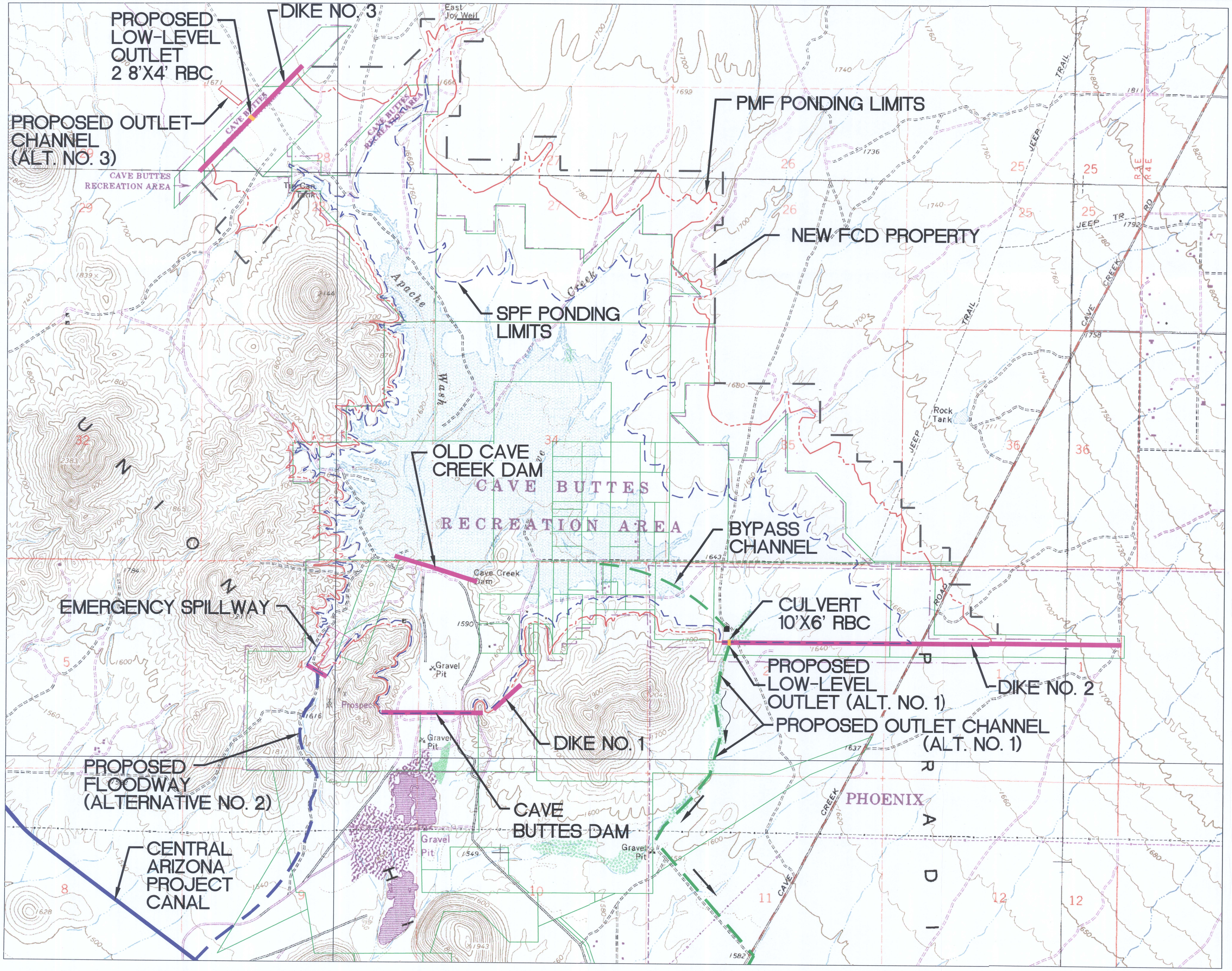
- 2 loss payments, each more than $1,000
- 3 or more loss payments, regardless of amount
- 2 Federal Disaster Relief payments, each more than $1,000
- 3 Federal Disaster Relief payments, regardless of amount
- 1 flood insurance claim payment and 1 flood disaster relief payment (including loans and grants), each more than $1,000

If your home is in a low-risk area, and one or more of the above conditions apply or you own a building other than a 1-4 family home that is located in a B, C, or X zone, you can still purchase flood insurance at the low-risk Standard Rates. For premium examples for $100,000 of coverage for a single-family home click here

*Contact your insurance agent for all the eligibility requirements for a PRP.

