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FUNCTIONAL REQUIREMENTS FOR WASTE AREA GROUPING 6 MONITORING STATION 3 UPGRADE

Oak Ridge National Laboratory

operated by
Martin Marietta Energy Systems, Inc.
Oak Ridge, Tennessee 37831

for the U. S. Department of Energy
under Contract No. DE-AC05-84OR21400

prepared by

Environmental Consulting Engineers, Inc.
Knoxville, Tennessee 37833

June 1991

MASTER

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APPROVAL OF FUNCTIONAL REQUIREMENTS FOR WASTE AREA GROUP 6 MONITORING STATION 3 UPGRADE

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1. INTRODUCTION

The Monitoring Station 3 (MS 3) and its corresponding watershed are shown in Figure 1.1. The purpose of upgrading MS 3 is to

- measure surface water outflow from the watershed;
- sample the water for transport of contaminants;
- provide adequate access to the Interim Waste Management Facility (IWMF).

The watershed is located within the Waste Area Grouping 6 (WAG 6). Closure activities in the WAG-6 area and the operation of IWMF are important contributing factors for the upgrade of MS 3. The rate of contaminant transport shall be determined by a combination of

- accurate discharge measurement;
- periodic sampling to determine contaminant concentration.

All the design and performance standards in this FRD are in accordance with the U.S. Department of Energy General Design Criteria No. 6430.1A (DOE, 1989).

