July 2000

Los Alamos
NATIONAL LABORATORY

Los Alamos National Laboratory
Los Alamos, New Mexico 87545

Date: July 18, 2000
In Reply Refer To: ESH-18/WQ&H:00-0237
Mail Stop: K497
Telephone: (505) 665-1859

LIBRARY COPY

Ms. Sandy Spon
Surface Water Quality Bureau
N.M. Environment Department
P.O. Box 26110
Santa Fe, NM 87502

Mr. Jim Wood
Regulatory Branch
Albuquerque District Corps of Engineers
4191 Jefferson Plaza NE
Albuquerque, NM 87109

SUBJECT: 404/401 APPLICATION FOR EMERGENCY CONTROL MEASURES TO REDUCE POTENTIAL FLOODING AND SOIL EROSION ON LANL PROPERTY DUE TO THE CERRO GRANDE WILDFIRE

HSWA LANL 9/P/00

Dear Ms. Spon and Mr. Wood:

On June 8, 2000 a copy of the joint 404/401 application and supplemental information for the Los Alamos National Laboratory's "Emergency Control Measures To Reduce The Potential For Flooding And Soil Erosion On LANL Property Due To The Cerro Grande Wildfire Project" was provided to your offices. The Laboratory's Emergency Rehabilitation Team, in cooperation with the United States Forest Service's Burned Area Emergency Rehabilitation (BAER) Team, made recommendations to reduce the potential for flooding and erosion expected with the start of the summer monsoon season. This project is necessary to control sediment transport from storm events, to help reduce flooding, and to reduce further fire threats. A permit application was submitted based on our conversations with the New Mexico Environment Department's Surface Water Quality Bureau and the U.S. Army Corps of Engineer (COE) representatives the week of May 23, 2000.

On June 23, 2000 Andrew Rosenau, COE, assigned Action No. 2000-00420 to this activity and authorized the work under Nationwide Permit (NWP) No. 37. Additionally, the New Mexico Environment Department's Surface Water Quality Bureau conditionally certified the Laboratory's activities under NWP No. 37 pursuant to Section 401 of the Clean Water Act, on the same day. On July 7, 2000 the Laboratory submitted a modification to the June 8, 2000 application. This application documented activities recommended by the BAER Team, and the major earth moving activities (Low-head weirs, flood retention structures, etc.) being conducted by the COE. The COE activities are being conducted according to NWP Nos. 3 and 18. Based on our conversations on July 17, 2000 regarding the July 7, 2000 application, I am providing a separate application for the Pajarito Canyon Flood Control Structure Project for your review (Enclosure 1). Enclosure 2 provides the details on the Pajarito flood retarding structure. It is my understanding that additional permit reviews and certifications may be required based on the July 7, 2000 application.

13413

TR

If you have any questions regarding this matter or need additional information, please contact Marc Bailey at (505) 665-8135 or Mike Saladen at (505) 665-6085.

Sincerely,

Mike Saladen for

Steven Rae
Group Leader
Water Quality and Hydrology Group

SR:MS/tml

Enclosures: a/s

Cy: A. Rosenau, USCOE, Albuquerque, New Mexico, w/enc.
J. Manger, USCOE, Albuquerque, New Mexico, w/enc.
J. Kieling, NMED-HRMB, Santa Fe, New Mexico, w/enc.
J. Vozella, DOE-LAAO, w/o enc., MS A316
M. Johansen, DOE-LAAO, w/enc., MS A316
T. Gunderson, DLDOPS, w/enc., MS A100
R. Burick, DLDOPS, w/o enc., MS A100
J. Covey, FWO-DO, w/enc., MS K492
T. George, FWO-DO, w/enc., MS K492
M. Alexander, ESH-18, w/enc., MS K497
M. Bailey, ESH-18, w/enc., MS K497
M. Saladen, ESH-18, w/o enc., MS K497
K. Mullen, ESH-18, w/o enc., MS K497
D. Nochumson, ESH-18, w/o enc., MS K497
D. Woitte, LC-GL, w/enc., MS A187
WQ&H File, w/enc., MS K497
CIC-10, w/enc., MS A150

JUL 2000
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ENCLOSURE 1

JOINT APPLICATION FOR DEPARTMENT OF THE ARMY PERMIT AND NM WATER QUALITY CERTIFICATION
(33 CFR 325) (33 CFR 325.2.6)

OMB APPROVAL NO. 0710-003
Expires October 1996

Public reporting burden for this collection of information is estimated to average 5 hours per response, including the time for reviewing instructions, searching existing data sources, gathering and maintaining the data needed, and completing and reviewing the collection of information. Send comments regarding this burden estimate or any other aspect of this collection of information, including suggestions for reducing this burden, to Department of Defense, Washington Headquarters Service Directorate of Information Operations and Reports, 1215 Jefferson Davis Highway, Suite 1204, Arlington, VA 22202-4302; and to the Office of Management and Budget, Paperwork Reduction Project (-710-0003), Washington, DC 20503. Please DO NOT RETURN your form to either of those addresses. Completed applications must be submitted to the District Engineer having jurisdiction over the location of the proposed activity.

PRIVACY ACT STATEMENT

Authority: 33 USC 401, Section 10; 1413, Section 404, Principle Purpose: These laws require permits authorizing activities in, or affecting, navigable waters of the United States, the discharge of dredged or fill material into Waters of the United States, and the transportation of dredged material for the purpose of dumping it into ocean waters. Routing Uses: Information provided on this form will be used in evaluating the application for a permit. Disclosure: Disclosure of requested information is voluntary. If information is not provided, however, the permit application cannot be processed nor can a permit be issued.

One set of original drawings or good reproducible copies which show the location and character of the proposed activity must be attached to this application (see sample drawings and instructions) and be submitted to the District Engineer having jurisdiction over the location of the proposed activity. An application that is not completed in full will be returned.

(ITEMS 1 THRU 4 TO BE FILLED BY THE CORPS)

1. APPLICATION NO.	2. FIELD OFFICE CODE	3. DATE RECEIVED	4. DATE APPLICATION COMPLETED

(ITEMS BELOW TO BE FILLED BY APPLICANT)

5. APPLICANT'S NAME Los Alamos National Laboratory	8. AUTHORIZED AGENT'S NAME AND TITLE (an agent is not required) N/A
6. APPLICANT'S ADDRESS MS K497, P.O. Box 1663, Los Alamos, NM 87545	9. AGENT'S ADDRESS N/A
7. APPLICANT'S PHONE NOS. W/AREA CODE a. Residence N/A b. Business (505) 665-8135	10. AGENT'S PHONE NOS. W/AREA CODE a. Residence N/A b. Business N/A

11. **STATEMENT OF AUTHORIZATION**

I hereby authorize, _____ To act in my behalf as my agent in the processing of this application and to furnish, upon request, supplemental information in support of this permit application.

N/A

APPLICANT'S SIGNATURE DATE

NAME, LOCATION, AND DESCRIPTION OF PROJECT OR ACTIVITY

12. PROJECT NAME OR TITLE (see instructions)
Pajarito Canyon Flood Control Structure Project: Flood Retarding Structure (FRS).

13. NAME OF WATERBODY, IF KNOWN (if applicable) Pajarito Canyon	14. PROJECT STREET ADDRESS (if applicable) N/A
15. LOCATION OF PROJECT Los Alamos (COUNTY) NM (STATE)	

16. OTHER LOCATION DESCRIPTIONS, IF KNOWN (see instructions)
The FRS is located in Pajarito Canyon, immediately downstream of the confluence of Twomile & Pajarito Canyons.See map 6/23/00.

17. DIRECTIONS TO THE SITE
Access to the FRS is approximately 400 feet downstream of the confluence of the two canyons. See attached draft design report.

18. Nature of Activity (Description of project, include all features)
Construct FRS to temporarily contain runoff from storms up to the 100-year flood plus sediment volume from the 100-year flood. The FRS should help slow flow velocity in the canyon, drop sediment, and reduce flooding downstream. The FRS was sited to provide (temporary) efficient storage volume while minimizing dam height and providing watershed control for two drainage basins upstream of Technical Area 18 (LANL) and Whiterock.

19. Project Purpose (Describe the reason or purpose of the project, see instructions)
Emergency work is needed to reduce the potential for flooding, erosion, and sediment transport expected with the monsoon season.

USE BLOCKS 20-23 IF DREDGED AND/OR FILL MATERIAL IS TO BE DISCHARGED

20. Project Schedule
Start Date June 2000 End Date June 2002

21. Reason(s) for Discharge

The FRS is designed to temporarily contain the runoff and sediment volume of a 100-year storm. Storms up to this frequency will be retained and slowly released through the outlet works to assist in minimizing flooding downstream of the FRS.

22. Type(s) of Material Being Discharged and the Amount of Each Type in Cubic Yards

See attached Draft Design Report: "Pajarito Canyon Flood Control Structure" prepared for the U.S. Army Corps of Engineers, dated July 11, 2000.

23. Surface Area in Acres of Wetlands or Other Waters Filled (see instructions)

No filling of wetlands is anticipated at this time

24. Is Any Portion of the Work Already Complete? Yes X No _____ IF YES, DESCRIBE THE COMPLETED WORK

ESH-ID Process/Excavation Permit review request completed June 28, 2000. Preliminary design work underway (75% plans and specifications). Batch plant 90% assembled on Pajarito Mesa; RCC transportation system 40% complete on first bench.

25. Addresses of Adjoining Property Owners, Lessees, Etc., Whose Property Adjoins the Waterbody (if more than can be entered here, please attach a supplemental list).

Bandelier National Monument boundary is due south.
Santa Fe National Forest boundary is due west
Los Alamos County boundary is approximately due north.
San Ildefonso Indian Reservation boundary is east.

26. Describe any adverse water quality impacts that may result from the proposed activity such as increased turbidity or erosion. How long will such impacts occur?

Work is conducted in ephemeral portion of Pajarito Canyon. Erosion control work may likely add to the already increased turbidity.

27. Describe methods to be used to prevent water quality impacts which could interfere with attainment of State designated fishery, recreation, irrigation, water supply or other uses.

All the Cerro Grande Wildfire related work is being conducted to prevent flooding and to control erosion. Extensive use of BMPs is taking place.

28. List of Other Certifications or Approvals/Denials Received from other Federal, State or Local Agencies for Work Described in This Application.

AGENCY	TYPE APPROVAL*	IDENTIFICATION NUMBER	DATE APPLIED	DATE APPROVED	DATE DENIED
N/A					

*Would include but is not restricted to zoning, building and flood plain permits.

29. Application is hereby made for a permit or permits to authorize the work described in this application. I certify that the information in this application is complete and accurate. I further certify that I possess the authority to udertake the work described herein or am acting as the duly authorized agent of the applicant.

SIGNATURE OF APPLICANT	DATE	SIGNATURE OF AGENT	DATE
[illegible] for Steven Rae	7/17/00		

The application must be signed by the person who desires to undertake the proposed activity (applicant) or it may be signed by a duly authorized agent if the statement in block 11 has been filled out and signed.

18 U.S.C. Section 1001 provides that: Whoever, in any manner within the jurisdiction of any department or agency of the United States knowingly and willfully falsifies, conceals, or covers up any trick, scheme, or disguises a material fact or makes any false, fictitious or fraudulent statements or representations or makes or uses any false writing or document knowing same to contain any false, fictitious or fraudulent statements or entry, shall be fined not more than $10,000 or imprisoned not more than five years or both.

U.S.GPO: 1994-520-478/83018

ENCLOSURE 2

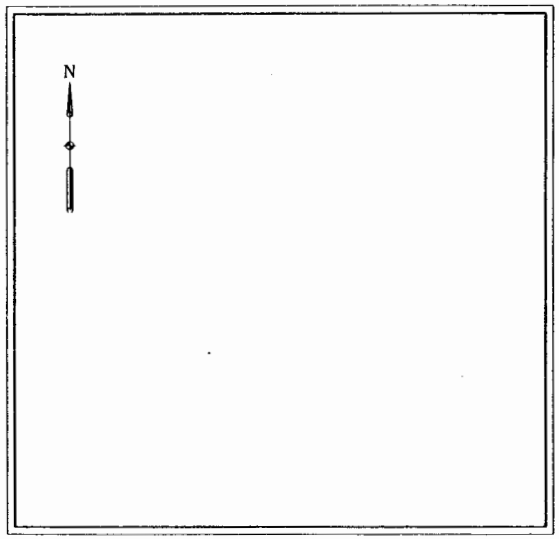
CONSTRUCTION DRAWINGS

FOR

PAJARITO CANYON RETAINING STRUCTURE
LOS ALAMOS, NEW MEXICO

PREPARED BY

URS Greiner Woodward Clyde



VICINITY MAP

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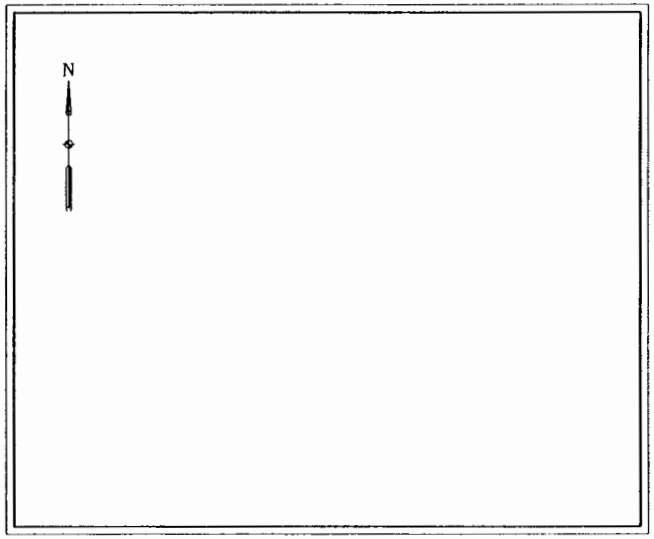

SITE PLAN

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INDEX OF DRAWINGS

DRAWING	DRAWING TITLE
1 OF 16	LOCATION PLAN AND LIST OF DRAWINGS
2 OF 16	GENERAL SITE PLAN
3 OF 16	EXCAVATION PLAN
4 OF 16	TYPICAL CROSS SECTION
5 OF 16	PROFILE OF DAM AND CONTRACTION JOINT DETAILS
6 OF 16	FOUNDATION TREATMENT DETAILS
7 OF 16	FOUNDATION EXCAVATION TREATMENT SECTION AND TYPICAL DETAILS
8 OF 16	PLACEMENT DETAILS
9 OF 16	RETAINING STRUCTURE CROSS SECTIONS
10 OF 16	RETAINING STRUCTURE CROSS SECTIONS
11 OF 16	RETAINING STRUCTURE ~~CROSS SECTIONS~~
~~12 OF 16~~	~~FINAL GRADING PLAN~~ Not Available
13 OF 16	SECTIONS AND DETAILS
14 OF 16	STILLING BASIN– SECTIONS AND DETAILS
~~15 OF 16~~	~~OUTLET WORKS – PLAN, SECTIONS AND DETAILS~~ Not Available
16 OF 16	INLET STRUCTURE TRASHRACK – PLAN AND SECTIONS

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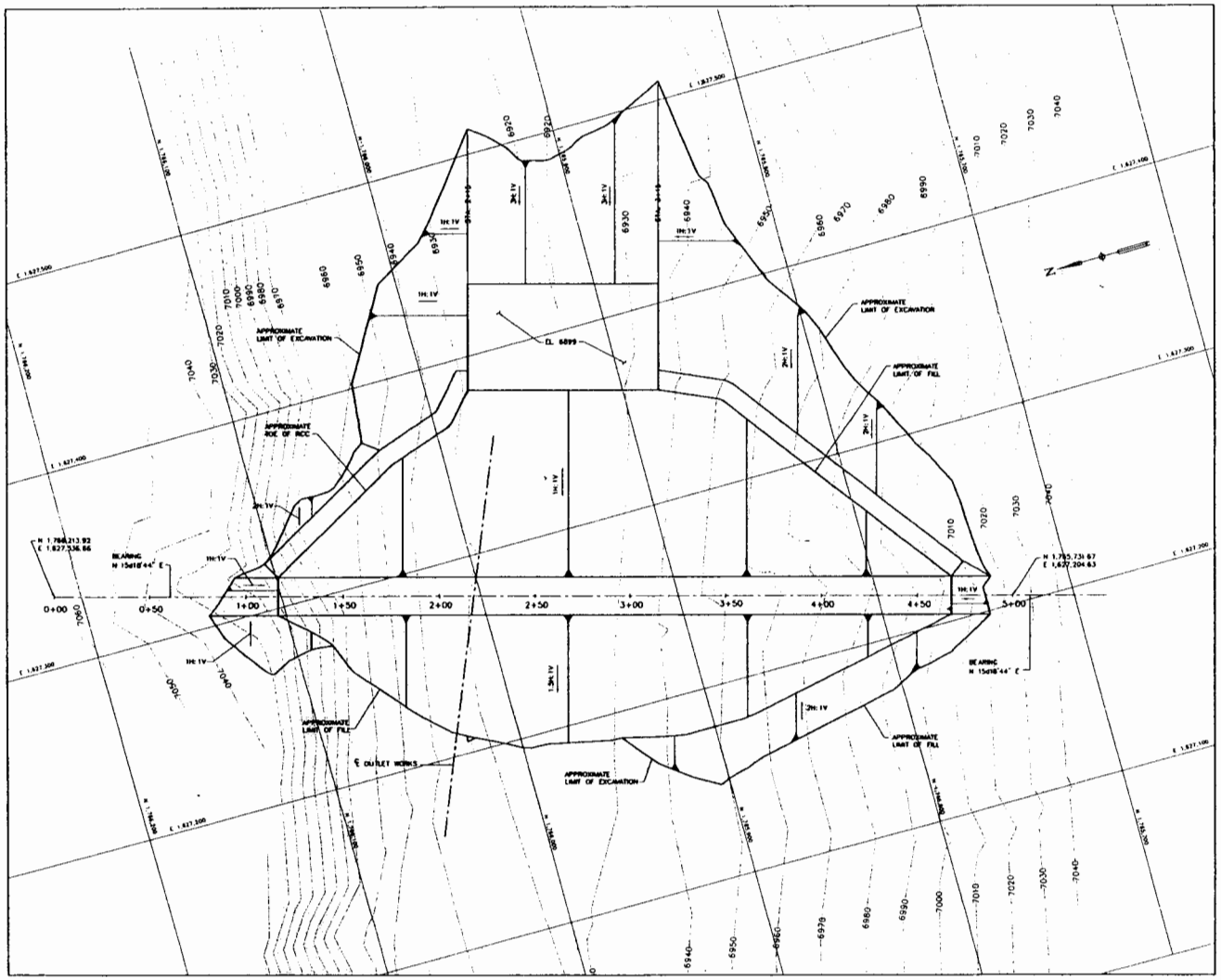
CHECKED: TEA
PEER REVIEWED: –
PROJECT MANAGER: CAT

PAJARITO CANYON LOS ALAMOS, NEW MEXICO

LOCATION PLAN AND LIST OF DRAWINGS

REVISION A
PROJECT E30001579.03
DRAWING 1



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PAJARITO CANYON — LOS ALAMOS, NEW MEXICO