Updated: 10/6/1998

Federal Emergency Management Agency



FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

STRUCTURES ASSESSMENT PROGRAM
ALTERNATIVES ANALYSIS
CAVE BUTTES DAM

F.C.D. CONTRACT NO. FCD 98-41

PCN PLAN .01.00

LEGEND

- PONDING LIMITS PMF (1674.1 FT)
- PONDING LIMITS SPF (1657.1 FT)
- EXISTING FCD PROPERTY EASEMENTS
- NEW FCD PROPERTY (ALTERNATIVE NO. 8)
- FLOODWAY (EMERGENCY SPILLWAY TO CAP CANAL (ALTERNATIVE NO. 2)

REFERENCE: USGS QUAD MAPS

1. UNION HILLS, ARIZONA 7.5 MINUTE SERIES PHOTOREVISED 1981
2. NEW RIVER SE 7.5 MINUTE SERIES PHOTOREVISED 1981

NOTE: PONDING LIMITS AND FCD PROPERTY/EASEMENTS ARE APPROXIMATE AND PROVIDED FOR ILLUSTRATION PURPOSES ONLY.

NORTH

2000 FT 0 FT 2000 FT

SCALE: 1 INCH = 1000 FEET

CONTOUR INTERVAL = 20' FEET

Kimley-Horn and Associates Inc

	BY	DATE
DESIGN	JTA	6/2000
DESIGN CHK.	RAE	6/2000
PLANS	PAC	7/2000
PLANS CHK.	RAE	7/2000

SUBMITTED BY: DRAFT DATE:

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

RECOMMENDED BY: DATE

APPROVED BY: DATE

CHIEF ENGINEER AND GENERAL MANAGER

SHEET 1 OF 2

EXHIBIT A

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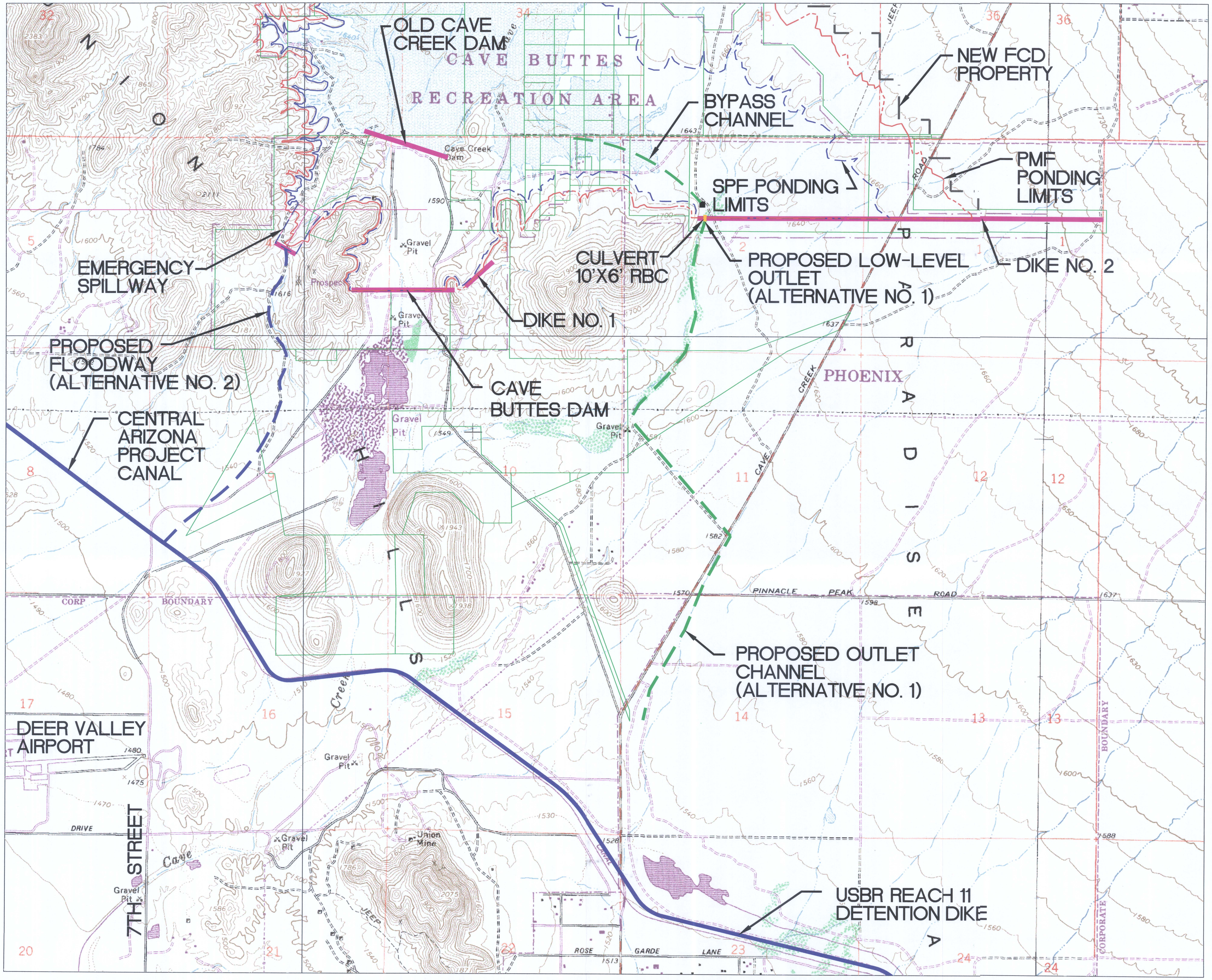

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

STRUCTURES ASSESSMENT PROGRAM
ALTERNATIVES ANALYSIS
CAVE BUTTES DAM
F.C.D. CONTRACT NO. FCD 98-41
PCN PLAN .01.00

LEGEND

PONDING LIMITS PMF (1674.1 FT)
PONDING LIMITS SPF (1657.1 FT)
EXISTING FCD PROPERTY/EASEMENT
NEW FCD PROPERTY (ALTERNATIVE NO. 8)
FLOODWAY (EMERGENCY SPILLWAY TO CAP CANAL) (ALTERNATIVE NO. 2)

REFERENCE: USGS QUAD MAPS

1. UNION HILLS, ARIZONA
   7.5 MINUTE SERIES
   PHOTOREVISED 1981

NOTE: PONDING LIMITS AND FCD PROPERTY/EASEMENTS ARE APPROXIMATE AND PROVIDED FOR ILLUSTRATION PURPOSES ONLY.

NORTH

2000 FT 0 FT 2000 FT

SCALE: 1 INCH = 1000 FT
CONTOUR INTERVAL = 20 FEET

Kimley-Horn and Associates Inc

	BY	DATE	
DESIGN	JTA	6/2000	FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
DESIGN CHK.	RAE	6/2000	RECOMMENDED BY: DATE
PLANS	PAC	7/2000	APPROVED BY: DATE
PLANS CHK.	RAE	7/2000	CHIEF ENGINEER AND GENERAL MANAGER
SUBMITTED BY: DRAFT	DATE:		SHEET 2 OF 2

EXHIBIT B

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FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

STRUCTURES ASSESSMENT PROGRAM ALTERNATIVES ANALYSIS POWERLINE, VINEYARD ROAD, AND RITTENHOUSE FLOOD RETARDING STRUCTURES

F.C.D. CONTRACT NO. FCD 98-41
PCN PLAN .01.00

REFERENCE: USGS QUAD MAPS

1. SUPERSTITION MTS. SW, ARIZONA 7.5 MINUTE SERIES PHOTOREVISED 1981
2. APACHE JUNCTION, ARIZONA 7.5 MINUTE SERIES PHOTOREVISED 1982
3. DESERT WELL, ARIZONA 7.5 MINUTE SERIES PHOTOREVISED 1981

