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DISCLAIMER

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TABLES

ACRONYMS

ANS	American National Standard
ANSI	American National Standard Institute
ASTM	American Society for Testing and Materials
cfs	cubic feet per second
COE	U.S. Army Corps of Engineers
CRWMS	Civilian Radioactive Waste Management System
DIRS	Document Input Reference System
DOE	Department of Energy
DTN	Data Tracking Number
ESF	Exploratory Studies Facility
EIS	Environmental Impact Statement
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
ft	feet
ft/s	feet per second
FY	Fiscal Year
HEC	Hydrologic Engineering Center
HEC-RAS	Hydrologic Engineering Center, River Analysis System
HMR	Hydrometeorological Report
ICD	Interface Control Document
LA	License Application
M&O	Management and Operating Contractor
NOAA	National Oceanic and Atmospheric Administration
NPP	North Portal Pad
NRC	Nuclear Regulatory Commission
NRCS	Natural Resources Conservation Service
OSA	Onsite Storage Area
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
QARD	Quality Assurance Requirements and Description
SCS	Soil Conservation Service

SDD	System Description Document
SN	Scientific Notebook
SRP	Standard Review Plan
TDMS	Technical Data Management System
USBR	U.S. Bureau of Reclamation
USDA	U.S. Department of Agriculture
YMP	Yucca Mountain Site Characterization Project

1. PURPOSE

The Hydrologic Engineering Studies for the North Portal Pad (NPP) and Vicinity were performed to provide design information for the proposed surface facilities for the project. Knowledge of flood potential and inundation extent is necessary in designing the surface facilities because of the risk of flood damage to the facilities, as well as the flood transport of radioactive materials away from the facilities. This hydrologic analysis is part of Work Package P4D1226TH2 under *Technical Work Plan for: Testing and Monitoring* (BSC 2001).

The surface facilities include an Onsite Storage Area (OSA) and railroad trestle alignments, which cross several natural channels that drain Midway Valley. Because of the critical nature of materials that will be at the site, analyses of runoff associated with a Probable Maximum Flood (PMF) event will indicate whether the surface facilities will allow safe conveyance of storm water runoff. The purpose of this study is to:

1. determine the magnitude and duration of runoff that would occur in Yucca Wash and Midway Valley Wash during a PMF event; and

2. determine flow characteristics during the PMF, including the maximum lateral extent of inundation and flow depths, velocities, sediment transport, and scour that may impact the surface facilities.

This study was conducted in response to Interface Control Document (ICD) No. 138 (CRWMS M&O 1998a,) and includes the following three parameters:

1. OSA Alternative Configurations
2. PMF of Onsite Storage Facility
3. PMF of Surface Facilities Design

For ease of discussion, these parameters have been designated, in this report, as ICD-138-1, ICD-138-2, and ICD-138-3, respectively. ICD138-1 requires an evaluation of alternative layout configurations for the OSA. ICD-138-2 is an analysis of the runoff associated with PMF events, to verify that surface facilities will allow the safe conveyance of storm water runoff around OSA. ICD-138-3 is an assessment of PMF water surface elevations for alternative designs of surface facilities, primarily located adjacent to NPP, including the railroad and highway embankment and bridge.

1.1 PREVIOUS INVESTIGATIONS

Four principal flood hazard analyses have been conducted for the Yucca Mountain Site Characterization Project (YMP) (Squires and Young 1984; Bullard 1992; Blanton 1992; Glancy and Beck 1998). In particular, the 1992 study by Blanton developed flood profiles and inundation maps for Midway Valley in the vicinity of the NPP. These flood hazard assessments were based on natural, unmodified topography. Since then, the area in the vicinity of the North Portal Pad has been modified by construction of the pad and placement of spoil from the Exploratory Studies Facility (ESF). Further modifications in the area are planned, including construction of a rail access to the pad and to an OSA (also known as the Centralized Interim Storage Facility). Figure 1 presents a map showing approximate locations of existing and proposed YMP facilities. Locations of the NPP and OSA shown on Figure 1 were transferred to topographic maps from the *Repository Surface Design Site Layout Analysis* (CRWMS M&O 1998b, pp. I-4 and I-8). The facilities include the NPP operations area, railroad/highway bridge, and the OSA.

The railroad access is planned to be constructed near the NPP, northeast across Midway Valley, as shown on Figure 1, and will impede the flow of storm water runoff through Midway Valley. At the OSA, fill material will be placed on the site and used for interim surface storage of nuclear waste material until construction of the permanent subsurface storage facilities are complete. This facility will intercept and redirect surface runoff in Midway Valley and possibly Yucca Wash.

The previous flood hazard evaluations did not consider flood potential associated with the existing and planned surface modifications. In addition, design of OSA should consider potential impacts on downstream facilities at NPP and rail access areas. The evaluations herein are to support: (1) siting and design of repository surface facilities, (2) surface facility design aspects of the repository License Application (LA), (3) providing information regarding surface water hydrology for inclusion in the Safety Analysis Report accompanying the License Application for the site as required by 10 CFR 63.21 (c) (1) (iii), and (4) evaluation of naturally occurring hazards in the Preclosure Safety Analysis of the Geologic Repository Operations Area as required by 10 CFR 63.112

1.2 SCOPE OF WORK

Development of this analysis is covered in the Technical Work Plan (BSC 2001). The initial entry for Scientific Notebook (Lee 1999, p. #1-1) provides the following tasks to be accomplished:

1. Collection, analysis, and documentation of characteristics of Midway Valley Wash from the headwaters to downstream from the surface facilities

2. Completion of PMF rainfall-runoff analyses of all areas tributary to OSA and railroad/highway bridge

3. Detailed routing analyses of the PMF hydrographs to determine critical flow characteristics at key locations upstream and pass the surface facilities (these flood routing analyses will consider the affects on flow characteristics of sediment that would be transported in the flow during a PMF event)

4. Detailed analyses of potential scour at the proposed railroad/highway bridge foundation.

Scope items 2, 3 and 4 directly respond to ICD-138-2 and 3. Scope item 1 is necessary to complete the other items. ICD-138-1 was responded to by routing predicted PMF flows around alternative OSA configurations.

Results of this study and design recommendations in this report are preliminary, having been prepared prior to the development of the site layout and grading plan for License Application design. Results of these studies should be continually reviewed against the evolving design effort.

2. QUALITY ASSURANCE

An AP-2.21Q Activity Evaluation was conducted for this Scientific Analysis and is included in the Technical Work Plan (BSC 2001, Attachment I). The evaluation concluded that this analysis is quality affecting and is therefore subject to the Quality Assurance Requirements and Description (QARD) (DOE 2002b).

This analysis does not involve items or natural barriers as defined in AP-2.22Q *Classification Criteria and Maintenance of the Monitored Geologic Repository Q-List* and therefore the quality level of these items is not applicable.

Data for the report was collected in an electronic format and thus the requirements given in AP-SV.1Q *Control of the Electronic Management of Information* apply. The Technical Work Plan (BSC 2001) indicates that controls shall be implemented as applicable. However, most of the controls mentioned in BSC 2001 are not needed as the input and outputs that constitute the electronic information for this analysis are printed immediately after use and pasted into Scientific Notebooks. Because of the brief residence time in electronic format before conversion into a hard copy, the requirements for protection from damage, retrievibility, storage, transfer, accuracy, security, and error free transmission are all met.

3. USE OF SOFTWARE

All software listed in Table 3-1 was obtained from Software Configuration Management. The software was appropriate for the applications described in this report, and the software was used within its range of validation. The computer used was a Compaq 486 with Windows 95 Operating System and is located in the URS office in Oakland, California.

Table 3-1. Software Usage

Name	CSCI Identifier
HEC-1 Version 4.0	30078-V4.0
HEC-RAS Version 2.1	30079-V2.1

A hard copy of the computer files are located in the Scientific Notebooks SN-M&O-SCI-005-VI (Lee 1999, Section 3), SN-M&O-SCI-010-VI (Palhegyi 1999, Section 3), and SN-M&O-SCI-042-V1 (Kramer 2002) prepared for these analyses.

3.1 PROBABLE MAXIMUM FLOOD CALCULATION

The U.S. Army Corps of Engineers' Hydrologic Engineering Center (HEC) HEC-1 computer software, Version 4.0 (U.S. Army Corps of Engineers 1990) was used to perform the rainfall-runoff simulations, with Probable Maximum Precipitation (PMP) amounts determined using procedures and data presented in the National Oceanic and Atmospheric Administration's Hydrometeorological Report No. 49 (HMR 49) (Hansen et. al. 1977). HEC-1 is the most widely used rainfall-runoff software among all the Federal Emergency Management Agency (FEMA) accepted hydrologic software. The software is used where regional flood flow frequency reports have not been provided or are not applicable (FEMA 2001b).

HEC-1 was designed to simulate the surface runoff response of a watershed to precipitation. The program represents the watershed as an interconnected network of hydraulic and hydrologic components. A component may be a sub-area of the watershed, river channel, reservoir, or diversion. Each component is described by its physical characteristics and mathematical relations that describe the pertinent hydrologic and hydraulic processes. In the HEC-1 software, the study area was divided into drainage sub-areas so that PMF hydrographs were calculated at key locations in the study area.

3.2 FLOOD INUNDATION CALCULATION

The U.S. Army Corps of Engineers, Hydrologic Engineering Center, River Analysis System software (HEC-RAS), Version 2.1 (Brunner 1997), was used for the flood inundation analysis. This program is designed for flood inundation studies and flood risk analysis. This software performs standard backwater computations to predict water surface elevations under steady gradually varied flow conditions. HEC-RAS is one of the FEMA nationally accepted computer software that can be used to estimate flood elevations (FEMA 2001a).

The computational procedure used in HEC-RAS is based on solution of the one-dimensional energy equation. Energy losses consist of surface roughness and expansion/contraction losses.

Energy loss by surface roughness is evaluated using Manning's equation and requires the user to define a roughness coefficient. The momentum equation is used in situations where flow is rapidly varied, such as hydraulic jumps and flow through bridges. A rigid channel boundary is used in the computations (i.e., channel cross section shapes do not change as a result of sediment deposition or scour).

3.3 OTHER SOFTWARE

Microsoft Excel 97 was used to perform support routine calculations such as those presented in Tables 4-2 and 6-2. The inputs and equations for the calculations are provided in the respective tables. Microsoft Excel 97 is an exempt software product in accordance with AP-SI.1Q, *Software Management*, Section 2.1.1.

4. INPUTS

4.1 DATA AND PARAMETERS

A distinction will be made in this analysis between inputs needed for the overall report and inputs needed to run the HEC-1 and HEC-RAS computer software. This distinction is made to clarify how the inputs were developed and their use. The qualification status of inputs used to the report are provided in the Document Input Reference System (DIRS) and in the Technical Data Management System (TDMS). Inputs to the computer programs were developed from inputs to the report, from a review of literature, or from other qualified data.

4.1.1 Inputs to the Report

4.1.1.1 Geographic Data of Watershed

Geographic data required to define the watershed at the project site are determined from Data Tracking Number (DTN)# MO9906COV98462.000, topographic contours developed for the Yucca Mountain Site Characterization Project. Measurements are taken from the contour maps to develop inputs for the computer software, as described in Section 4.1.2.

4.1.1.2 PMP Hyetograph

Using HMR 49 (Hansen et al. 1977) procedures with the geographic data (namely the area and location of watershed) described in Section 4.1.2.1, the local storm PMP for the NPP tributary drainage area was calculated to be 13.2 inches with a duration of 6 hours (Lee 1999, p.#2-5). The general storm PMP corresponding to the larger regional watershed was estimated to be about 5 inches for a 6-hour duration (Lee 1999, p.#2-5). The local storm has a higher precipitation intensity; thus, the local storm PMP of 13.2 inches was used in these studies because it results in the most critical PMF.

Two temporal distributions of PMP are recommended in HMR 49. The U.S. Weather Bureau's temporal distribution places the maximum intensities in the middle of the storm period. The U.S. Army Corps of Engineer's (COE) temporal distribution has the peak occurring later in the storm and hence is more conservative because the influence of initial rainfall abstractions will gradually dissipate later into the storm period. This distribution was therefore chosen for this study, since a high level of assurance against potential damage from extreme runoff is warranted. Figure 2 (from Kramer, 2002, pp. 31-32) presents the precipitation hyetograph developed for this study based on HMR 49, which is a plot of rainfall intensity versus time. A 3-minute increment was chosen for the hyetograph because it governs the time step used to calculate the flood hydrograph ordinates and therefore the accuracy in describing the rising limb of the hydrograph.

4.1.2 Inputs to the Computer Software

4.1.2.1 Physical Properties of Watersheds

The tributary drainage area for the NPP under existing conditions is about 4.2 square miles and centered approximately at Latitude 36°52′ and Longitude 116°29′, as determined from DTN# MO9906COV98462.000, topographic contours developed for the Yucca Mountain Site Characterization Project. The larger regional watershed tributary to the surface facilities at the project site encompasses Yucca Wash, Midway Valley Wash, Drillhole Wash and Boundary Ridge and has an area of 31.2 square miles and is estimated from the same source. These geographic data were used to obtain the PMP values.

The study area was divided into 7 sub-areas to provide information on flows at several key locations. The delineation of these sub-areas is illustrated on Figure 3 (from Lee 1999, p. #4-23). Figure 4 is a schematic diagram of the watershed network showing how the sub-areas are connected by the channels. The flow bifurcation at the basin outlet of sub-area D3 is handled by a diversion component, designated as DIV on Figure 4, in the HEC-1 setup. As a result of flow splitting, part of the runoff from D3 will combine with the hydrographs of S1 and ND4 at flow concentration point CP2, while part of it (i.e. the DIV component) will combine with the hydrograph of ND2 directly at flow concentration point CP3. The OSA was included in sub-area FAC, which denotes the boundaries of modified topography along the periphery interceptor ditch.

Physical properties, including area, main channel length, highest and lowest elevations of the basin, were obtained from the topographic data, DTN# MO9906COV98462.000 and are summarized in Table 4-1. The size of each sub-area was estimated by planimeter measurement. Note that the sum of these sub-areas is 5 mi^2, larger than the calculated tributary area for NPP because (1) the two sub-areas, FAC and NP1 do not contribute to the flow at NPP, and (2) the area of ND4 is slightly adjusted for the post-construction condition to account for the presence of OSA. Use of a smaller area for PMP estimate will result in a slightly more intense local storm event, and hence will provide a more conservative estimate of the flood elevations.

The area of each sub-basin was a direct input for the HEC-1 software while the other parameters were intermediate data for subsequent computation of inputs for the program.

Table 4-1. Characteristics of Sub-Areas Used in HEC-1 Software

Sub-Area Name (See Figures 3 And 4)	Area ($Mile^2$)	Flow Path Length (Miles)	Elevation At Head Of Basin (Feet)	Elevation At Basin Outlet (Feet)	Average Sub-Area Channel Slope (Feet/Mile)[a]	Average Channel Slope (%)
ND1	1.31	3.835	5,295	3,670	423.7	8.0
ND2	0.60	2.083	4,775	3,724	504.5	9.6
ND4	0.86	2.689	4,950	3,875	399.7	7.6
D3	1.18	3.239	5,760	3,905	572.8	10.8
S1	0.42	1.705	4,760	3,796	565.6	10.7
NP1	0.26	0.947	3,770	3,670	105.6	2.0
FAC (Overland Flow Element)	0.37	Not applicable	Not applicable	Not applicable	Not applicable	Not applicable

a. Average slope calculated as the difference in elevation between channel headwater and basin outlet divided by flow path length.

Source: Lee 1999, p. #4-37

4.1.2.2 Lag Time

Lag time is the time difference between the occurrence of the center of mass of excess rainfall and the peak of the unit hydrograph. It is commonly used to define the shape of the unit hydrograph for a particular drainage basin. There are several formulae available to calculate lag time. Five commonly used formulae were evaluated for these studies. Lag times estimated using each formula and the sub-area characteristics used to calculate the lag times are summarized in Table 4-2. Area, channel length and channel slope are taken from Table 4-1.

Table 4-2. Comparison Between Different Lag Time Formulae

Basin Name	A= Area (mi^2)	L=Total channel length (mi)	L_{ca}[a] (mi)	S= Slope (ft/mi)	Basin Factor, $LL_{ca}/S^{0.5}$	USBR Lag[b] (C=1.1) (hrs)	Snyder Lag[c] (C_t=1.8) (hrs)	Snyder Lag[c] (C_t=2.2) (hrs)	Kirpich[d] Lag (hrs)	Williams[e] Lag (hrs)	SCS[f] Lag (hrs)
ND1	1.31	3.84	1.70	423.7	0.32	0.75	3.16	3.86	0.43	1.32	1.25
ND2	0.60	2.08	0.83	504.5	0.08	0.47	2.12	2.60	0.25	0.75	0.70
ND4	0.86	2.69	1.32	399.7	0.18	0.62	2.63	3.21	0.33	0.97	0.97
D3	1.18	3.24	1.52	572.8	0.21	0.65	2.90	3.55	0.33	1.06	0.94
S1	0.42	1.70	0.75	565.5	0.05	0.42	1.94	2.37	0.20	0.62	0.56
NP1	0.26	0.95	0.44	105.6	0.04	0.38	1.38	1.69	0.25	0.51	0.81

Source: Lee 1999, p. #4-50

a. L_{ca} = length along the flow path from the basin outlet to the point opposite the centroid of the basin area.

b. $\mathrm{Lag} = C\left(\frac{LL_{ca}}{\sqrt{S}}\right)^{0.33}$, where C is a coefficient, L and L_{ca} are in miles and S is in ft/mi (Duncan et al. 1987, pp. 29-38).

c. $\mathrm{Lag}=C_t(LL_{ca})^{0.30}$,where C_t is a coefficient, L and L_{ca} are in miles (Linsley et al. 1982, p. 223).

d. Lag=0.60*Tc, where $Tc = 0.0078L^{0.77}S^{-0.385}$, and is the time of travel from the farthest point in the watershed to the outlet in minutes, L is in miles and S is in ft/ft (Maidment 1993, pp. #9-16, and #9-23).

e. Lag = 0.60 * Tc, where $Tc=21.3LA^{-0.1}S^{-0.2}$ in minutes, L is in miles, A is in mi^2, and S is in ft/ft (Maidment 1993, pp. #9-16, #9-17, and #9-23).

f. $\mathrm{Lag} = \frac{L^{0.8}(St+1)^{0.7}}{1900S^{0.5}}, St = \frac{1000}{CN} - 10$. A conservatively high value of curve number (CN) of 80 was estimated for the sub-areas. L is in feet and S is in percent. (Viessman et al. 1997, pp. 139-140)

In general, lag time values computed using the U. S. Bureau of Reclamation (USBR) empirical formula (Duncan et al. 1987, pp. 29-38) were the smallest, and therefore, most conservative (i.e., produce the largest peak flow). The only exception is the Kirpich formula (Maidment 1993, p. #9.16), but this formula was derived based on data from small agricultural drainage areas, which do not resemble conditions at the study site. The Williams (Maidment 1993, pp. #9.16 and 9.17) and SCS (Viessman et al. 1997, pp. 139-140) formulae could be applicable to the study site but they would result in less conservative peak flows. Based on the comparisons given in Table 4-2, the USBR empirical formula for lag time was used in the HEC-1 software.

4.1.2.3 Physical Properties of Interconnecting Channels

Physical characteristics of the natural channels are required for the HEC-1 software to perform hydrograph routing between flow concentration points and are shown in Table 4-3. The Muskingum-Cunge method (U.S. Army Corps of Engineers 1990, pp. 40-41) was used to route flows in natural channels and requires the length, slope, and dimensions of the channels which were obtained from the topographic maps (DTN# MO9906COV98462.000).

Manning's n in HEC-1 software represents the composite roughness of the main channel and flood plains. A value of 0.055 was selected for the natural channels, based on recommended values in Chow (1959 pp. 101-123), which includes the effects of channel bottom, degree of irregularities, and the vegetation density. The selected value is consistent with roughness coefficients reported in other studies conducted for the same general drainage area. For example, the range of Manning's n used for Drillhole Wash and Yucca Wash was between 0.030 to 0.055 (Blanton 1992; Squires and Young 1994), and was 0.038 to 0.055 for Topopah Wash (Christensen and Spahr 1980). Note that different values of Manning's n were specified in the HEC-RAS software, as discussed in Section 5.9. However, the results from HEC-1 are relatively insensitive to the choice of Manning's n within the appropriate range, with less than 0.1% difference in the estimated discharge when the Manning's n was lowered from 0.055 to 0.045.

Note that ND4S1 is not included in Table 4-3 because it is not an existing channel and is therefore discussed in Section 5.4.

Table 4-3.Characteristics of Routing Channels Used in the HEC-1 Software

Channel Name (Refer To Figures 3 & 4 For Locations)	Length (feet)	Slope (ft/ft)	Manning's n	Shape	Bottom Width (feet)	Side Bank Slope (h:1)
D3CP2	2400	0.0421	0.055	TRAP	50	0
CP2CP3	2250	0.0325	0.055	TRAP	300	30
DIVCP3	4650	0.0389	0.055	TRAP	75	15
CP3CP4	2300	0.0261	0.055	TRAP	450	60
FACCP5	3500	0.0352	0.055	TRAP	100	10

Source: Lee 1999, p. #4-37

4.1.2.4 Percolation Rate

Except for impervious surfaces, a certain amount of precipitation will be continuously lost through infiltration into the ground. The rate of infiltration depends on soil type and degree of saturation of the soil. When saturated, soils have a minimum and relatively uniform infiltration rate. In-situ percolation tests can be used to estimate the infiltration rate during storm events.

The percolation rate of soils at the study site was obtained from DTN# SNF29041993001.002, percolation test data at the ESF Muck Storage Area. These data are presented in Table 4-4. Except for an extraordinarily high percolation rate for Test No. 6, percolation rates ranged from 1 to 3 inches per hour with an average of 1.8 inches per hour (note, each test was conducted in a different bore hole). Test 6 was excluded in the calculation of percolation rate because it would skew the average and lead to a higher percolation rate which would in turn result in a smaller flood flow. For each test, the soil was presoaked for 4 hours before running the percolation test.