GENERAL SITE PLAN

REVISION: A
PROJECT: E30001579.03
DRAWING: 2

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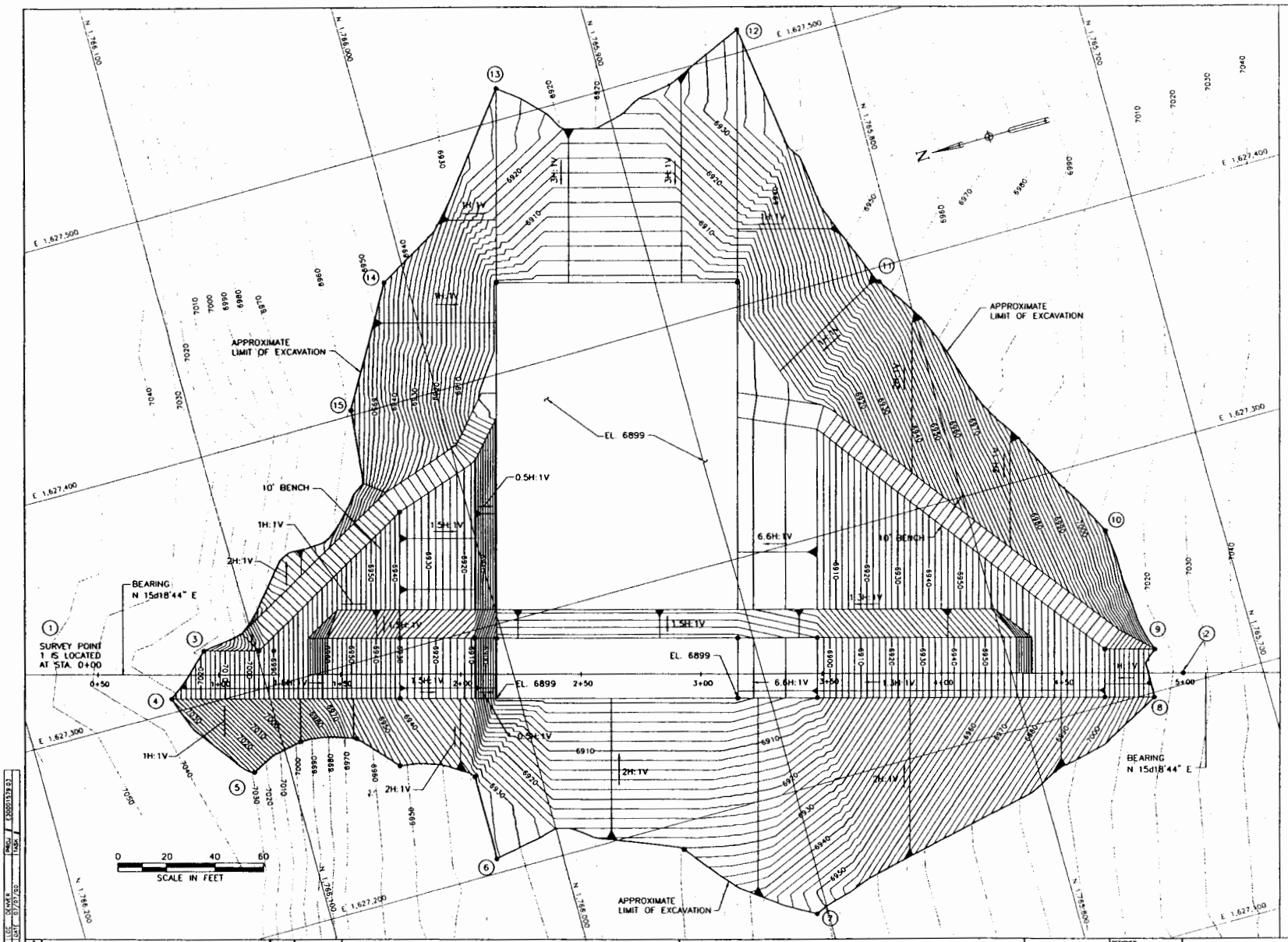

EXCAVATION CONTROL POINT TABLE

POINT NO.	NORTHING	EASTING
1	N 1766213.92	E 1627336.66
2	N 1765731.67	E 1627204.63
3	N 1766120.72	E 1627321.51
4	N 1766138.83	E 1627305.72
5	N 1766114.28	E 1627267.01
6	N 1766026.83	E 1627205.84
7	N 1765904.56	E 1627148.78
8	N 1765745.90	E 1627198.15
9	N 1765740.36	E 1627217.37
10	N 1765746.82	E 1627270.64
11	N 1765810.41	E 1627394.22
12	N 1765840.04	E 1627509.37
13	N 1765942.87	E 1627512.50
14	N 1766009.15	E 1627448.56
15	N 1766035.82	E 1627401.95

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PAJARITO CANYON — LOS ALAMOS, NEW MEXICO

EXCAVATION PLAN

REVISION: A
PROJECT: E30001579.03
DRAWING: 3

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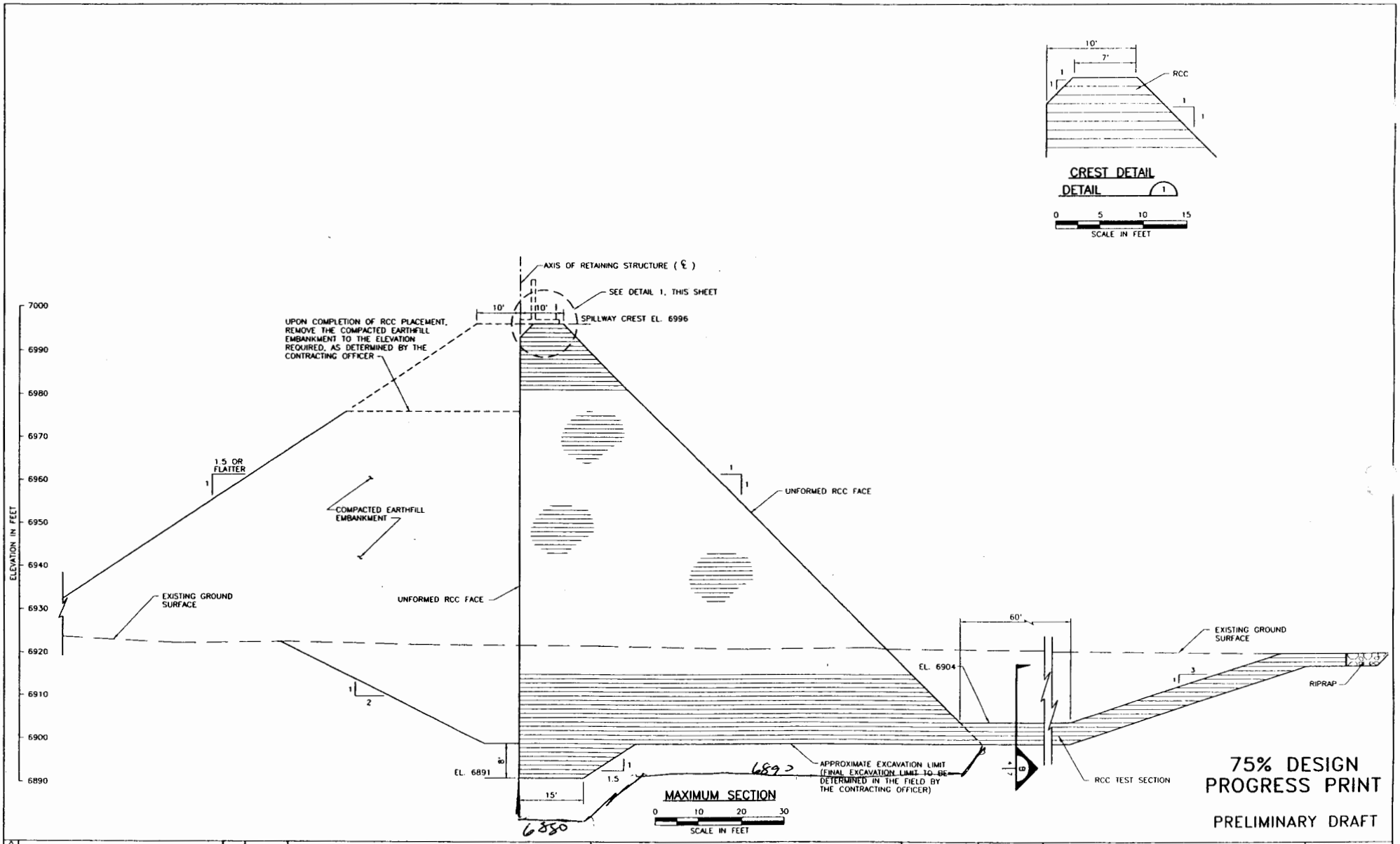
DATE | 07/07/00 | TASK



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TYPICAL CROSS SECTION

REVISION A
PROJECT E300015790 3
DRAWING 4

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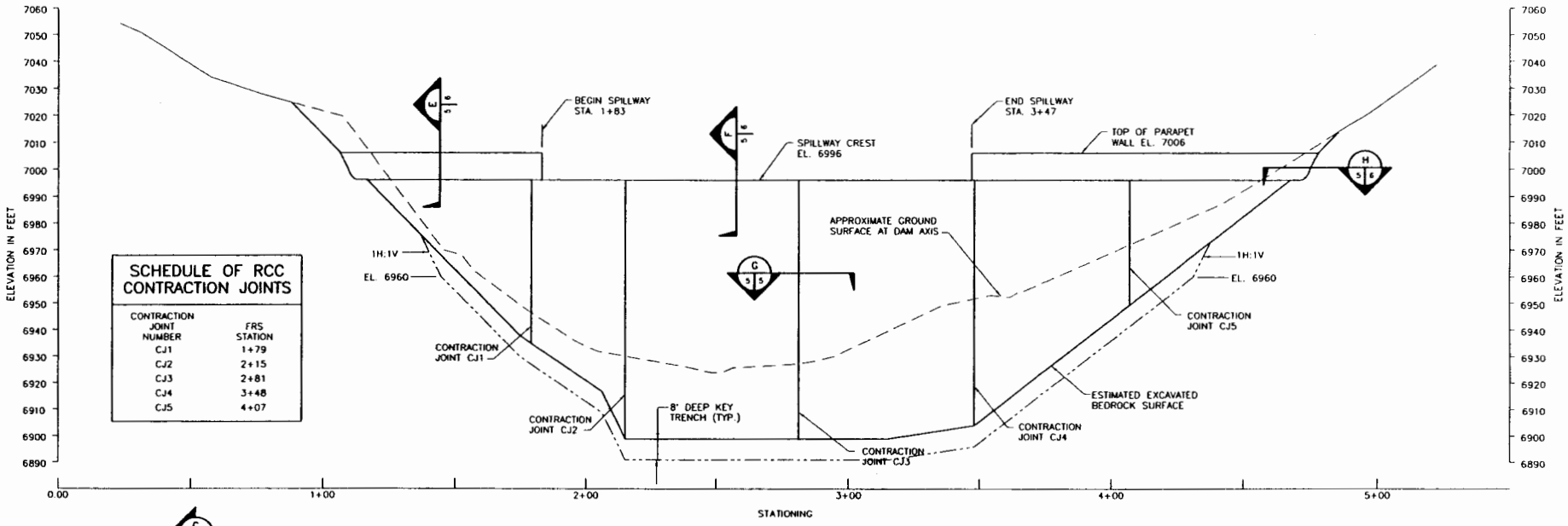
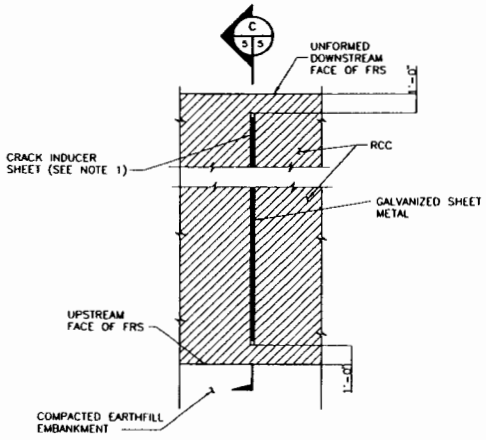


SCHEDULE OF RCC CONTRACTION JOINTS

CONTRACTION JOINT NUMBER	FRS STATION
CJ1	1+79
CJ2	2+15
CJ3	2+81
CJ4	3+48
CJ5	4+07

PROFILE ALONG DAM AXIS



SECTION G/5|5
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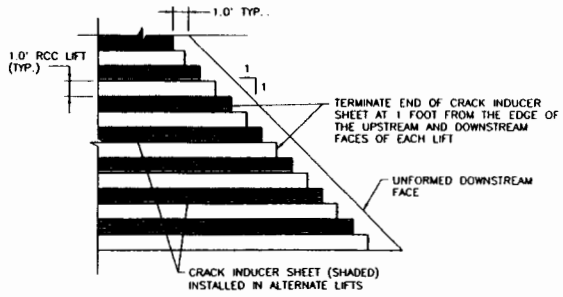
RCC CONTRACTION JOINT DETAIL



SECTION C/5|5
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NOTES:

1. RCC CONTRACTION JOINTS SHALL BE CONSTRUCTED BY PLACING CRACK INDUCER SHEETS IN A VERTICAL PLANE IN THE RCC STRUCTURE, FROM THE BASE OF THE STRUCTURE TO THE CREST.
2. CONTRACTION JOINT LOCATIONS ARE APPROXIMATE. FINAL CONTRACTION JOINT LOCATIONS SHOULD BE DETERMINED BY THE CONTRACTING OFFICER IN THE FIELD AFTER COMPLETION OF THE FOUNDATION EXCAVATION.
3. CONTRACTION JOINT CJ1 SHALL BE POSITIONED AT THE LOCATION OF THE OUTLET STRUCTURE.
4. TWO ADDITIONAL CONTRACTION JOINTS MAY BE ADDED AT THE WELDED TUFF/ UNWELDED TUFF INTERFACE.

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PROFILE OF STRUCTURE AND SECTIONS

REVISION: A
PROJECT: E30001579.03
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PAJARITO-25 | LOC: DENVER | PROJ: E30001579.03
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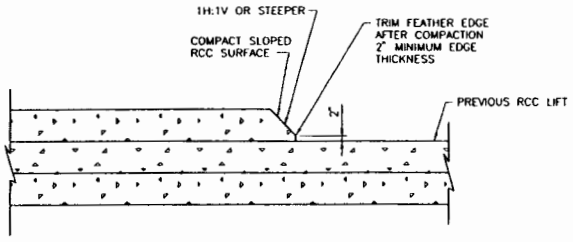

TYPICAL EDGE JOINT TREATMENT

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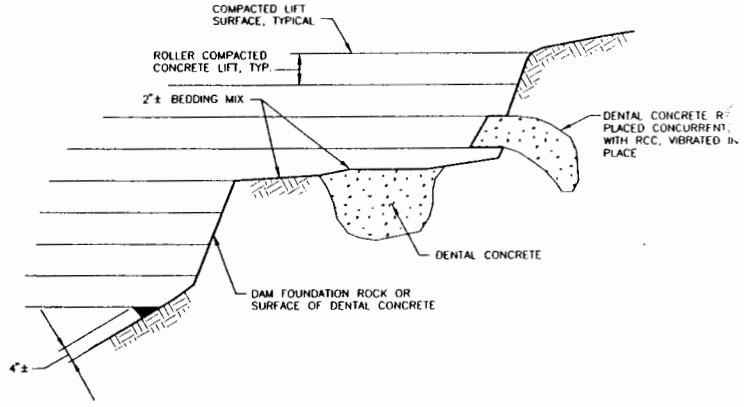

FOUNDATION PREPARATION FOR ABUTMENTS

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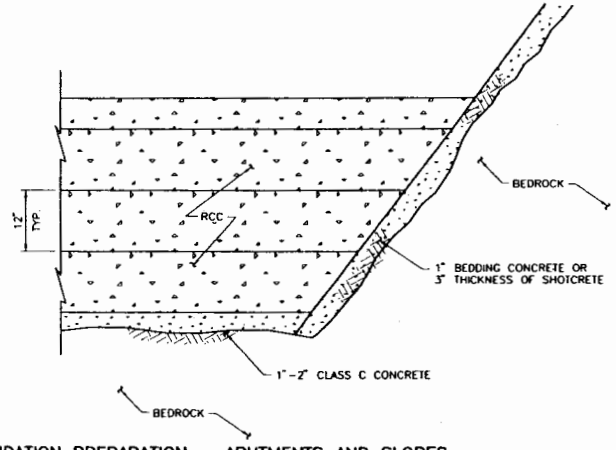

FOUNDATION PREPARATION – ABUTMENTS AND SLOPES

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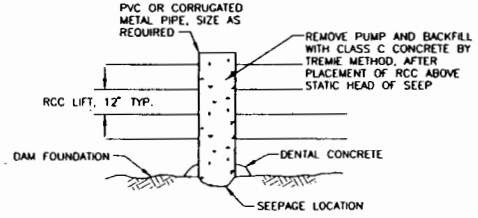

DEWATERING SUMP FOR ISOLATED SEEPS IN DAM FOUNDATION

NOT TO SCALE

FOUNDATION TREATMENT PROCEDURES ON ROCK SURFACES

1. ABUTMENTS WILL BE PREPARED BY A COMBINATION OF ROCK EXCAVATION, GROUT ENRICHED RCC, SHOTCRETE, AND DENTAL CONCRETE TO A SMOOTHNESS AND UNIFORMITY GENERALLY SUITABLE FOR PLACEMENT OF ROLLER COMPACTED CONCRETE.
2. ALL NEAR HORIZONTAL OR SLIGHTLY SLOPING ROCK SURFACES (INCLUDING THE ENTIRE STARTER SECTION IN THE VALLEY BOTTOM) SHALL BE PREPARED BY A COMBINATION OF ROCK EXCAVATION AND DENTAL CONCRETE TO A SURFACE ROUGHNESS SUITABLE FOR RCC PLACEMENT.
3. FOUNDATION TREATMENT METHODS WILL BE IN ACCORDANCE WITH DETAILS SHOWN ON THIS SHEET.
4. TREATMENT OF ROCK FRACTURES SHALL BE COMPLETED IN ADVANCE OF DENTAL CONCRETE PLACEMENT.
5. SEE SPECIFICATIONS FOR REQUIREMENTS FOR FOUNDATION PREPARATION.
6. GROUT ENRICHED RCC, SHOTCRETE, AND DENTAL CONCRETE SHALL BE PLACED IN THE FOUNDATION ONLY AT LOCATIONS DIRECTED BY THE ENGINEER.
7. DENTAL CONCRETE SLOPES STEEPER THAN 1V:1.5H SHALL BE FORMED.
8. FRACTURES IN FOUNDATION ROCK SMALLER THAN 1/4" WIDE SHALL BE CLEANED PRIOR TO PLACING BEDDING MIX.
9. MULTIPLE STARTER SECTIONS MAY BE NECESSARY DEPENDING ON FOUNDATION CONDITIONS.

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1	PAJARITO-05	LOC	DENVER
		DATE	07/27/00
		PROJ.	E30001579.03
		TASK	

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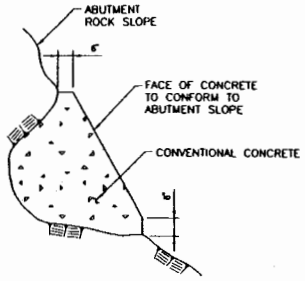
PROJECT MANAGER:	PAK

PAJARITO CANYON — LOS ALAMOS, NEW MEXICO

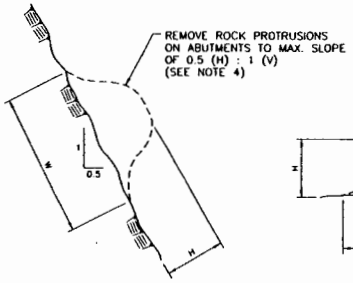
FOUNDATION TREATMENT DETAILS

REVISION: A
PROJECT: E30001579.03
DRAWING: 6



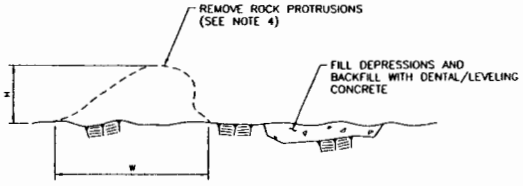
TREATMENT OF OVERHANGS ON ABUTMENT

NOT TO SCALE



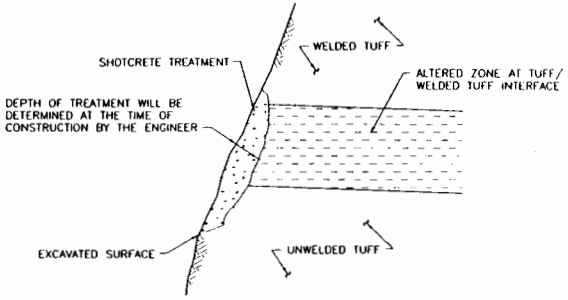
TREATMENT OF ROCK PROTRUSIONS ON ABUTMENTS

NOT TO SCALE



EXCAVATION AND TREATMENT AT FOUNDATION SURFACE

NOT TO SCALE



AT FOUNDATION SURFACE

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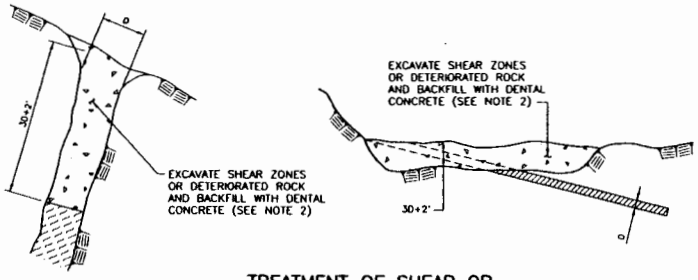

TREATMENT OF SHEAR OR DETERIORATED ROCK ZONES

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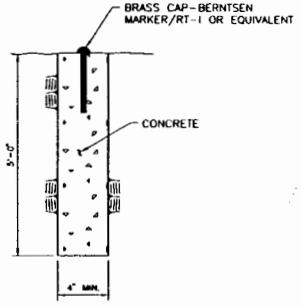

SURVEY MONUMENT

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(REFER TO NOTE 6)

NOTES:

1. FOUNDATION TREATMENT DETAILS SHOWN ON THIS DRAWING ARE TYPICAL OF TREATMENT WHICH MAY BE REQUIRED AND ARE SUBJECT TO CHANGE AS REQUIRED BY ENGINEER.
2. ALL TREATMENT DEPTHS ARE GIVEN AS GUIDELINES AND ARE SUBJECT TO CHANGE AS REQUIRED BY ENGINEER.
3. EXCAVATION FOR RCC DAM FOUNDATION TO EXTEND TO IN-PLACE, SLIGHTLY WEATHERED OR UNWEATHERED BEDROCK AS DETERMINED BY THE ENGINEER.
4. REMOVE ROCK PROTRUSIONS GREATER THAN 1 FOOT HIGH IF WIDTH (W)/ HEIGHT (H) $\leq$ 3.
5. SLAB THICKNESS MEASURED PERPENDICULAR TO SLOPE.
6. INSTALLATION OF TWO SURVEY MONUMENTS WILL BE REQUIRED ON THE DAM CONTROL LINE. LOCATION OF MONUMENTS TO BE DETERMINED BY ENGINEER.