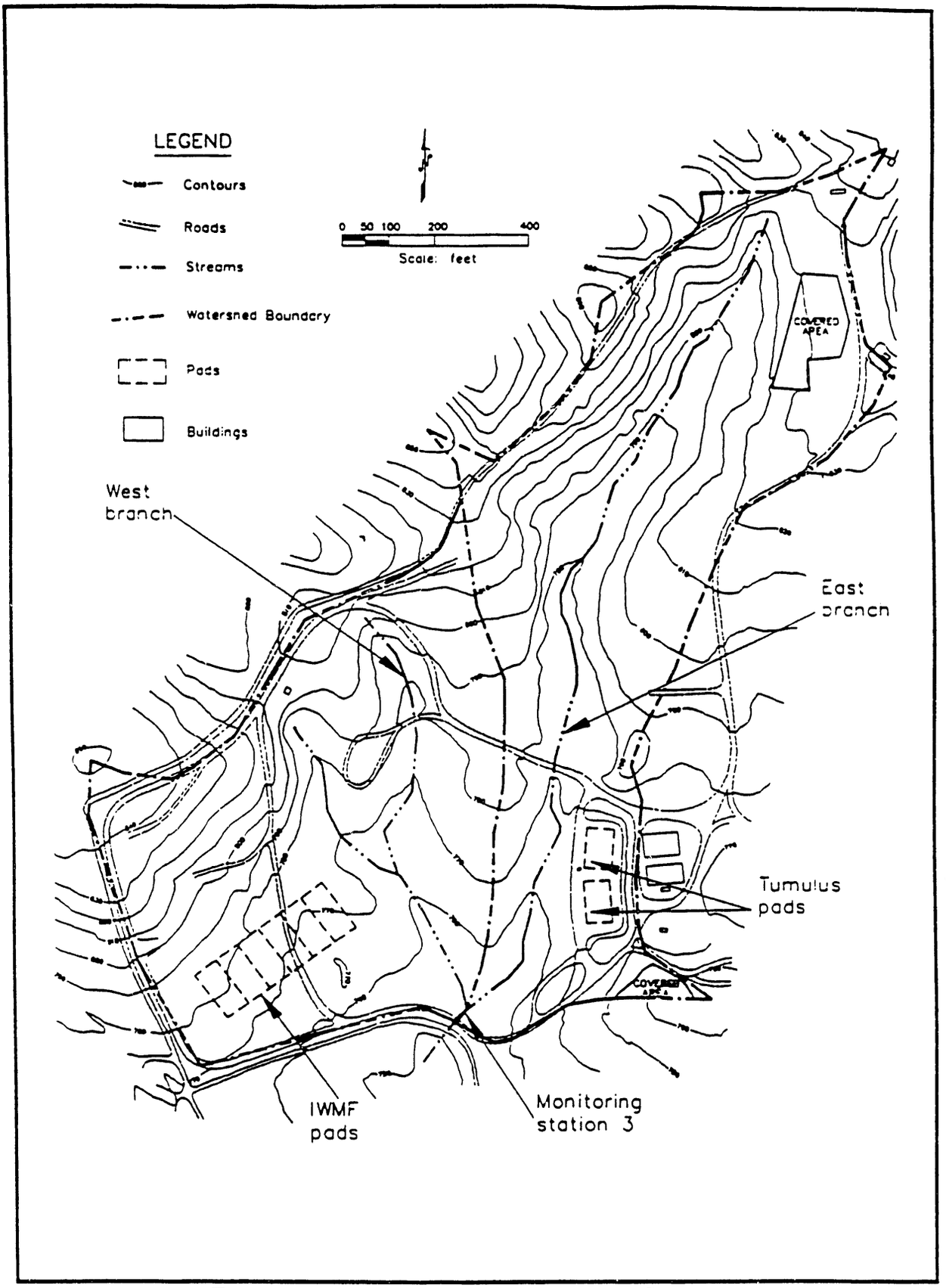

Fig. 1.1. The Watershed of Monitoring Station 3.

2. PROJECT DEFINITION

The surface of WAG 6 may undergo many changes in the future as part of the closure activities. Changes in surface characteristics will cause changes in runoff characteristics of the MS 3 watershed. Appropriate assumptions are to be made in this project regarding the future surface conditions of the watershed. The extent to which the assumed surface conditions will affect the project objectives (defined in Section 1.0) is to be identified.

The purpose of this FRD is to establish performance requirements for MS 3 consisting of the following:

1. The expected range of discharges passing through the station and the required accuracy levels for discharge measurements.
2. The equipment required to measure the expected discharges at the required accuracy levels.
3. The sampling requirements for monitoring water quality.
4. The hydraulic requirements for the discharge conveyance structure to be located under the IWMF access road.
5. The design loads to be used for the bridge over the IWMF access road.

3. MISSION

The missions of the upgraded Monitoring Station 3 are:

1. To monitor water discharges during the WAG 6 closure and post-closure activities;
2. To monitor water discharges resulting from the operation of the IWMF; and
3. To provide an upgraded structure to allow passage of waste vehicles to IWMF over the road and unobstructed flow of water under the road.

4. REQUIREMENTS AND SPECIAL ISSUES

4.1 REQUIREMENTS AND SPECIAL CONSTRAINTS

4.1.1 DOE Design Storm Requirements

Design storm requirements for stormwater management systems at DOE sites were published in DOE 6430.1A in 1989 (DOE, 1989). That document states that open channel stormwater conveyance systems shall be sized to accommodate the peak flow from the 25-year, 6-hour design storm, while checking for the 25-year, 24-hour storm.

Because it is necessary to define the range of storm flows for which accurate discharge measurements will be required, 6-hour design storms have been developed for several frequencies, including the 2, 5, 10, 25, 50 and 100-year events. The procedure for developing the design storms and determining the outflow hydrographs has been shown in Appendix A. This procedure is provided as reference material and is not meant as the only possible procedure for developing design storms.

4.1.2 Monitoring Equipment Requirements

4.1.2.1 Monitoring constraints and regulations

In order to define functional requirements for the discharge and water quality monitoring station, many factors were reviewed. First, pertinent published regulations and policies concerning monitoring requirements were reviewed. The review revealed that there are no current regulatory requirements for discharge and/or water quality monitoring at MS 3. However, with the impending stormwater National Pollutant Discharge Elimination System (NPDES) regulations, and the potential long-term monitoring requirements after WAG 6 closure, there is certain future need for both discharge and water quality monitoring.