Table 4-4.Results from Percolation Tests at the ESF Muck Storage Area

Test	Infiltration (min/in)	Infiltration Rate (in/hr)
1	20	3
2	40	1.5
3	60	1
4	30	2
5	40	1.5
6	6.7	9
Average (excluding Test No. 6)		1.8

Source: DTN# SNF29041993001.002

Other data exist that provided estimates on surface flux rates in other areas in the vicinity of the project site. DTN# GS960908312212.009 gives the surface fluxes at 14 different locations in Midway Valley ranging from 0.25 to 6.72 inches per hour, with the average being 2.3 inches per hour. DTN# GS950308312213.004 includes the surface fluxes at 2 locations in Fortymile Wash and one location in Pagany Wash, which is part of the upper terraces that drain to Midway Valley. Since the infiltrometer test only lasted for about 1 to 2 hours in Fortymile Wash, it might not be indicative of the ongoing infiltration rate under saturated conditions. The surface flux rate measured at Pagany Wash was 1.65 inches per hour. However, data from these two sources were not directly employed in the calculation of percolation rate for the HEC-1 software as the soil saturation conditions were unknown for these tests. Nonetheless, the surface flux rates presented in these sources are generally consistent with the percolation rate of 1.8 inches per hour.

4.1.2.5 Cross Sections

Cross sections were developed from DTN# MO9906COV98462.000 and from data published in an earlier study (Blanton 1992, Figures 3 to 9) for key locations in the study area, with modifications to reflect new facilities that were not included in the previous study or shown on the topographic contour maps. Sections were created along the main drainage channel from downstream of the proposed railroad bridge to upstream from the NPP area, as well as along the western and eastern boundaries of the OSA. Figure 5 (Palhegyi 1999, pp. #7-27 to 7-29) shows the locations of cross sections used in this study. The cross sections near the NPP are plotted looking downstream in Figures 6 through 11. Because the main channel splits into two branches near the NPP area, the cross-section is divided into the main channel (designated 2700a, 2200a and 1800a) and the side channel on the west flood plain (designated 2700b, 2200b and 1800b) at Stations 2700, 2200 and 1800. Figures 12 through 15 show the cross sections along the western and eastern boundaries of the OSA (Kramer 2002, pp. 273-275 and 282-284).

4.2 CRITERIA

The NRC siting criteria related to the potential floodway are identified in Section 1.1. Criteria from the *Monitored Geologic Repository Site Layout System Description Document* (SDD) (CRWMS M&O 1999a) are applicable to this analysis and relevant sections are presented below.

4.2.1 Elevation of Waste Handling Facilities

The layout shall locate the surface waste handling facilities above the probable maximum flood (CRWMS M&O 1999a, Section 1.2.1.6). It is therefore determined that a PMF analysis is warranted for the design of the NPP and other associated surface facilities.

4.2.2 Nominal Pad Slope

The nominal slope within the pad areas shall be between 2 percent and 3 percent (CRWMS M&O 1999a, Section 1.2.1.7) to ensure proper drainage from the site. The slope of the OSA was assumed to be 1.3 percent in the HEC-1 software based on the design layout (CRWMS M&O 1998b), as described in Section 5.4, because these criteria have not been established at the time of this analysis. In view of the non-conformance, a sensitivity simulation was performed to assess the impact of the pad slope to the peak flows (Kramer 2002, pp. 10-30). Since the resulting peak flows are essentially the same as those in the original analyses, it is concluded that the analysis is insensitive to the small difference in the pad slope and the results will be unaffected had the new criterion been incorporated in the simulations.

4.2.3 Configuration of Pads

The configuration of pads shall prevent pooling of water (CRWMS M&O 1999a, Section 1.2.1.8). Based on this statement, the NPP and OSA areas were specified to slope in the downstream direction.

4.2.4 Site Drainage to Protect Ramp and Other Features

Site drainage shall protect the ramp, ramp portal, shaft, and shaft collar areas from water inflow as a result of the probable maximum flood (CRWMS M&O 1999a, Section 1.2.1.9). As such, one objective of this analysis is to assess the flooding potential at the NPP under the PMF conditions.

4.2.5 Routing of Stormwater

Site drainage shall contain and route stormwater from natural surface water drainage ways around surface facilities and provide water drainage for systems located on pads (CRWMS M&O 1999a, Section 1.2.1.10). Therefore, the diversion channels around the NPP and the OSA areas were included in the software to convey the flood flows.

4.2.6 Runoff

Site drainage shall be designed for the runoff accumulated from the storms identified (below) (CRWMS M&O 1999a, Section 1.2.1.11). The focus of this hydrologic study is on the North Portal Pad area and therefore the PMF conditions were considered.

Table 4-5. Criteria for Selection of Design Storm from SDD

Site Drainage Area	Design Storm
North Portal Pad and associated restricted area, South Portal, and Shaft Areas.	Probable Maximum Storm
Remaining areas not listed above.	100-year, 6-hour storm

4.3 CODES AND STANDARDS

Section 2.4.3 of NUREG-0800 (NRC 1987): This document sets forth the acceptance criteria for the standard review plan (SRP) section on hydrometeorological design basis for determining the extent of flood protection required. It specifies that publications of the National Oceanic and Atmospheric Administration (NOAA) and the Corps of Engineers may be used to estimate the PMF discharge and water level condition at the site. As such, NOAA's HMR-49 procedures were adopted to calculate the PMP values and the Corps' HEC-1 and HEC-RAS computer programs were selected to evaluate the peak discharges and flooding conditions at the NPP area.

ANSI/ANS-2.8-1992: The American National Standard (ANS) for Determining Design Basis Flooding at Power Reactor Sites defined PMF as "the hypothetical flood that is considered to be the most severe reasonably possible, based on comprehensive hydrometeorological application of probable maximum precipitation and other hydrologic factors favorable for maximum flood runoff such as sequential storms and snowmelt". The definition of PMF in this analysis conforms to that adopted by the ANS.

5. ASSUMPTIONS

Conservative assumptions were made whenever data were not available to accurately estimate hydrologic or hydraulic characteristics of a component of the drainage system. Key assumptions in these analyses are presented in this section.

5.1 CHARACTERISTICS WITHIN THE WATERSHED

The PMP is assumed to be spatially uniform during the entire duration of the storm. In addition, the soils are assumed to be fairly homogeneous throughout the site albeit the channel and flood plain could have different properties. Because the size of the Midway Valley Wash watershed is relatively small, the assumption of spatial homogeneity is deemed appropriate and is used throughout the analyses. The sensitivity analyses in Section 6.2.4 provide some error bounds on the PMF estimate by varying the inputs in HEC-1 software regarding soil properties, and therefore no further confirmation is anticipated for this assumption.

5.2 LAYOUT OF SURFACE FACILITIES

The locations of the surface facilities, including the NPP, OSA and the bridge/railroad trestle are assumed to follow the *Repository Surface Design Site Layout Analysis* (CRWMS M&O 1998b) available at the time of these analyses. This assumption is used throughout because the analyses were dependent on the configuration of the surface facilities in relation to the flood plain.

The surface facilities shown in the *Engineering Files for Site Recommendation* (CRWMS M&O 2000a, Figure I-19), in the *Final Environmental Impact Statement for a Geologic Repository for the Disposal of Spent Nuclear Fuel and High-Level Radioactive Waste at Yucca Mountain, Nye County, Nevada* (DOE 2002a, Figure S-7), and in the *Repository Surface Design Engineering Files Report* (CRWMS M&O 1999b, Figure IV-3) show that the NPP and railroad trestle are in approximately the same positions as those that were used in this analysis (Figure 1). The shape and location of the OSA are defined as an approximate outline in CRWMS M&O 2000a, DOE 2002a, and CRWMS M&O 1999b and are thus only shown conceptually. For this analysis it is assumed that the OSA shown in Figure 1 is adequate as this represents the most detailed depiction of the area, and takes existing topography into consideration.

Although the final design of the surface facilities is not completed, the results from these analyses are valid as long as the locations and dimensions of the NPP and OSA are approximately the same. In fact, the predicted discharge and flood elevation at the surface facilities will not be changed if the NPP is extended in the downstream direction while the upstream boundaries are fixed. Likewise, a wider pad perpendicular to the flow direction will not affect the local results at the pad as long as it does not encroach onto the next channel. In addition, the OSA could be eliminated and not invalidate the analysis for the NPP. The OSA pad interferes with the channel flow and causes the maximum flood elevation to be higher than that without the pad. The elimination of the OSA thus makes this analysis conservative. The *Repository Surface Design Engineering Files Report Supplement* (CRWMS M&O 2000b, Attachment II) indicates that the OSA has been eliminated.

The location of the bridge/railroad trestle can impact the water surface elevation since streamlines converge as the flow approaches the restricted section at the bridge. However, the flood elevation and in turn the area flooded will not be significantly different if the alignment of the bridge/railroad trestle is moved up or down the channel.

This assumption does not require further confirmation.

5.3 SURFACE DRAINAGE AT NPP

An existing man-made channel is located along the northern boundary of the NPP and collects runoff from Exile Hill, and from Midway Valley Wash upstream from the NPP. The reach bordering the northern periphery of the NPP has a 20-foot wide bed, 2.25:1 side slopes, and an average depth of 6 feet as determined from the 1994 survey, DTN# RA950000000002.001. However, the existing alignment of the channel encroaches into the proposed NPP as opposed to diverting the flood flows around the pad. It is therefore assumed in these analyses that the existing channel will be realigned and/or reconstructed to conform to the proposed surface layout. A 20-wide, 2:1 side slopes channel was input into the software with a bottom slope of 0.5% based on the geometry of the partially built channel and was also assumed to be built against the edge of the proposed NPP to provide a conservative estimate of the flood elevation at the NPP. This assumption is used in Section 6.3.1.

The predicted maximum flood elevation at the northern boundary of the NPP is independent of the channel configuration because the total head was used to estimate the water surface elevation as discussed in Section 6.3.1. Unless a substantially smaller channel is installed, the predicted water profile alongside the channel will also be valid. It is logical to assume that the channel will be constructed to handle the anticipated flows above the NPP and the dimensions chosen for the channel in this analysis are reasonable for the anticipated flows. As such, this assumption does not require further confirmation.

5.4 SURFACE DRAINAGE AT OSA

The OSA is assumed to be constructed above the PMF water surface elevations such that flood flows from the upstream watershed will not inundate the OSA and instead be conveyed around the facility. This assumption will ensure compliance with the criterion described in Section 4.2.1. Furthermore, given the desired function of the interim storage area, the pad is assumed in the software as having impervious concrete or asphaltic surface, with drainage inside the sub-area serviced by a 10-foot wide manmade concrete-lined channel sloping at a gradient of 1.3% (as discussed in Section 4.2.2). Since the drainage arrangement inside the OSA is not expected to affect the PMF at the surface facilities, this assumption does not require further confirmation. This assumption is used in Section 6.2.2.

An interceptor ditch is anticipated to be constructed around the OSA to meet the drainage requirement discussed in Section 4.2.5. It is assumed to be rip-rapped with a Manning’s n of 0.04 (Chow 1959, pp. 111-112) and have a cross-sectional configuration as shown in Figure 16 based on some preliminary design concepts (Kramer 2002, pp. 265-289). The gradient of the ditch is assumed to be 0.5% along the northern boundary because the natural gradient is small.

This gradient matches the gradient of the existing channel constructed next to the NPP (see Section 5.3). The combined northern and western branch of the ditch is designated as ND4S1 in the HEC-1 software, as shown in Figure 4, with an average channel slope of 0.8% (to match the terrain). The assumptions are used in Sections 6.2.2 and 6.3.2. The interceptor ditch will be sized to accommodate the expected runoff from the area above the OSA. The assumptions made about its configuration are reasonable for the anticipated runoff. Therefore, this assumption does not need further confirmation.

5.5 GEOMETRY OF RAILROAD BRIDGE

The design for the railroad/highway bridge had not been developed when this analysis was conducted. Bridge dimensions are assumed on the basis of typical concrete freeway bridge designs. The assumed bridge dimensions are 4 feet diameter piers spaced 50 feet apart, all supporting a 6-foot thick box girder type bridge. The same design was used for both the railroad and the highway bridge. The combined width of the railroad and highway bridge is assumed to be 250 feet. The length of the bridge is assumed to be long enough to span the width of the main channel along the proposed alignment. This assumption is used in Sections 6.3.1, 6.3.4, and Attachment I.

As discussed in Section 5.2, the results of the analyses are valid if the final alignment of the railroad bridge is approximately the same as in this study. Also, the bridge dimensions in the final design do not need to match exactly to those used in the assumption as long as similar degree of obstructions to the flow is presented by the bridge and its piers.

This assumption does not require further confirmation.

5.6 UNIT HYDROGRAPH

The transformation of rainfall excess to sub-area runoff is governed by the shape of the unit hydrograph, which in turn is determined by the size, shape, topography, and hydraulic roughness of the sub-area.

No data are available on unit hydrograph characteristics specific to the Yucca Wash or Midway Valley Wash areas. Therefore, the Natural Resources Conservation Service (NRCS) Dimensionless Unit Hydrograph (U.S. Army Corps of Engineers 1990, pp. 23-24) is assumed in the current analyses to accurately depict the runoff response to excess rainfall. The basis for this assumption is that it is applicable to small drainage areas, such as the ones used in these studies. This method utilizes one parameter, the lag time, to define the shape of the unit hydrograph. No confirmation is required for this assumption as it only represents a method for defining the shape of the hydrograph. This assumption is used in Section 6.2.2.

5.7 CHANNEL LOSSES

Based on field observations (Christensen and Spahr 1980, pp. 4-5; Squires and Young 1984, pp. 6-11), channel beds in the study area are mostly gravelly sand, as would be expected in alluvial fan flood plain areas. Significant flow losses are expected as the runoff is conveyed down the channels, thereby reducing peak flows. Similar phenomenon of gradually disappearing flows was reported in Fortymile Wash in an independent study (Glancy and Beck 1998, pp. 56-57). Other channel loss data were also reported in the literature (Savard 1998, pp. 12-20). However, channel losses are assumed to be insignificant in this study because the amount of loss is indeterminate, and inclusion of losses would not be conservative. Since this assumption leads to the maximum peak flow, no confirmation is required. This assumption is used in Section 6.2.2.

5.8 INFILTRATION AND INTERCEPTION LOSSES

Different options are available in HEC-1 to simulate losses resulting from infiltration and initial abstraction (precipitation that is stored in depressions and intercepted by vegetation, structures, etc.). These losses represent the portion of precipitation not contributing to direct runoff and it is assumed that they can be approximated by an initial and a uniform loss rate for simplicity and reasonable accuracy. This assumption is used in Section 6.2.2.

5.8.1 Initial Abstraction

Initial rainfall loss is the volume of rainfall that is lost before any runoff commences. Initial rainfall loss includes interception (wetting), depression storage, and rainfall required to saturate the uppermost layer of soil before ponding at the surface occurs. Initial rainfall loss varies with the intensity of rainfall and antecedent moisture deficit of the soils, and thus, is to a certain extent, storm-specific.

A storm that occurred at the site in July 1985 was reported to produce only slight runoff after nearly 1 inch of precipitation (Bullard 1992, p. 7), indicating an initial abstraction of about 1 inch. An initial abstraction of 1 inch was selected for these analyses. This may not be the actual amount of losses during the PMF event; however, results from sensitivity analyses (Section 6.2.4) indicate that the peak flow at the NPP would be minimally affected by this parameter as long as its value is less than 2 inches, which equals the cumulative precipitation before the peak rainfall commences. As such, this assumption of initial abstraction does not require confirmation.

5.8.2 Infiltration Rate

The percolation rate of 1.8 inches per hour discussed in Section 4.1.2.4 represents an estimate of the saturated permeability of the soil which would be the lower bound of the infiltration rate during a PMP event. On the other hand, the tests were conducted using 4-foot deep slotted pipes that permitted horizontal movement of water out of the borehole, whereas during a storm event there will be downward infiltration only. As such, one-dimensional infiltration during a storm may be slower than the 3-dimensional rate measured during the tests.

Infiltration rate is related to the Hydrologic Soil Group of the soil. No soil surveys are available for Nye County in the YMP area. Although DTN# GS960408312212.005 provides soil classifications for the soils in the YMP area based on soil taxonomy, correlation between that classification system and the hydrologic soil group cannot be established. Therefore, the soils mapped by the U.S. Department of Agriculture, Natural Resource Conservation Service (NRCS) (formerly the Soil Conservation Service) in neighboring counties were examined (Borup and

Bagley 1976, Table 8; Candland 1980, Tables 7 and 8). Details of the data found in these reports are included in Appendix II to this report. Soils on the outwash plains, alluvial fans, and flood plains in the neighboring areas were classified as generally belonging to Hydrologic Soil Groups B and D, with saturated permeabilities from 0.6 to 2 inches per hour. The particle size distribution presented in the soil surveys was compared to particle size distributions for soils in Midway Valley (DTN# GS921283114220.014). In general, Midway Valley soils contain less fines and more gravels than soils described in the soil surveys, which means the soils at the site are probably more pervious, and therefore, have greater permeabilities.

Considering all these factors, a constant infiltration loss rate at 1.5 inches per hour is assumed for this study as it is considered a conservative estimate of the uniform loss rate for the YMP area. The influence of this selection on the peak flood estimate was addressed through sensitivity analyses as presented in Section 6.2.4, and thus no confirmation is required.

5.9 MANNING'S ROUGHNESS COEFFICIENTS

Manning's roughness coefficients have to be defined in HEC-RAS to evaluate energy loss due to friction. Three different values for Manning's n are assumed in the analyses to calculate a lower limit, upper limit, and best estimate of PMF flow conditions. The assumed values are used in Sections 6.3.1 and 6.3.2. Since a range of values for the roughness coefficients was evaluated in this study, this assumption requires no further confirmation.

5.9.1 Clear Water Flow (Lower Limit)

Typically, a single value for Manning's n is used for the main channel, and a different coefficient is used for the flood plains. Manning's n for clear flow conditions is based on typical values published in Chow (1959 pp. 101-123). A Manning's n value of 0.035 was used for the channel and a value of 0.05 was used for the flood plain. These values were selected based on ground cover and surface features (Christensen and Spahr 1980, pp. 4-5; Squires and Young 1984, pp. 6-10) observed at the site in January 1999 and were consistent with values used in previous studies.

5.9.2 High Sediment Transport (Best Estimate)

During the PMF, the bed form is assumed to be continually changing and transporting large quantities of sediment, and may uproot plants and carry them and debris in the flow. This process will have the effect of increasing surface roughness. Simons and Senturk (1992, pp. 197-371) lists Manning roughness coefficients for these forms and suggests increasing Manning's n by 0.02 over the value that would be appropriate for clear water conditions.

The degree of obstructions to the flow can also increase the Manning's n value. Obstructions include such things as debris deposits, exposed roots, floating vegetation that snag on downstream vegetation, and boulders. To account for obstructions under high sediment transport conditions, a value of 0.02 was added to the clear water value using recommended values from Chow (1959, pp. 101-123). The correction to the roughness coefficient due to sediment and debris transport is summarized in Table 5-1.

Table 5-1. Corrections to Clear Water Manning's n Values to Account for Sediment and Debris Transport

n Components	Description	Values Selected
n_0	clear water value	0.05
n_1	sediment transport and bed forms	0.02
n_2	obstructions and debris	0.02
n	total roughness coefficient	0.09

Source: Palhegyi 1999, pp. #3-44 to #3-47.

5.9.3 Mudflow (Upper Limit)

Under extreme sediment and debris transport conditions, a mudflow phenomenon may result in which the concentration of sediments in water is greater than 20 percent by volume. A mudflow behaves differently than clear water in that it has higher viscosity and internal shear stress.

Costa (1997, pp. 21-30) calibrated the National Weather Service flood routing software DAMBRK using the Manning's roughness coefficient as the calibration variable to simulate observed mudflows. Calibration was made at several sites having field data to predict travel time and inundation area. Although a range of n values from 0.07 to 0.35 was identified for individual site calibrations, Costa concluded that a constant Manning's n value of 0.16 provides the best fit when all data are combined and this is the value adopted in HEC-RAS for both the channel and the flood plains under mudflow conditions.

5.10 FLOW BULKING

The water surface elevations are unknown and thus a water surface is required to be defined at the stream boundary to start the calculation. The normal method from the HEC-RAS program is chosen, as the boundary is relatively far from the area of interest and the exact water surface at the boundary does not need to be precisely defined. Flow bulking is the increase in flow volume due to the entrainment of air and sediment in the flowing water. Flow bulking can cause an increase in water depth over what would occur under clear water conditions. Bulking must be considered when the concentration of air or sediment becomes large enough to increase the depth of flow beyond that determined from a clear water analysis.

O'Brien (1993, pp. 264-269) recommends using a sediment concentration of 10 to 15 percent by volume to account for sediment bulking in desert flows. This sediment load is equivalent to increasing the clear water flow rate by 10 to 15 percent. O'Brien suggests that the frontal wave can contain 20 to 30 percent sediment concentration by volume. However, he provides no basis for these percentages and these recommendations could not be verified with another source.

Schick (1995, pp. 209-218) studied alluvial fan flooding in Eilat, Israel, and reported average flood flow suspended sediment concentrations ranging from 15,000 to 40,000 milligrams per liter (mg/l) with a few cases as high as 100,000 mg/l. Typical range of sediment concentrations in rivers range from a few hundred to as high as 50,000 mg/l. Using Schick's highest reported concentration of 104,000 mg/l suggests a 4 percent increase in volume over clear water with no sediment present.

Flow bulking due to air entrainment (i.e., "white water") was also considered. Chow (1959, pp. 33-36) provides a relationship to compute entrainment of air in high flows. The Douma equation is a function of channel velocity and depth. Based on this relationship, the concentration of air entrained in the PMF appears negligible, and concentrations of air must be on the order of 25 percent by volume or more to affect depth of flow.

Review of the literature suggests that bulking may not be a significant factor affecting PMF flows at YMP. This is because a PMF will have too much water for bulking to be significant. The summary above suggests that bulking the PMF flow by 4 to 10 percent would be more than adequate, and a bulking factor of 10 percent is assumed. Therefore in this study, flows are increased by 10% to account for bulking for conservatism. This assumption is used in Sections 6.3.1 and 6.3.2. Since the choice of this parameter is based primarily on literature, no confirmation is required.

5.11 BOUNDARY CONDITIONS FOR THE HEC-RAS SOFTWARE

Normal flow conditions are specified at the upstream and downstream boundaries in the HEC-RAS software (i.e. the water surface slope is assumed to be equal to the channel ground surface slope). Ground slope, or channel slope, were determined from the topographic maps, DTN# MO9906COV98462.000. This is a reasonable assumption as the boundaries are not control sections, and is used in Sections 6.3.1 and 6.3.2. Since this assumption is only a method for specifying boundary conditions in the software, no confirmation is required.