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FOUNDATION EXCAVATION TREATMENT SECTION AND TYPICAL DETAILS

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SHEET 7 OF 16

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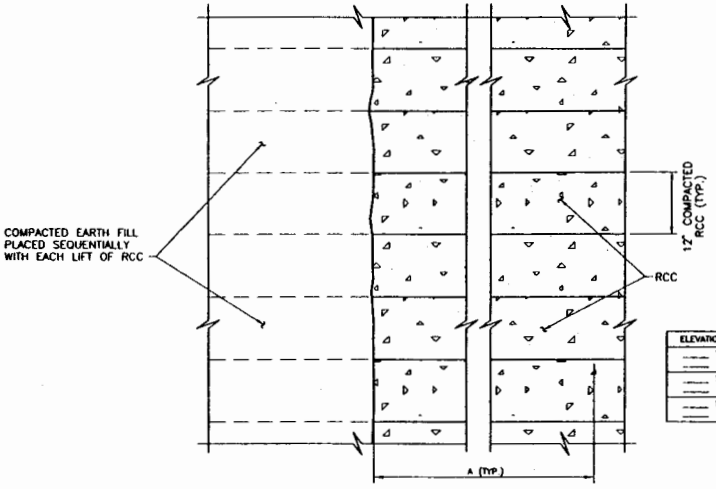

ELEVATION	A MEASUREMENT
TO	12 FT.
TO	10 FT.
TO	10 FT.

UPSTREAM RCC DETAIL

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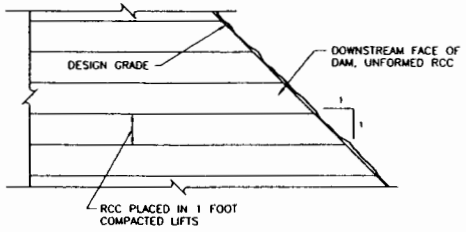

DOWNSTREAM DAM FACE

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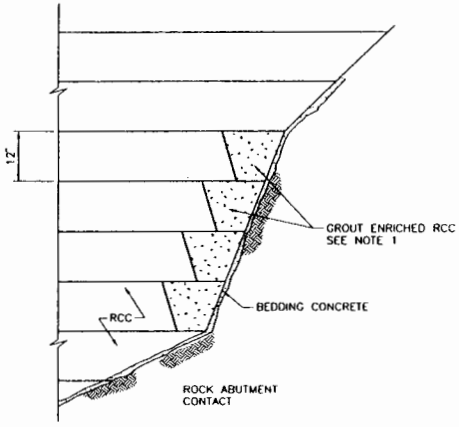
SCALE IN FEET



ALTERNATE ROCK ABUTMENT TREATMENT FOR OVERSTEEPENED ABUTMENTS

NOT TO SCALE

NOTE:

1. OVERSTEEPENED ABUTMENT CONTACTS SHALL HAVE GROUT ENRICHED RCC PLACED ADJACENT TO THE ABUTMENT SLOPE AS DETERMINED BY THE ENGINEER. THE PROCEDURE FOR THE CONSTRUCTION OF GROUT ENRICHED RCC IS AS FOLLOWS:
   a.) PUMP GROUT ON THE PREVIOUS RCC AND ABUTMENT ROCK JUST AHEAD OF RCC.
   b.) PLACE RCC LIFT OVER THE GROUT.
   c.) PUDDLE GROUT ON TOP OF FRESH UNCOMPACTED RCC.
   d.) VIBRATE THE GROUT AND RCC TOGETHER WITH A 2-INCH DIAMETER INTERVAL VIBRATOR.
   e.) COMPACT GROUT ENRICHED RCC WITH APPROPRIATE COMPACTION EQUIPMENT

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PLACEMENT DETAILS

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SHEET 8 OF 16

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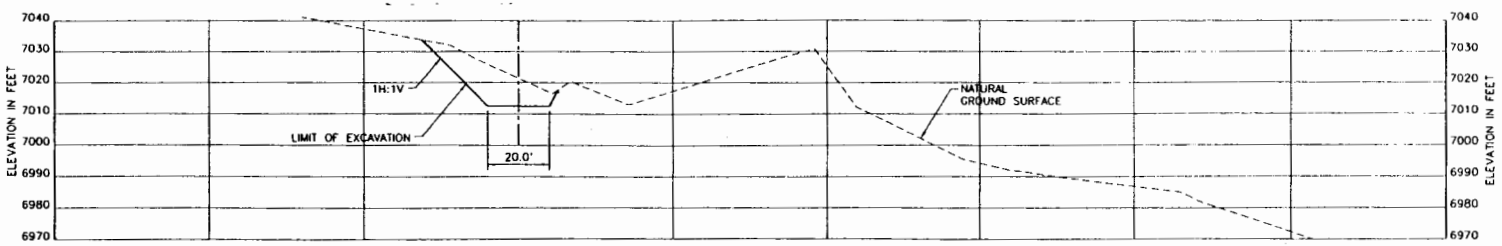

SECTION AT STATION 1+00

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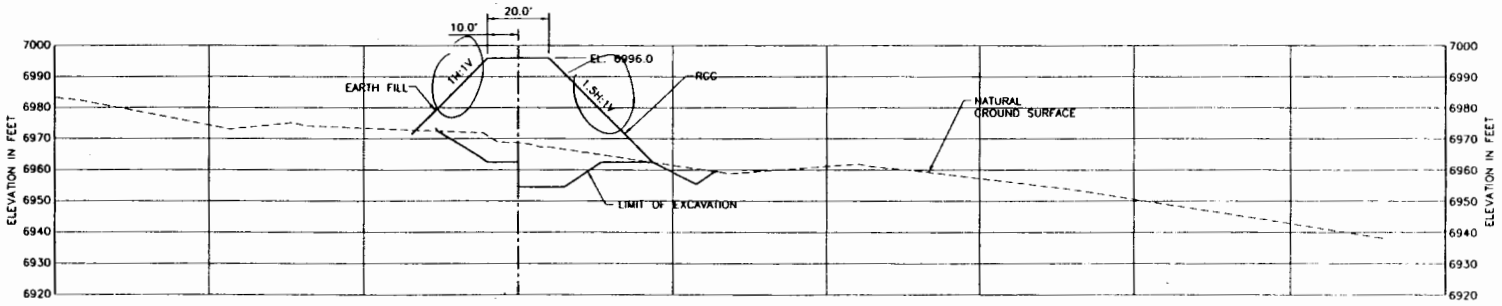

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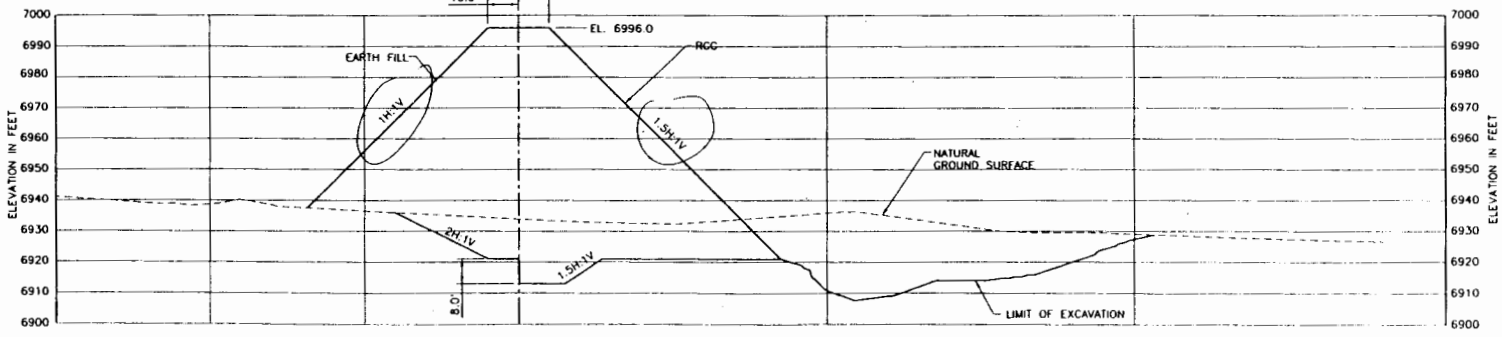

SECTION AT STATION 2+00

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RETAINING STRUCTURE CROSS SECTIONS

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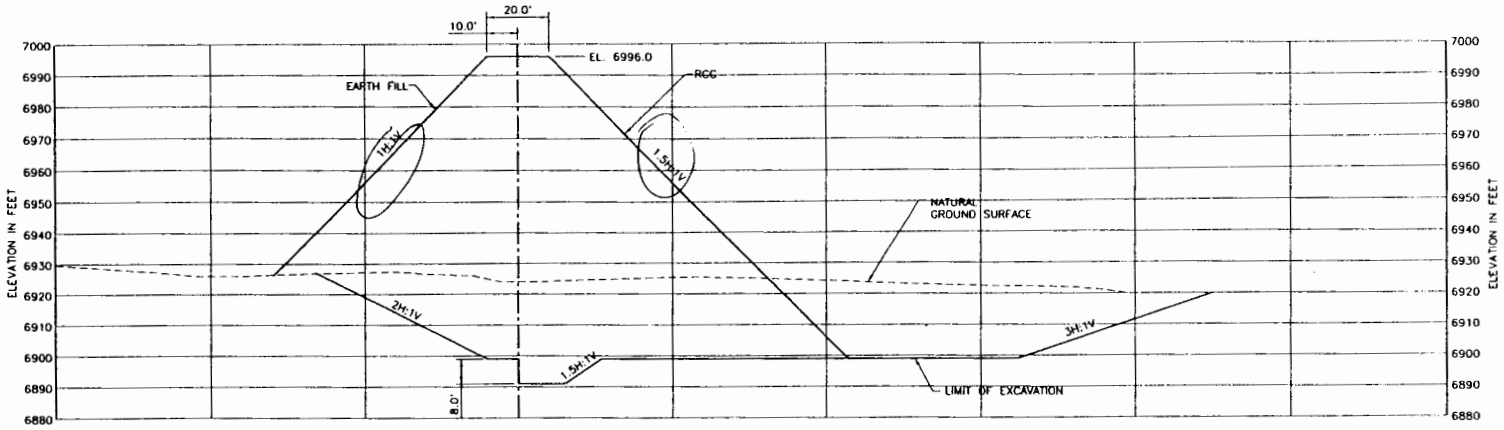

SECTION AT STATION 2+50

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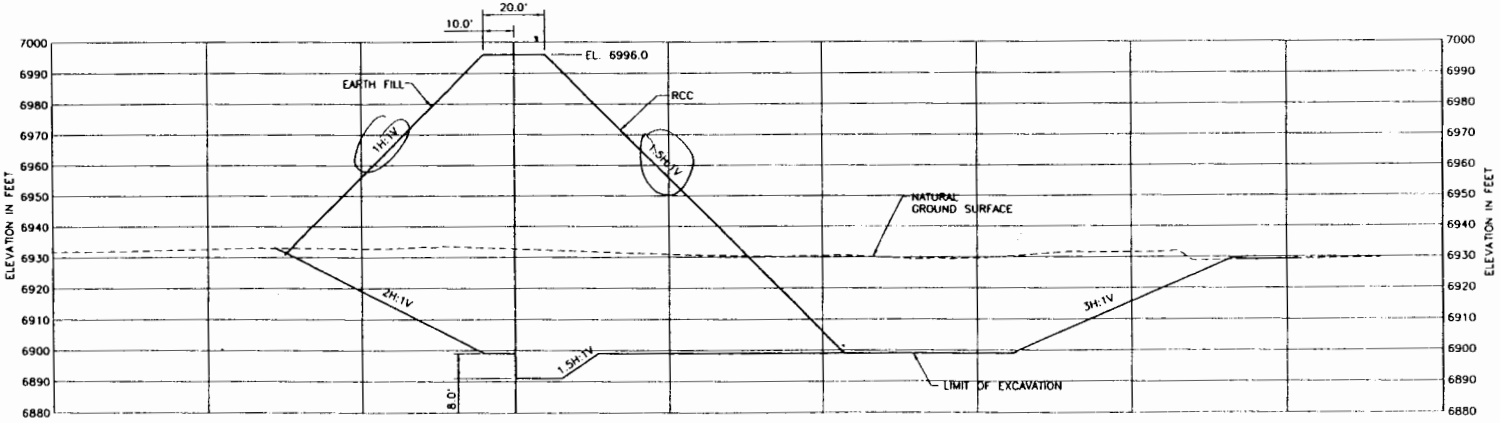

SECTION AT STATION 3+00

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RETAINING STRUCTURE
CROSS SECTIONS

REVISION: A
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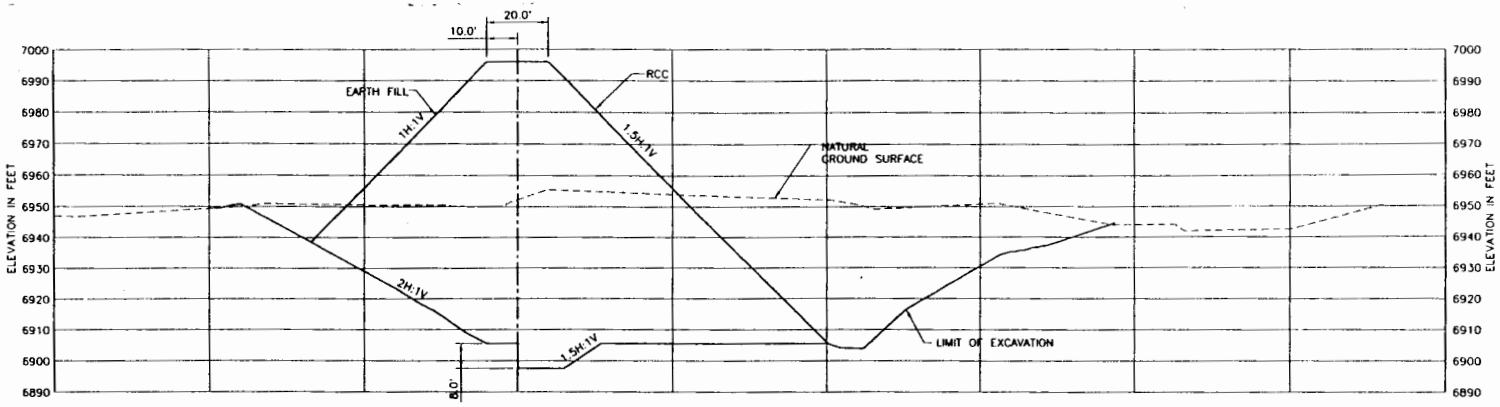

SECTION AT STATION 3+50

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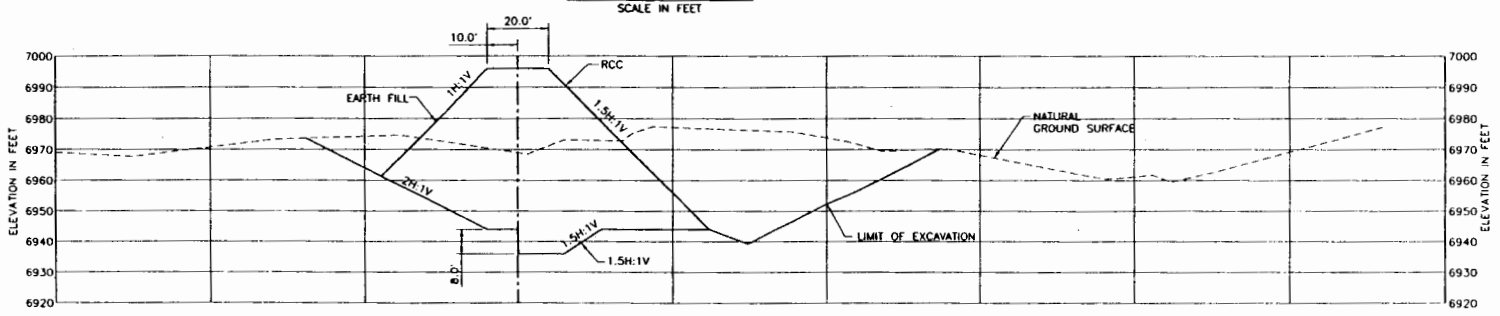

SECTION AT STATION 4+00

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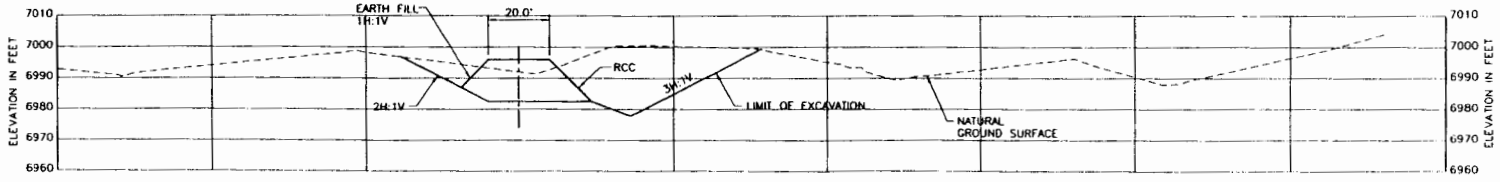

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RETAINING STRUCTURE CROSS SECTIONS

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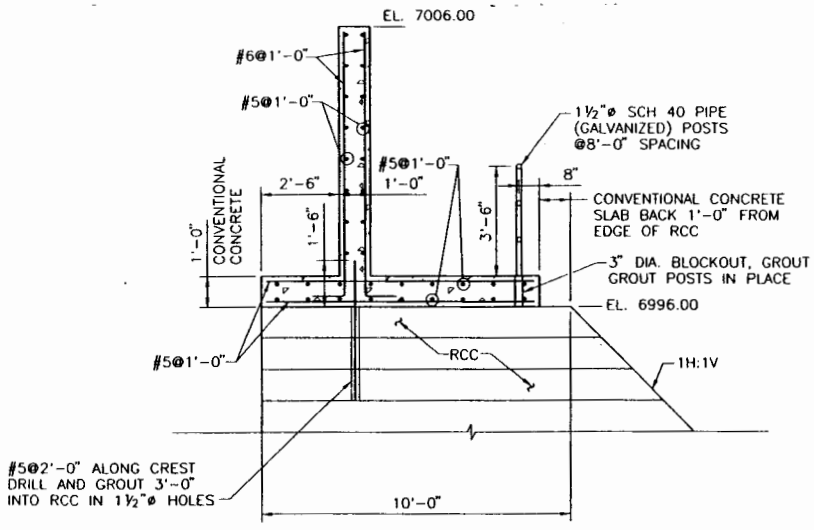

DAM CREST PARAPET WALL (TYP.)