The Tumulus I and II pads are in the MS 3 watershed (Figure 1.1), and runoff from these projects is included in the current NPDES permit for the Oak Ridge National Laboratory (ORNL). Environmental Monitoring and Compliance (EMC) staff is currently preparing a permit modification for the IWMF runoff and pad drainage. Both the Tumulus and IWMF sites are upstream of the MS 3, and their permit constraints do not directly impact data requirements at MS 3.

Interviews were conducted with ORNL personnel to obtain further regulatory information, general policies, research needs, and planned activities which may impact the MS 3 monitoring requirements. Two meetings were held with EMC staff, one with Charlie Valentine and another with John Murphy. Engineering requirements were obtained during an interview with Dirk van Hoesen. Mr. Van Hoesen also provided ECE with several documents as references. In addition, meetings were held with Roger Clapp and Dennis Borders of the Environmental Sciences Division (ESD). In these meetings, present and future policies and research needs were discussed. The current emphasis toward flumes and away from weirs as flow measuring devices was identified.

From these review efforts, it was determined that stormwater NPDES, closure/post closure monitoring, and general research needs are applicable and should drive the design efforts. These requirements dictate that both accurate discharge measurements over the widest possible range and flow proportional automatic water quality sampling are needed. The following subsections outline the specific requirements.

4.1.2.2 Discharge measurement requirements at ORNL

To meet the requirements for discharge monitoring at MS 3 given in the previous section, the following constraints shall be observed:

1. To minimize the problem of handling of contaminated sediments, avoid weirs or other devices that cause upstream sediment deposition.
2. Use a structure that will miss no more than 2 1/2 percent of the highest and 2 1/2 percent of the lowest discharges.
3. Use a structure that can measure the discharge as accurately as possible within the entire range of its operation.
4. Use a structure that can be monitored by an automatic recorder or logger.

As stated previously, there are no reliable flow frequency data available for the MS 3 watershed. Therefore, identification of the upper and lower 2 1/2 percent of the discharges is not possible. Fortunately, however, there are flow measuring devices that can measure a large range of flow rates very accurately, as given in Section 4.1.3.

4.1.2.3 Water quality sampling requirements at ORNL

In view of the water quality monitoring requirements outlined in Section 4.1.2.1, the following shall be observed:

1. The structure shall be self cleaning, so that sediment and debris are not accumulated.
2. The sampling equipment shall allow automatic sampling.
3. The sampling equipment shall allow flow proportional sampling.
4. The sampling equipment shall be capable of refrigerating sample volumes of several liters.
5. Adequate security shall be provided for the sampling and storage equipment.
6. Any information collected from the flow measurement equipment shall be compatible with the collecting organization's other existing hardware.

4.1.3 Monitoring Equipment Selection

4.1.3.1 Flow measurement equipment

A review of the capabilities and specifications of several flow measuring flumes indicates that a 4.5-ft H flume can be used to monitor discharge at MS 3 (USDA, 1962). Dimensions of the 4.5 ft H flume are given in Figure 4.1. The discharge rating table for this flume is attached in Appendix B. Analytic equations for the flume discharge rating are given in Ackers et al. (1978). The flume is capable of accurately measuring discharges from 0.0031 cfs (at a head of 0.02 ft) to 84.0 cfs (at a head of 4.49 ft). Based on the discussion in Appendix A (Section 8.1), and the constraints in Section 4.1.2.2, this range is adequate to measure the required range of discharges.

The H flume is an accurate device for flow measurement and shall, therefore, be used for this application. The rating table given for the H flume (Appendix B) has been developed for the flume operating in free condition, i.e., water flowing in a free nappe from the flume crest. However, the H flume can operate satisfactorily with a certain degree of submergence. Studies have shown that for a submergence of 50%, the flume accuracy drops less than 3% (USDA, 1962). The definition sketch of flume submergence and the effect of submergence on accuracy is attached in Appendix C. Requirements for water level determinations and installation to avoid large submergences are given in Section 4.1.3.3.