5.12 SCOUR MECHANISM

The total scour occurring at a bridge comprises long-term degradation, contraction scour and local scour at piers and abutments. Long-term degradation is considered insignificant based on the degree of incision of the existing channels observed during the site reconnaissance and is therefore neglected. Secondly, it is assumed that the various scour components develop independently, i.e., the potential local scour is added to the contraction scour without considering the effects of contraction scour on the channel and bridge hydraulics. This assumption is valid provided that local scour is the dominant scour component. Furthermore, the PMF condition is assumed to be the critical flow that has the greatest scour potential as the water flow is greatest during the PMF event. The assumptions for scour analysis are used in Section 6.3.4 and no confirmation is required as they are part of the methodology used for computing scour.

5.13 MEAN SEDIMENT SIZE

The mean sediment grain size, D50 value, was assumed to range from 1.5 mm to 5 mm upon review of sediment data collected in channels upstream of the potential railroad/highway bridge (see Appendix III for details). The data indicate that the grain sizes are variable but coarse. Samples were collected following procedures that comply with the American Society for Testing and Materials (ASTM) procedure ASTM D422-63, *Standard Test Method for Particle-Size Analysis of Soils*. The lab was unable to perform ASTM D422-63 and requested a change to

ASTM C136-96a, *Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates*. As a result, the original sample sizes were too small and did not conform to ASTM C136-96a requirements. The scour analysis is not sensitive to the value of D50 within the specified range, which represents coarse sand to fine gravel and is considered appropriate for the bottom sediments in the channel based on field observations. This assumption is used in Section 6.3.4 and further confirmation is not required as long as the assumption in Section 5.5 is correct.

6. SCIENTIFIC ANALYSIS DISCUSSION

The results of the flood analyses are discussed in this section. Section 6.1 describes the hydrologic background at the project site. Section 6.2 provides an estimate of the magnitude of runoff at pertinent locations within the Midway Valley Wash during a PMF event. The peak discharges are subsequently used in the flood routing described in Section 6.3 to determine the flow characteristics under the PMF conditions, including extent of inundation, flow depths, velocities and scour that may impact the surface facilities.

6.1 BACKGROUND

Yucca Mountain is located in the upper Amargosa River basin in southwest Nevada, about 100 miles northwest of Las Vegas. Most of the onsite facilities associated with the Yucca Mountain Site Characterization Project (YMP) are located in Midway Valley Wash and portions of Yucca Wash. Both are tributaries to Fortymile Wash (located downstream from the project site).

Midway Valley Wash drains approximately 4.2 square miles above the NPP site (Lee 1999, p. #2-5). The wash has a dendritic-like drainage pattern consisting of one large primary drainage channel and several smaller tributary channels. The primary channel is the largest most pronounced channel in Midway Valley Wash. The tributary channels are smaller. There are also smaller branches that appear to have been formed by a separation in the flow from the primary channel. The primary channel joins Yucca Wash above its confluence with Fortymile Wash south of Alice Hill. Midway Valley Wash receives runoff from numerous, mostly unnamed washes from the surrounding hills.

Although Midway Valley Wash is not identified as a true alluvial fan in that it does not have a single topographic apex and a fan shape, it has many other alluvial fan-like characteristics as described below. The surface of Midway Valley Wash is made up of unconsolidated sedimentary deposits and has multiple active channels. Flooding in an alluvial fan is a function of both water and sediment transport. Flooding typically begins at the highest point where flow is last confined, then spreads out as sheet flow and debris slurries, and can follow one or more multiple channels. Floodwaters can carry large amounts of sediment that can be deposited and fill an otherwise deep channel, forcing flows over banks and in new directions. The unconsolidated nature of sediments in Midway Valley Wash will likely allow rapid changes in width, depth, and location of channels during a PMF event.

The NPP is located along Exile Hill and is constructed mostly on fill material. This pad encroaches into Midway Valley Wash and could potentially restrict flood flows. The proposed railroad/highway bridge provides access to the pad. The railroad/highway enters Midway Valley Wash at its confluence with Yucca Wash south of Alice Hill and continues north along Alice Hill (south of the NPP), turns northwest, and continues on to the operations area. The railroad/highway will cross Midway Valley Wash primarily on an engineered embankment and partially on a bridge, forcing flood waters into a relatively narrow channel flowing near NPP. The proposed OSA is located in the northern portion of Midway Valley Wash. At its proposed location, the OSA will restrict and re-route flood flows in this portion of Midway Valley.

6.2 PROBABLE MAXIMUM FLOOD ANALYSIS

The PMF analysis consists of two parts: (1) calculation of probable maximum precipitation (PMP) and (2) using the PMP in a rainfall-runoff software to estimate the PMF at key locations in the watershed.

6.2.1 PMP Determination

The PMP for the site was determined using procedures described in HMR 49 (Hansen et al. 1977), which is considered to provide the best estimate of PMP potential (Cudworth 1992, p. 43). The method takes into account the meteorological conditions and atmospheric processes in a region, moisture-maximized rains of record, and broad-scale terrain features among other factors to determine a theoretically maximum amount of precipitation for a region or a local watershed. Using the watershed size and geographical location determined in Section 4.1.2.1, the estimated local storm PMP is 13.2 inches over a duration of 6 hours, which has a higher precipitation intensity than the general storm PMP, and is therefore used in the PMF determination. The temporal distribution of the total precipitation as presented in Figure 2 (Section 4.1.1.2) was developed based on recommendations in HMR 49.

6.2.2 PMF Determination

The appropriateness of HEC-1 to evaluate PMF discharges was discussed in Section 3. It is adopted in this study to simulate the surface runoff response of the Midway Valley Wash to the hyetograph developed for the local storm PMP. HEC-1 represents the watershed as an interconnected network of hydraulic and hydrologic components. As stated in Section 4.1.2.1, the study area was divided into 7 sub-areas to provide information on flows at several key locations based on the proposed surface layout (Section 5.2). The sub-areas characteristics including basin size (from Section 4.1.2.1), unit hydrograph as defined by the lag time (see Sections 4.1.2.2 and 5.6), and infiltration losses (from Section 5.8) were input into the software, along with the properties of channels that connect the sub-areas (see Sections 4.1.2.3 and 5.7).

The ICD calls for an evaluation of alternative layouts for the OSA (ICD-138-1). The OSA is to be constructed in Midway Valley Wash about 1 mile north of the North Portal Pad. Midway Valley Wash above the OSA drains about 0.86 mi^2 (Lee 1999, p. #4-24). Preliminary design concepts of the OSA call for a large fill area that occupies most of the wash cross section. The ICD also calls for an evaluation of surface facilities associated with the OSA to verify that they can safely convey the PMF around the OSA (ICD-138-2). These surface facilities also have not yet been designed and assumptions were made in these analyses, as described in Section 5.4. A detailed configuration of the OSA layout is not critical in the analysis presented in this report. Important aspects of the OSA design are its placement in Midway Valley Wash, as discussed in Section 5.2, and its elevation relative to the water surface elevations during the PMF.

In Alternative 1, it was assumed that the orientation of the OSA would be such that flows from Sub-area ND4 (Figure 3) would be intercepted at the north boundary in a new surface drainage channel and be conveyed westward along the north boundary to the west side of OSA and released back into the natural channel. Flows would then continue in the natural channel and combined with runoff from Sub-areas S1 and D3 at Flow Concentration Point CP2 and eventually towards the NPP area. This alternative closely follows the existing drainage patterns. In Alternative 2, a new surface channel is also provided, but the orientation of OSA would be such that intercepted flow from Sub-area ND4 is conveyed east to Yucca Wash and outside the study area. This means that the runoff from Sub-area ND4 would not be included in the runoff that ultimately goes to the NPP area.

Results of the rainfall-runoff analyses are summarized in Table 6-1. These results reflect the HEC-1 input parameters and assumptions discussed in Sections 4 and 5 and are herein referenced as base case conditions.

6.2.2.1 Alternative 1 Results

As previously described, in Alternative 1, runoff from Sub-area ND4 is conveyed to the west side of OSA and combined with runoff from Sub-areas S1 and D3. This alternative results in the maximum flow along the west side of the OSA (CP2) and at the NPP area (CP4) and railroad/highway bridge (CP6).

Table 6-1. Results of PMF Analysis

Flow Location (Figure 3)	Base Case Peak Flow	
	Alternative 1 (incld. ND4) (cfs)	**Alternative 2 (excld. ND4) (cfs)**
ND4	4,350	0
CP2	12,160	7,860
CP3	15,410	11,240
CP4	20,940	16,720
CP5	3,600	3,600
CP6	23,950	19,780

Source: Lee 1999, Attachment, pp. #1-19 to #1-21 and #14-17 to #14-19

The estimated peak PMF flow at the periphery of OSA at flow Concentration Point CP2 is 12,160 cfs. The maximum discharge at the proposed NPP, Concentration Point CP4, is about 20,940 cfs. Runoff from the OSA as well as the small sub-basin just south of OSA (designated as NP1 in Figure 3) is calculated to be 3,600 cfs. This flow is combined with the flow in the main drainage channel for a total flow of 23,950 cfs under the railroad/highway bridge.

6.2.2.2 Alternative 2 Results

In Alternative 2 it is assumed that runoff from Sub-area ND4 is conveyed east along the north boundary of OSA and discharged into Yucca Wash. The peak flow from Sub-area ND4 is 4,350 cfs. This diversion results in the calculated peak flow at location CP4 being 16,720 cfs which, when combined with flow from other sub-areas results in a peak flow of 19,780 under the railroad/highway bridge. It should be pointed out that the elimination of Sub-area ND4 will not reduce the peak flow at the bridge by 4,350 cfs because the peak flow at a concentration point is not the arithmetic sum of the peak flows from all the upstream sub-areas owing to the difference in lag times of the sub-areas.

6.2.3 Previous Studies

In a previous PMF study for the Yucca Mountain Site (Bullard 1992, Table 2) the PMF discharges were estimated to be on the order of 33,000 cfs. The difference between the current study and the previous study is largely associated with the choice of the C coefficient used in the lag time formulation. Review of the earlier study indicated that the selection of C values was made to give theoretical PMF flows that were larger than the estimated historical flows for 5 historical floods in the southwest desert region. Resulting C values ranged from 0.5 to 1.6 and historic peak flows were about 64% to 88% of the calculated PMF peak flows. The author subsequently used C value of 0.5 in the PMF prediction of the Yucca Mountain site (Bullard 1992, p. 6). Because PMF peak flows must be larger than historic flows, the low C values used in the analyses may have been unrealistically low to compensate for over-estimates of the historical flows or inaccuracies in other parameters used in the PMF analysis.

In the present analyses, a C value of 1.1 was selected (representing the mid-point of the range mentioned above). In the USBR formula, the value of the C coefficient was calibrated using 162 reconstructed flood hydrographs from natural basins throughout the contiguous United States west of the Mississippi River. Thirty-eight watersheds within the Great Basin were analyzed and the resulting calibrated C values ranged from 0.75 to 2.03 (Duncan et al. 1987, Fig 3-5). The Nevada Test Site is located within the Great Basin. Since a shorter lag time will result in a higher peak flow in the basin, a conservatively low C value of 1.1 was adopted for these studies. Only 4 of the 38 calibrated C values for the Great Basin were less than 1.1.

In the USBR formula, the C coefficient is approximately equal to 26 times the average Manning's n value for the basin, which represents the hydraulic roughness characteristics of the drainage network and includes both channel and flood plain roughness as well as drainage-network densities. As discussed in Section 5.9.1, the Manning's n for the drainage basin, for clear water conditions, was estimated to be 0.035 for the channel and 0.05 for the flood plain. Manning's n values of 0.035 and 0.05 correspond to C values of 0.91 and 1.3, respectively. For flow conditions that include consideration of sediment transport, the C value adopted for these studies would theoretically be larger, the lag time longer, and the estimated peak discharges less. However, the C value of 1.1 was held constant for all analyses, including those that considered sediment flow and mudflow conditions.

Although use of a C value equal to 0.5 for these studies would increase the calculated PMF peak flow and make it more conservative, the rationale for such a selection is not justified by available data. Based on the USBR relationship, a C value of 0.5 would be equivalent to a Manning's roughness coefficient of 0.019. This value for roughness coefficient is unrealistically low and therefore considered inappropriate for the watershed.

6.2.4 Sensitivity Analyses

Analyses were performed to evaluate the sensitivity of study results to variations in the initial rainfall loss, uniform rainfall loss rate, and lag time parameters. Comparisons were made on the peak discharge at the proposed NPP, at Concentration Point CP4. The results of these sensitivity analyses are summarized in Table 6-2 and are discussed below. Run 1 in Table 6-2 represents the base case conditions.

An initial rainfall loss of 1 inch was used for the base case conditions discussed in Section 5.8.1. Since the soils are mainly gravelly silt and remain unsaturated during the dry summer season when most thunderstorms occur in this area, an appreciable amount of initial precipitation would be needed to saturate the surface stratum. Higher losses of 2 and 3 inches were therefore examined. Calculated peak flows remained essentially unchanged until the initial loss exceeded 3 inches, when some portion of the peak precipitation was extracted as an initial loss. Runs 2 and 3, as presented in Table 6-2, reflect changes in peak flow of -0.4% and -7.2%, respectively.

A uniform rainfall loss rate of 1.5 inches per hour was used in the base case based on the assumption described in Section 5.8.2. Uniform loss rates between 0.05 and 2 inches per hour were simulated to evaluate the sensitivity of results to this parameter. The resulting peak flows ranged from 19,870 cfs to 24,820 cfs, where the high value is 18.5% above the base case peak flow and was produced with a loss rate of 0.05 inch/hour. As expected, peak flows increased with lower loss rates. However, given the percolation test data provided in Section 4.1.2.4, a uniform loss rate of 1.5 inch per hour is considered to be a reasonable conservative estimate.

The NRCS (SCS) loss method (U.S. Army Corps of Engineers 1990, pp. 18-19) was also investigated, in which loss is governed by the curve number of the soil. This method simulates a continuously decreasing infiltration loss. The curve number is selected based on the soil type and vegetation cover. For desert regions with moderately permeable soils and poor to fair sageous cover, curve numbers ranges from about 72 to 85 (USDA 1986, Table 2-2d). Peak flows predicted using this method were modeled and, as shown in Table 6-2, gave results that are comparable to the uniform loss rate method, with less than 10% change in the peak flow from the base case.

Lag time was calculated using the USBR formula (Duncan et al. 1987, pp. 29-38) with a C coefficient equal to 1.1. The lag time was found to be the most sensitive parameter in the determination of peak flow. Two sensitivity runs were performed (runs no. 12 and 13 in Table 6-2) using C coefficients of 0.5 and 1.3. The low value (C=0.5) was selected because it was used in a previous study (Bullard 1992, p. 6). The peak flow was greatly increased to 35,000 cfs, which is 67% higher than the base case. However, it is believed that a C value of 0.5 is unrealistically low for the reasons discussed in Section 6.2.3. On the other hand, a C value of 1.3 corresponds to a Manning's n of 0.05, which is the flood plain roughness used in this study. As shown in Table 6-2, the peak PMF flow is –11.9% lower than the base case conditions when a C value of 1.3 is used.

Table 6-2. Sensitivity Analyses of Input Variables on Peak Flow

(Shaded values are those changed from the base case)

Run No.	Loss Method	Initial Rainfall Loss (in)	Uniform Loss Rate (in/hr)	Curve Number	Lag Time Coefficient	Peak Q At CP4 (cfs)	Volume Of Runoff (Acre-feet)	% Runoff of Total Rainfall	Time Of Peak (hour)	% Change In Q From Base
1	uniform	1	1.5	-	1.1	20,940	1880	61.2	3.9	0.0
2	uniform	2	1.5	-	1.1	20,860	1853	60.4	3.9	-0.4
3	uniform	3	1.5	-	1.1	19,440	1730	56.4	3.9	-7.2
4	uniform	1	0.05	-	1.1	24,820	2794	91.0	3.9	18.5
5	uniform	1	0.5	-	1.1	23,620	2387	77.8	3.9	12.8
6	uniform	1	1	-	1.1	22,280	2098	68.4	3.9	6.4
7	uniform	1	2	-	1.1	19,870	1722	56.1	3.9	-5.1
8	SCS	-	-	72	1.1	19,470	2201	71.7	4	-7.1
9	SCS	-	-	77	1.1	20,930	2370	77.2	3.9	-0.1
10	SCS	-	-	79	1.1	21,500	2436	79.4	3.9	2.6
11	SCS	-	-	85	1.1	23,000	2628	85.6	3.9	9.8
12	uniform	1	1.5	-	0.5	35,000	1879	61.2	3.5	67.1
13	uniform	1	1.5	-	1.3	18,450	1879	61.2	4	-11.9

'-' = parameter not directly input into the HEC-1 software

Source : Lee 1999, p. #7-2

6.3 FLOOD INUNDATION

This section discusses the extent of flood inundation at the YMP site under PMF conditions and assesses whether the surface facilities will allow safe conveyance of stormwater runoff with minimal risk of flooding. Flood inundation potential of the NPP, the railroad/highway bridge, and OSA are discussed. There are two primary objectives of the flood inundation study:

1. Predict flood depths, velocities, and extent of inundation at NPP during the PMF.
2. Evaluate potential flood risk at OSA and recommend alternatives.

6.3.1 PMF Flow Characteristics at NPP

HEC-RAS software was employed using the cross-sectional data described in Section 4.1.2.5 and the assumed diversion channel geometry described in Section 5.3 to predict the flow characteristics in the vicinity of the NPP area during the PMF conditions. The assumed surface roughness coefficients (Manning's n) from Section 5.9 were input for the 1) clear water flow, 2) high sediment transport flow, and 3) mudflow scenarios. Only the base case peak flows determined for Alternative 1, in which flows in sub-area ND4 would be diverted around the west side of OSA, were evaluated. This would provide a realistic and yet conservative estimate of the inundation extent. A bulking factor of 10 percent as discussed in Section 5.10 was applied to the values presented in Table 6-1. Therefore, the bulked flow of 23,040 cfs at CP4 was specified for the reach upstream from the railroad bridge, and the combined bulked flow of 26,340 cfs at CP6 was specified at the railroad bridge.

Downstream from Station NPP2700, the main channel is divided into two channels separated by a low ridge. Depending on the flow conditions, there is a potential for flow bifurcation if the water surface elevation is higher than the ridge. Once flow splitting occurs, the flow characteristics in the side channel will be governed by its own energy grade line and independent of the main channel flow until it reunites with the main channel. Therefore, a separate HEC-RAS setup was used to simulate the flow in the side channel on the west flood plain. As shown in Figure 5, this side channel is intercepted by the man-made canal along the northern boundary of the NPP as shown in the cross section at Station NPP1800. The man-made canal discharges into the main channel upstream of the railroad bridge.

Split flows were proportioned based on the flow area in the two channels at the point of bifurcation. Bifurcation begins when the ridge separates the flow in the two channels. This was determined from the HEC-RAS setup developed for the main channel.

Table 6-3 summarizes the results of the flood routing analysis. Water surface elevations and flow velocities at selected cross sections are presented for the various scenarios. For cross sections at Stations NPP2700, NPP2200 and NPP1800, the water surface elevation was predicted from either the main channel setup or the west flood plain setup, whichever resulted in a more conservative estimate.

The effect of wind waves has not been included in this analysis. The transitory nature of the flooding, short fetch distance, and relatively shallow water depth make additional height of water from wind action to be very minimal.

Table 6-3. Flood Inundation Results

Cross Sections	Peak PMF Flow (cfs)	Clear Water Analysis: Flow Velocity (ft/s)	Clear Water Analysis: Water Surface Elevation* (ft)	High Sediment Transport Analysis: Flow Velocity (ft/s)	High Sediment Transport Analysis: Water Surface Elevation* (ft)	Mud Flow Analysis: Flow Velocity (ft/s)	Mud Flow Analysis: Water Surface Elevation* (ft)
North Portal Pad Area							
NPP3500	23,040	23.5	3729.3	11.0	3731.2	7.1	3733.1
NPP3200	23,040	23.5	3718.7	10.4	3721.9	6.9	3724.1
NPP2700	23,040	24.4	3706.7	11.8	3709.2	7.9	3710.7
NPP2200	23,040	19.5	3690.4	10.4	3692.3	6.3	3693.2
NPP1800	23,040	22.8	3676.3	7.9	3684.2	5.2	3684.7
NPP1700	23,040	12.6	3676.3	7.1	3678.1	4.3	3680.6
NPP1500	26,340	11.5	3675.1	10.5	3675.2	4.7	3679.5
NPP1400	26,340	10.0	3672.8	7.8	3673.7	3.7	3678.9
NPP1250	Bridge						
NPP1100	26,340	12.3	3663.4	11.9	3663.4	8.8	3664.7
NPP1000	26,340	21.3	3655.5	9.4	3659.0	6.3	3660.8
NPP0	26,340	19.7	3632.0	9.2	3633.7	5.9	3635.3

* Assumed North Portal Pad elevation = 3681.5 feet. Pad located at cross section NPP1800
Source: Kramer 2002, pp. 43-44.

6.3.1.1 Clear Water Flow Conditions

The northeast boundary of the NPP is located at about cross section at Station NPP1800 and the proposed railroad/highway bridge is located at about cross section at Station NPP1250. The results indicate that flow is supercritical upstream from these surface facilities. Velocities between cross sections at Stations NPP1800 and NPP3500 range from 19.5 ft/s to 24.4 ft/s. The flow immediately upstream from the bridge between cross sections at Stations NPP1400 and NPP1700 is subcritical, with velocities ranging from 10.0 ft/s to 12.6 ft/s. The depth of water upstream from the bridge ranges from 6 feet to a maximum of 11 feet at the bridge section. The assumed bridge design slows the flood flow, but does not cause flooding of the facility.

Figure 17 (from Palhegyi 1999, p. #3-101) shows the extent of flood inundation. The PMF flow is contained within the primary flood channel boundaries and does not cause flooding of the North Portal Pad.

Under this scenario, flooding of the North Portal Pad would not be expected.

6.3.1.2 High Sediment Transport Flow Conditions

The analysis of high sediment transport flow conditions provides a conservatively high, yet realistic, estimate of water surface elevations that may occur during a PMF. Sediment movement creates various streambed configurations or forms depending on the strength of flow and size of bed material. Bed forms are irregularities larger than the largest size particle forming the bed material. At low flows, the bed material can be stationary. As flow increases, sediment begins to move and bed forms develop. During high flows these bed forms can be several feet in height, but typically not higher than 50 percent of the flow depth.