SECTION E 5 6

0 2 4 6

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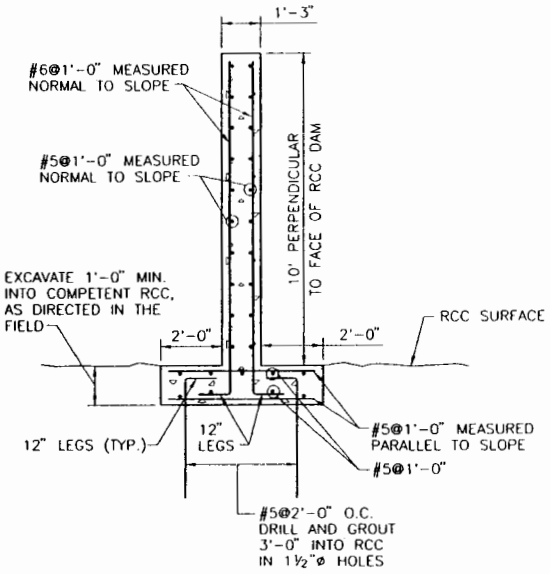

TRAINING WALLS (TYP.)

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SECTIONS AND DETAILS

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SHEET 13 OF 16

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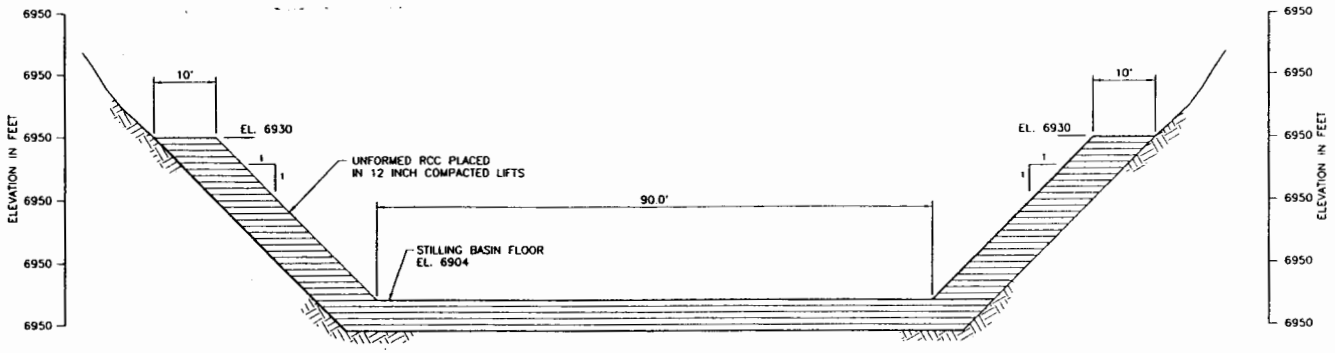

RCC STILLING BASIN

SECTION (B 4/6)
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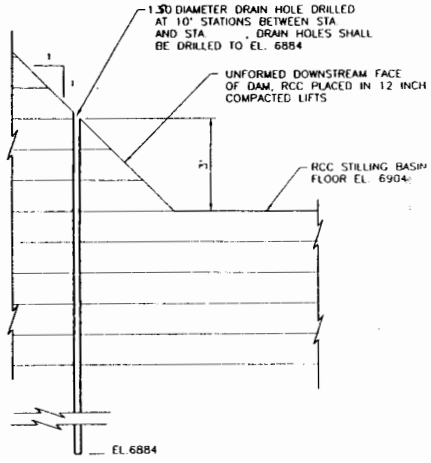
SCALE IN FEET



DRAIN HOLE DETAIL

(1 2/6)
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PAJARITO CANYON — LOS ALAMOS, NEW MEXICO

STILLING BASIN
SECTIONS AND DETAILS

REVISION: A
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SHEET 14 OF 16

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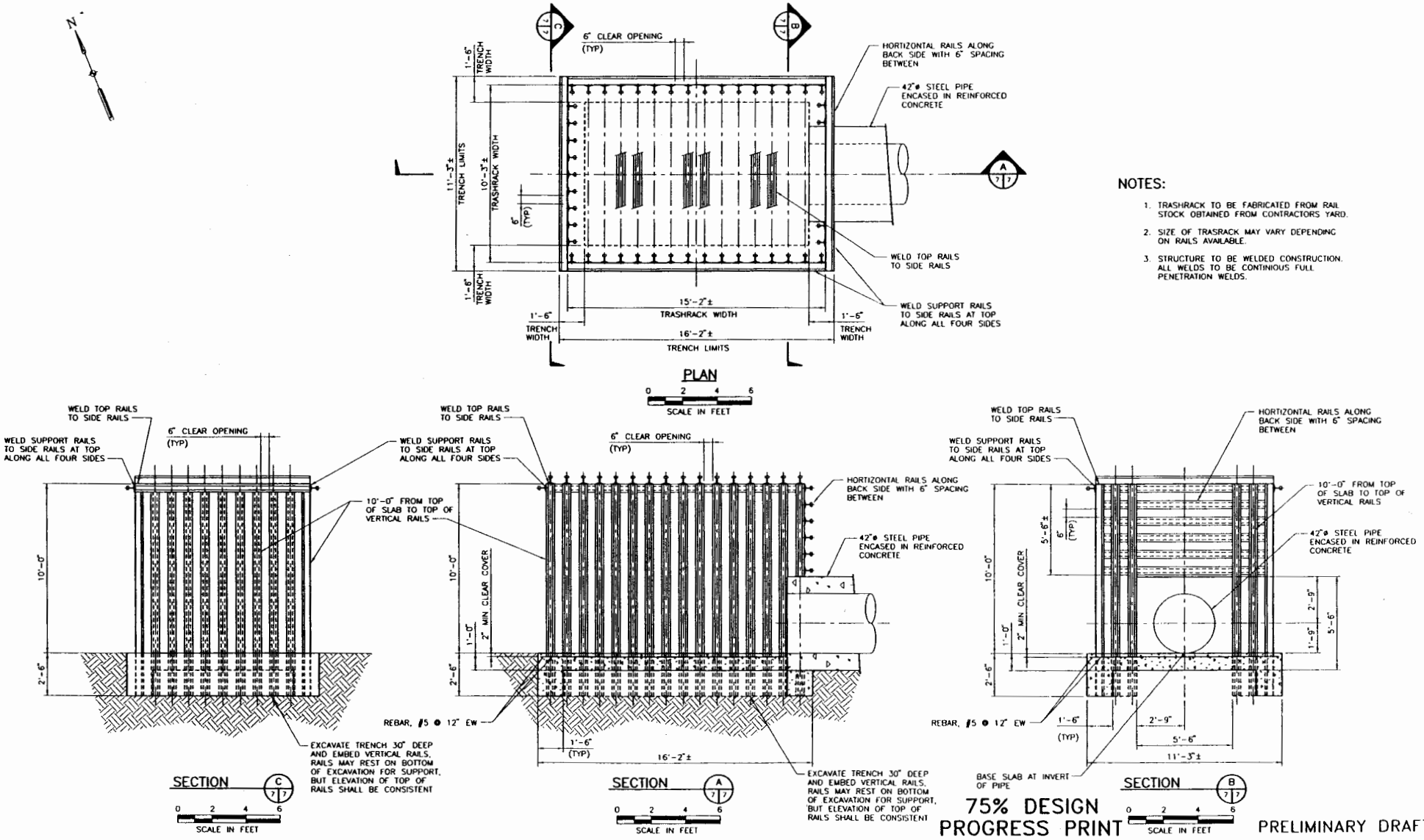

NOTES:

1. TRASHRACK TO BE FABRICATED FROM RAIL STOCK OBTAINED FROM CONTRACTORS YARD.
2. SIZE OF TRASRACK MAY VARY DEPENDING ON RAILS AVAILABLE.
3. STRUCTURE TO BE WELDED CONSTRUCTION. ALL WELDS TO BE CONTINIOUS FULL PENETRATION WELDS.

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PAJARITO CANYON — LOS ALAMOS, NEW MEXICO

INLET STRUCTURE TRASHRACK PLAN AND SECTIONS

REVISION A
PROJECT E30001579.03
DRAWING 16
SHEET 16 OF 16

DRAFT DESIGN REPORT

PAJARITO CANYON FLOOD CONTROL STRUCTURE

Prepared for
US Army Corps of Engineers

July 11, 2000

URS

4582 S. Ulster Street
Denver, CO 80237
E3.00001579.03

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This report will serve as a guideline and documentation for the design of the flood retarding structure (FRS-dam). The dam will be designed and constructed rapidly this report is intended to evolve throughout the design and construction of the project. It is likely that design requirements, design criteria, and construction details will change during the design and construction of the dam. This document will also form the basis of a final report documenting the design and construction of the dam.

The Pajarito FRS is located on Pajarito Creek, immediately downstream of the confluence of Twomile canyon and Pajarito Canyon. The dam was sited to provide efficient storage volume while minimizing dam height and providing watershed control for two large drainage basins upstream of TA18 and White Rock. Access to the dam is located about 400 feet downstream of the confluence of the two canyons at a location where the valley is relatively narrow, taking advantage of the topography to minimize the crest length and material volume for the dam.

Several dam types were considered for the FRS, including earth, earth and rock, and concrete. Considering that the dam must be able to handle the probable maximum flood, a spillway for this flood of 16,000 cubic feet per second would need to be very large and constructed of concrete to resist erosion from the flood flow. Of the three dam types considered, a concrete dam provides the ability to construct the spillway over the top of the dam and to have high resistance to erosion from flood flows. Typical of many concrete dams built today, roller compacted concrete was chosen as the construction methodology as it is a very efficient means of concrete construction. Roller compacted concrete is concrete that has a reduced moisture content, reduced sufficiently such that it behaves as a moist, sand and gravel fill. This material is mixed in conventional concrete batch plants or continuous mix pugmills transported to the site and placed in horizontal lifts similar to placement of soil in embankment fills. The lifts are compacted to a high density, the RCC sets like concrete, creating a mass concrete structure.

The FRS is designed to contain the runoff from storms up to the 100-year flood plus sediment volume from the 100-year flood. Storms up to this frequency interval will be retained and slowly released through the outlet works to assist in minimizing flooding downstream of the FRS. The spillway is designed to pass flows greater than the 100-year event up to and including the probable maximum flood. The spillway is approximately 150 feet wide and 10.5 feet deep and is located across the crest of the dam. Control walls will be constructed along both sides of the spillway to control the flow away from the abutments and to direct it into the stilling basin. The stilling basin downstream of the dam will be designed to provide the hydraulic jump and still the flows over the spillway resulting in a quesant flow downstream of the dam.

The outlet works is needed to minimize dam height while providing the needed downstream flood protection and to release water at a controlled rate. The reservoir will be drained within a 96-hour period to avoid water rights concerns and to release water at a controlled rate. The outlet works will be approximately a 42-inch diameter pipe located near the invert of the natural channel. A trashrack designed to dewater 6-inch diameter and larger debris. These retained materials would be removed during normal operation and maintenance after a flood event.

The FRS will be founded on volcanic tuff exposed in the foundation in the valley bottom and valley walls. The tuff appears to be relatively "tight" and since storage is to last for a period of up to only 96 hours, seepage through the foundation and along the FRS foundation contact is expected to be minimal and of little consequence to overall dam safety and performance.

The right side of Pajarito Valley along this stretch is comprised of a series of landslides and toppled rock blocks. As the reservoir is used for flood storage, these materials, along the valley wall, can become wetted and may sluff or slide into the stored water. Movement of the materials is not expected to affect the overall performance of the FRS, but may reduce the volume of the storage basin and may need to be removed as part of the regular operation and maintenance activities.

1.1 BACKGROUND

The forest fire surrounding Los Alamos has created the danger of flash flooding to the City and Los Alamos National Laboratory. This report documents the design and construction of the flood retarding structure (dam) to be built to protect the City and National Laboratory from flash flooding in Pajarito Canyon.

1.2 OBJECTIVE OF DESIGN AND INTENDED DESIGN LIFE

To design a dam to be rapidly designed and constructed to store the volume of a 100-year frequency storm. The dam is planned to be constructed within about one month of the Notice to Proceed for consideration. The dam is planned to be a temporary flood control structure with an intended design life of 15 to 20 years.

1.3 DESIGN BUILD APPROACH TO CONSTRUCTION

The dam will be designed and constructed as a design-build effort between Sundt Construction (Sundt) and URS Corporation (URS). URS will be the independent design engineer responsible for the design and field inspection, Sundt will be the contractor responsible for construction of the dam. The work will be accomplished under an IDIQ contract between the U.S. Army Corps of Engineers (COE) and Sundt. URS is subcontracted to Sundt for the design of the dam and engineering services during construction.

1.4 DESIGN TEAM AND PEER REVIEW OF DESIGN

The dam is being designed by the URS Denver Office. The URS (formerly Woodward-Clyde) peer reviewers were not involved directly in the design of the project and will function as independent reviewers.

The URS Design/Construction Team consists of the following lead engineers. Resumes are included in Appendix C.

- Terrence Arnold, P.E., URS Corporation (Lead Engineer for Preparation of Plans and Technical Specifications)
- Stan Ellis, URS Corporation (Project Construction Engineer)
- Daniel Johnson, P.E., URS Corporation (Dam Design)
- Guy Lund, P.E., URS Corporation (Structural Analysis)
- Salvatore Todaro, P.E., URS Corporation (Project Design Manager)

The design of the dam will be peer reviewed by a team of engineers, consisting of the following individuals:

- Howard Boggs, P.E., Independent Consultant (Stability and Structural Analysis)
- Mike Forrest, P.E., URS Corporation (Geotechnical Engineering – RCC Dam Design)
- Mark Schmoll, URS Corporation (Engineering Geology)

- Michael Stevens, Independent Consultant (Hydrology and Hydraulic Engineering)
- Steve Tatro, P.E., will conduct a review for the COE

1.5 DAM TYPE

The dam will be a gravity structure constructed of roller compacted concrete (RCC).

1.6 DAM CROSS SECTION

The RCC dam cross section was selected so that the dam can be rapidly constructed with the available equipment, material, and construction forces. An earthen embankment will be constructed upstream of the RCC dam to serve as an upstream form for the RCC. The embankment will be left in place after construction of the RCC dam. The RCC dam will have a vertical upstream surface and downstream sloped surface of 1.0 horizontal to 1.0 vertical (1:1). The RCC slope of 1:1 was selected because of the rounded aggregate and concerns with the ability of the RCC to be easily constructed at a steeper slope without the use of concrete formwork.

1.7 AGGREGATE FOR RCC

The aggregate to be used for the RCC is a 1-inch maximum size, road base type material. The aggregate was selected by Sundt for reasons of expediency and availability. The fine aggregate content (minus number 4) will be approximately 50 percent. Refer to Chapter 7 for the properties and gradation of the RCC aggregate.

2.1 LOCAL GEOLOGY

2.2 SUBSURFACE EXPLORATION PROGRAM

The subsurface exploration program consisted of drilling five borings within the proposed footprint of the flood retarding structure and excavating four test pits along the FRS centerline (Figure 1). The borings were logged by Jon Marin, a geologist working for SAIC, a subcontractor to LANL, and were observed by Bob Waddell, an engineer with URS. David Simpson, a senior engineering geologist with URS, along with Mr. Waddell, identified the boring locations, depths, and sampling intervals. The drilling was performed on June 30 and July 1, 2000. Stewart Brothers Drilling Company drilled the borings with a 4-wheel-drive, buggy-mounted CME 750 drill rig using 8-inch diameter hollow stem augers. Samples were collected with two types of samplers. An SPT sampler was driven into the formation with an auto-hammer (assumed to generate energy equivalent to a 140-pound hammer falling 30 inches), and a 3-1/2-inch inside diameter, Moss sampler was advanced inside the augers as the augers progressed downward. Each SPT sample was saved in a plastic bag for later inspection and laboratory testing. The Moss sampler contained thin-walled stainless steel or brass liners that relatively undisturbed samples were pushed up into as the augers and sampler moved downward. These liners were removed from the sampler upon completion of drilling each 2-1/2-foot-long run and were capped to preserve the in situ moisture content of the sample. The borings were drilled to depths of 31.5 to 60 feet.

A falling head test was performed in boring TH-1, located near the upstream toe area in the middle of the canyon bottom, to provide in situ permeability information.

The three test pits were excavated by Sundt on July __, 2000 with a Hitachi EX330 excavator (similar to CAT 245) using a 3-foot wide bucket. Two of the pits were excavated on the right abutment above TH-4, and one of the pits was excavated on the left abutment above TH-2 and TH-3 close to the centerline of the flood retarding structure (Figure 3-1).

2.2.1 RESULTS OF SITE RECONNAISSANCE AND SUBSURFACE EXPLORATION

The FRS site is underlain by weathered Bandelier Tuff bedrock. In the canyon bottom the tuff is generally mantled by about 10 to 15 feet of sandy silt to silty sand alluvium. The alluvium is slightly thicker beneath the lower portion of the right abutment.

Colluvium, toppled tuff boulders, and landslide debris mantle the slopes of both abutments. This material is generally composed of silty sand/sandy silt soil and gravel to very large boulder-size pieces of Bandelier tuff. The upper portion of the left abutment is a steep cliff, about 60 feet high, of welded tuff. Only about the bottom 10 feet of the cliff will be within the footprint of the flood retarding structure. Steep cliffs are absent from the right abutment in the vicinity of the flood retarding structure site. Rounded moderately sloping outcrops of tuff are present near the top of the canyon on the right abutment, more than 100 feet above the top of the flood retarding structure.

None of the Bandelier Tuff encountered in any of the borings drilled during this abbreviated exploration program could be considered to be strong rock. It could easily be drilled with hollow stem augers to depths of as much as 60 feet and could be sampled with standard soil sampling techniques. It was extremely weathered when first encountered below the alluvium or colluvium and became less weathered with increasing depth. The upper 10 to 15 feet of this material was clayey and classified as clayey sand (SC) or sandy clay (CL). With depth the unwelded tuff became less clayey and is classified as silty sand (SM) (check lab data to confirm). All of the tuff within the borings and test pits was relatively soft, could easily be excavated, and samples could be crushed by hand. The exploratory boring and test pit data is summarized in Table 2-1.

The welded tuff exposed in the cliff on the left abutment is probably five to ten times stronger than the unwelded tuff below that underlies most of the flood retarding structure footprint. The welded tuff is typically jointed with vertical and near vertical joints on roughly 2-foot to 8-foot spacing and ranging from less than ¼ inch to more than 3 inches in width. However, the weaker unwelded tuff that provides most of the flood retarding structure foundation does not appear to support open fractures. Groundwater levels were checked in borings TH-1, -2, -3, and –4 approximately 48 and 72 hours after drilling and all holes were dry to the bottom, although the stream had been flowing shortly after the completion of drilling.

The elevation of the top of the tuff beneath the lower portion of the right abutment, as identified in boring TH-4, is almost the same as the elevation to the top of the tuff found in borings TH-1 and TH-2, even though TH-4 was drilled about 22 feet up slope from TH-1 and TH-2, drilled in the canyon bottom (Figure 2-2). The landslide debris and colluvium that mantle the bedrock on the upper portion of the right abutment overlies alluvium near the toe of that abutment. The geological profile at the dam axis is shown on Figure 2-1.

Table 2-1

Summary of Subsurface Exploratory Data

Boring No.	Location	Total Depth (Ft)	Depth to Tuff Bedrock (Ft)	Approx. Depth to Foundation (Ft)
TH-1	canyon bottom upstream toe	60.0	10	20 to 25
TH-2	canyon bottom on axis of dam	46.5	10	25
TH-3	left abutment near upstream toe	31.5*	8.5	15
TH-4	right abutment near axis of dam	51.5	31.5	50
TH-5	toe of right abutment near upstream toe	36.5**	22	22

* Boring TH-3 was drilled from an excavated pad; the collar elevation was about 3 feet below grade.