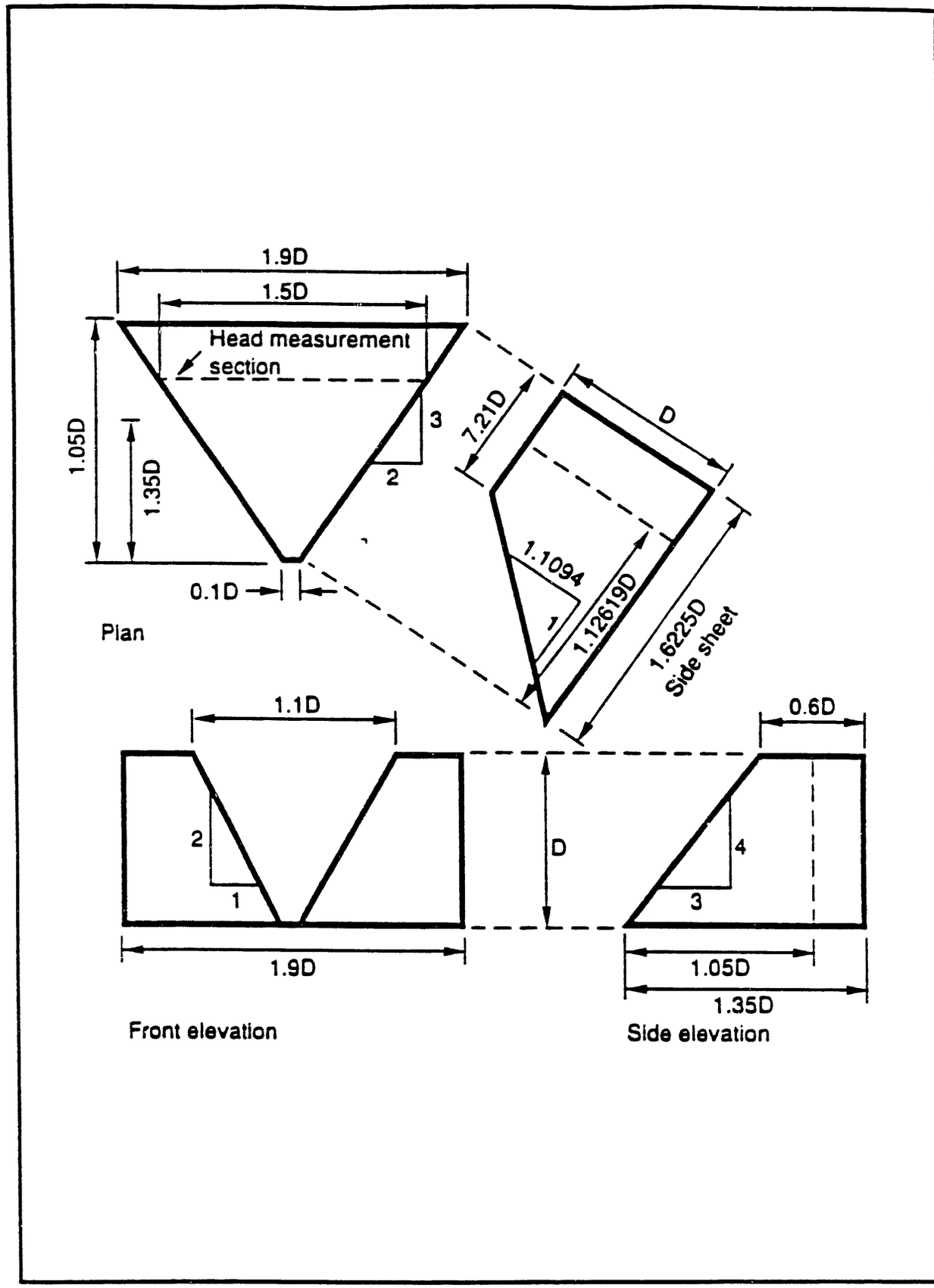

Fig. 4.1. H flume dimensions, D = 4.5 Ft. (Grant, 1989).