Results of analyses using a Manning's n value that was increased to account for sediment and debris transport are also presented in Table 6-3. The results indicate that flow is subcritical at all cross-sections with lower velocities and larger depths than the clear water flow scenario. Velocities between cross-sections at Stations NPP1400 and NPP3500 range from 7.1 ft/s to 11.8 ft/s. The depth of water upstream from the bridge ranges from 8 feet to 12 feet.

Figure 18 shows the extent of flood inundation under high sediment flow conditions. In the lower reaches below the facility, the PMF is contained within the main flood channel boundaries. However, upstream from the facility the PMF flow has the potential to be divided into two channels, the main channel and a smaller side channel located to the west of the main branch.

The NPP is not flooded from high water elevations in the main channel. However, potential flooding is possible from flows in the side channel on the west flood plain.

The potential flood flow west of the primary channel is a small percentage of the total PMF, approximately 1980cfs. The calculated water surface elevation of this flow is assumed to be the same as the energy grade line and is approximately 3684.2 feet when it reaches the man-made canal at the western boundary of the NPP. This water surface elevation is about 2.7 feet higher than the proposed NPP elevation, and it decreases as the water is conveyed downstream by the canal. Since the flow enters the man-made canal at an approximately 90-degree angle, the flow will lose its momentum and all the kinetic energy will be converted to potential energy. Therefore, the predicted water surface elevation at section NPP1800 corresponds to the energy grade line at the NPP. In addition, it is possible that the loss of momentum will be accompanied by a deposition of sediment load in the canal, thus resulting in a higher water surface elevation than that presented in Table 6-3.

6.3.1.3 Mudflow Conditions

The analysis of mudflow conditions provides an estimate of the maximum water surface elevations that may occur during a PMF. This analysis is based on a study published by Costa (1997) on "Hydraulic Modeling for Lahar Hazards at Cascades Volcanoes". It is unlikely that Midway Valley Wash would experience a true mudflow because of the large flow of water during a PMF. Mudflows, as defined in this analysis, contain about 20% percent solids by volume, which means a significant volume of sediment would have to be present in 26,000 cfs of peak water flow to create the mudflow conditions. Although debris flows have been documented in the Yucca Mountain area (Coe et al. 1995, p. 1), the total cumulative rainfall for that event was 2.7 inches for the first 24 hours and .8 inches the next day, substantially less than the 13 inches of PMP used in this analysis.

As expected with increasing Manning's n, flow becomes subcritical with much slower velocities and larger flow depths than the clear water and high sediment transport scenario. Velocities between cross sections 1400 and 3500 range from 3.7 ft/s to 7.9 ft/s. The depth of water upstream from the railroad/highway bridge ranges from 9 feet to 17 feet.

Figure 19 shows the extent of flood inundation under mudflow conditions. The PMF inundates more of the flood plain west of the main channel. Potential flooding of the North Portal Pad surface facilities is greater than under the clear water and high sediment transport conditions.

Flooding of the NPP may not occur from high water elevations in the main channel adjacent to the North Portal Pad, although the water surface is projected to be the same as the elevation of the NPP (3681.5 feet). However, potential flooding appears possible due to flood inundation of Midway Valley Wash upstream from the pad and flood flow in the side channel in the west flood plain.

The estimated potential flood flow from the flood plain is 2,340 cfs, and has an elevation of 3,684.7 feet when it reaches the NPP canal, which is about 3.2 feet higher than the NPP elevation.

6.3.2 Flood Potential at OSA

The peak discharge that needs to be intercepted and routed around the OSA by a new diversion channel under PMF conditions will be approximately 4,790 cfs, with a 10 percent bulking factor applied to the flow at location ND4 given in Table 6-1.

The design of OSA and associated drainage facilities had not been developed at the time of these analyses. Cross sections described in Section 4.1.2.5 and assumptions made in Section 5.4 with regard to the man-made channel geometry were used to evaluate the flooding potential of different alternative layouts. Section 6.2.2 discusses the results of Alternative 1 and Alternative 2, which differ from each other in the direction the interceptor ditch drains. As presented in Table 6-1, the peak flow approaching the NPP will be reduced by approximately 20 percent if the interceptor ditch is aligned to drain eastward and discharges into Yucca Wash.

In both alternatives, the OSA is assumed to be constructed above the PMF water surface elevations such that flood flows from the upstream watershed will not inundate the OSA. Uniform flow is assumed along the northern boundary of the man-made triangular channel, and the depth of water is estimated to be about 7.5 feet (Kramer 2002, p. 265). This means that the top of the floodwall should be at least 8.5 feet higher than the bottom of the channel to provide one foot of freeboard.

Along the western boundary of the interceptor ditch, maximum flow will occur under Alternative 1 arrangement, where runoff from Sub-area ND4 is conveyed to the west side of the OSA and combined with runoff from Sub-areas S1 and D3 at Concentration Point CP2. A HEC-RAS setup was developed for the channel west of the OSA using a bulked flow of 13,380 cfs. Only the high sediment flow scenario was considered as it is the most likely condition under a PMF. Results indicated that the maximum water depth would be about 9 feet in the channel adjacent to OSA western boundary (Kramer 2002, p. 269). To allow for one foot of freeboard, the top of floodwall should be more than 10 feet above the bottom of the adjacent channel.

Along the eastern boundary of the interceptor ditch, maximum flow will occur under Alternative 2 arrangement in which runoff from Sub-area ND4 is diverted eastward to Yucca Wash and combined with runoff from the small catchment that drains to the natural channel adjacent to and east of the OSA. The peak discharge from the small catchment was estimated from that of Sub-area ND4 by proportioning the drainage areas and was approximately 420 cfs before bulking (Kramer 2002, p. 267). To be conservative, the total peak discharge in the channel was obtained by summing the flows from ND4 and the small catchment. Therefore, a bulked flow of 5,240 cfs was input into the HEC-RAS software for the natural channel adjacent to the eastern boundary of OSA. The predicted maximum depth is about 8 feet (Kramer 2002, p. 281) which requires that the top of the floodwall be at least 9 feet above the bottom of the channel to provide minimal freeboard.

6.3.3 Previous Studies

The inundation maps produced by the 1992 flood hazard assessment differ from Figures 17 to 19 presented in Section 6.3.1. Apart from the fact that the previous study was based on natural, unmodified topography, it is also because Blanton (1992, pp. 1-3) applied a factor of 2.0 to the PMF to account for bulking caused by sediment and debris (i.e., he doubled the PMF flow rate), whereas a 10 percent factor was used in this study. He did not provide any quantitative rationale for this value but based it on engineering judgment after considering the natural ground condition within the small drainage basins and the steepness of the drainage slopes (Blanton 1992, pp. 2-3). This bulking factor is apparently intended to account for all changes in flow regime resulting from a high sediment load. In the current study, Manning's n was increased and a smaller bulking factor was included to account for changes in flow regime due to high sediment load. It is therefore concluded that the approach taken by this analysis provides a realistic yet conservative estimate of the inundation extent under PMF conditions.

6.3.4 Scour Analyses

In addition to protection against flooding from peak flows during a PMF event, the potential for scour at the foundation of bridges should also be considered in the design of surface facilities. The depth of scour at the railroad bridge was estimated using procedures described in the Federal Highway Administration’s Hydraulic Engineering Circular No. 18 (HEC-18) (Richardson et al. 1993, pp. 27-44). The PMF is assumed to be the critical flow that has the greatest scour potential (Section 5.12).

Total scour at a bridge is caused by three different mechanisms: long-term aggradation and degradation, contraction scour, and local scour. Long-term aggradation and degradation are processes in which the bottom of the channel is continually adjusting to reach a state of equilibrium. Both are considered insignificant based on the degree of incision of the existing channels observed during the site reconnaissance and are therefore neglected.

Contraction scour occurs when the flow area of a channel is decreased from the normal either by a natural constriction or by a structure. The decrease in flow area is accompanied by an increase in average velocity and bed shear stress, and hence there is an increase in stream power at the contraction and more bed material is transported through the contracted reach than is transported into the reach. Contraction scour can also be caused by the approaches to a bridge cutting off the overland flow that normally goes across the flood plain during high flow. This latter case causes clear-water scour at the bridge section because the overland flow normally does not transport any bed sediments. For the high sediment flow scenario, the mean velocity immediately upstream of the bridge at NPP1400 is estimated to be 7.8 ft/s (see Table 6-3). Using the Neill's and Laursen's equations provided in HEC-18 with a median particle size D50 of 1.5 to 5 mm (Section 5.13), the critical velocity for beginning of motion is determined to range from 2.9 to 4.3 ft/s (Mineart 1999, p. #5-4). Since the mean velocity is greater than the critical velocity, the flow is transporting bed material and live-bed contraction scour will exist. The modified Laursen's equation for live-bed contraction scour is:

$y_2/y_1 = (Q_2/Q_1)^{6/7} (W_1/W_2)^{k1}$ (Richardson et al. 1993, Eq. 16, p. 33)

$y_s = y_2 - y_1$ (Richardson et al. 1993, Eq. 17, p. 33)

where y_s = average scour depth

y_1 = average depth in the upstream main channel, ft

y_2 = average depth in the contracted section, ft

W_1 = bottom width of the upstream main channel, ft

W_2 = bottom width of the main channel in the contracted section, ft

Q_1 = flow in the upstream channel transporting sediment, cfs

Q_2 = flow in the contracted channel, cfs

k_1 = exponent based on ratio of shear velocity to fall velocity of bed material

The ratio of flows in the upstream and contracted section is unity. The ratio of bottom widths is computed to be 1.087 based on the pier spacing and diameter given in Section 5.5. Using the flow depth and slope of the energy grade line at Station NPP1400 and the median particle size D50 (Section 5.13), k_1 is calculated as 0.69. Substituting the appropriate values into the equation, the contraction scour is estimated to be about 0.7 feet (Kramer 2002, pp. 8-9).

HEC-18 recommends using the CSU equation to calculate pier scour:

$y_s/a = 2.0\, K_1 K_2 K_3 (y_1/a)^{0.35} FR^{0.43}$ (Richardson et al. 1993, Eq. 22, p. 39)

where the required inputs are:

K_1 = correction factor for pier nose shape = 1.0
K_2 = correction factor for angle of attack=1.0
K_3 = correction factor for bed condition=1.2 (small to medium dunes)
a = width of pier (Section 5.5)
y_1 = water depth
y_s = scour depth
FR = Froude number (Kramer 2002, p. 39)

The sediment transport case was used for scour analysis because the worst case scenario for scour at the bridge piers is sediment flow at a high velocity, as opposed to the clear water and mudflow cases (see Table 6-3, Flood Inundation Results). FHWA (Richardson et al. 1993, pp. 38-39) recommends that the ratio of y_s/a be limited to 2.4. Application of this criterion results in a scour depth of 9.6 feet.

Based on the assumption that the scour components develop independently (Section 5.12), the contraction scour and the local scour are added to give a total scour depth of 10.3 feet at the railroad bridge under PMF conditions.

7. CONCLUSIONS

The following conclusions and recommendations are based on the results of the analyses described above, reflecting high sediment and debris transport conditions (the "best estimate" of flow conditions). Additionally, the following recommendations are, in some cases, based on assumed locations and configurations of proposed facilities and pertain specifically to the 1998 design. Results of these studies should be reviewed and refined during the LA and subsequent design efforts. Results of these analyses have been submitted to the TDMS in DTN #MO0209EBSPMFSD.029.

7.1 CONCLUSIONS

1. The peak runoff during a PMF that needs to be intercepted and routed around the northern boundary of OSA in a new man-made channel will be approximately 4,800 cfs. The maximum flows along the western and eastern boundary of the OSA are 13,400 cfs and 5,200 cfs, respectively. Predicted water depth ranges from 7.5 to 9 feet along the interceptor ditch.

2. The peak flow at the proposed new railroad/highway bridge will be either 21,800 cfs or 26,300 cfs during a local storm PMF event (discharges include 10 percent bulking by entrained air and sediment), depending on whether the flows are diverted around the OSA to Yucca Wash or toward the North Portal Pad. The bulked peak flow approaching the NPP area, depending upon the diversions, are 18,400 cfs or 23,000 cfs.

3. Approximately 23,000 cfs of water will flow in the main channel upstream of the railroad/highway bridge and the NPP area when the runoff from Sub-area ND4 is diverted towards the NPP. Depending upon the sediment load in the channel, up to 10 percent of the flow may be diverted into the side channel on the west flood plain and flow directly toward the NPP into the existing man-made canal. This flow will enter the canal at an approximately 90^{o} angle to the canal. The predicted water surface elevation at the NPP is about 3684.2 feet, 2.7 feet higher than the proposed NPP elevation. Moreover, the canal may fill with sediment at this location, further raising the water surface elevation, and flow will discharge onto the NPP unless improvements are provided to prevent flooding.

4. Potential scour at the proposed railroad/highway bridge during a local storm PMF could be as much as 11 feet.

7.2 RECOMMENDATIONS

1. Runoff from Sub-area ND4 should be routed to the east side of the OSA and discharged into Yucca Wash to minimize hydraulic loading on facilities adjacent to the NPP. This will require construction of a canal along the north perimeter of OSA similar to the concept presented in Figure 16, with a gradient of 0.5%. A floodwall may be required if the OSA elevation is less than 10 feet above the bottom of the adjacent natural channels, to prevent overtopping and provide minimal freeboard.

2. A wall, approximately 6 feet high, should be constructed along the perimeter of the NPP in and just upstream from the area where flows in the sub-channel in the western flood plain intersect the NPP. This recommendation is based on the high sediment flow conditions with 3 feet of freeboard, assuming a pad elevation of 3681.5 feet.

3. The proposed new railroad/highway bridge should be designed to accommodate a peak discharge of 26,300 cfs (which includes bulking) and a potential scour depth of 11 feet.

4. Periodic inspections and cleaning out of the flow diversion channels should be planned for the repository operating period, to ensure efficiency of these features.

7.3 LIMITATIONS

1. Because of the unconsolidated nature of sediments in Midway Valley Wash, the width and depth of channels can change rapidly during a PMF event, and multiple new flow paths may be opened up. This cannot be adequately addressed by the flood routing analysis as a rigid channel bed is implicitly assumed in the HEC-RAS software, and there are numerous possibilities of new flow paths.

2. The scour depth was estimated assuming uniform soil property with depth, but in reality, the erosion may stop if a harder stratum is encountered depending on the erodibility of the bed material.

3. Results of this study and design recommendations in this report are based on the *Repository Surface Design Site Layout Analysis* (CRWMS M&O 1998b) and have been prepared prior to the development of the site layout and grading plan for License Application design. In the event the final design of the surface facilities deviates substantially from the proposed layout used in this study, the analyses should be updated to reflect the changes. Also see the limitations discussed in Assumption 5.2.

8. INPUTS AND REFERENCES

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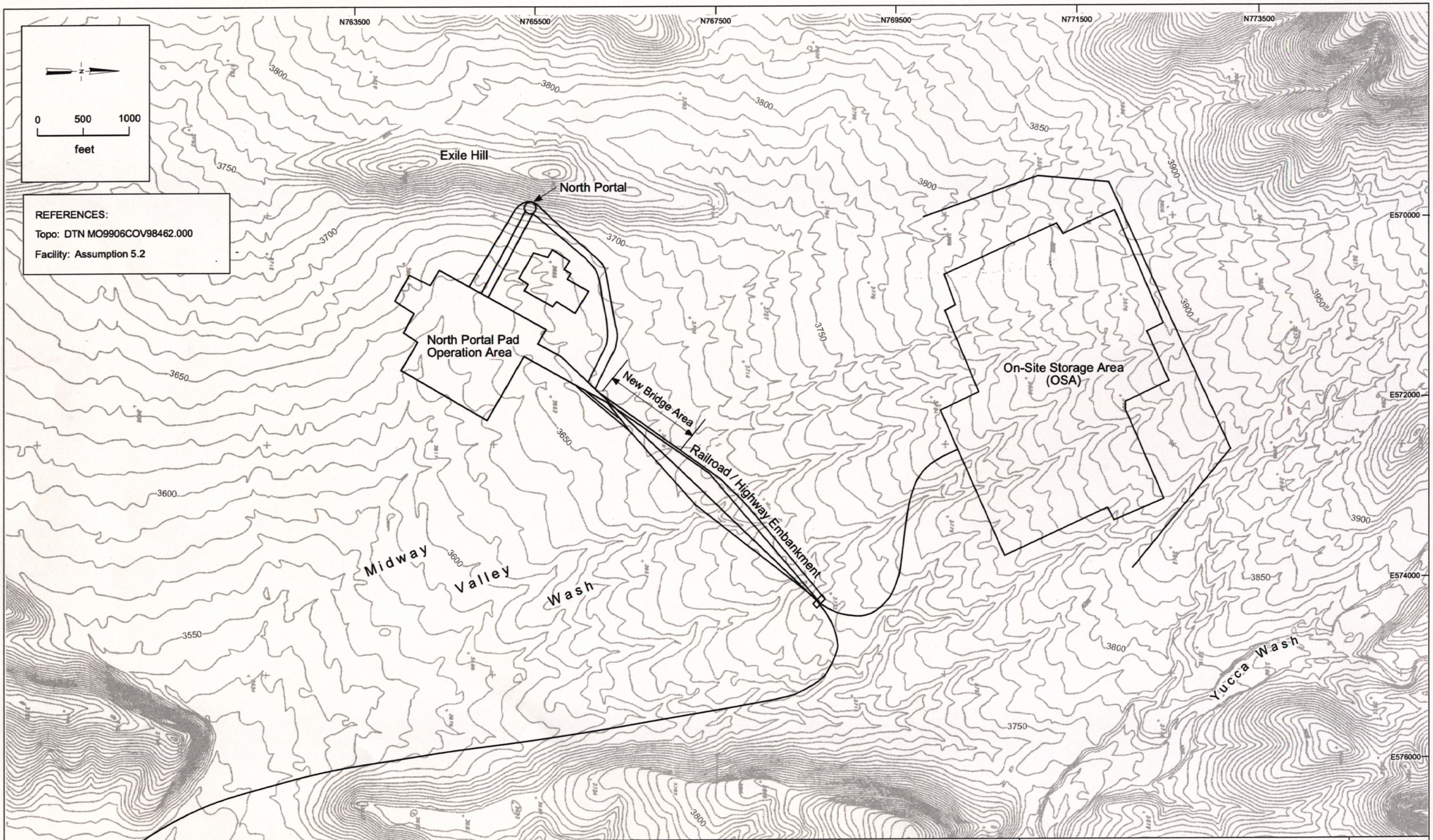
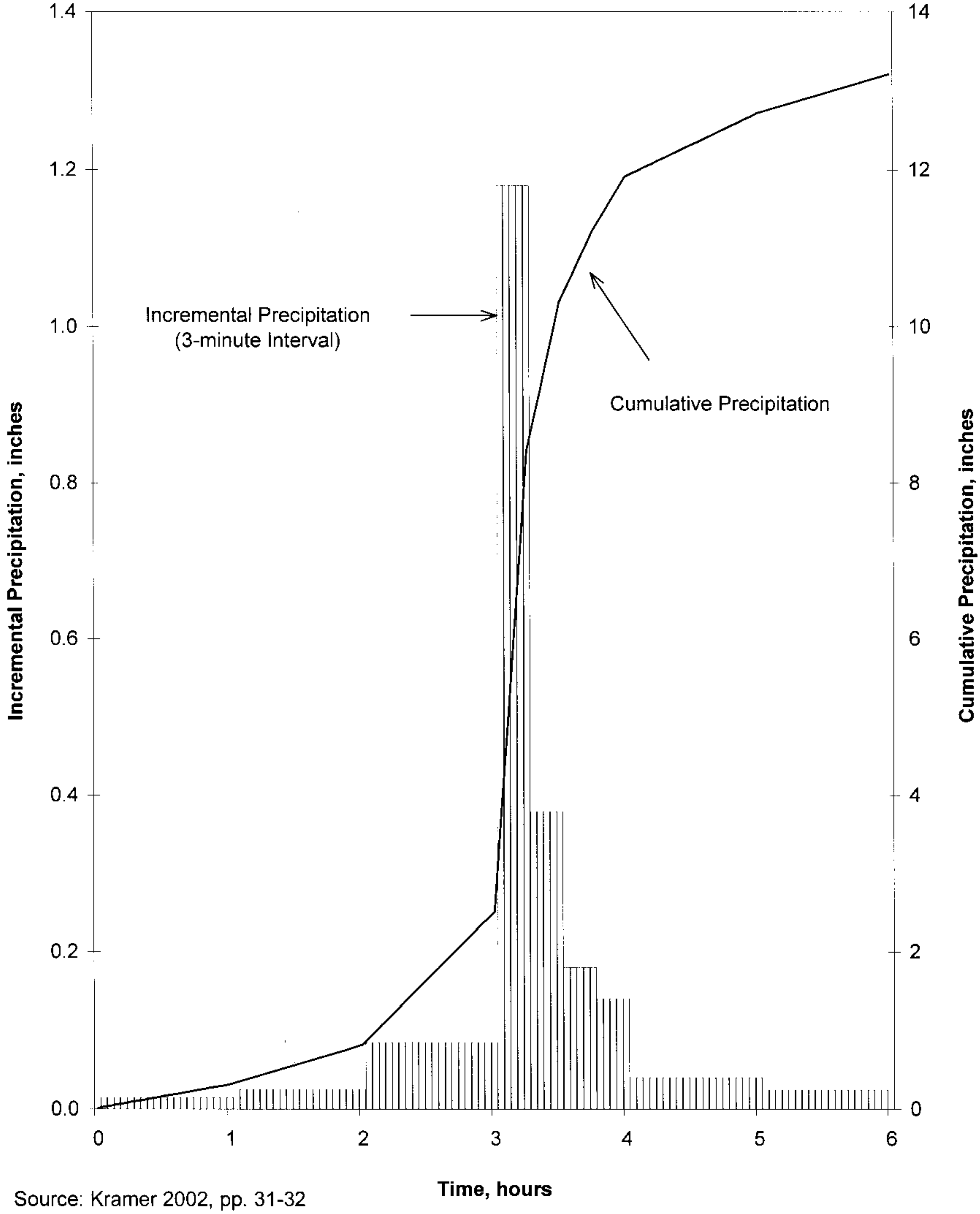
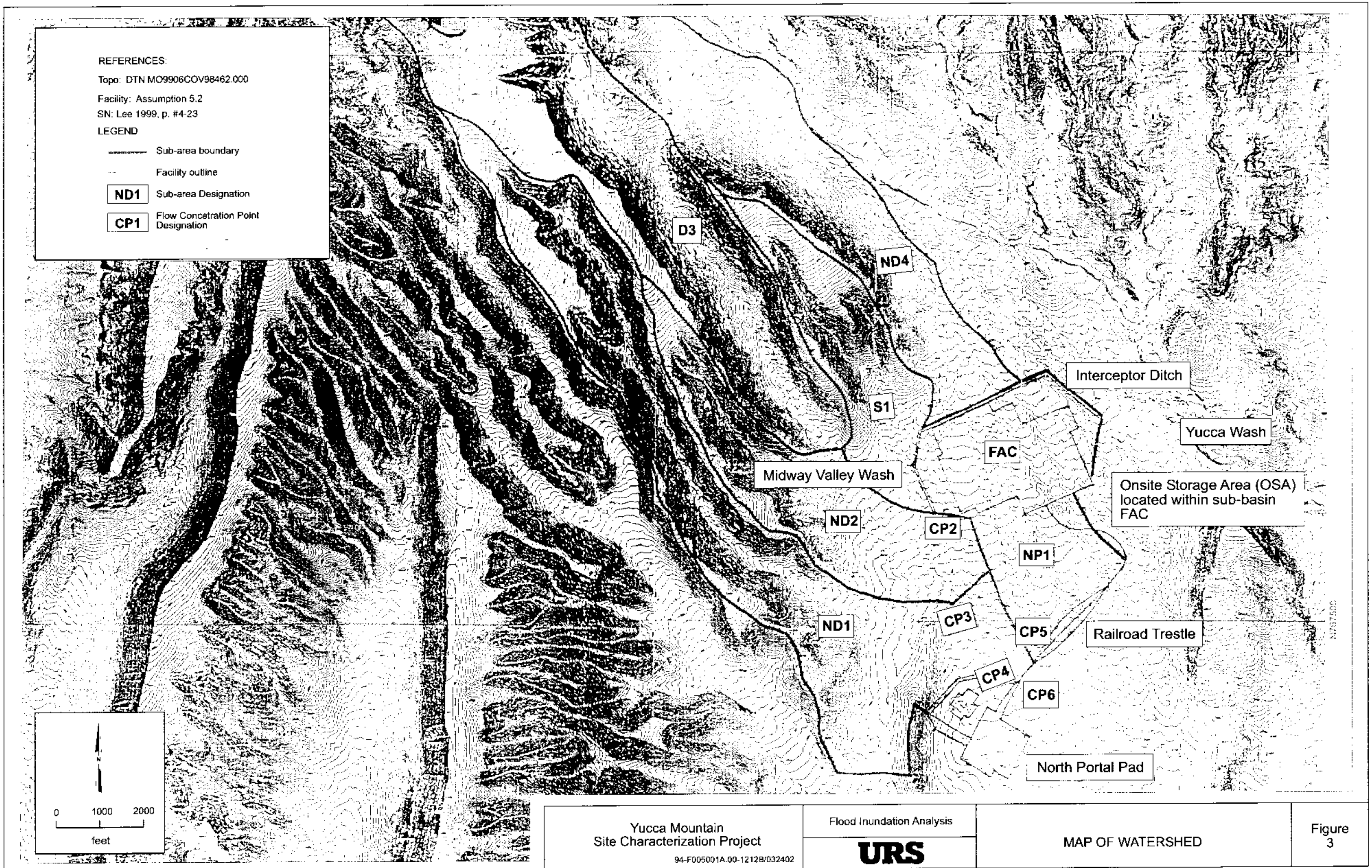