EXISTING FEATURES

- FLOOD RETARDING STRUCTURE (FRS)
- CENTRAL ARIZONA PROJECT CANAL
- PRINCIPAL SPILLWAY
- EMERGENCY SPILLWAY
- RESERVOIR PONDING LIMITS (AS-BUILT PLAN)

NO. ALTERNATIVES

1. SEGMENTATION
2. UPGRADE TO HIGH HAZARD DAM (SEE REPORT)
3. MODIFICATIONS TO IMPROVE PERFORMANCE (SEE REPORT)
4. DETENTION BASINS
5. LEVEE/FLOODWAY SYSTEM
6. DISCHARGE TO CAP CANAL
7. UPSIZE POWERLINE FLOODWAY (SEE REPORT)

NORTH

2000 FT 0 FT 2000 FT

SCALE: 1 INCH = 1000 FT
CONTOUR INTERVAL = 20 FEET

Kimley-Horn and Associates Inc

	BY	DATE
DESIGN	RAE	1/2001
DESIGN CHK.	WJM	1/2001
PLANS	WJM	1/2001
PLANS CHK.	RAE	1/2001

SUBMITTED BY: DRAFT DATE:

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

RECOMMENDED BY: DATE

APPROVED BY: DATE

CHIEF ENGINEER AND GENERAL MANAGER

SHEET 1 OF 1

EXHIBIT C

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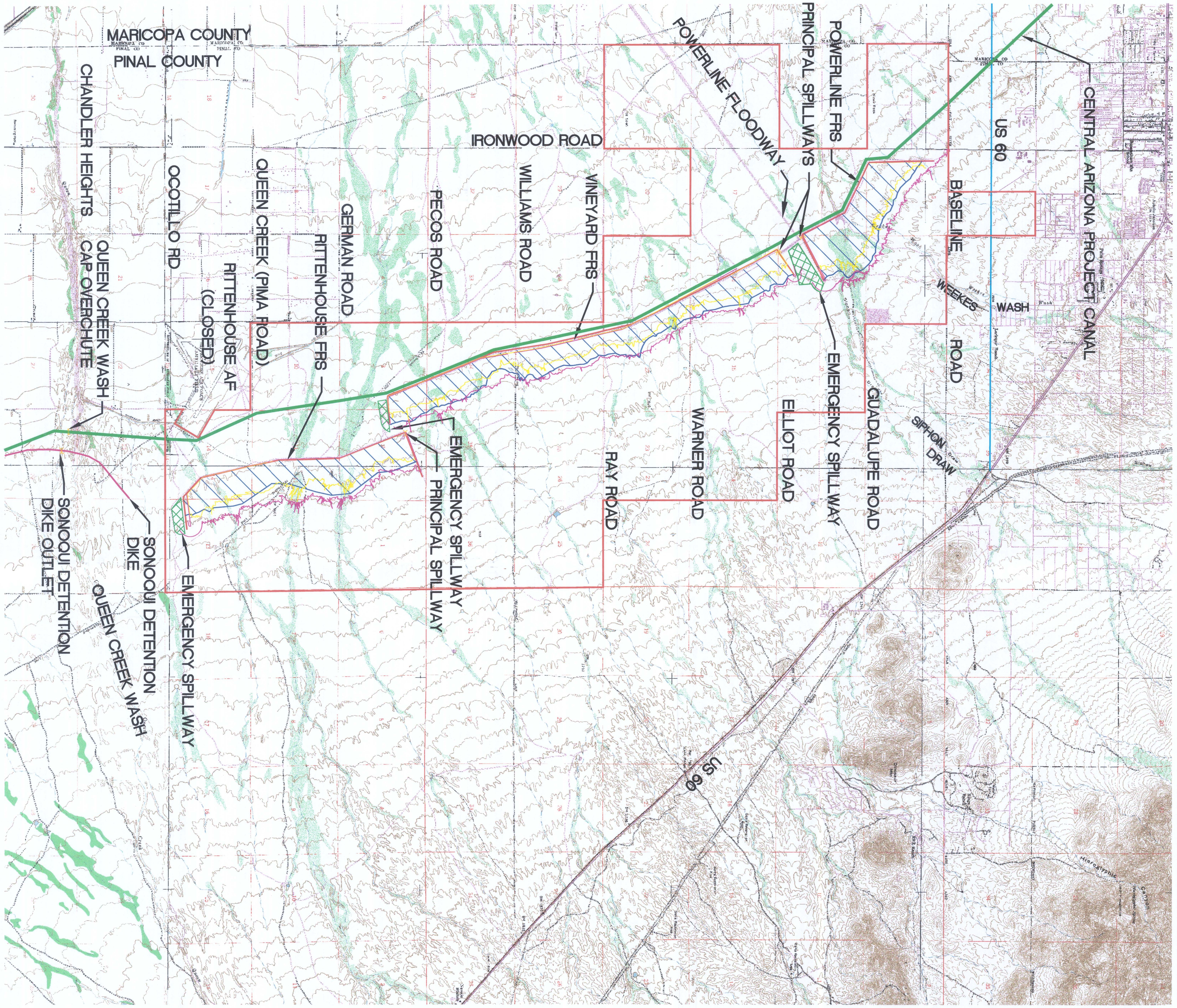