** Boring TH-5 was drilled from a road; the collar elevation was about 2 to 3 feet below grade.

2.3 LABORATORY TESTING PROGRAM AND RESULTS

2.3.1 Geological Mapping and Geologic Sections

2.3.2 Recommended Excavation

2.4 RECOMMENDED FOUNDATION TREATMENT

It does not appear that much foundation shaping will be required. Because the rock is weak, the contractor should not have difficulty providing an acceptably shaped foundation (i.e., one that is free from steps, sharp offsets, vertical or overhanging sections, or deep cavities) with standard excavation techniques and equipment. It also appears that only small amounts of dental concrete and slush grout will be required, due to the apparently massive nature of the unwelded tuff foundation. However, if shear zones or intensely fractured zones are exposed during excavation, they could be overexcavated to a depth equal to three times their width and backfilled with dental concrete prior to covering with RCC.

The geologic contact between the welded tuff on the left abutment and the underlying unwelded tuff was not exposed prior to construction. This surface will need to be examined following foundation excavation. If gravel or other highly permeable material along this contact, some amount of overexcavation or pressure grouting may be required to prevent seepage and potential piping of the foundation materials.

2.5 FOUNDATION CONSIDERATIONS

1) Erosion along the tuff/concrete contact must be prevented. The foundation material is only weakly cemented and would probably be easily erodible. Seepage along the top of the foundation must be prevented.

2) Differential settlement of portions of the flood retarding structure may occur due to varying strengths of the foundation. If this occurs, seepage down through cracks or construction joints in the flood retarding structure to the foundation could occur leading to erosion of the foundation via piping.

3) It is likely that foundation excavation on the left abutment will lead to undercutting of the welded tuff unit near the top of the abutment. If the tuff is undercut, the foundation excavation should be extended up through the tuff to a daylight point somewhere on the bench above. Overhangs in the foundation will not be allowed due to the difficulty of getting adequate contact between the flood retarding structure construction materials and

the foundation. Removal of the welded tuff may be difficult due to the steepness of the cliff that this rock forms at the top of the abutment.

4) Because of the steepness of the right abutment, it is likely that the required relatively deep foundation excavation will result in a cut that daylights well above the top of the flood retarding structure.

5) The contractor must exercise care when performing the final clean up of the foundation. The rock is weak and cannot be cleaned with the same amount of energy that a stronger rock might require. If too much energy is used, the foundation will easily be excavated, instead of cleaned off.

6) Large amounts of water may be required to pre-moisten the tuff foundation before the initial lift of concrete is placed against it.

3.1 REGIONAL

3.2 LOCAL

3.3 LOCAL FAULTING

4.1 DISCUSSION OF ASSUMPTION FOR ANALYSIS

Los Alamos Dam is designed to be a Roller Compacted Concrete (RCC) flood control dam. Figure 5-1 shows the dimensions for the maximum overflow section of the dam. The maximum height of the dam is 106 feet from the base of the shear key to the crest of the dam. The crest thickness is 10 ft at elevation 6996 feet. The upstream face of the dam is vertical. The downstream face slopes at 1.0 horizontal (H):1.0 vertical (V).

The shear key, located at the upstream heel of the dam, extends 8 feet into the foundation and provides additional resistance against sliding. The shear key configuration consists of a 15 foot wide base and a downstream slope of 1.5 H:1.0 V.

The dam has been designed without internal or foundation drains.

The dam is a flood control facility, therefore, the normal operating condition for the dam is empty (no reservoir). There is no tailwater for the normal operating condition. The reservoir level for the 100-year flooding event is at the spillway crest elevation, which corresponds to the top of the maximum section, elevation 6996.0. The corresponding tailwater surface for the 100-year flood event is at elevation 6926.0, and corresponds to the original streambed elevation. The reservoir level corresponding to the probable maximum flooding (PMF) event is at elevation 7006.0. The corresponding tailwater surface for the PMF event is elevation 6936.0.

The foundation at Los Alamos Dam has been classified as a tuff.

4.2 MATERIAL PROPERTIES

The material properties used in this study were developed based on published data for roller compacted concrete (RCC), similar foundation rock, and standard engineering practice. The material properties are summarized in Table 1. The compressive strength of the RCC was based on a 360-day design strength of 2000 lb/in^2 *[1]*. The tensile strength of the RCC was computed from the modulus of rupture, as recommended in the U.S. Army Corps of Engineers guidelines *[7,8]*. The shear strength of the RCC was assumed equal to 10 percent of the compressive strength *[2,3]*. The unit weight was based on data from similar RCC dams *[2]*. The instantaneous modulus of elasticity of the RCC concrete was based on published data for similar RCC dams *[2,4]*. The sustained modulus of elasticity was assumed equal to 67 percent of the instantaneous value, in accordance with the U.S. Bureau of Reclamation (USBR) guidelines which states that the sustained modulus should be taken as 60 to 70 percent of the instantaneous modulus of elasticity *[5]*.

The foundation properties were based on data for similar rock types *[6]*.

Table 4-1 Material Property Assumptions	
Properties	**Values**
RCC Properties:	
Compressive Strength	2,000 lb/in^2
Tensile Strength	222 lb/in^2
Shear Strength	200 lb/in^2
Modulus of Elasticity	
Sustained (Static)	1,800,000 lb/in^2
Instantaneous (Dynamic)	2,700,000 lb/in^2
Unit Weight	150 lb/ft^3
Internal Angle of Friction	38.8 degrees
Foundation Properties:	
Deformation Modulus	200,000 lb/in^2
Unit Weight	90 lb/ft^3
Internal Angle of Friction	35 degrees
Cohesion	0 lb/in^2
Bearing Capacity	lb/in^2

4.3 LOADING CONDITIONS

The loading conditions developed for this study are based on the U.S. Army Corp of Engineers criteria for concrete Gravity Dams *[7]*. The usual load corresponds to the normal operating conditions for the dam. The unusual load corresponds to the 100-year flood condition. Two extreme load combinations were evaluated, one corresponding to the PMF event and the other corresponding to the maximum credible earthquake (MCE) event. The loading conditions evaluated are summarized below:

4.4 USUAL LOADING CONDITION NO. 1 (USLC-1)

- Gravity
- Embankment elevation 6992.0
- Normal empty reservoir

4.5 UNUSUAL LOADING CONDITION NO. 1 (UNLC-1)

- Gravity
- Embankment elevation 6992.0
- Reservoir water surface corresponding to the 100-year flood event, elevation 6996.0
- Tailwater elevation 6926.0
- Uplift

4.6 EXTREME LOADING CONDITION NO. 1 (EXLC-1)

- Gravity
- Embankment elevation 6992.0
- Reservoir water surface corresponding to the PMF event, elevation 7006.0
- Tailwater elevation 6936.0
- Uplift

4.7 EXTREME LOADING CONDITION NO. 2 (EXLC-2)

- Gravity
- Embankment elevation 6992.0
- Normal empty reservoir
- MCE

The gravity load for the dam unit weight of concrete equal to 150 lb/ft^3. The foundation rock located below the dam downstream of the shear key used a unit weight of equal to 90 lb/ft^3.

The reservoir and tailwater loads were applied as hydrostatic pressures to the upstream and downstream face of the dam using the unit weight of water equal to 62.5 lb/ft^3 *[5]*.

The equivalent saturated horizontal fluid pressure of reservoir sedimentation was assumed to be 85 lb/ft^3 *[5]*, and simulates an active pressure coefficient for the embankment of about 0.36.

The dam was design without an internal foundation drain system. The uplift pressure distribution for an uncracked base condition was assumed to vary linearly from full reservoir pressure at the upstream heel of the dam, to the tailwater pressure at the downstream toe of the dam. If the results from the analysis indicated that tensile stresses would develop at the dam/foundation contact, then the base was assumed to crack. For a cracked base condition the uplift pressure was assumed to be equal to the full reservoir pressure within the cracked portion of the base, then vary linearly from full reservoir at the crack tip to the tailwater pressure at the downstream toe of the dam.

4.8 CRITERIA

The structural stability analysis of Los Alamos Dam evaluated the following three conditions. First, that the dam is safe against overturning along any horizontal plane within the structure and the assumed failure plane at the base of the dam. Second, that the dam is safe against sliding along any horizontal plane within the structure and the assumed failure plane at the base of the dam. Finally, that the computed stresses within the concrete and foundation are less than the allowable limits of the material.

4.9 OVERTURNING

The dam is considered safe against overturning is it satisfies moment equilibrium. The USCOE criteria determines the overturning stability by determining the location of the resultant force at the assumed plane. The required location of the resultant force for overturning stability, as required by the USCOE, is summarized in Table 5-2.

Table 5-2
Overturning Resultant Location

Load Case	Location
Usual	Middle 1/3 of base
Unusual	Middle 1/2 of base
Extreme	Within base

In addition to location of the resultant force, the moment equilibrium was evaluated using the cracked base method of analysis. The normal stress distribution was computed to determine if tensile stresses occurred along the base of the dam. A crack was assumed to develop if the results indicated that tensile stresses would develop, since the dam/foundation contact is not capable of developing tensile stress. If cracking occurs at the base the result is an increase in the uplift load, and a decrease in the area of contact along the dam/foundation interface. To satisfy moment equilibrium, the crack base analysis had to shoe that a crack would stabilize, and not propagate through the thickness of the base.

4.10 SLIDING STABILITY

To satisfy sliding stability the computed factor of safety must be greater than the allowable limits set forth in the USCOE guidelines *[7]*. The minimum allowable sliding factors of safety used in these analyses are summarized in Table 5-3.

Table 5-3 Minimum Allowable Sliding Factors of Safety	
Load Case	**Sliding Stability**
Usual	2.0
Unusual	1.7
Extreme	1.3

4.11 ALLOWABLE STRESSES

The allowable stresses for the RCC were established based on the assumed material properties and the minimum allowable stress, as required in the USCOE criteria. The allowable shear capacity was computed using the USBR criteria *[5]*. The allowable foundation bearing capacity was computed based on the ultimate bearing capacity and the assumed factors of safety of 4.2, 2.7, and 1 for the usual, unusual and extreme loading conditions, respectively *[5]*. The allowable stresses are summarized in Table 5-4.

Table 5-4 Allowable Stress				
Load Case	**RCC Compressive Stress (lb/in^2)**	**RCC Tensile Stress (lb/in^2)**	**RCC Shear Stress (lb/in^2)**	**Foundation Bearing Capacity (lb/in^2)**
Usual	600	0	67	
Unusual	1000	95	100	
Extreme – static	180	159	200	
Extreme – dynamic	1800	238	200	

4.12 METHODS OF ANALYSES

For the structural analysis of Los Alamos Dam, the loading conditions were evaluated using the gravity method of analysis. This was considered appropriate because the loading conditions all consisted of static loads.

The gravity method of analysis is a two-dimensional evaluation which assume that the load is transferred vertically down to the foundation without transfer of load in the horizontal direction. This method also assumes that the concrete in the dam is a homogeneous, isotropic and

uniformly elastic material, and that the normal stresses along any horizontal plane are linearly distributed.

The stress distribution across any horizontal plane were computed using Equation 1 shown below:

$$\sigma = \frac{P}{A} \pm \frac{M_C \times \bar{c}}{I}$$

Equation 1

where:
- σ = stress,
- P = Summation of vertical forces, including uplift,
- A = Area of uncracked base,
- M_C = Summation of moment about to centroid of the uncracked base,
- c = Distance from base centroid to extreme fiber
- I = Base moment of inertia about centroid.

The stability factor of safety for the analysis was computed using Equation 2 shown below:

$$Q = \frac{c \times A + F_V \times Tan(\phi)}{F_H}$$

Equation 2

where:
- Q = Sliding Factor of Safety,
- c = Foundation cohesion,
- A = Area of uncracked base,
- F_V = Summation of vertical forces, including uplift,
- Tan(Φ) = Coefficient of friction along sliding plane,
- F_H = Summation of horizontal forces at assumed contact.

4.13 OVERTURNING RESULTS

The stress distribution along the base of the dam, and selected horizontal planes within the dam were evaluated for moment equilibrium. The plane of failure at the base of the dam was assumed to be inclined from the base of the shear key to the toe of the dam. The dip of the plane is about 5 degrees upstream. The horizontal planes were selected at approximately 25-, 35-, 50-, and 75-percent of the dam height. The results are shown in Figures 2 through 5 for the usual, unusual and two extreme loading conditions, respectively.

The results for load combinations USLC-1, UNLC-1 and EXLC-2 show that the assumed plane are all in compression, indicating that the resultant location is within the middle third of the base and satisfies overturning criteria. Because the entire planes are in compression, cracking will not occur and the assumed uplift load will not increase. These loading conditions are stable against overturning.

The initial results for EXLC-1, the PMF loading condition, indicated that tensile stresses would develop at the upstream heel of the dam. Since the foundation is not capable of developing tensile stress, the upstream heel would be expected to crack. A cracked base analysis showed that the crack at the base of the dam would extend about 13.9 feet downstream (14 percent of the base thickness). The crack stabilizes and does not propagate through the thickness of the base, and thus satisfies moment equilibrium. The resultant force location is within the middle half of the base, and thus satisfies overturning criteria. The base is considered safe against overturning for the assumed load EXLC-2.

The vertical stress distribution results from load EXLC-2 for the horizontal plane within the dam are all in compression. This indicates that the resultant location is within the middle third of the base and satisfies overturning criteria. Because the entire planes are in compression, cracking will not occur and the assumed uplift load will not increase. The load condition is stable against overturning.

4.14 SLIDING STABILITY RESULTS

The sliding stability factors of safety at the base of the dam were computed using the results from the gravity analyses. For the sliding stability computations along the plane at the base of the dam the horizontal and vertical forces were transformed into normal and parallel forces to the inclined plane. For the horizontal planes within the dam the vertical and horizontal forces were used in the sliding stability computations. The initial sliding factor of safety was computed using the parameters defined in Table 1, which conservatively assumed that cohesion was equal to zero (0) lb/in^2. If the sliding factor of safety was less than the minimum allowable, then computations were performed to determine the minimum cohesive value required to satisfy the stability criteria.

The results for usual loading condition 1 are summarized in Table 4-5. The results from the stability analysis show that the sliding factor of safety is greater than the required minimum value of 2.0 for all assumed plane at the base of the dam and within the concrete. The dam is considered stable for the assumed usual loading condition.

Table 4-5 Usual Load Combination No. 1 Sliding Stability Factor of Safety							
	Dam Forces				Computed	Allowable	
Elevation	Normal (kips)*	Thrust (kips)	Friction Angle (kips)	Uncracked Length (ft)	Factor of Safety	Factor of Safety	Required Cohesion (lb/in^2)
6971.5	-81.8	9.5	38.8	34.5	6.9	2.0	--
6947.0	-253.6	45.5	38.8	59.0	4.5	2.0	--
6922.5	-515.4	108.7	38.8	83.5	3.8	2.0	--
6898.0	-865.4	198.8	35	103.0	3.1	2.0	--
Inclined Base	-976.6	114.1	35	103.4	6.0	2.0	--

* (+) positive stress denotes tension, (-) negative stress denotes compression.

The results for unusual loading condition 1 are summarized in Table 4-6. The results from the stability analysis show that the sliding factor of safety is less than the required value of 1.7 for most of the assumed planes. As previously mentioned these sliding factors of safety were computed using the conservative assumption that cohesion is equal to zero. The required cohesive values for the required stability factor of safety are summarized in the table, and indicate that with very little cohesion the dam will satisfy criteria. The foundation strength parameters should be verified as soon as possible to confirm the stability of the section.

Table 4-6
Unusual Load Combination No. 1
Sliding Stability Factor of Safety

Elevation	Dam Forces		Friction Angle (kips)	Uncracked Length (ft)	Computed	Allowable	Required Cohesion (lb/in^2)
	Normal (kips)*	Thrust (kips)			Factor of Safety	Factor of Safety	
6971.5	-55.4	23.5	38.8	34.5	1.9	1.7	--
6947.0	-163.2	97.8	38.8	59.0	1.3	1.7	4
6922.5	-314.9	222.8	38.8	83.5	1.1	1.7	10
6898.0	-476.4	375.0	35	103.0	0.9	1.7	20
Inclined Base	-561.3	340.6	35	103.4	1.2	1.7	13

* (+) positive stress denotes tension, (-) negative stress denotes compression.

The results for extreme loading condition 1 are summarized in Table 4-7. The results from the stability analysis show that the sliding factor of safety is less than the required value of 1.3 for most of the assumed planes. As previously mentioned these sliding factors of safety were computed using the conservative assumption that cohesion is equal to zero. The required cohesive values for the required stability factor of safety are summarized in the table, and indicate that with very little cohesion the dam will satisfy criteria. The foundation strength parameters should be verified as soon as possible to confirm the stability of the section.

Table 4-7 Extreme Load Combination No. 1 Sliding Stability Factor of Safety							
Elevation	Dam Forces		Friction Angle (kips)	Uncracked Length (ft)	Computed	Allowable	Required Cohesion (lb/in^2)
	Normal (kips)*	Thrust (kips)			Factor of Safety	Factor of Safety	
6971.5	-44.6	38.8	38.8	34.5	0.9	1.3	3
6947.0	-144.8	128.4	38.8	59.0	0.9	1.3	6
6922.5	-297.5	269.1	38.8	83.5	0.9	1.3	10
6898.0	-454.3	444.5	35	103.0	0.7	1.3	18
Inclined Base	-436.4	417.8	35	103.4	0.7	1.3	18

* (+) positive stress denotes tension, (-) negative stress denotes compression.

The results for extreme loading condition 2 are summarized in Table 8. The results from the stability analysis show that the sliding factor of safety is less than the required value of 1.3 in the lower portion of the dam, only. As previously mentioned these sliding factors of safety were computed using the conservative assumption that cohesion is equal to zero. The required cohesive values for the required stability factor of safety are summarized in the table, and indicate that with very little cohesion the dam will satisfy criteria. The foundation strength parameters should be verified as soon as possible to confirm the stability of the section.

Table 4-8 Extreme Load Combination No. 2 Sliding Stability Factor of Safety							
Elevation	Dam Forces		Friction Angle (kips)	Uncracked Length (ft)	Computed	Allowable	Required Cohesion (lb/in^2)
	Normal (kips)*	Thrust (kips)			Factor of Safety	Factor of Safety	
6971.5	-81.8	47.4	38.8	34.5	1.4	1.3	--
6947.0	-253.6	158.2	38.8	59.0	1.3	1.3	--
6922.5	-515.4	332.7	38.8	83.5	1.2	1.3	2
6898.0	-865.4	569.9	35	103.0	1.1	1.3	10
Inclined Base	--	--	--	--	--	--	--

* (+) positive stress denotes tension, (-) negative stress denotes compression.

4.15 STRESS RESULTS

The computations for the gravity method of analysis are contained in Attachment A. The vertical stress distributions between the upstream and downstream face of the maximum section of dam were computed at different elevations. The elevations were selected to obtain horizontal planes at the base, and approximately 25-, 35-, 50-, and 75-percent of the dam height. The results from the analyses for the usual, unusual, and extreme loading conditions are summarized in Tables 9, 10, 11, and 12, respectively.

Table 4-9 Usual Load Combination No. 1 Vertical Stress		
Elevation	Upstream (lb/in^2)	Downstream (lb/in^2)
75 percent height, El. 6971.5	-29	-4
50 percent height, El. 6947.0	-50	-10
25 percent height, El. 6922.5	-70	-16
Base of Dam, El, 6898	-86	-31
Inclined Plane from Shear Key to Toe	-100	-25

* (+) positive stress denotes tension, (-) negative stress denotes compression.

The results from the usual loading combination show that the entire base of the dam and the selected horizontal planes all remain in compression. The computed stress results are less than the allowable compressive strength of the RCC (600 lb/in^2). These results show that the stresses in the dam and the resultant at the dam/foundation interface satisfy the criteria for new gravity dam design.