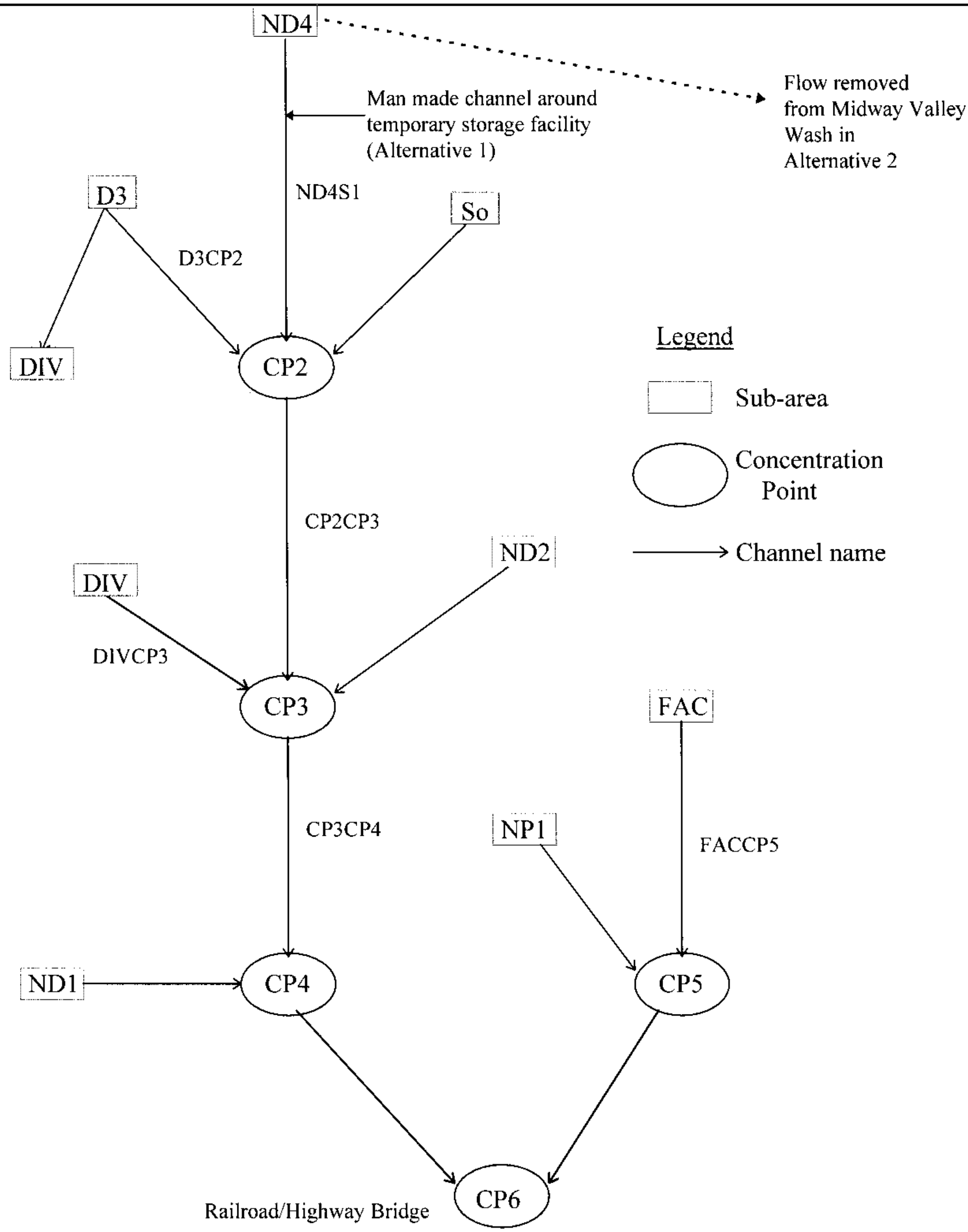
Yucca Mountain Site Characterization Project 94-F005001A.00-1212B/032402	Flood Inundation Analysis URS	NORTH PORTAL PAD, RAILROAD/HIGHWAY BRIDGE, AND ON-SITE STORAGE FACILITIES	Figure 1

Figure 2 Incremental and Cumulative Precipitation Using COE Engineering Manual Distribution



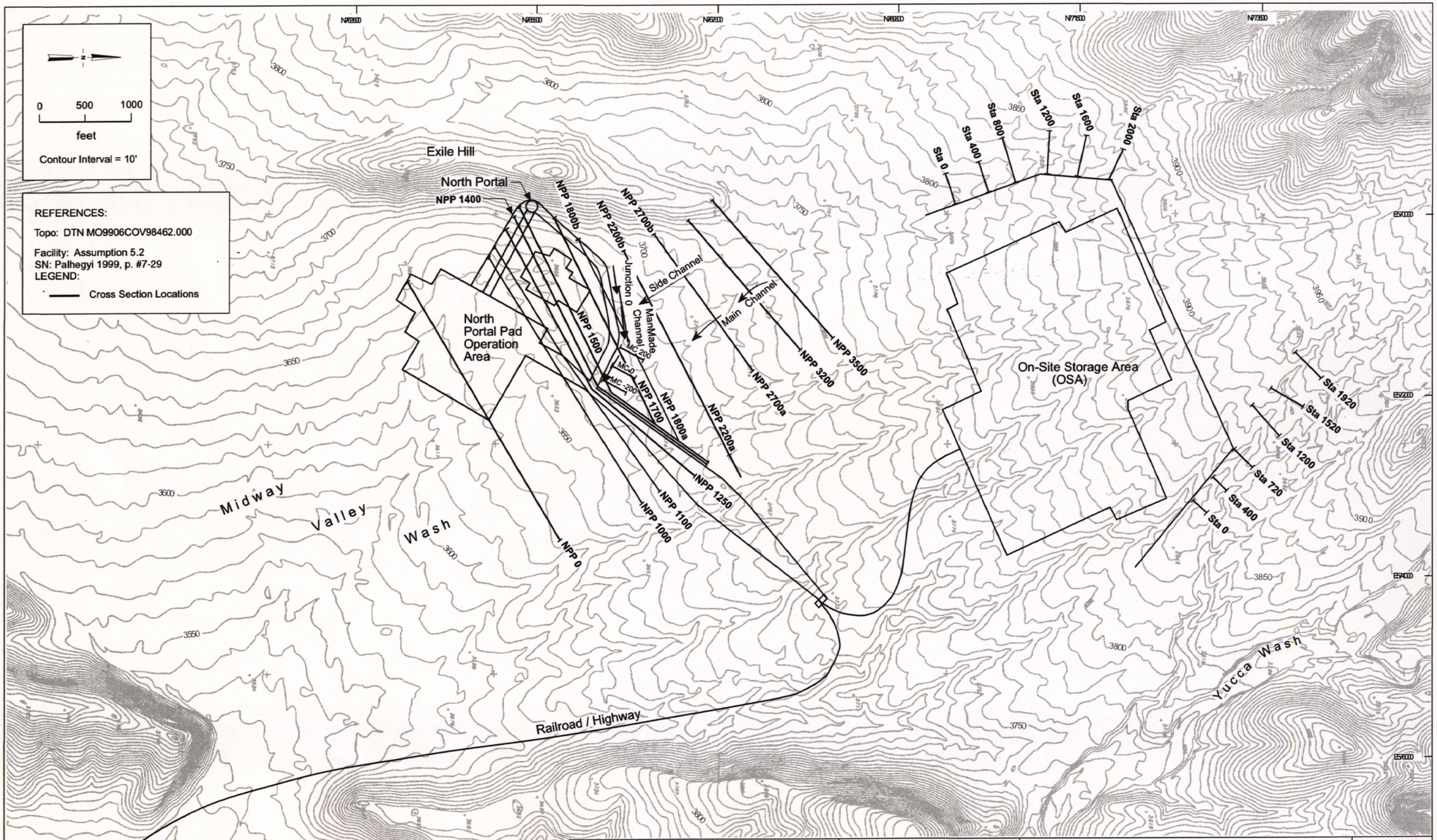
Source: Kramer 2002, pp. 31-32


REFERENCES:
Topo: DTN MO9906COV98462.000
Facility: Assumption 5.2
SN: Lee 1999, p. #4-23
LEGEND
Sub-area boundary
Facility outline
ND1 Sub-area Designation
CP1 Flow Concetration Point Designation
D3
ND4
Interceptor Ditch
S1
Yucca Wash
FAC
Midway Valley Wash
Onsite Storage Area (OSA) located within sub-basin FAC
ND2
CP2
NP1
ND1
CP3
CP5
Railroad Trestle
CP4
CP6
North Portal Pad
0 1000 2000
feet
Yucca Mountain
Site Characterization Project
94-F005001A.00-1212B/032402
Flood Inundation Analysis
URS
MAP OF WATERSHED
Figure 3



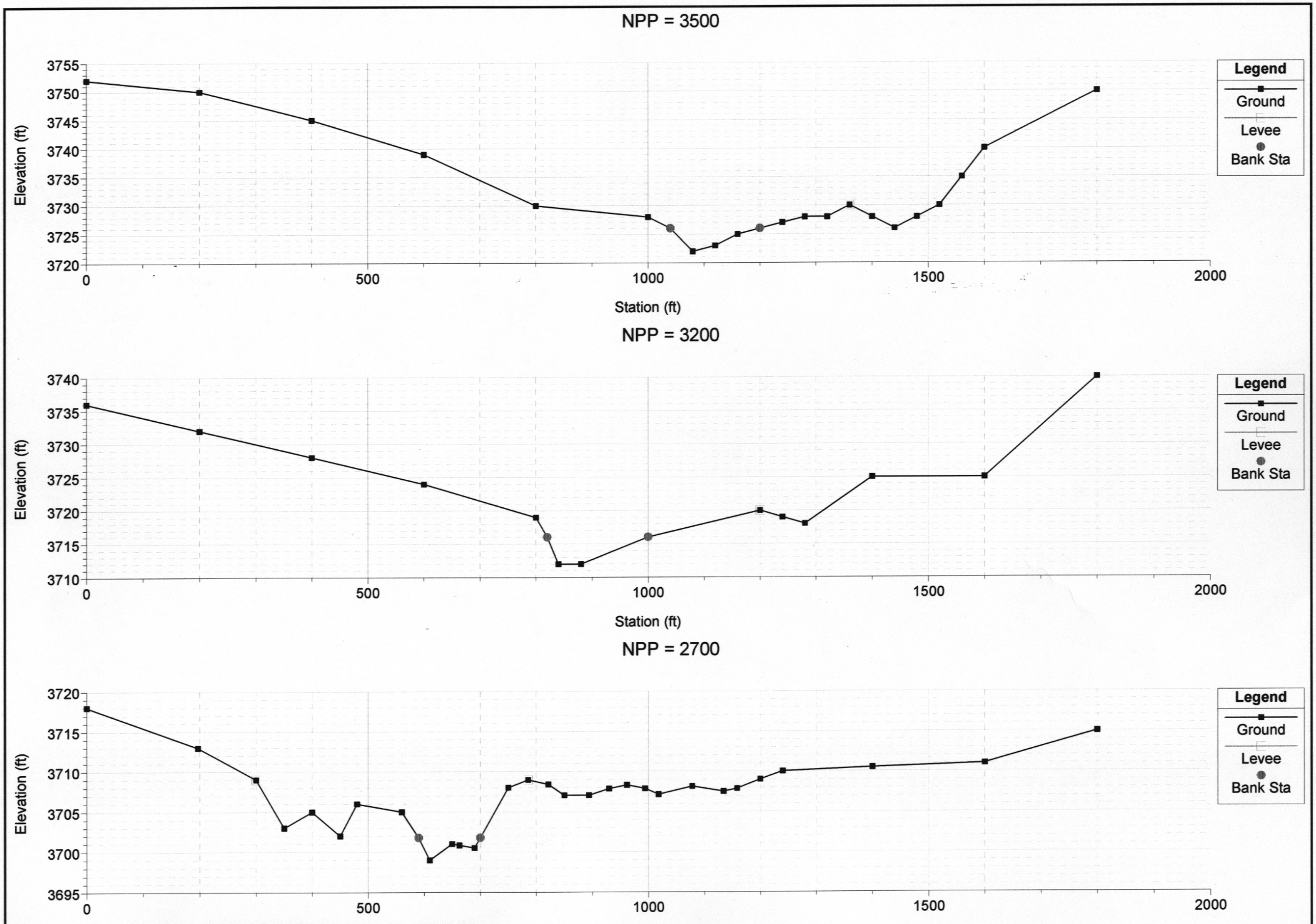
Notes:
DIV = diversion
See Figure 3 for the locations of sub-areas and flow concentration points.
Source: Lee 1999, p.#4-22

Figure 4 Schematic Diagram of Watershed Network



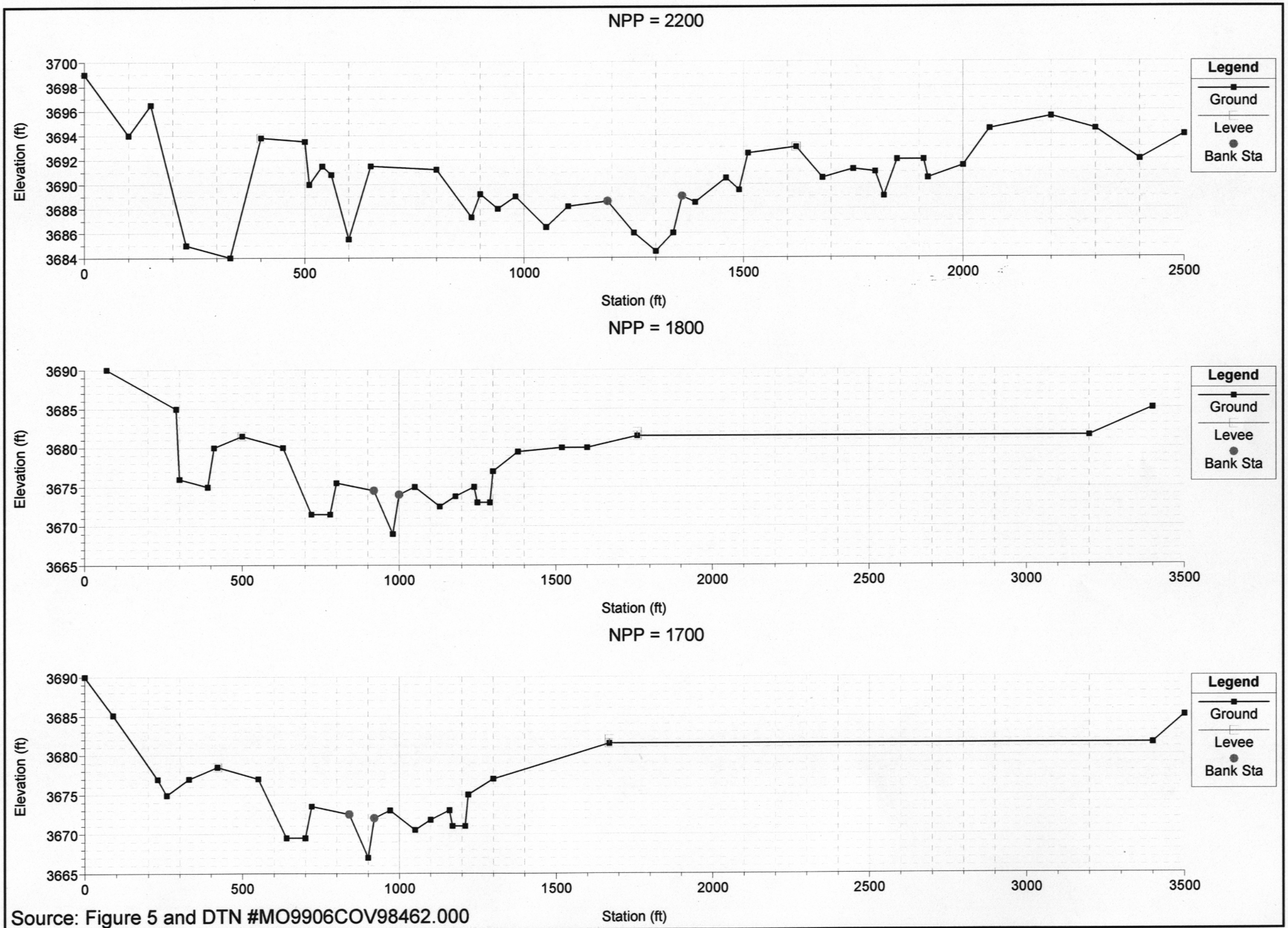
Yucca Mountain Site Characterization Project 94-F005001A.00-1212B/032402	Flood Inundation Analysis URS	LOCATION OF HEC-RAS CROSS-SECTIONS	Figure 5

Figure 6 Cross-Section Geometry for Stations 3500, 3200 and 2700 Used in the HEC-RAS Software for the PMF Flood Inundation Analysis at the NPP



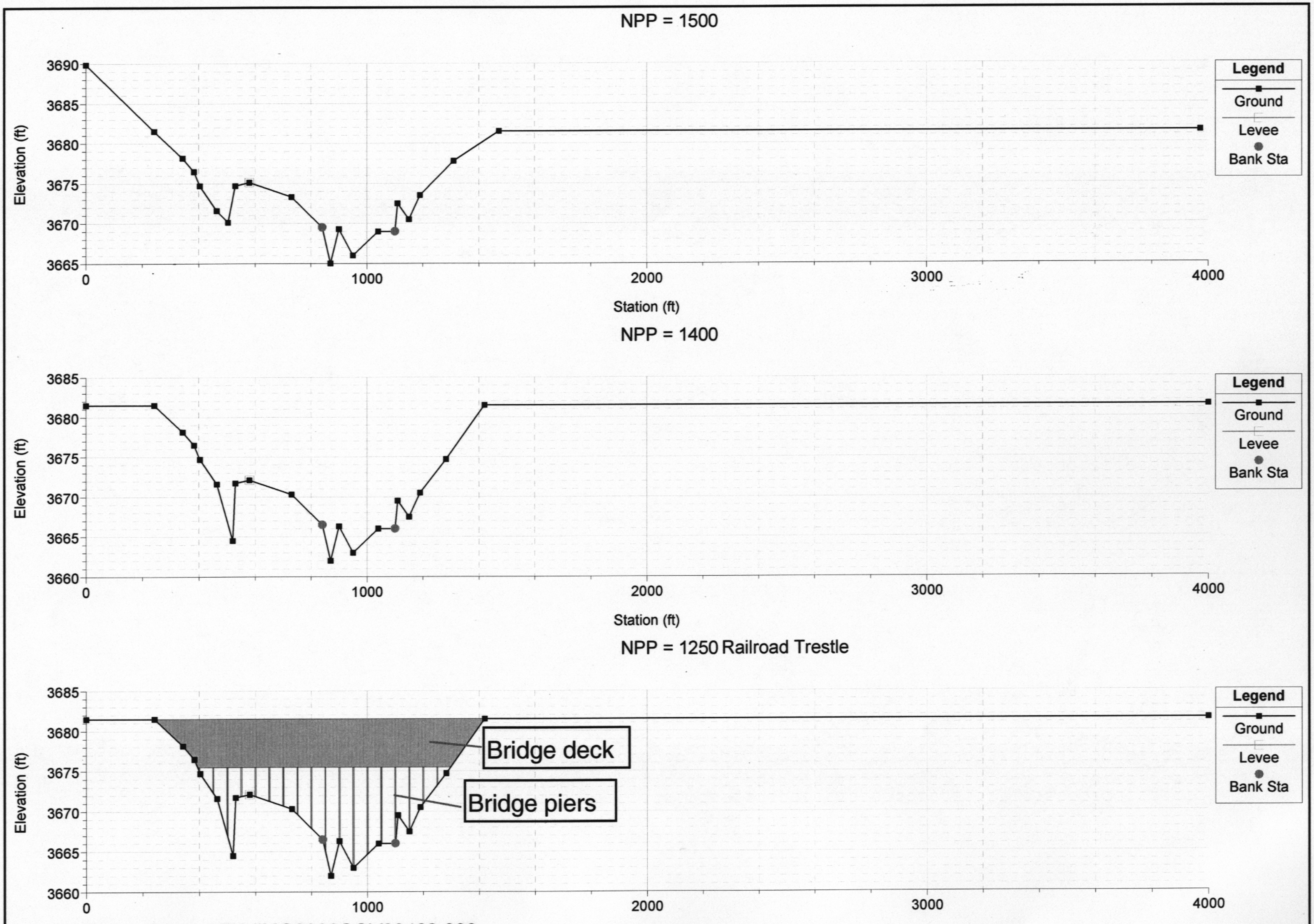
Source: Figure 5 and DTN #MO9906COV98462.000