4.1.3.2 White Oak Lake headwater elevation assessment

Large storms occurring on the MS 3 watershed are likely to be accompanied by high levels in White Oak Lake. Consideration of expected White Oak Lake levels was therefore used in the selection of the appropriate discharge measurement device at the MS 3.

Normal pool for White Oak Lake is elevation 744.0 (Tschantz, 1987). It has been conservatively assumed that 25-year frequency events will occur simultaneously on the MS 3 watershed and the 6.01 square mile White Oak Lake watershed. Based on the White Oak Dam discharge frequency curve presented by the Corps of Engineers, the 25-year peak dam discharge is about 900 cfs (COE, date unknown). From a previously developed rating curve (Tschantz, 1987) White Oak Lake headwater elevations corresponding to a discharge of 900 cfs would be between elevations 751 and 754, depending upon the position of the two lift gates at the dam.

White Oak Lake was recently as high as elevation 751 (personal communication, Dennis Borders) following heavy rains in February, 1991. While these storms brought several inches of rain over a three day period, they were not particularly intense over short durations.

Based on the considerations discussed above, the elevation of White Oak Lake selected for the 25-year design storm is 752.0. Discharge capacity at the White Oak Dam at this elevation is about 1600 cfs. However, there is no formal policy for operation of lake levels in large storms. It is possible that much smaller storm events could create backwater problems at the discharge measuring station if runoff is temporarily stored in White Oak Lake.

4.1.3.3 Installation of H flume and box culvert

As described in Section 4.1.3.1, an H flume 4.5 feet deep will provide reliable discharge measurement accuracy over the required flow range. The H flume was developed in the 1930's to measure runoff from small agricultural watersheds. When properly constructed and installed, the H flume has proven to be a highly accurate flow measuring device.

In order to effect minimum disturbance of the ground on the upstream side of the IWMF access road, the flume shall be placed at the downstream end of a box culvert under the IWMF access road. The box culvert shall be designed to provide both the required structural integrity and adequate approach conditions to the H flume. The upstream end of an H flume 4.5 feet deep is 8.55 feet wide. The box culvert should therefore be at least 8.55 feet wide and at least 4.5 feet deep (inside dimensions). Because the watershed is within a fenced, limited public access area, adequate measures against public access should be taken in accordance with DOE 6430.1 A (DOE, 1989).

Ideally, the flume should be installed so that it can discharge freely with no downstream submergence. However, submergence of 30 percent has less than 1 percent effect on the calibration (USDA, 1962; also, see Appendix C). As shown in Appendix B, the 25-year design flow of 71.4 cfs can be carried at a head of 4.2 feet in the H flume. If minimum cover depths over the culvert are desired, the floor of the flume may be placed 1.26 feet (30 percent of 4.2 feet) below the design lake elevation of 752.0. The flume floor would therefore be at elevation 750.74, which may be below the existing ground surface elevation. The flume floor may be placed at a higher elevation, which would reduce the degree of submergence, and provide better accuracy at higher flows. Schematics of the H flume, box culvert approach section, and installation profile are shown in Figures 4.2, 4.3, and 4.4.

The approach section upstream of the H flume should be at least 22.5 feet long, with a longitudinal bottom slope of 2 percent (Figures 4.2, 4.4). The length of the box culvert will be dependent upon construction details which are beyond the scope of this document. If the box culvert length is more than 22.5 feet, the remaining upstream portion of the culvert should be sloped to ensure adequate flow capacity in the 25-year storm, and to minimize the excavation required during construction. To minimize the geomorphologic impact of the proposed structure on the upstream channel, the upstream invert elevation of the box culvert shall be as close to preconstruction grade