Table 4-10
Unusual Load Combination No. 1
Vertical Stress*

Elevation	Upstream (lb/in^2)	Downstream (lb/in^2)
75 percent height, El. 6971.5	-14	-9
50 percent height, El. 6947.0	-18	-20
25 percent height, El. 6922.5	-20	-32
Base of Dam, El, 6898	-16	-48
Inclined Plane from Shear Key to Toe	-13	-59

* (+) positive stress denotes tension, (-) negative stress denotes compression.

The results from the unusual loading combination show that the entire base of the dam and the selected horizontal planes all remain in compression. The computed stress results are less than the allowable compressive strength of the RCC (1000 lb/in^2). These results show that the stresses in the dam and the resultant at the dam/foundation interface satisfy the criteria for new gravity dam design. These results also show that the dam/foundation interface will remain uncracked, and thus, the assumed uplift load will remain unchanged.

Table 4-11 Extreme Load Combination No. 1 Vertical Stress *		
Elevation	**Upstream** (lb/in^2)	**Downstream** (lb/in^2)
75 percent height, El. 6971.5	-3	-15
50 percent height, El. 6947.0	-5	-29
25 percent height, El. 6922.5	-6	-44
Base of Dam, El, 6898	-1	-61
Inclined Plane from Shear Key to Toe	0	-67

* (+) positive stress denotes tension, (-) negative stress denotes compression.

The results from extreme loading condition show that the selected horizontal planes all remain in compression and that the computed stress results are less than the allowable compressive strength of the RCC (1800 lb/in^2). These computed results indicate that the stresses within the dam will satisfy the criteria for new gravity dam design.

Initial computations of the stress distribution along the inclined slope from the upstream shear key to the downstream toe of the dam indicated that the stress upstream heel was tension. The allowable tensile stress along the dam/foundation contact was assumed to be zero, therefore, these results indicated that a crack could develop at the upstream heel, along the assumed inclined sliding plane. As discussed previously, the uplift load along the base increases due to a cracked base condition. Therefore, it was determine necessary to perform a cracked base analysis along the potential inclined plane under the dam.

An evaluation of the cracked base was performed as part of the analysis. The results indicate that the crack length will extend to about 13.9 feet from the heel, or equal to about 14 percent of the base thickness. The results from the analysis show that the resultant force falls within the limits of the base, therefore, satisfying the USCOE resultant criteria.

Table 4-12 Extreme Load Combination No. 2 Vertical Stress*		
Elevation	**Upstream** (lb/in^2)	**Downstream** (lb/in^2)
75 percent height, El. 6971.5	-15	-18
50 percent height, El. 6947.0	-25	-35
25 percent height, El. 6922.5	-33	-53
Base of Dam, El, 6898	-35	-82
Inclined Plane from Shear Key to Toe	--	--

* (+) positive stress denotes tension, (-) negative stress denotes compression.

The results from the unusual loading combination show that the horizontal bases within the dam all in compression. The computed stress results are less than the allowable compressive strength of the RCC (2000 lb/in^2). These results show that the stresses in the dam and the resultant at the dam/foundation interface satisfy the criteria for new gravity dam design. These results also show that the dam/foundation interface will remain uncracked, and thus, the assumed uplift load will remain unchanged.

Summary and Conclusions

A structural analysis was performed for the maximum overflow section of Los Alamos Dam. The gravity method of analysis was used to evaluate the behavior due to the usual, unusual and extreme loading conditions. The structural behavior was evaluated based on the allowable stresses of the concrete. The stability of the dam was evaluated for moment equilibrium and sliding stability.

The results these studies show that the all vertical stresses in the dam are all compressive, and less than the maximum allowable compressive stresses of the RCC.

The gravity results for the usual static loading conditions show that the base of the dam will remain in complete compression.

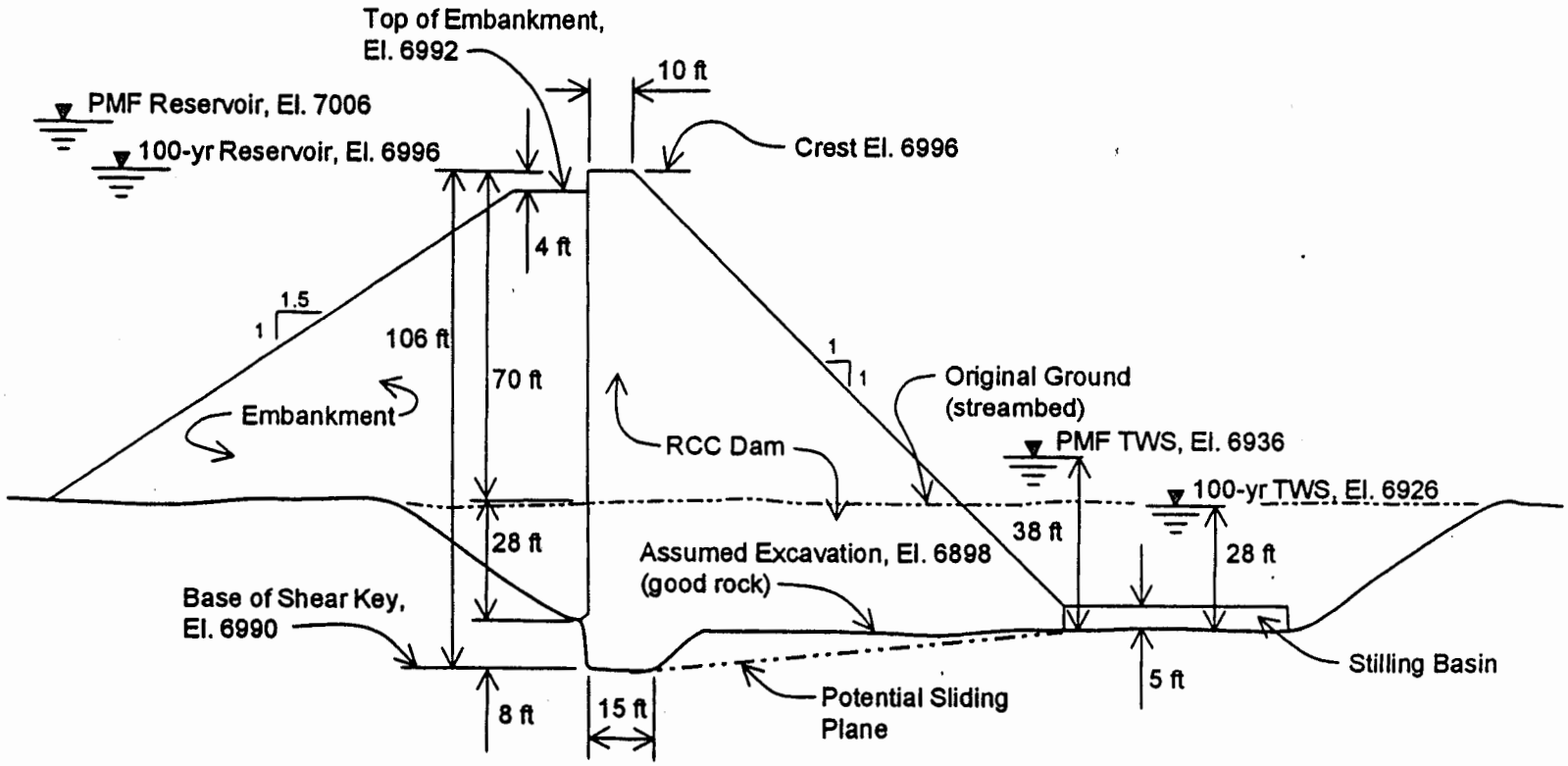
The gravity results from unusual (100-year flood) loading condition show that the base of the dam will remain in complete compression, which indicates that the base of the dam will not crack and there will not be a change in the assumed uplift conditions.

The results from the extreme (PMF) loading condition show that the base of the dam develops tension at the upstream heel. Since the dam/foundation contact is not capable of developing tension the base was assumed to crack and the uplift load was increased to account for the cracked section of the base. The results from the cracked base analysis shows that the crack does not propagate through the thickness of the dam, and the resultant force falls within the limits of the base. This satisfies the USCOE criteria for cracked base analysis.

The sliding stability computations showed that the dam satisfies the minimum allowable sliding stability criteria for the usual loading conditions. However, the dam requires cohesions in the foundation rock to satisfy the requirements for the unusual and extreme loading conditions. Based on these studies, the dam is considered safe for the all assumed loading conditions with the foundation drains in full operation.

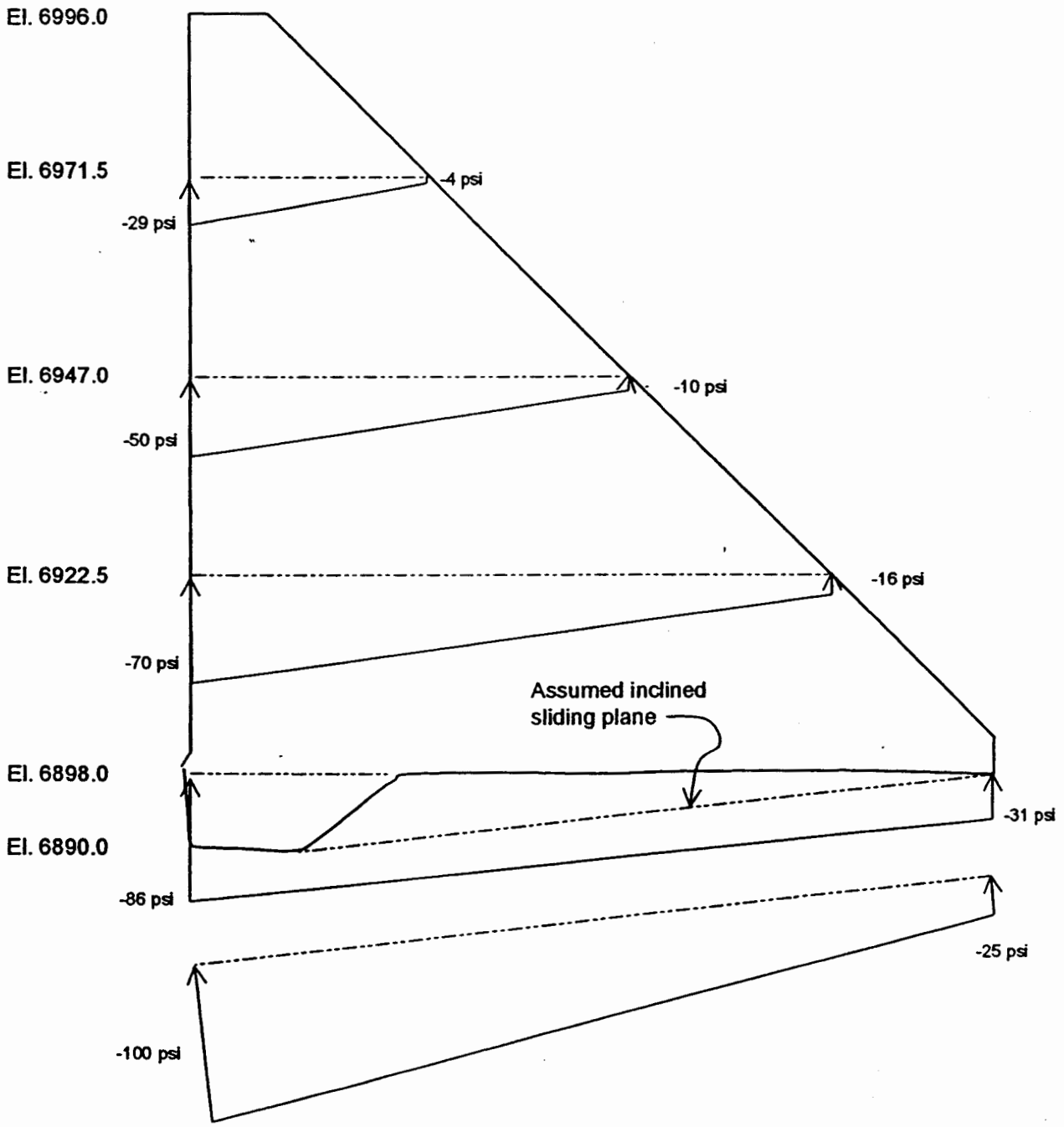
4.16 DESIGN CRITERIA

4.17 RESULT OF ANALYSIS



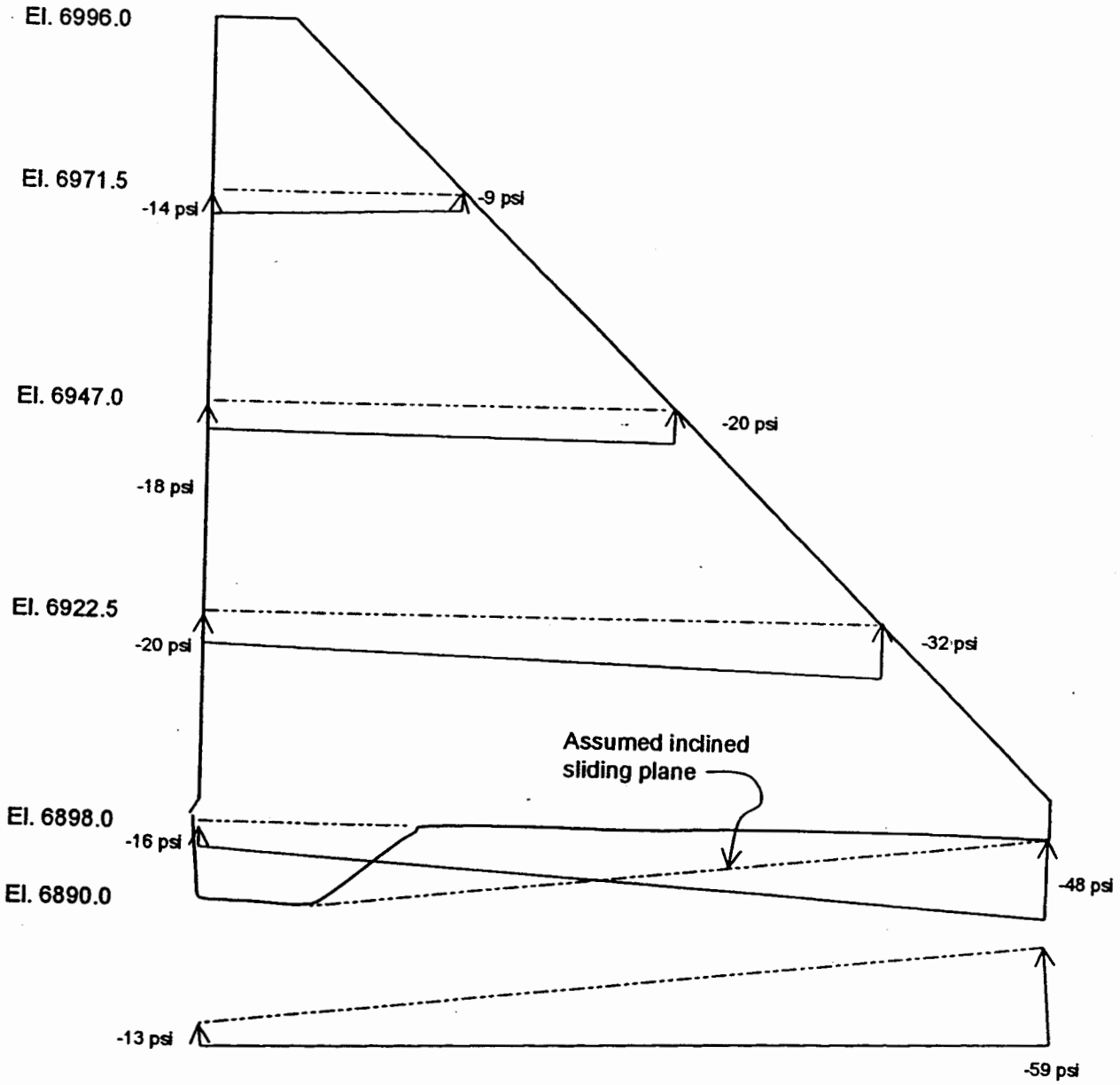
LOS ALAMOS DAM
Maximum Section

Figure 4-1



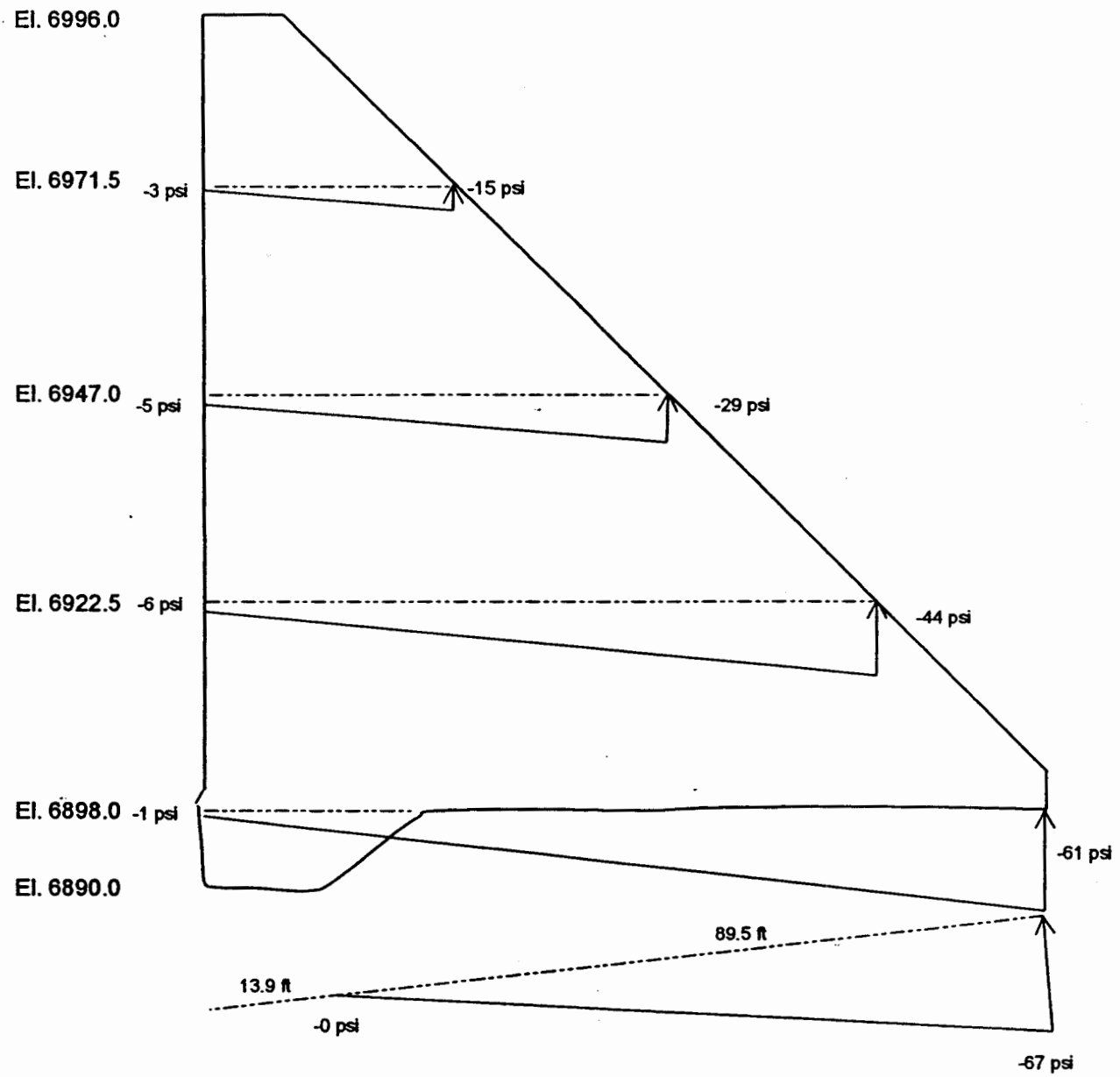
LOS ALAMOS DAM
Usual Load Combination No. 1
Stress Distribution