Figure 7 Cross-Section Geometry for Stations 2200, 1800 and 1700 Used in the HEC-RAS Software for the PMF Flood Inundation Analysis at the NPP



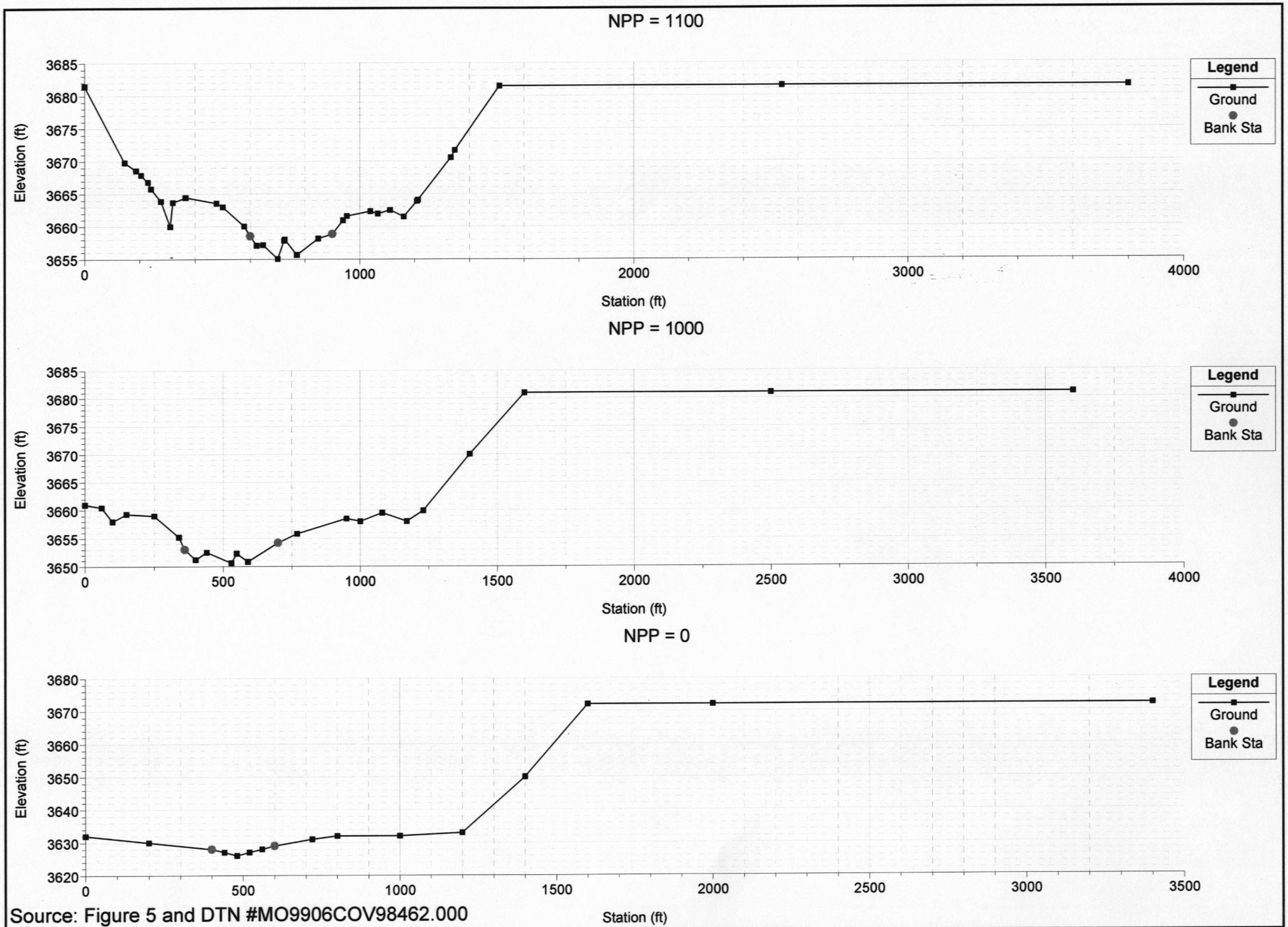
Source: Figure 5 and DTN #MO9906COV98462.000

Figure 8 Cross-Section Geometry for Stations 1500, 1400 and 1250 Used in the HEC-RAS Software for the PMF Flood Inundation Analysis at the NPP



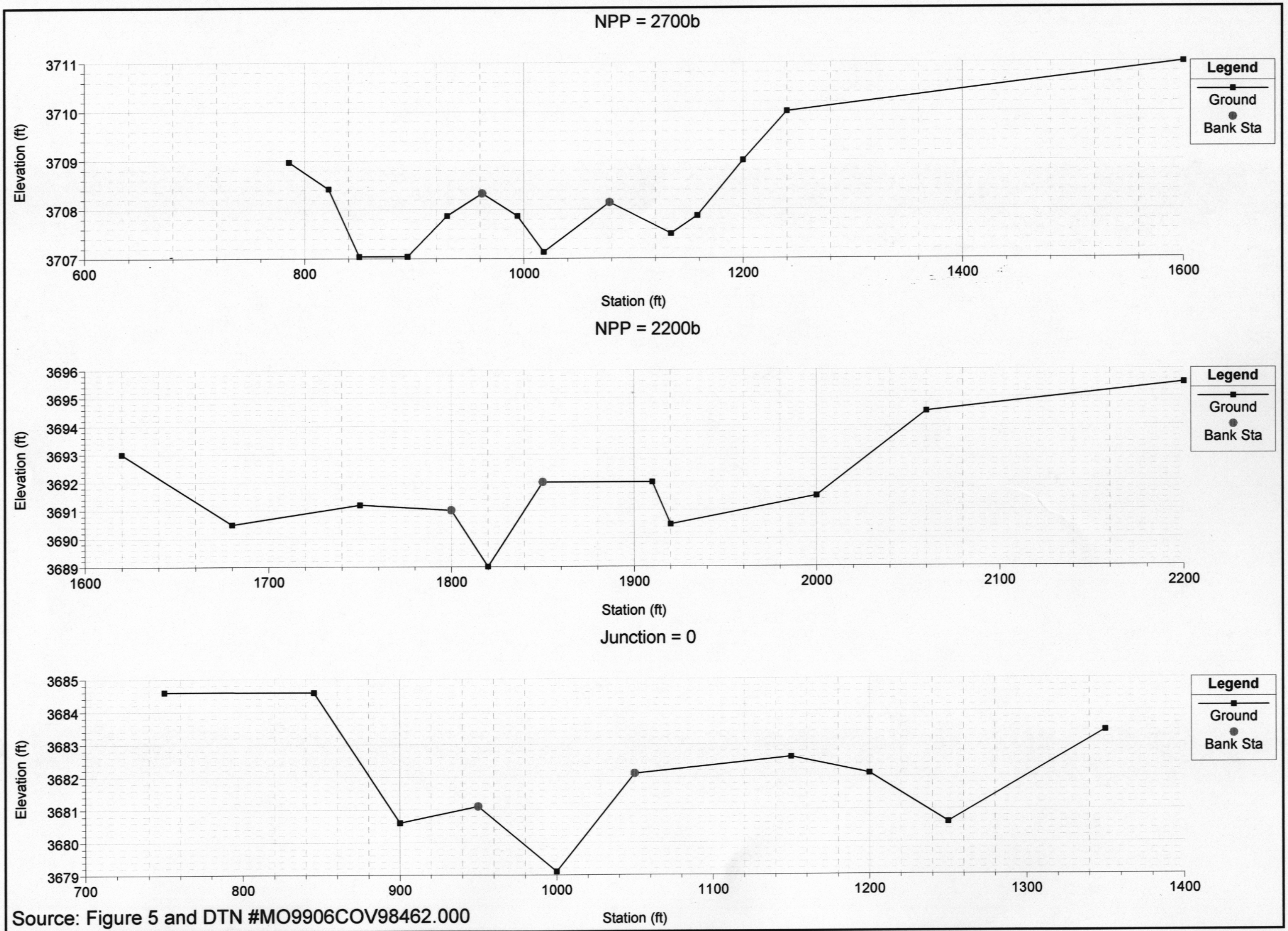
Source: Figure 5 and DTN #MO9906COV98462.000

Figure 9 Cross-Section Geometry for Stations 1100, 1000 and 0 Used in the HEC-RAS Software for the PMF Flood Inundation Analysis at the NPP



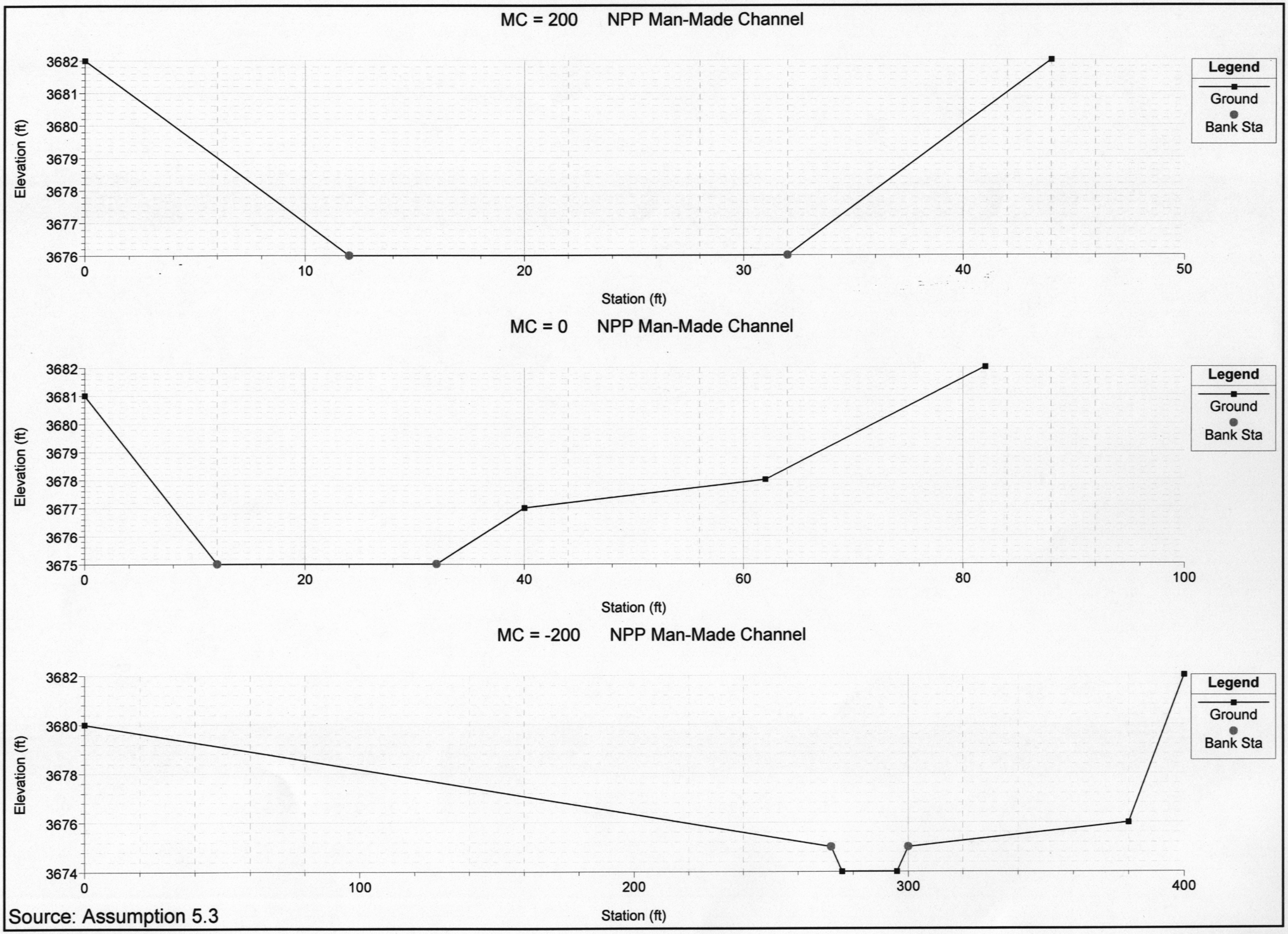
Source: Figure 5 and DTN #MO9906COV98462.000

Figure 10 Cross-Section Geometry for Stations 2700b, 2200b and Junction 0 Used in the HEC-RAS Side-Channel Branch for the PMF Flood Inundation Analysis at the NPP



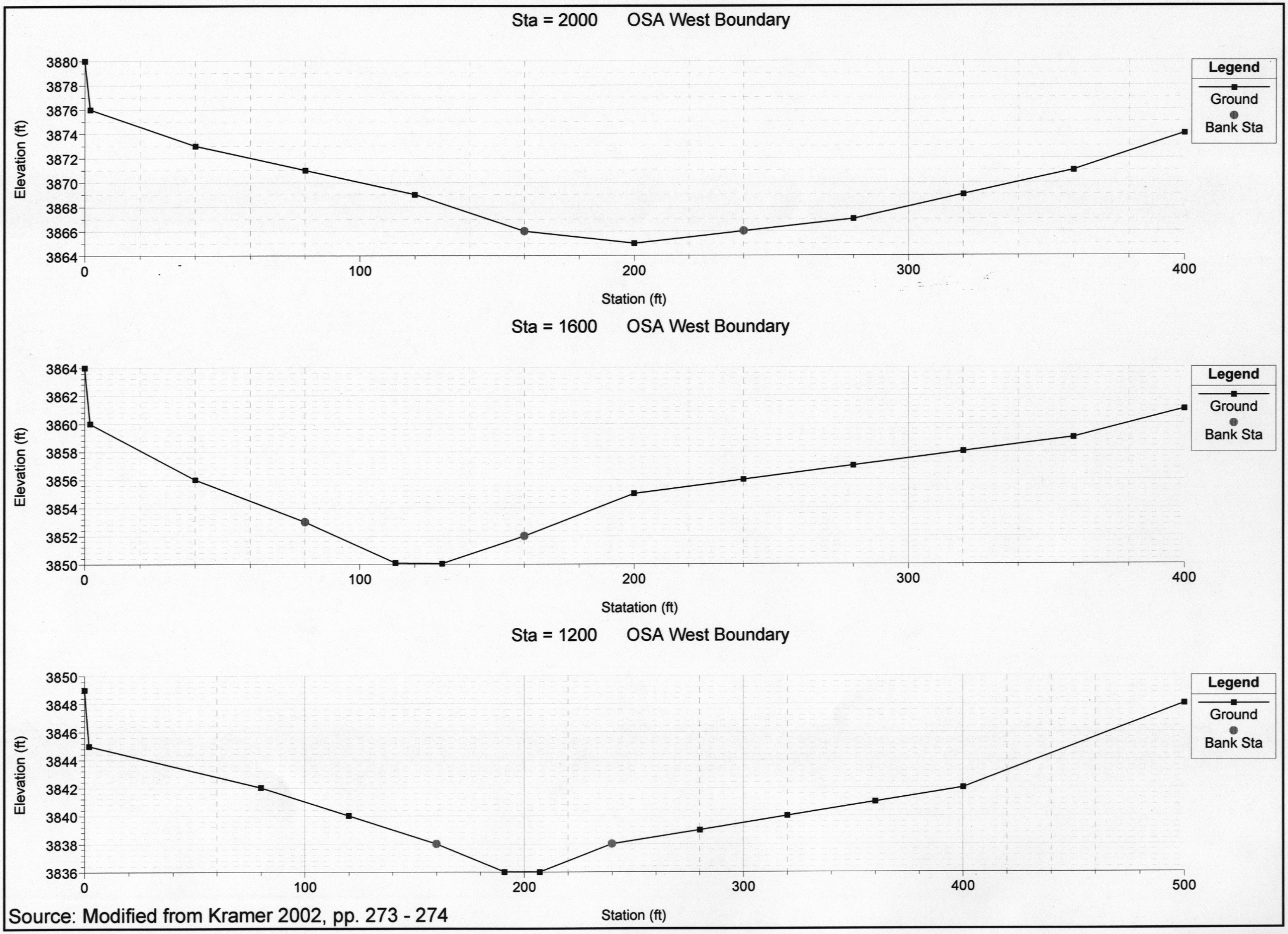
Source: Figure 5 and DTN #MO9906COV98462.000

Figure 11 Cross-Section Geometry for Stations 200, 0 and -200 Used in the HEC-RAS Man-Made Channel Branch for the PMF Flood Inundation Analysis at the NPP



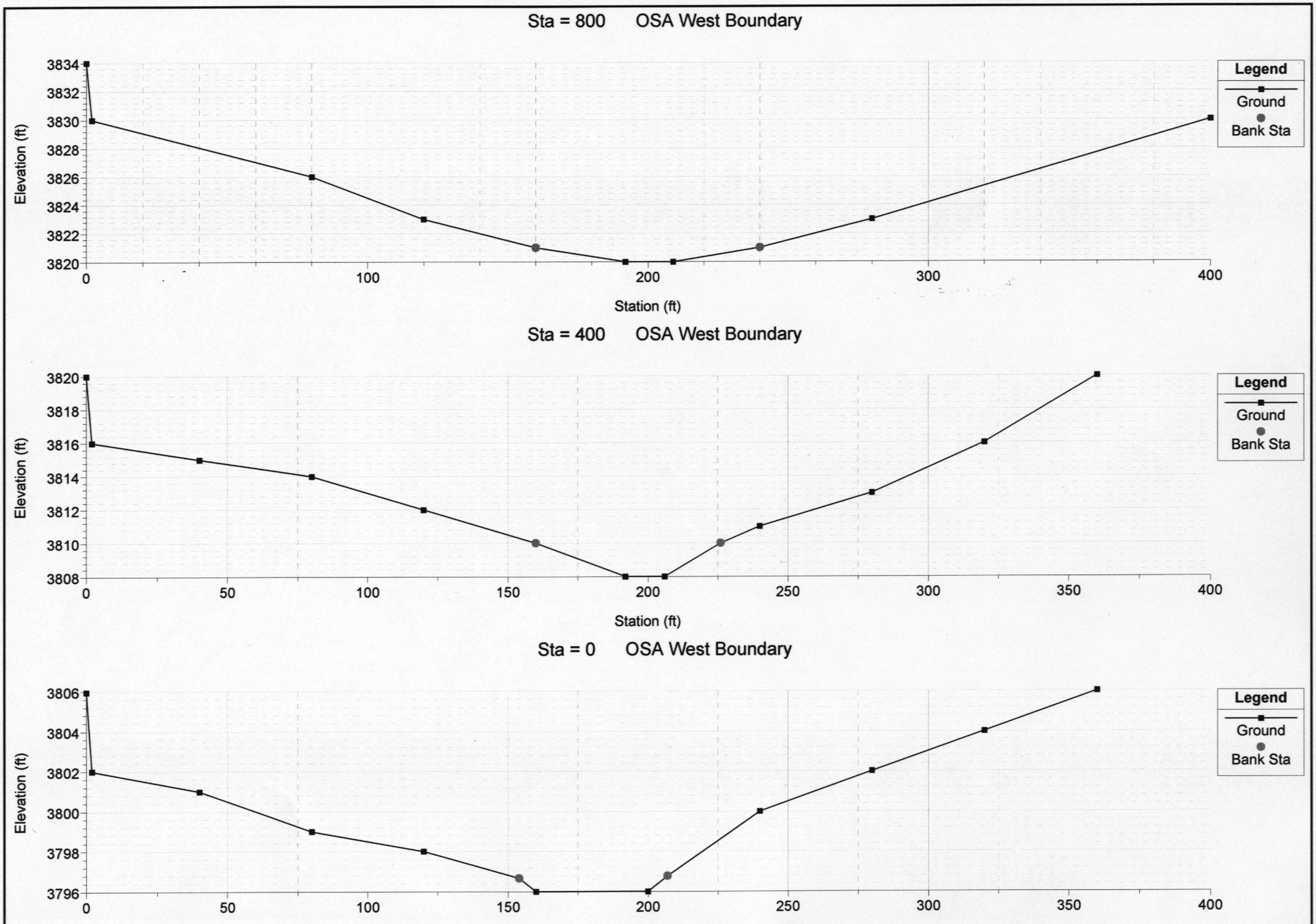
Source: Assumption 5.3

Figure 12 Cross-Section Geometry for Stations 2000, 1600 and 1200 Used in the HEC-RAS Software for the PMF Flood Inundation Analysis at the OSA