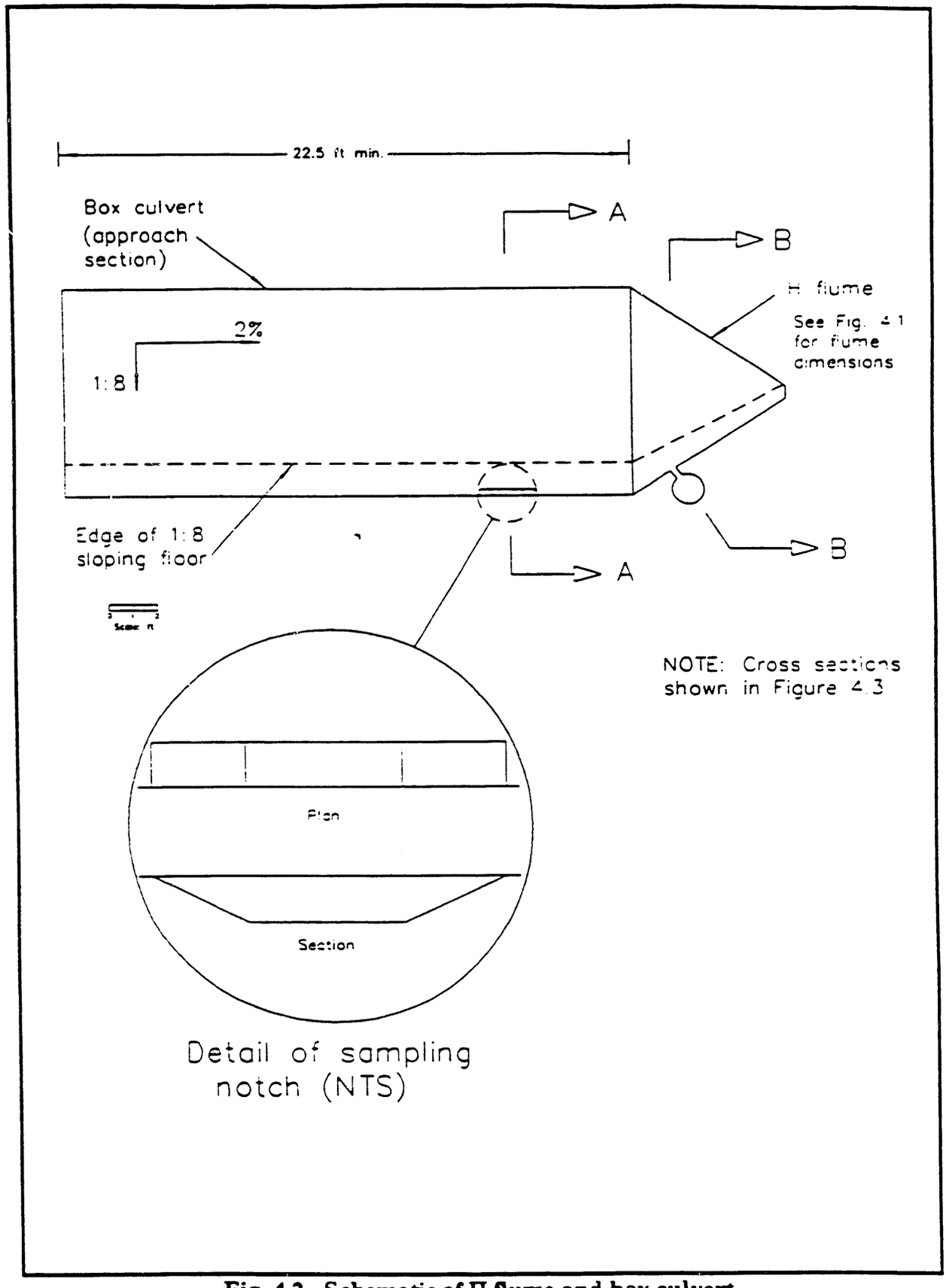

Fig. 4.2. Schematic of H flume and box culvert.