Figure 4-2



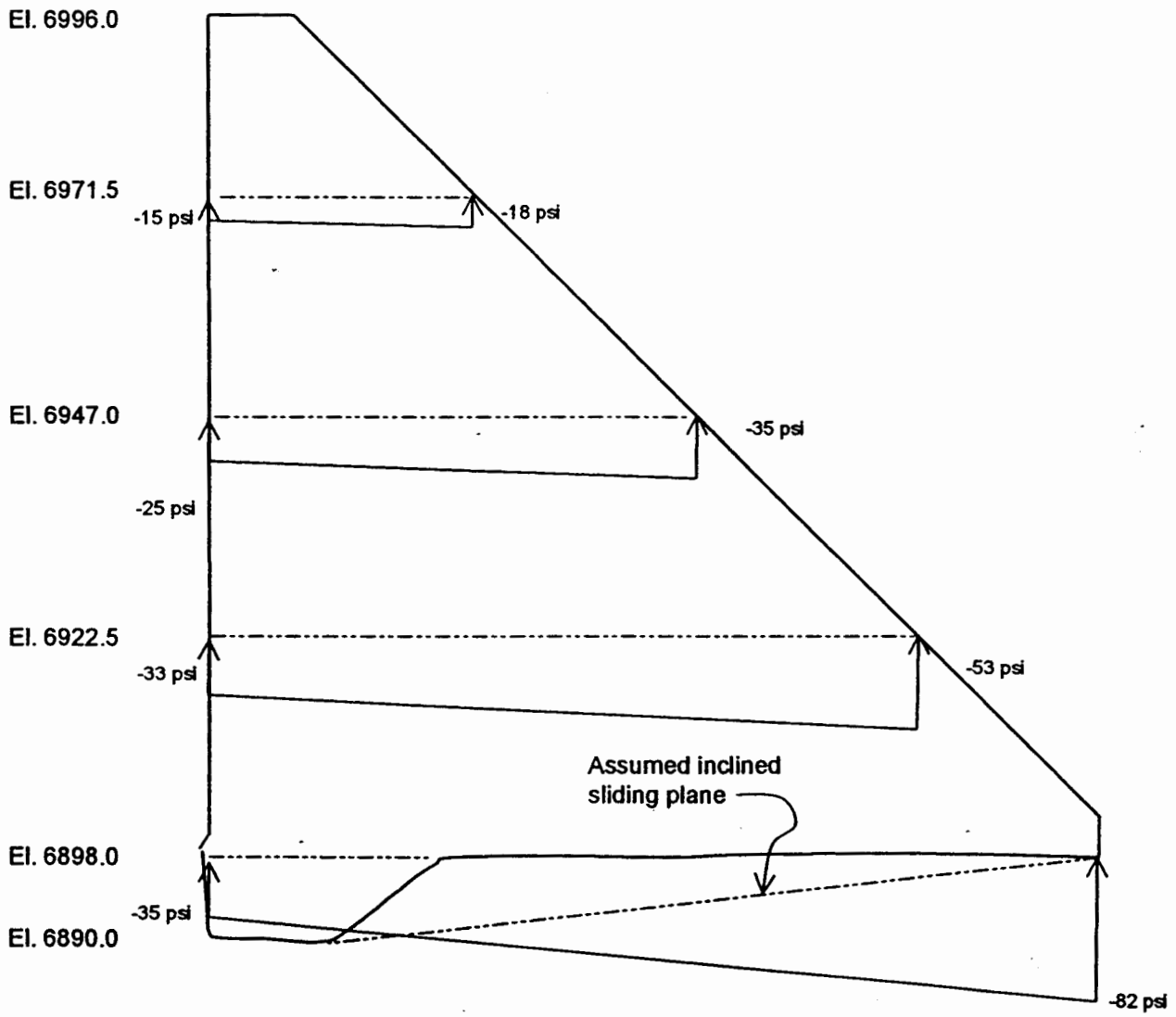
LOS ALAMOS DAM
Unusual Load Combination No. 1
Stress Distribution

Figure 4-3



LOS ALAMOS DAM
Extreme Load Combination No. 1
Stress Distribution

Figure 4-4



LOS ALAMOS DAM
Extreme Load Combination No. 2
Stress Distribution

Figure 4-5

Project: LOS ALAMOS
Feature: Maximum Section
Detail: Extreme Loading Condition - Probable Maximum Flood (PMF) Load
File: EXLC1a

Job No:
By: G.S.Lund
Chkd.By:
Date:
Time:

Input the type of measurement system to be used ('english' , 'metric' , or 'SI'):
english

[1]. Gravity Cross-section:

(a) Overall Dimensions, Elev., Width, & Crest Station:

Dam Crest Elevation =	6996.000	feet
Section Elevation =	6898.000	feet
Base Elevation =	6898.000	feet
Crest Station =	25.00	feet
Crest Thickness =	10.000	feet
Width =	1.000	feet
Section Elevation Length =	103.000	feet
Base Elevation Length =	103.000	feet
Conversion Length =	12.000	feet
Drain Distance from D/S Toe of Dam (Section El.) =	0.000	feet
Drain of U/S Uplift Effectiveness =	0%	

For Drain Distance Enter the following values:
D>0 assumes drained condition with drains located at 'D' from Toe.
D=0 assumes undrained uplift condition.
D<0 assumes no uplift condition.

For Drain Effectiveness Enter the following values:
for D<0 ,E is unused.
for D=0 ,E is percent reduction for all uplift pressure along base
for D>0 ,E is percent reduction for uplift pressure at drain location HW - e (HW - TW)

(b) Section Slopes, Upstream and Downstream Face

A negitive upstream slope signifies a ledge at the designated elevation, a length equal to the slope number.

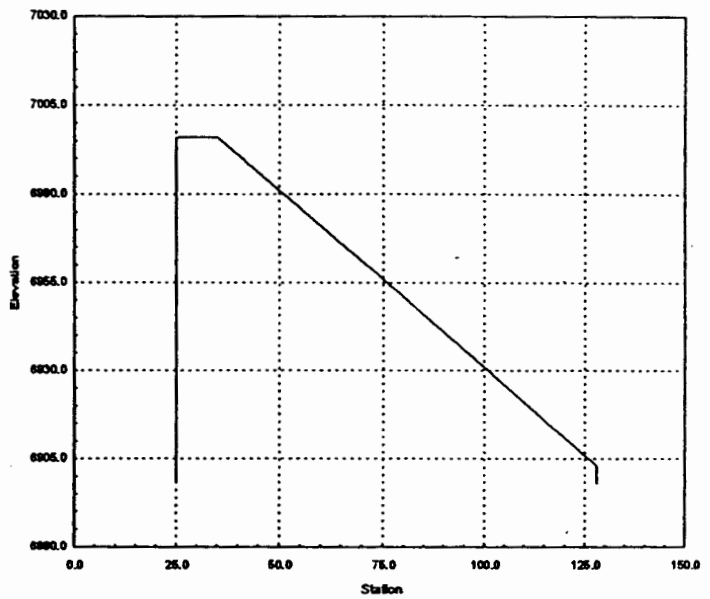
Upstream Slope

Slope (H/V) =	0.000	
Slope Top Elevation =	6996.000	feet

Downstream Slopes

Slope 1 (H/V) =	1.000	
Slope 1 Elevation =	6996.000	feet
Slope 2 (H/V) =	1.000	
Slope 2 Elevation =	6979.667	feet
Slope 3 (H/V) =	1.000	
Slope 3 Elevation =	6963.333	feet
Slope 4 (H/V) =	1.000	
Slope 4 Elevation =	6947.000	feet
Slope 5 (H/V) =	1.000	
Slope 5 Elevation =	6930.667	feet
Slope 6 (H/V) =	0.000	
Slope 6 Elevation =	6903.000	feet

LOS ALAMOS DAM
Maximum Section



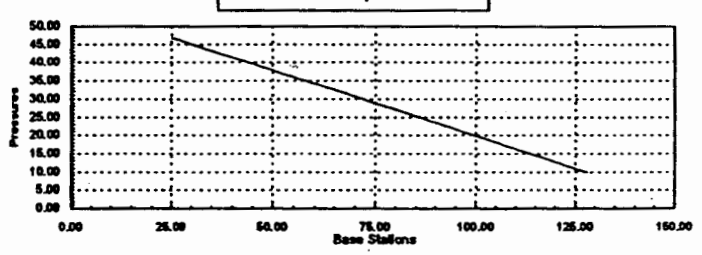
[2]. Material Properties:

Concrete Unit Weight =	150.00	lb/ft^3
Water Unit Weight =	62.50	lb/ft^3
Ice Force =	5000.00	lb/ft^2
Silt (horizontal) Unit Weight =	85.00	lb/ft^3
Silt (vertical) Unit Weight =	120.00	lb/ft^3
Coefficient of Internal Friction =	35.000	degrees
Foundation Cohesion =	0.000	lb/in^2
Tensile Strength at Section Elev. =	0.000	lb/in^2
Safety Factor =	3.000	

The uplift forces are based on the FERC criteria stated in "Chapter 3 - Engineering Guidelines for the Evaluation of Hydropower Projects".

Pressure @ Section's Upstream Heel =	46.88	lb/in^2
Pressure @ Section's Drain Location =	9.90	lb/in^2
Pressure @ Section's Downstream Toe =	9.90	lb/in^2

Uncracked Base Uplift Distribution

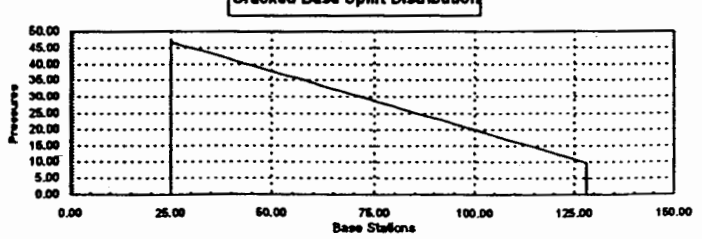


[3]. Reservior and Tailwater Elevations:

Reservoir Water Surface Elevation =	7006.000	feet
Reservoir Silt Elevation =	6992.000	feet
Ice Thickness =	0.000	feet
Tailsilt Elevation =	6898.000	feet
Tailwater Elevation =	6920.800	feet
Tailwater Coefficient =	100%	
Post Tension Anchor Force =	0.00E+00	lbs.
Post Tension Distance from Dam's Base Toe =	103.000	feet

The tailwater coefficient is input as a percentage. It is used to reduce the effective tailwater load on the downstream face of the structure due to several characteristics(eg. air entrainment, velocity, etc.). The uplift pressure at the toe is not effected by the coefficient.

Cracked Base Uplift Distribution



[4]. Earthquake Parameters:

U/S Slope Earthquake Coefficient, Ce =	0.735	
D/S Slope Earthquake Coefficient, Ce =	0.500	
Horizontal Ground Acceleration =	0.000	g
Vertical Ground Acceleration =	0.000	g

Project:	LOS ALAMOS	Job No:	Date:
Feature:	Maximum Section	By: G.S.Lund	Time:
Detail:	Extreme Loading Condition - Probable Maximum Flood (PMF) Load	Chkd.By:	
File:	EXLC1a		

[5]. External Forces on Thrust Block:

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
i) Thrust Block Gravity Load:							
(a) Gravity Forces:	(865,425)	–	66.86	35.51	(57,866,019)	0	(13,296,631)
(b) Gravity Earthquake Inertia Forces:	0	0	–	–	0	0	0
(c) Post Tension Anchor Forces:	0	–	103.00	–	0	–	0
ii) Reservoir ,Silt, and Ice:							
(a) Reservoir Forces:	0	361,375	0.00	35.47	0	12,817,588	12,817,588
(b) Silt Forces:	0	99,396	0.00	31.37	0	3,118,030	3,118,030
(c) Ice Forces:	–	0	–	108.00	–	0	0
(d) Reservoir Psuedo-static Forces:	0	0	–	0.00	0	0	0
(e) Silt Psuedo-static Forces:	0	0	–	0.00	0	0	0
iii) Tailwater & Tailsilt:							
(a) Tailwater Forces:	(9,900)	(16,244)	5.95	7.61	(58,908)	(123,619)	327,334
(b) Tailsilt Forces:	0	0	0.00	0.00	0	0	0
(c) Tailwater Psuedo-static Forces:	0	0	–	0.00	0	0	0
(d) Tailsilt Psuedo-static Forces:	0	0	–	0.00	0	0	0
Sub-Total without Uplift	(875,325)	444,527			(57,924,926)	15,812,000	2,966,321
iv) Uplift Forces:							
(a) Uplift Forces (uncracked base):	421,013	–	62.68	–	26,388,580	–	4,706,436
Total with Uplift	(454,313)	444,527			(31,536,347)	15,812,000	7,672,757

[6]. Stress Analysis:

Uncracked Base Analysis

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
(a) Without Uplift	(875,325)	2,966,321	46.9	(59.0)	11.7	(47.4) (compression)	(70.7) (compression)
(b) With Uplift	(454,313)	7,672,757	0.0	(30.6)	30.1	(0.5) (compression)	(60.8) (compression)

Cracked Base Analysis

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.00	695,250	0.00	0.00	0.00	35,805,375	(180,075)	(22,119,551)	122.84	(22,119,551)	(6,307,552)	uncracked
20.60	640,403	20.60	10.30	728.48	35,428,638	(234,923)	(22,496,289)	85.46	(20,076,585)	(4,264,585)	uncracked
41.20	585,555	41.20	20.60	5827.88	34,298,426	(289,770)	(23,626,500)	60.94	(17,657,234)	(1,845,234)	uncracked
61.80	530,708	61.80	30.90	19669.09	32,414,740	(344,618)	(25,510,186)	43.12	(14,861,499)	950,501	uncracked
82.40	475,860	82.40	41.20	46623.02	29,777,580	(399,465)	(28,147,346)	29.26	(11,689,380)	4,122,620	uncracked
103.00	421,013	103.00	51.50	91060.58	26,386,945	(454,313)	(31,537,981)	17.92	(8,140,877)	7,671,123	uncracked
[illegible] 103.00	421,013	103.00	51.50	91060.58	26,386,945	(454,313)	(31,537,981)	17.92	(8,140,877)	7,671,123	uncracked

Note: Depress Cntrl-C for refined Cracked Base Analysis Macro.
Depress Cntrl-R to reset cracked base analysis.

[7]. Final Stress Distribution:

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
(454,313)	(31,537,981)	15,812,000	103.00	51.50	91061	7,671,123	(30.6)	30.1	(0.5) (compression)	(60.8) (compression)

[8]. Stability Analysis:

1. Sliding-Stability:

$$F.S. = \frac{(C \times A) + (P - U)\tan(phi)}{V}$$

$$F.S. = \frac{0 + 318,113}{444,527}$$

F.S. = 0.72

2. Overturning Moment:

$$F.S. = \frac{\text{Sum Resisting Moment (toe)}}{\text{Sum Overturning Moment (toe)}}$$

$$F.S. = \frac{-58,048,545}{42,322,564}$$

F.S. = 1.37

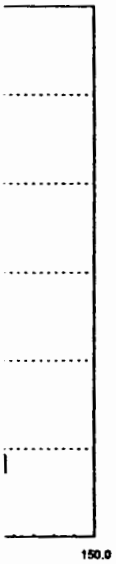
3. Floatation:

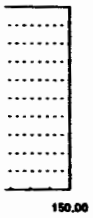
$$F.S. = \frac{\text{Sum Weight}}{\text{Sum Uplift}}$$

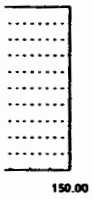
$$F.S. = \frac{-875,325}{421,013}$$

F.S. = 2.08

e (HW - TW)







(compression)
(compression)
(compression)
(compression)
(compression)
(compression)

(compression)

Project: LOS ALAMOS
Feature: Maximum Section
Detail: Extreme Loading Condition - Probable Maximum Flood (PMF) Load
File: EXLC1b

Job No:
By: G.S.Lund
Chkd.By:
Date:
Time:

Input the type of measurement system to be used ('english' , 'metric' , or 'SI'):
english

[1]. Gravity Cross-section:

(a) Overall Dimensions, Elev., Width, & Crest Station:

Dam Crest Elevation =	6996.000	feet
Section Elevation =	6922.500	feet
Base Elevation =	6922.500	feet
Crest Station =	25.00	feet
Crest Thickness =	10.000	feet
Width =	1.000	feet
Section Elevation Length =	83.500	feet
Base Elevation Length =	83.500	feet
Conversion Length =	12.000	feet
Drain Distance from D/S Toe of Dam (Section El.) =	0.000	feet
Drain of U/S Uplift Effectiveness =	0%	

For Drain Distance Enter the following values:
D>0 assumes drained condition with drains located at 'D' from Toe.
D=0 assumes undrained uplift condition.
D<0 assumes no uplift condition.

For Drain Effectiveness Enter the following values:
for D<0 ,E is unused.
for D=0 ,E is percent reduction for all uplift pressure along base
for D>0 ,E is percent reduction for uplift pressure at drain location HW - e (HW - TW)

(b) Section Slopes, Upstream and Downstream Face

A negitive upstream slope signifies a ledge at the designated elevation, a length equal to the slope number.

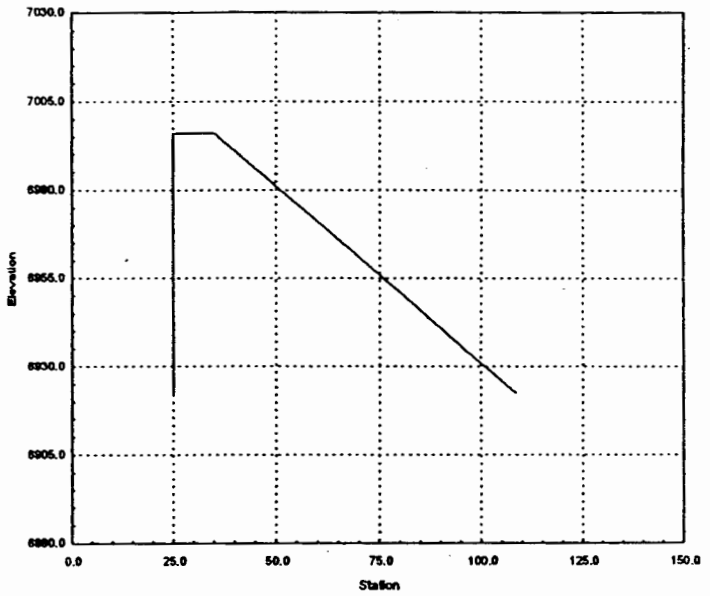
Upstream Slope

Slope (H/V) =	0.000	
Slope Top Elevation =	6996.000	feet

Downstream Slopes

Slope 1 (H/V) =	1.000	
Slope 1 Elevation =	6996.000	feet
Slope 2 (H/V) =	1.000	
Slope 2 Elevation =	6983.750	feet
Slope 3 (H/V) =	1.000	
Slope 3 Elevation =	6971.500	feet
Slope 4 (H/V) =	1.000	
Slope 4 Elevation =	6959.250	feet
Slope 5 (H/V) =	1.000	
Slope 5 Elevation =	6947.000	feet
Slope 6 (H/V) =	1.000	
Slope 6 Elevation =	6927.500	feet

LOS ALAMOS DAM
Maximum Section



[2]. Material Properties:

Concrete Unit Weight =	150.00	lb/ft^3
Water Unit Weight =	62.50	lb/ft^3
Ice Force =	5000.00	lb/ft^2
Silt (horizontal) Unit Weight =	85.00	lb/ft^3
Silt (vertical) Unit Weight =	120.00	lb/ft^3
Coefficient of Internal Friction =	38.750	degrees
Foundation Cohesion =	0.000	lb/in^2
Tensile Strength at Section Elev. =	0.000	lb/in^2
Safety Factor =	3.000	

The uplift forces are based on the FERC criteria stated in "Chapter 3 - Engineering Guidelines for the Evaluation of Hydropower Projects".