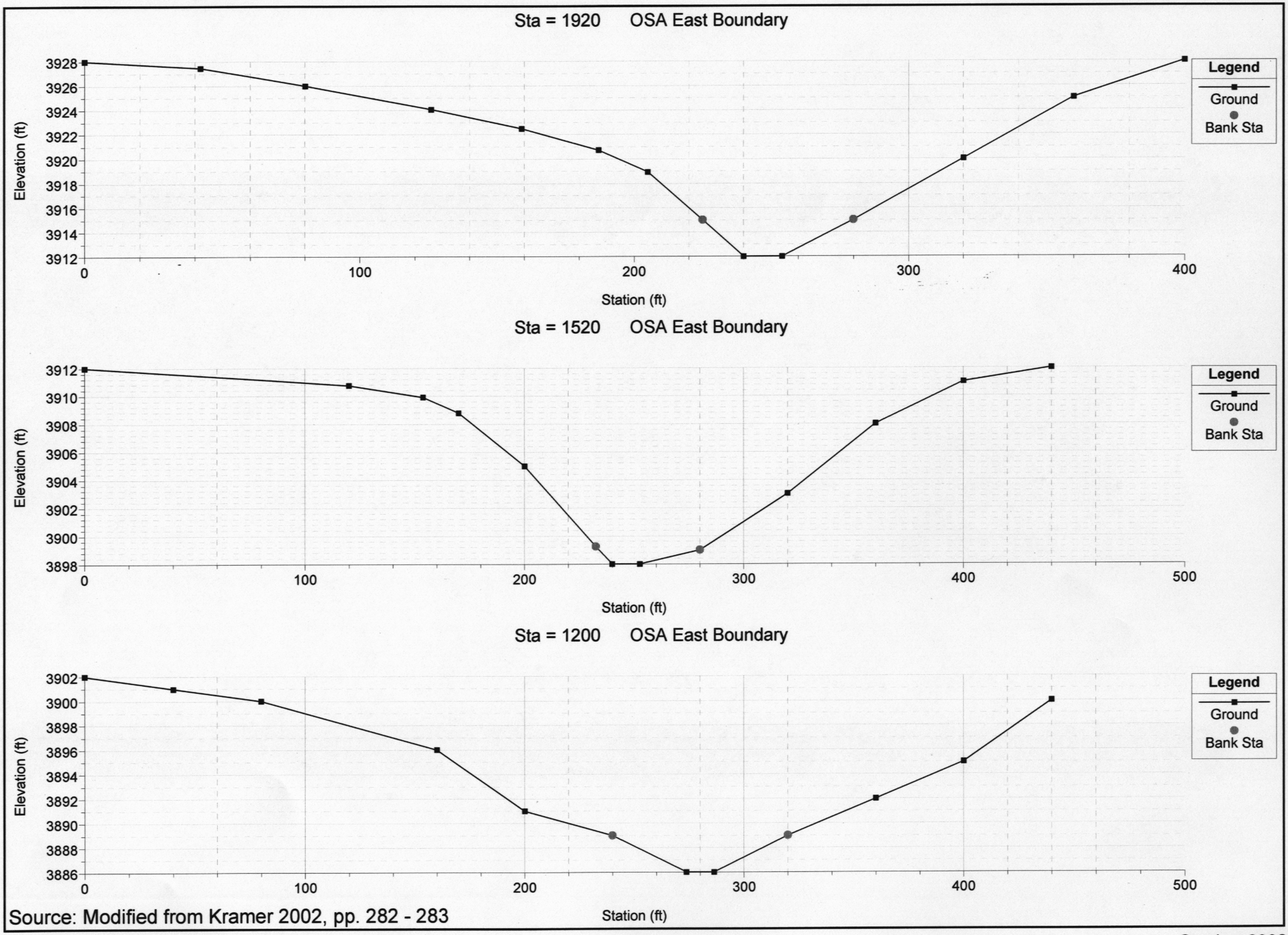
Source: Modified from Kramer 2002, pp. 273 - 274

Figure 13 Cross-Section Geometry for Stations 800, 400 and 0 Used in the HEC-RAS Software for the PMF Flood Inundation Analysis at the OSA



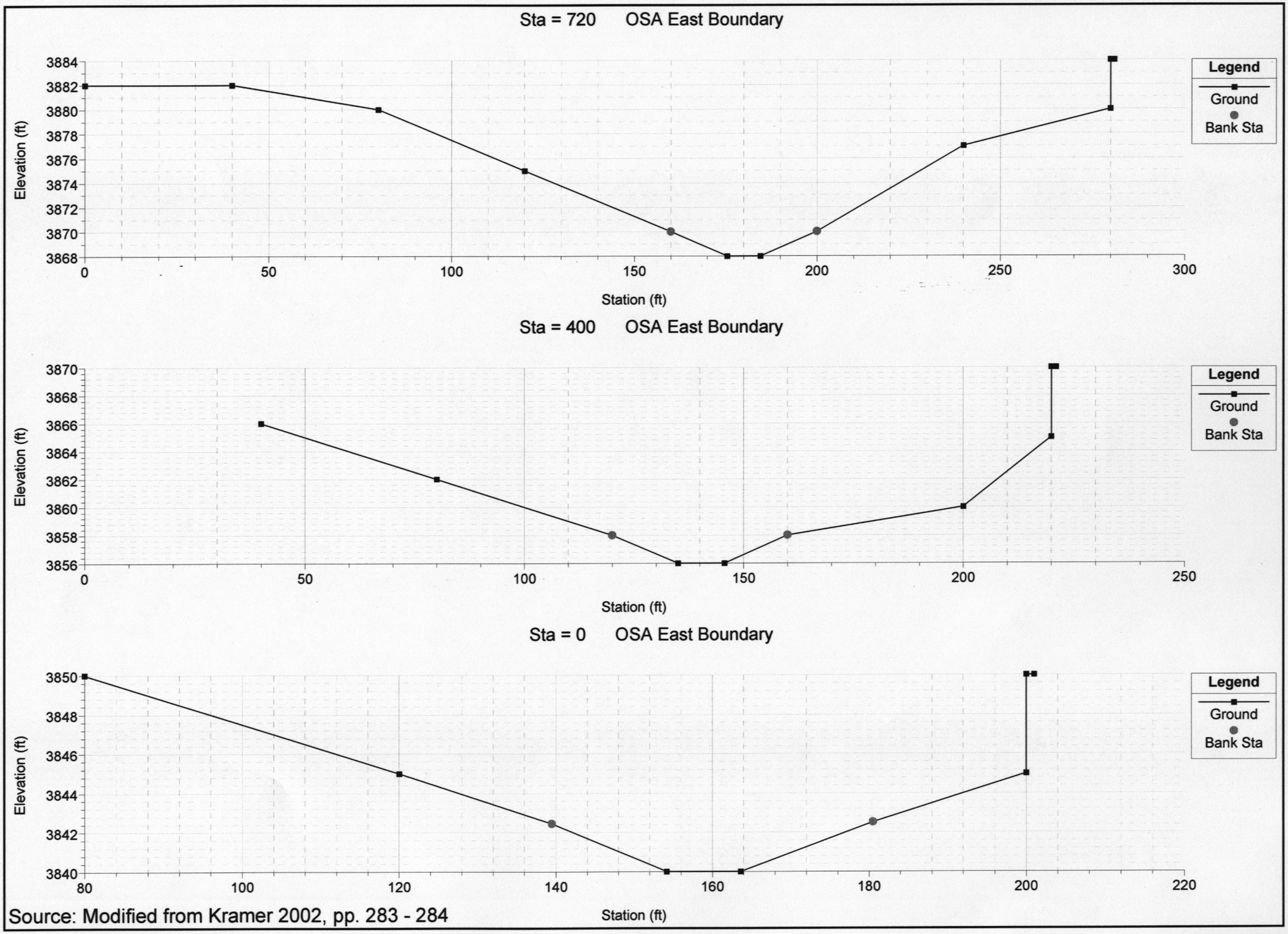
Source: Modified from Kramer 2002, pp. 274 - 275

Figure 14 Cross-Section Geometry for Stations 1920, 1520 and 1200 Used in the HEC-RAS Software for the PMF Flood Inundation Analysis at the OSA



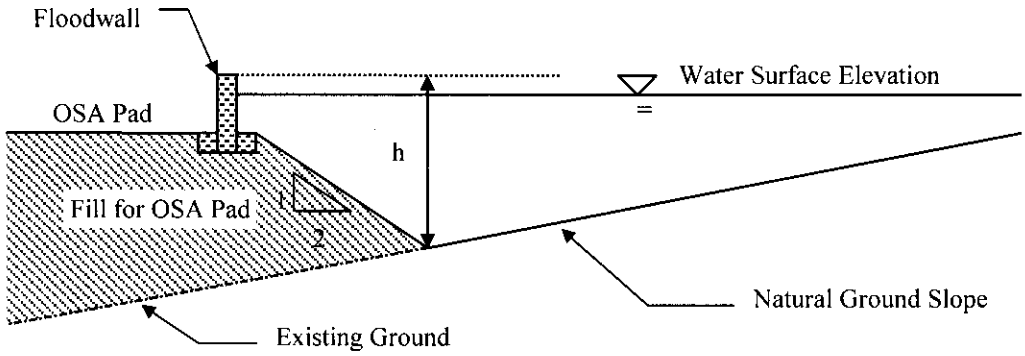
Source: Modified from Kramer 2002, pp. 282 - 283

Figure 15 Cross-Section Geometry for Stations 720, 400 and 0 Used in the HEC-RAS Software for the PMF Flood Inundation Analysis at the OSA



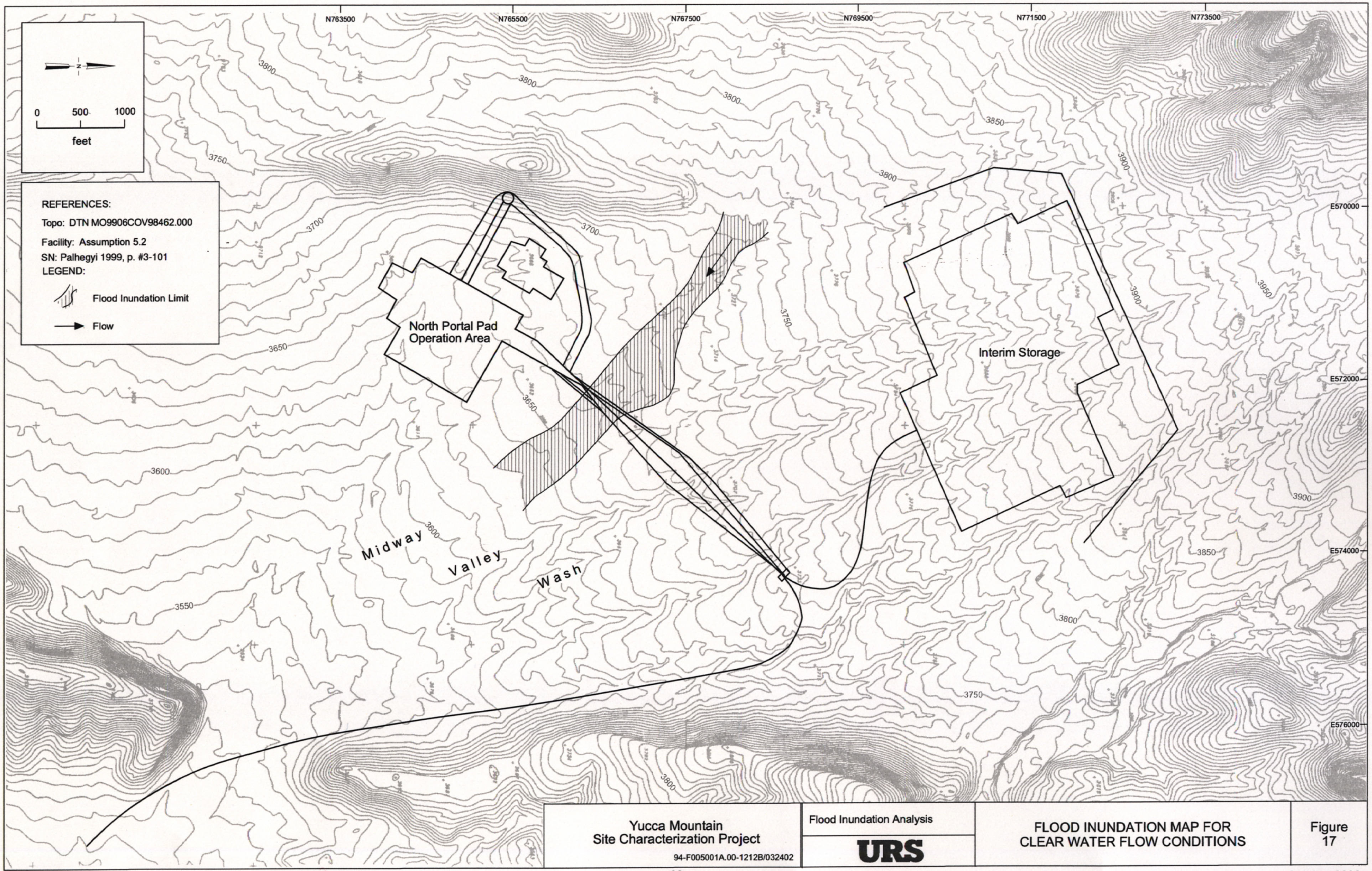
Source: Modified from Kramer 2002, pp. 283 - 284

Figure 16 Concept for Man-Made Channel Along North Boundary of OSA

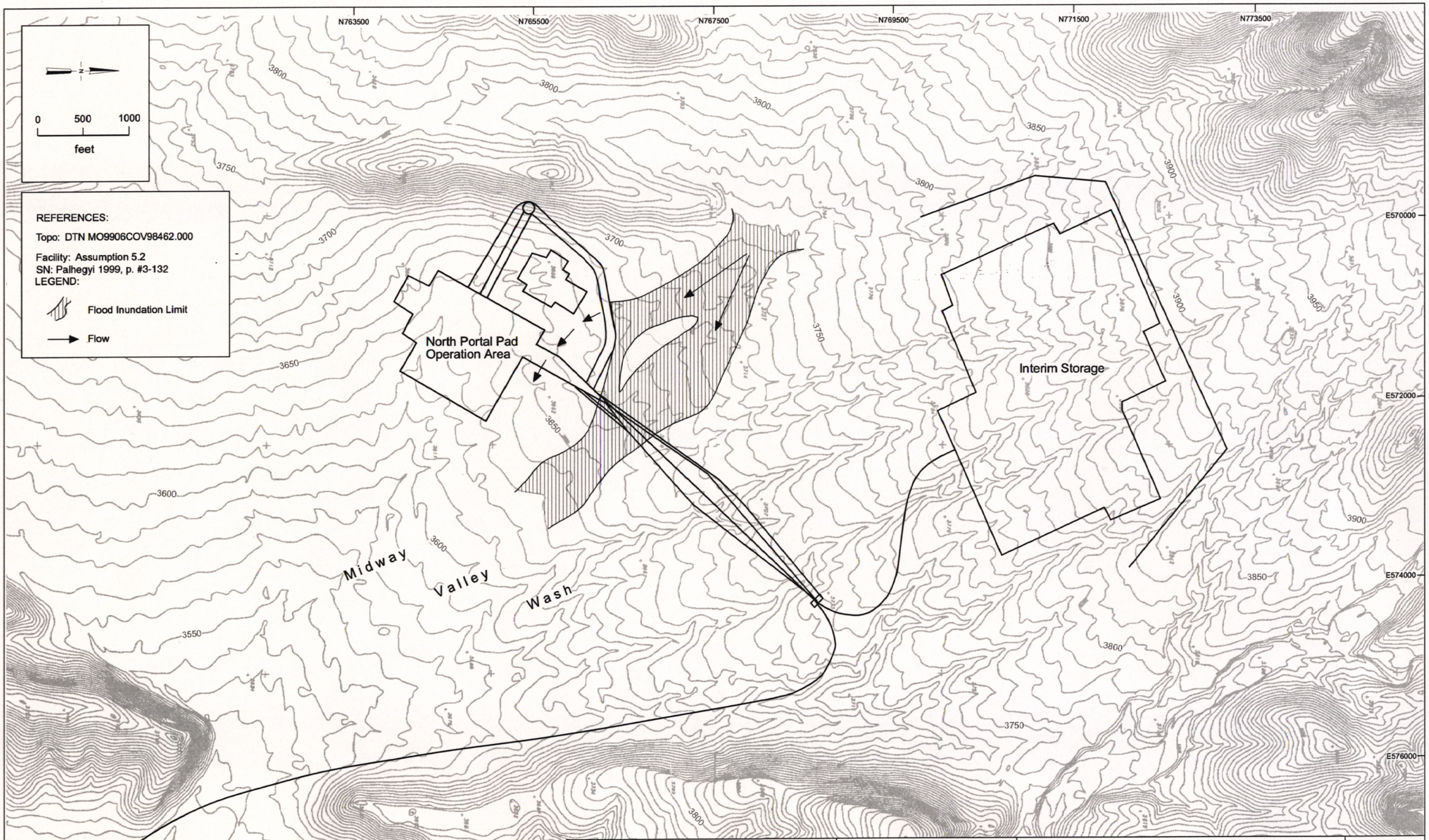


h = height of floodwall above bottom of channel

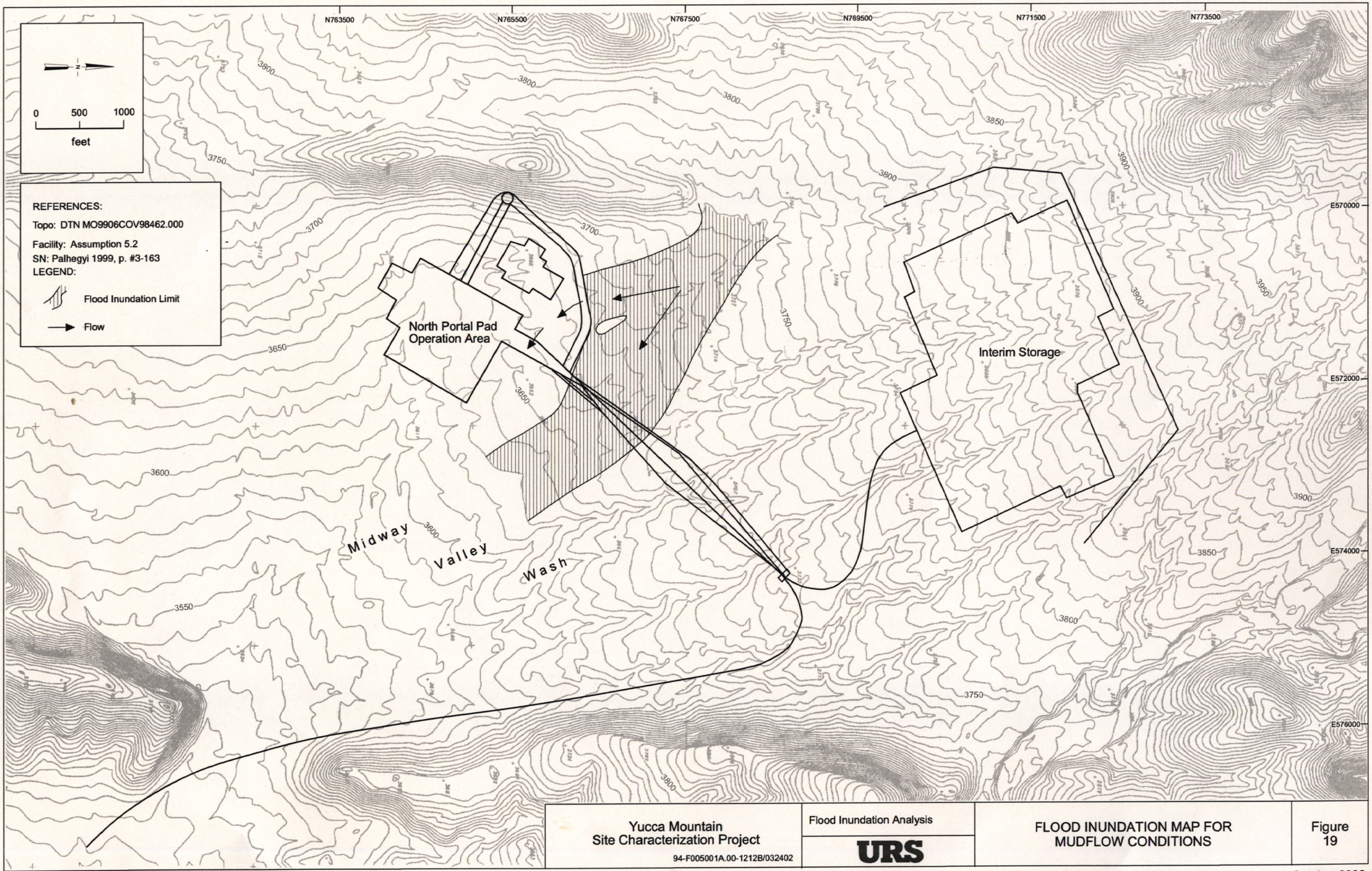
Source: Assumption 5.4



Yucca Mountain Site Characterization Project 94-F005001A.00-1212B/032402	Flood Inundation Analysis URS	FLOOD INUNDATION MAP FOR CLEAR WATER FLOW CONDITIONS	Figure 17



Yucca Mountain Site Characterization Project 94-F005001A.00-1212B/032402	Flood Inundation Analysis URS	FLOOD INUNDATION MAP FOR HIGH SEDIMENT FLOW CONDITIONS	Figure 18



Yucca Mountain Site Characterization Project	Flood Inundation Analysis 94-F005001A.00-1212B/032402 URS	FLOOD INUNDATION MAP FOR MUDFLOW CONDITIONS	Figure 19

APPENDIX I

EXAMPLE PROBLEM

I.1 PMP Determination Using HMR49

The following procedures illustrate the determination of local-storm PMP. Similar procedures apply to general-storm PMP but using different figures and tables of HMR49 (Hansen et al. 1977)

Step 1

Delineate the tributary drainage area for the NPP and adjacent surface facilities on the topographic maps. Estimate the area using a planimeter. Determine the longitude and latitude of the center of the watershed, and the minimum elevation of the watershed.

Step 2

Obtain the average 1-hr 1-mi^2 PMP from Figure 4.5 of HMR49 based on the longitude and latitude of the watershed. No altitude adjustment is required as the elevation of the watershed is less than 5000 feet.

Step 3

Obtain the average 6 to 1-hr ratio for drainage from Figure 4.7 of HMR49 based on the longitude and latitude of the watershed.

Step 4

Find the durational variation factors for ¼ hours to 6 hours from Table 4.4 of HMR49 using the 6 to 1-hr ratio obtained from Step 3.