REFERENCE: USGS QUAD MAPS

1. SUPERSTITION MTS. SW, ARIZONA
7.5 MINUTE SERIES
PHOTOREVISED 1981
2. APACHE JUNCTION, ARIZONA
7.5 MINUTE SERIES
PHOTOREVISED 1982
3. DESERT WELL, ARIZONA
7.5 MINUTE SERIES
PHOTOREVISED 1981

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

STRUCTURES ASSESSMENT PROGRAM ALTERNATIVES ANALYSIS POWERLINE, VINEYARD ROAD, AND RITTENHOUSE FLOOD RETARDING STRUCTURES

F.C.D. CONTRACT NO. FCD 98-41

PCN PLAN .01.00

EXISTING FEATURES

- FLOOD RETARDING STRUCTURE (FRS)
- CENTRAL ARIZONA PROJECT CANAL
- PRINCIPAL SPILLWAY
- EMERGENCY SPILLWAY
- RESERVOIR PONDING LIMITS (AS-BUILT PLAN)
- STATE LAND DEPARTMENT LEASE NO 09-3681 19,072 ACRES
- 100-YR PONDING LIMITS (FCD)
- PROBABLE MAXIMUM FLOOD PONDING LIMITS (FCD)

NOTE:

PONDING LIMITS ARE DERIVED FROM AS-BUILT CONSTRUCTION PLANS FOR POWERLINE, VINEYARD ROAD, AND RITTENHOUSE FRS. AND FROM H.I.S. LAYERS PROVIDED BY THE DISTRICT FOR EACH STRUCTURE FOR THE 100-YEAR AND PMF PONDING LIMITS. THE 100-YEAR AND PMF PONDING ELEVATIONS WERE PROVIDED TO THE DISTRICT BY KHA WHICH WERE OBTAINED FROM PREVIOUS FLOOD CONTROL DISTRICT STUDIES.

NORTH

2000 FT 0 FT 2000 FT

SCALE: 1 INCH = 1000 FT

CONTOUR INTERVAL = 20 FEET

Kimley-Horn and Associates Inc

	BY	DATE
DESIGN	RAE	1/2001
DESIGN CHK.	WJM	1/2001
PLANS	WJM	1/2001
PLANS CHK.	RAE	1/2001

SUBMITTED BY: DRAFT DATE:

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

RECOMMENDED BY: DATE

APPROVED BY: DATE

CHIEF ENGINEER AND GENERAL MANAGER

SHEET 1 OF 1

EXHIBIT D

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