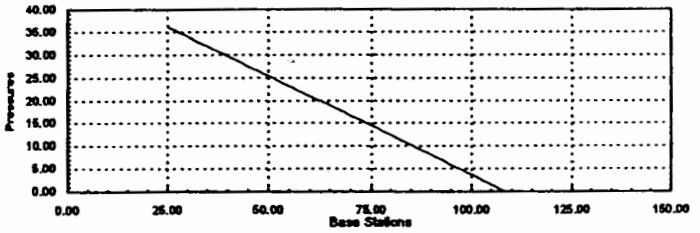
Pressure @ Section's Upstream Heel =	36.24	lb/in^2
Pressure @ Section's Drain Location =	0.00	lb/in^2
Pressure @ Section's Downstream Toe =	0.00	lb/in^2

[3]. Reservior and Tailwater Elevations:

Reservoir Water Surface Elevation =	7006.000	feet
Reservoir Silt Elevation =	6992.000	feet
Ice Thickness =	0.000	feet
Tailsill Elevation =	6922.500	feet
Tailwater Elevation =	6922.500	feet
Tailwater Coefficient =	100%	
Post Tension Anchor Force =	0.00E+00	lbs.
Post Tension Distance from Dam's Base Toe =	83.500	feet

The tailwater coefficient is input as a percentage. It is used to reduce the effective tailwater load on the downstream face of the structure due to several characteristics(eg. air entrainment, velocity, etc.). The uplift pressure at the toe is not effected by the coefficient.

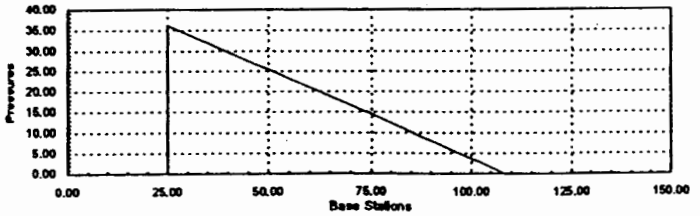
Uncracked Base Uplift Distribution



[4]. Earthquake Parameters:

U/S Slope Earthquake Coefficient, Ce =	0.735	
D/S Slope Earthquake Coefficient, Ce =	0.500	
Horizontal Ground Acceleration =	0.000	g
Vertical Ground Acceleration =	0.000	g

Cracked Base Uplift Distribution



[5]. External Forces on Thrust Block:

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
i) Thrust Block Gravity Load:							
(a) Gravity Forces:	(515,419)	–	55.31	27.12	(28,507,428)	0	(6,988,696)
(b) Gravity Earthquake Inertia Forces:	0	0	–	–	0	0	0
(c) Post Tension Anchor Forces:	0	–	83.50	–	0	–	0
ii) Reservoir ,Silt, and Ice:							
(a) Reservoir Forces:	0	214,758	0.00	27.14	0	5,829,471	5,829,471
(b) Silt Forces:	0	54,339	0.00	23.20	0	1,260,569	1,260,569
(c) Ice Forces:	–	0	–	83.50	–	0	0
(d) Reservoir Psuedo-static Forces:	0	0	–	0.00	0	0	0
(e) Silt Psuedo-static Forces:	0	0	–	0.00	0	0	0
iii) Tailwater & Tailsilt:							
(a) Tailwater Forces:	0	0	0.00	0.00	0	0	0
(b) Tailsilt Forces:	0	0	0.00	0.00	0	0	0
(c) Tailwater Psuedo-static Forces:	0	0	–	0.00	0	0	0
(d) Tailsilt Psuedo-static Forces:	0	0	–	0.00	0	0	0
Sub-Total without Uplift	(515,419)	269,097			(28,507,428)	7,090,040	101,345
iv) Uplift Forces:							
(a) Uplift Forces (uncracked base):	217,883	–	55.66	–	12,127,968	–	3,031,360
Total with Uplift	(297,536)	269,097			(16,379,461)	7,090,040	3,132,705

[6]. Stress Analysis:

Uncracked Base Analysis

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
(a) Without Uplift	(515,419)	101,345	36.2	(42.9)	0.6	(42.3) (compression)	(43.5) (compression)
(b) With Uplift	(297,536)	3,132,705	0.0	(24.7)	18.7	(6.0) (compression)	(43.5) (compression)

Cracked Base Analysis

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.00	435,766	0.00	0.00	0.00	18,193,215	(79,653)	(10,314,214)	129.49	(10,314,214)	(3,224,173)	uncracked
16.70	392,189	16.70	8.35	388.12	17,950,563	(123,230)	(10,556,866)	77.32	(9,527,898)	(2,437,857)	uncracked
33.40	348,613	33.40	16.70	3104.98	17,222,607	(166,806)	(11,284,822)	50.95	(8,499,157)	(1,409,117)	uncracked
50.10	305,036	50.10	25.05	10479.29	16,009,347	(210,383)	(12,498,082)	34.36	(7,227,992)	(137,952)	uncracked
66.80	261,459	66.80	33.40	24839.80	14,310,783	(253,959)	(14,196,646)	22.50	(5,714,402)	1,375,638	uncracked
83.50	217,883	83.50	41.75	48515.24	12,126,915	(297,536)	(16,380,514)	13.30	(3,958,388)	3,131,652	uncracked
[illegible]											
83.50	217,883	83.50	41.75	48515.24	12,126,915	(297,536)	(16,380,514)	13.30	(3,958,388)	3,131,652	uncracked

Note: Depress Cntrl-C for refined Cracked Base Analysis Macro.
Depress Cntrl-R to reset cracked base analysis.

[7]. Final Stress Distribution:

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
(297,536)	(16,380,514)	7,090,040	83.50	41.75	48515	3,131,652	(24.7)	18.7	(6.0) (compression)	(43.5) (compression)

[8]. Stability Analysis:

1. Sliding-Stability:

$$F.S. = \frac{(C \times A) + (P - U)\tan(phi)}{V}$$

$$F.S. = \frac{0 + 238,798}{269,097}$$

F.S. = 0.89

2. Overturning Moment:

$$F.S. = \frac{\text{Sum Resisting Moment (toe)}}{\text{Sum Overturning Moment (toe)}}$$

$$F.S. = \frac{-28,507,428}{19,216,955}$$

F.S. = 1.48

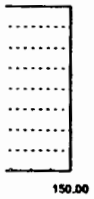
3. Floatation:

$$F.S. = \frac{\text{Sum Weight}}{\text{Sum Uplift}}$$

$$F.S. = \frac{-515,419}{217,883}$$

F.S. = 2.37

e (HW - TW)



(compression)
(compression)
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(compression)

5.1 100-YEAR FLOOD (HYDROGRAPH)

The 100-year flood at the reservoir is produced from runoff from three primary watersheds upstream of the reservoir. Using the U.S. Army COE HEC-1 hydrologic model, the inflow and outflow hydrographs from the reservoir with a 42" RCP outlet are presented in Figure 6.1. As stated in section 2, the reservoir capacity was designed to store up to the 100-year storm, using the outlet works to release water. An spillway will be used for events larger than the 100-year.

100-year, 6-hour Event
Total Rainfall = 2.77 inches

Location	Peak Discharge (cfs)
Inflow to reservoir	2806
Outflow from reservoir	343

5.2 100-YEAR DEBRIS VOLUME

The 100-year, 6-hour debris volume will be assumed to be one-third of the total volume entering the reservoir, or 182 AC-FT. Using the U.S. Army COE HEC-1 hydrologic model, a 100-year, 6-hour storm event was modeled assuming 182 AC-FT of storage was filled initially to represent the potential sediment load. This resulted in a required storage elevation of 7005.5 ft.

However, as the watershed has time to revegetate, the volume of sediment entering the reservoir during the 100-year event will decrease.

5.3 500-YEAR FLOOD (HYDROGRAPH)

The 500-year, 24-hour flood at the reservoir is produced from runoff from three primary watersheds upstream of the reservoir. Using the U.S. Army C.O.E HEC-1 hydrologic model, the inflow and outflow hydrographs from the with a 42" RCP outlet works and the spillway height set to the 100-year water surface elevation of 6996.7 ft are presented in Figure 6.2. As stated in Section 2, the reservoir capacity was designed to store the 100-year, 6-hour storm and less using only the outlet works to release water. The spillway will be used for events larger than the 100-year event.

500-year, 24-hour Event
Total Rainfall = 3.66 inches

Location	Peak Discharge (cfs)
Inflow to reservoir	3441
Outflow from reservoir	1328

5.4 PROBABLE MAXIMUM FLOOD

The Probable Maximum Flood (PMF) for the reservoir was computed using USBR HMR 55-A. The U.S. Army COE HEC-1 hydrologic model was used to model the PMF for the watershed.

Using the 100-year, 6-hour event spillway height of 6996.7 ft and a 42" RCP outlet works, a water surface elevation at the dam will be approximately 7007 ft, with approximately a maximum of 16,000-cfs over the spillway.

PMF, 24-hour Event
Total Rainfall = 23 inches

Location	Peak Discharge (cfs)
Inflow to reservoir	17094
Outflow from reservoir	16943

Figure 5-1
100-year Inflow and Outflow Hydrographs at the Reservoir using calibrated hydrology from E.S/ LANL

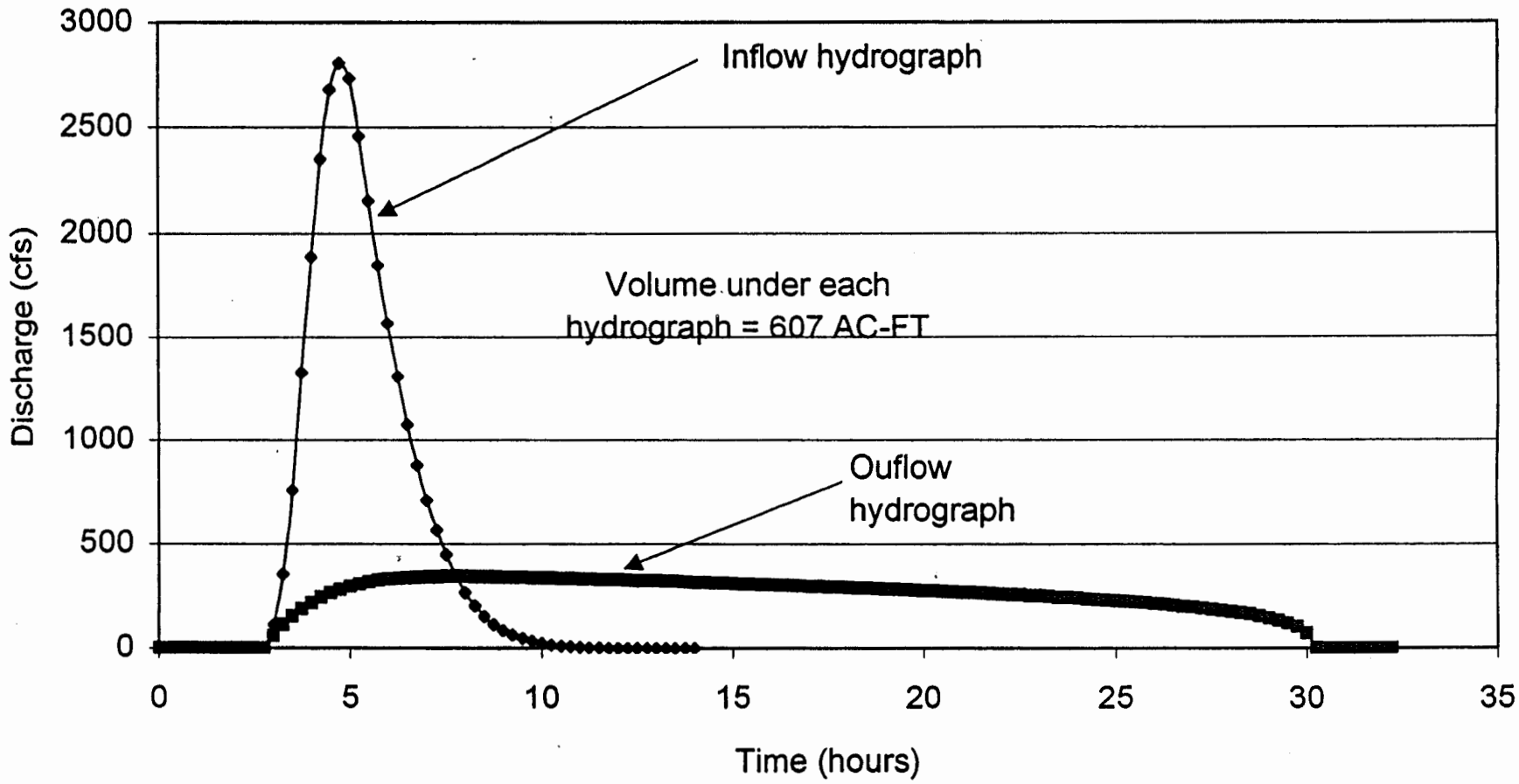
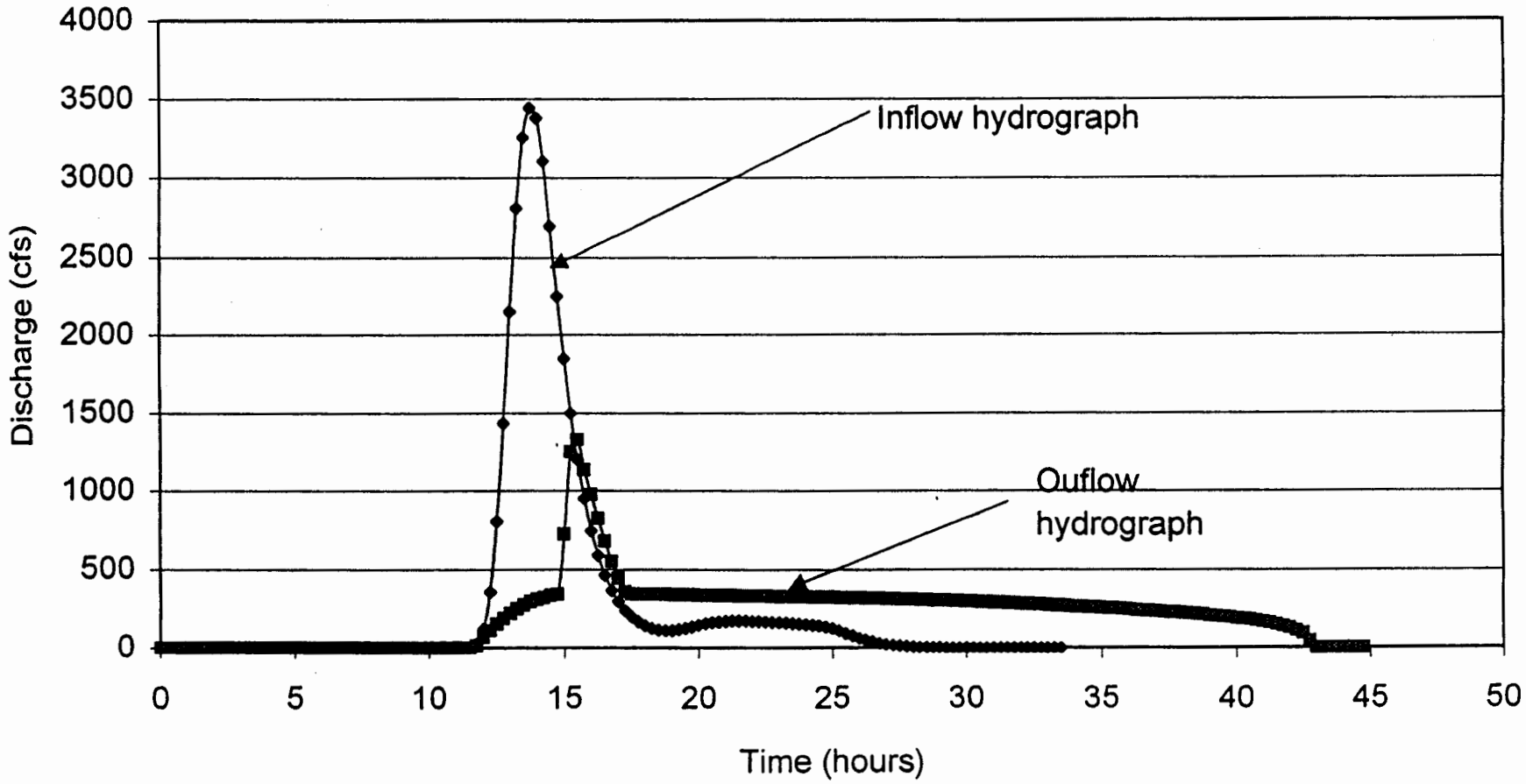


Figure 5-2
500-year Inflow and Outflow Hydrographs at the Reservoir using calibrated hydrology from E.S/ LANL



6.1 RESERVOIR AREA CAPACITY DATA

The reservoir area-capacity curve used in the C.O.E. HEC-1 model was developed from a 2' contour topographic map. This information was used by the HEC-1 model to determine storage volume in the reservoir.

6.2 FLOOD ROUTING 100-YEAR, 500-YEAR, PMF

Flood routing was performed by the U.S. Army C.O.E. HEC-1 hydrologic model. The models used a 15-minute time step in all HEC-1 simulations.

6.3 TAILWATER STUDY (AT STILLING BASIN)

A tailwater study was performed at the stilling basin, located downstream of the dam. The tailwater was predicted to be approximately 20-ft above the channel invert at the stilling basin location. This analysis was performed by utilizing a one-inch equals 200-ft topographic map with two-foot contours. Cross-sections were extrapolated from the topographic map at the stilling basin location and at ten separate locations downstream for approximately two-miles. These cross-section were input into the U.S. Army C.O.E. HEC-RAS model. The model assumed a normal water surface slope of 0.02 ft/ft, which is approximately the channel slope. The model was executed with a 16,000-cfs discharge. This discharge resulted in a 20-ft depth at the stilling basin.

6.4 SPILLWAY HYDRAULICS (CHUTE WATER SURFACE PROFILE)

A large portion of the downstream RCC face of the dam is used for the spillway chute floor. The centerline of the stilling basin is aligned with the center axis of the valley so spillway flows are directed in the center of the stream. The spillway is designed for the PMF with a peak of 16,000 ft^3/s. The tailwater elevation for this flow is 6940.0 feet.

6.4.1 Crest Configuration

The top crest width of the RCC dam is 10 feet; its elevation is 6996.0 feet. The upstream edge of the crest is cambered to help direct the flow over the crest. The upstream earth embankment is sloped from the camber to the reservoir. The discharge coefficient for this crest shape is taken as 3.10, from the U.S. Bureau of Reclamation Design of Small Dams, for a crest with zero approach depth.

6.4.2 Crest Length

The top of the dam suitable for passing flows is approximately 164 feet long. Consideration was given to using this entire length as the crest to pass the PMF peak. This results in a maximum head of 7.0 feet on the crest. Calculations to determine the path of the water down the face of the dam from this wide crest are difficult to predict. The contact of the dam and the left abutment makes an angle of 115 degrees with axis of the stilling basin. The water must turn from coming straight down the 1-to-1 slope of the dam onto the 1-to-1 (approximately) face of the left

7.1 MATERIAL PROPERTIES

7.2 RCC MIXES

7.3 RESULTS OF TESTS

7.4 RECOMMENDED MIX DESIGN

8.1 PLAN FOR RCC TEST SECTION

8.2 RESULTS OF TEST

- Drawing 1 – Cover Sheet
- Drawing 2 – Damsite Topography and Survey Control
- Drawing 3 – Excavation Plan
- Drawing 4 – Foundation Treatment
- Drawing 5 – Dam Plan
- Drawing 6 – Dam Sections, Details
- Drawing 7 – Dam Details
- Drawing 8 – Spillway Details
- Drawing 9 – Outlet Tower Plan and Sections
- Drawing 10 – Outlet Tower – Details
- Drawing 11 – Outlet Tower – Reinforcement
- Drawing 12 – Standard Reinforcement Drawing

[1] Hansen, Kenneth D., William G. Reinhardt, Roller Compacted Concrete Dams. McGraw-Hill, Inc. 1991.

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[3] FERC Guidelines. Federal Energy Regulatory Commission, Office of Hydropower Licensing, "Engineering Guidelines for the Evaluation of Hydropower Projects", Washington D.C., April 1991

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[5] U.S. Department of the Interior, Bureau of Reclamation, Engineering Monograph No. 19, Design Criteria of Concrete Arch and Gravity Dams. Denver, GPO, 1977.

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[8] Chopra, Anil K., Gregory Fenves. "Simplified Analysis for Earthquake Resistant Design of Concrete Gravity Dams", University of California, Berkeley, UCB/EERC - 85/10, June 1986

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