Step 5

Calculate the 1-mi^2 PMP for durations ¼ hours to 6 hours by multiplying the 1-hr 1- mi^2 PMP from Step 2 to the durational variation factors from Step 4.

Step 6

Obtain the areal reduction factors for durations ¼ hour to 6 hours from Figure 4.9 of HMR49 based on the watershed size.

Step 7

Multiply the areal reduction factors from Step 6 to the 1- mi^2 PMP from Step 5 to get the areal reduced PMP for durations ¼ hours to 6 hours.

Step 8

Calculate the incremental PMP by successive subtraction of the areal reduced PMP from Step 7 for different durations.

Step 9

Arrange the time sequence of hourly increments PMP and the four largest 15-minute increments according to Tables 4.7 and 4.8 of HMR49, respectively. Two sequencing are offered in Table 4.7, and the COE pattern is selected to produce more conservative results. The resulting hyetograph is shown on Figure 2 of this report.

Step 10

Determine the general-storm PMP following similar procedures in HMR49. However, Yucca Wash, Midway Valley Wash, Drillhole Wash and Boundary Ridge are included as the tributary watershed and therefore the general-storm PMP has a larger area.

I.2 PMF Analysis Using HEC-1

Step 1

Compare the local-storm PMP and general-storm PMP, and select the one with a higher rainfall intensity to be used as rainfall input in HEC-1.

Step 2

Transfer the Surface Design Site Layout (CRWMS M&O 1998b) onto the topographic map as shown on Figure 3 of this report. Construct a network of hydrologic (basins) and hydraulic (interconnecting channels) components by delineating the sub-areas in a way such that they combine at pertinent locations within Midway Valley Wash. The resulting network encompasses 7 sub-areas and Figure 4 shows how they are connected and combined at the OSA boundary (flow concentration point CP2), NPP (CP4) and railroad bridge (CP6).

Step 3

Measure the area, total length of flow path, and length of flow path from basin outlet to the centroid of each sub-area from the topographic map. Compute the basin lag time using the five commonly used formulae. Calculations are illustrated below for Sub-area ND1:

Measured length of flow path, L = 3.835 mi

Measured length of flow path to centroid, L_{ca} = 1.70 mi

Measured elevation at head of basin, ELH = 5295 ft

Measured elevation at basin outlet, ELO = 3670 ft

Calculated average slope of channel, $S = \frac{ELH - ELO}{L}$

$$S = \frac{(5295 - 3670)\text{ft}}{3.835\text{mi}}$$

$$S = 423.7 \text{ ft/mi}$$

Lag time (USBR formula), $\text{lag} = C\left(\frac{LL_{ca}}{\sqrt{S}}\right)^{0.33}$

$$\text{lag} = 1.1 \cdot \left(\frac{3.835 \times 1.70}{\sqrt{423.7}}\right)^{0.33}$$

$$\text{lag} = 0.75 \text{ hrs}$$

Equations for the other lag time formulae are given in Table 4-2 of this report. Lag time for all the sub-areas except FAC is computed in this manner and summarized in Table 4-2. Because FAC has an asphaltic/concrete surface, it experiences overland flow, and overland flow length, representative slope, roughness coefficient are specified in lieu of basin lag time.

Step 4
Select the uniform loss method for initial abstraction and continuous infiltration losses. Use the same loss rates for all the sub-areas except that Sub-area FAC was specified to be 100% impervious which means no losses would occur in this overland flow element.

Step 5
Input the area, lag time (or corresponding parameters for overland flow for Sub-area FAC), and loss rates of each sub-area into HEC-1. Runoff from sub-areas is combined and routed based on the network constructed for the watershed. For instance, runoff from ND4 is routed via man-made channel ND4S1, and runoff from D3 is routed via natural channel D3CP2 to combine with runoff from S1 at the basin outlet of S1, designated as Flow Concentration Point CP2. The Muskingum-Cunge method and Kinematic wave method were used for natural and man-made channels, respectively. Size and slope of the natural channels are measured from the topographic map, and assumptions were made for man-made channels.

Step 6
Run the HEC-1 software and produce outputs in the form of flood hydrographs at each sub-area and flow concentration point.

I.3 Flood Inundation Analysis Using HEC-RAS

The following procedures exemplify the flood inundation analysis performed at the NPP under clear-water flow conditions. Other HEC-RAS simulations discussed in Section 6.2 were conducted in a similar manner.

Step 1
Obtain the peak flow impacting the NPP and the railroad bridge at Flow Concentration Points CP4 and CP6 from the HEC-1 software. Apply a 10% bulking factor and use the bulked flows in the HEC-RAS software.

Step 2
Develop cross-sections for the main channel and flood plain in the vicinity of the surface facilities at the North Portal Pad. The reach begins upstream of the NPP and ends downstream of the railroad bridge. Figure 5 of this report shows the locations of the cross-sections used in the software, and Figures 6 to 10 present the cross-sections developed from the topographic map. The bulked flow of 23,040 cfs was applied upstream of and including cross-section NPP1700, while 26,340 cfs was specified for the downstream cross-sections to reflect the combined flows from Sub-areas FAC and NP1.

Step 3
For each cross-section, define the limits of the main channel. Apply a Manning's n of 0.035 for the main channel and 0.05 for the flood plain.

Step 4
Define the railroad bridge at Section NPP1250. Input the deck elevation, bridge width, pier locations and pier diameter from Assumption 5.5.

Step 5

Run the HEC-RAS software by specifying normal flow conditions at the upstream and downstream boundaries. Check to see if flow bifurcation occurs upstream of the NPP. If it occurs, determine the amount of flow conveyed by the side channel in the west flood plain and run a separate HEC-RAS setup to obtain the water surface elevation in the side channel when the flood flow reaches the NPP. Compare the predicted water surface elevation with the proposed pad elevation to decide if a floodwall is needed.

Step 6

Retrieve the flow characteristics including water surface elevation, flow velocity, and flow depth at each cross-section from the software outputs.

I.4 Scour Analysis Using HEC-18

Step 1

Obtain the flow velocity and flow depth at the railroad bridge from the HEC-RAS software for the high sediment flow conditions.

Step 2

Calculate the critical velocity for initiation of scour using the average of Neill's and Laursen's equations. Different assumed particle sizes can be calculated similarly.

From HEC-RAS, $y_1 = 11.49$ ft

Assume mean particle size, $D_{50} = 1.5$ mm $= 0.0049$ ft

Critical velocity, $V_c = 11.235 y_1^{1/6} D_{50}^{1/3}$

$$V_c = 11.235 \times 11.49^{1/6} \times 0.0049^{1/3}$$

$$V_c = 2.9 \text{ft/s}$$

Step 3

Compare the mean velocity from HEC-RAS at the bridge to the critical velocity for initiation of scour. Since the mean velocity of 8.1 ft/s is greater than the critical velocity of 2.9 ft/s, live-bed contraction scour is expected to occur. It is estimated using the equation for live-bed contraction scour from HEC-18:

$$Q_1 = Q_2$$

$$W_1 = 600 \text{ ft}$$

$$W_2 = 600-600/50*4$$

$$y_2/y_1 = (Q_2/Q_1)^{6/7} (W_1/W_2)^{k1}$$

$$\therefore \quad y_2/y_1 = 1^{6/7}(600/552)^{0.69}$$

$$y_2 = 1.059*11.49 = 12.2 \text{ ft}$$

Scour depth, $y_s = y_2 - y_1$

$$y_s = 12.2 - 11.49 = 0.7 \text{ ft}$$

Step 4

Compute the pier scour using the CSU equation:

$y_s/a=2.0\ K_1\ K_2\ K_3\ (y_1/a)^{0.35}\ FR^{0.43}$

where a = pier diameter = 4 ft

$K_1 = 1.0$

$K_2 = 1.0$

$K_3 = 1.2$

FR = 0.48 (from HEC-RAS output at the bridge section)

$$\therefore \quad \frac{y_s}{a} = 2\times1\times1\times1.2\times\left(\frac{11.49}{4}\right)^{0.35}\times0.48^{0.43}$$

$y_s/a = 2.53$

Since HEC-18 specified an upper limit of 2.4 for the ratio y_s/a, the estimated scour depth is adjusted to 9.6 feet.

Step 5

Add the contraction scour to the local pier scour to obtain the total scour depth of 10.3 feet at the bridge.

APPENDIX II

SOIL DATA FROM PREVIOUS HYDROLOGIC INVESTIGATIONS

The following tables summarize available soil data from watersheds adjacent to the YMP study site and from the study site. The soil associations listed below are a subset of the soils included in each of the references. The subsets are selected based on the description of the soils that appear to be similar to the soils at the YMP site. Hydrologic soil groups are not provided for Meadow Valley, and therefore they were assigned to the soil associations using the classifications given in Exhibit A of "Urban Hydrology for Small Watersheds" (USDA 1986). The composition of each soil association was computed from the sieve analysis results included in the source tables, with fines defined as the portion passing No. 200 sieve and gravel as the portion retained on No. 4 sieve. The Midway Valley Wash samples are from the channel.

Table II-1. Soils Data from Watersheds Adjacent to YMP Area.

Soil Series	Symbol	Hydrologic Soil Group	% Gravel	% Sand	% Fines	Permeability (in/hr)
Big Smoky Valley (Candland 1980, Tables 7 and 8)						
Soils on Valley Fill, Outwash Plains, Alluvial Fans and Aprons						
Dobel-Bluewing	DN	D	35	15	50	0.2-0.6
Belted gravelly loamy sand	BHC	D	20	50	30	0.6-2
Stumble loamy fine sand	STC	A	5	75	20	2.0-6.0
Tybo-Stumble	TS	D	25	45	30	2.0-6.0
Badland-Pintwater	BB	A				
Tipperary fine sand	TGE	A	0	75	25	>20.0
Yomba gravelly sand	Ym	C	25	40	35	0.6-2.0
Old Camp-Rock outcrop	OD	D	70	12	18	0.6-2.0
Osobb-Gabbs	OT	D	65	15	20	0.6-2.0
Yomba-Playas	YO	C	25	40	35	0.6-2.0
Timper gravelly sandy loam	TEB	D	20	30	50	2.0-6.0
Mazuma fine sandy loam	McA	C	35	25	40	2.0-6.0
Meadow Valley (Borup and Bagley 1976, Table 8)						
Soils on Upper Terraces and Alluvial Fans						
Lize-Tica	LT	B-D	40	25	35	0.6-2.0
Denmark gravelly loam	DMD	D	35	25	40	2.0-6.0
Ursine	UR	D	40	20	40	0.6-2.0
Linco-Acana	LC	B-D	35	35	30	0.6-2.0
Acana gravelly sandy loam	ACC	D	37	38	25	0.2-6.0
Minu stony sandy loam	MVC	D	35	25	40	0.6-2.0
Soils on Flood Plains, Lower Terraces and Alluvial Fans						
Geer-Heist	GM	B	0	20	80	0.6-2.0
Patter-Geer	PN	B	2	43	55	0.6-2.0
Patter-Heist	PO	B	0	30	70	0.2-0.6
Holtle loam	HOC	B	2	33	65	0.6-2.0
Fanu	FAC	B	30	40	30	2.0-6.0
Poorma	PTB	B	5	20	75	0.6-2.0

Table II-2. Previous Soil Data from YMP.

Exploration Designation	Depth (ft)	Soil Classification	% Gravel	% Sand	% Fines
North Ramp Surface Facility (DTN GS921283114220.014)					
TP-11	0-2.6	GM	52	30	18
TP-19	0-2	SC	20	46	34
TP-21	0-1.4	SM	31	46	23
TP-25	0-1.2	GM	40	37	23
TP-28	0-1.7	GM	45	36	19
TP-29	0-1.7	SM	32	34	34
Midway Valley Wash (Gradation Testing, Mineart 1999, pp. #4-14 to 4-33)					
00541811		SC	12	42	46
00541818		SP-SC	32	57	11

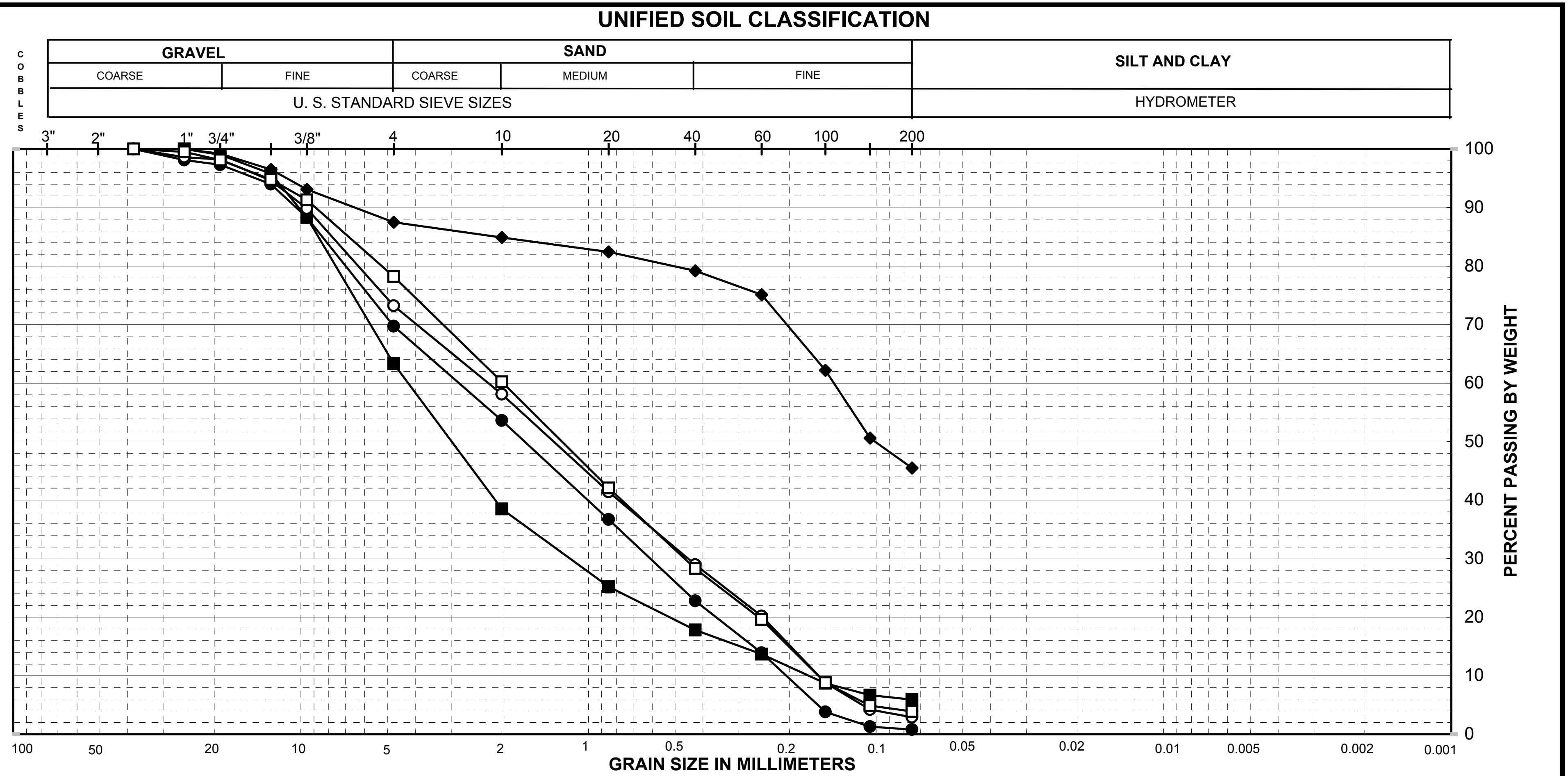
APPENDIX III

SOIL GRADATION TESTS, 1999

Nine bulk soil samples collected from Midway Valley Wash were analyzed for particle size distribution. Data are representative of flood plain and stream channel surface soils. Samples 00541811 and 00541818 represent flood plain soils. The remainder of the samples represents stream channels deposits. These results are excerpted from Scientific Notebook SN-M&O-SCI-013-V1 (Mineart 1999, pp. #4-14 to 4-33).

The particle-size distribution test procedure was ASTM C136-96a and ASTM C117-95, *Standard Test Method for Materials Finer than 75μm (No. 200) Sieve in Mineral Aggregates by Washing*. The sieved material is washed through the No. 200 sieve. Note that, due to the small size of the bulk samples used for shipping and handling convenience, the mass of the samples used in the sieve analyses is less than the minimum mass required in ASTM C 136-96a. In addition, the YMP procedure *Laboratory Geotechnical Testing of Soil Samples*, NWI-SPO-003Q Rev 0 endorses ASTM D 422-63, but does not mention ASTM C 136-96a.

Therefore, these particle size test results are considered unqualified data, and will not be entered into the YMP Technical Data Management System. The particle size analyses were only used to generally classify the soil samples actually recovered for this preliminary investigation, and were not used as a basis for design recommendations in this report. The results from the flood plain soils are shown as corroborative information for comparison purposes in Table II-2.



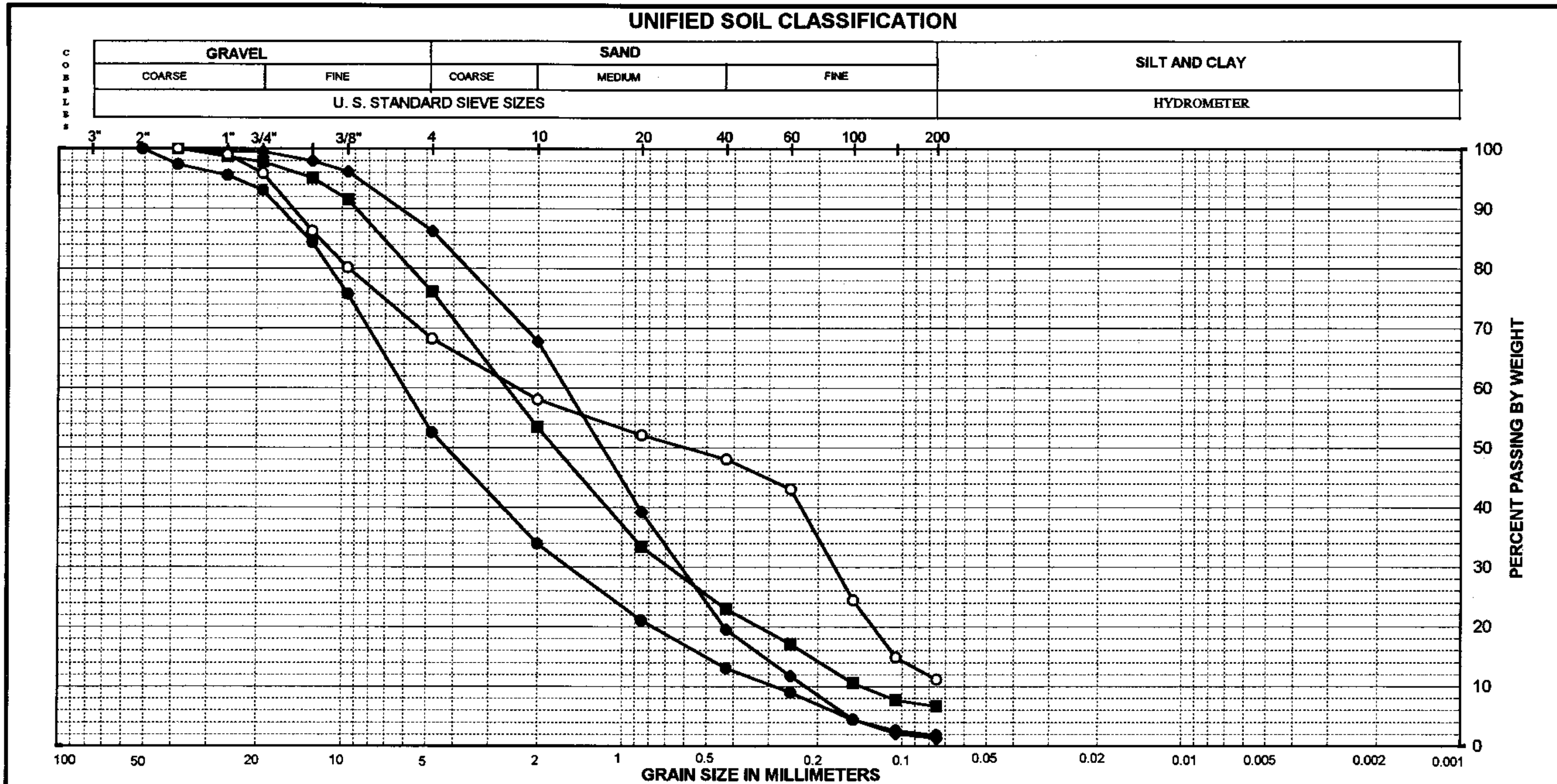
Exploration No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	CF	Description and Classification	D_{60}	D_{30}	D_{10}	Cu	Cc
00541810	p. 89		●					**Reddish brown poorly graded Sand with gravel (SP)**	2.9	0.6	0.2	14.5	0.6
00541811	p. 91		◆					**Tannish brown clayey Sand (SC)**					
00541812	p. 90		■					**Brown well-graded Sand with silt and gravel (SW-SM)**	4.2	1.2	0.18	23.3	1.9
00541813	p. 92		❍					**Brown poorly graded Sand with gravel (SP)**	2.3	0.45	0.17	13.5	0.5
00541814	p. 93		□					**Brown poorly graded Sand with gravel (SP)**	2	0.45	0.17	11.8	0.6

PROJECT NAME: Yucca Mountain Project

Source: Mineart 1999, p. # 4-14

PARTICLE-SIZE DISTRIBUTION CURVES FOR SAMPLES 00541810 - 00541814

Figure: III-1



Exploration No.	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	CF	Description and Classification	D_{60}	D_{30}	D_{10}	Cu	Cc
00541815	p. 94		●					**Brown well-graded Sand with gravel (SW)**	6	1.7	0.3	20.0	1.6
00541816	p. 95		◆					**Brown poorly graded Sand (SP)**	1.8	0.6	0.22	8.2	0.9
00541817	p. 96		■					**Brown well-graded Sand with silt and gravel (SW-SM)**	2.7	0.69	0.15	18.0	1.2
00541818	p. 97		○					**Brown poorly graded Sand with silty clay and gravel (SP-SC)**	2.4	0.18	0.07	34.3	0.2

PROJECT NAME: Yucca Mountain Project

Source: Mineart 1999, p. #4-15

PARTICLE-SIZE DISTRIBUTION CURVES FOR SAMPLES 00541815 - 00541818

Figure: III-2