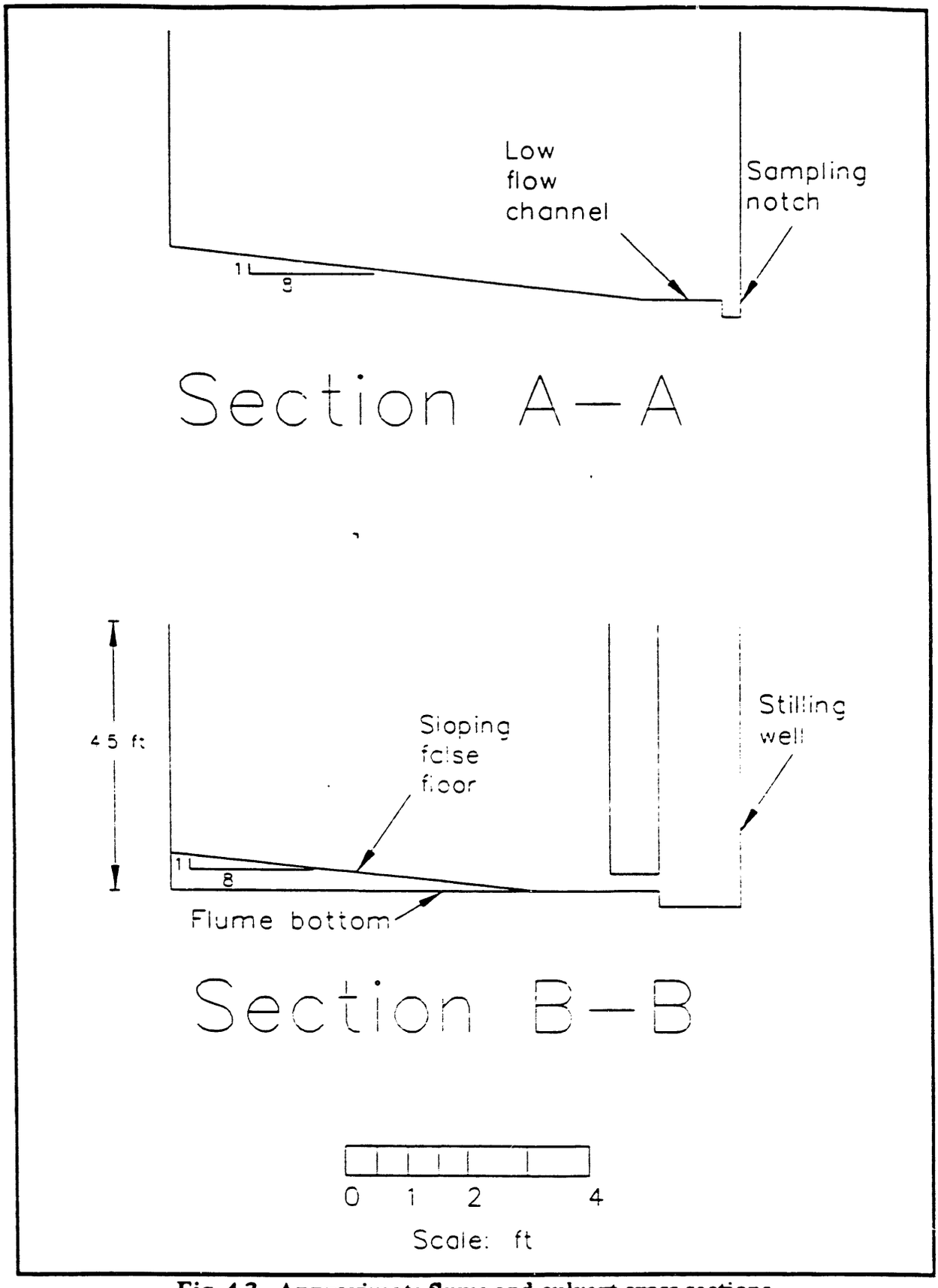

Fig. 4.3. Approximate flume and culvert cross sections.

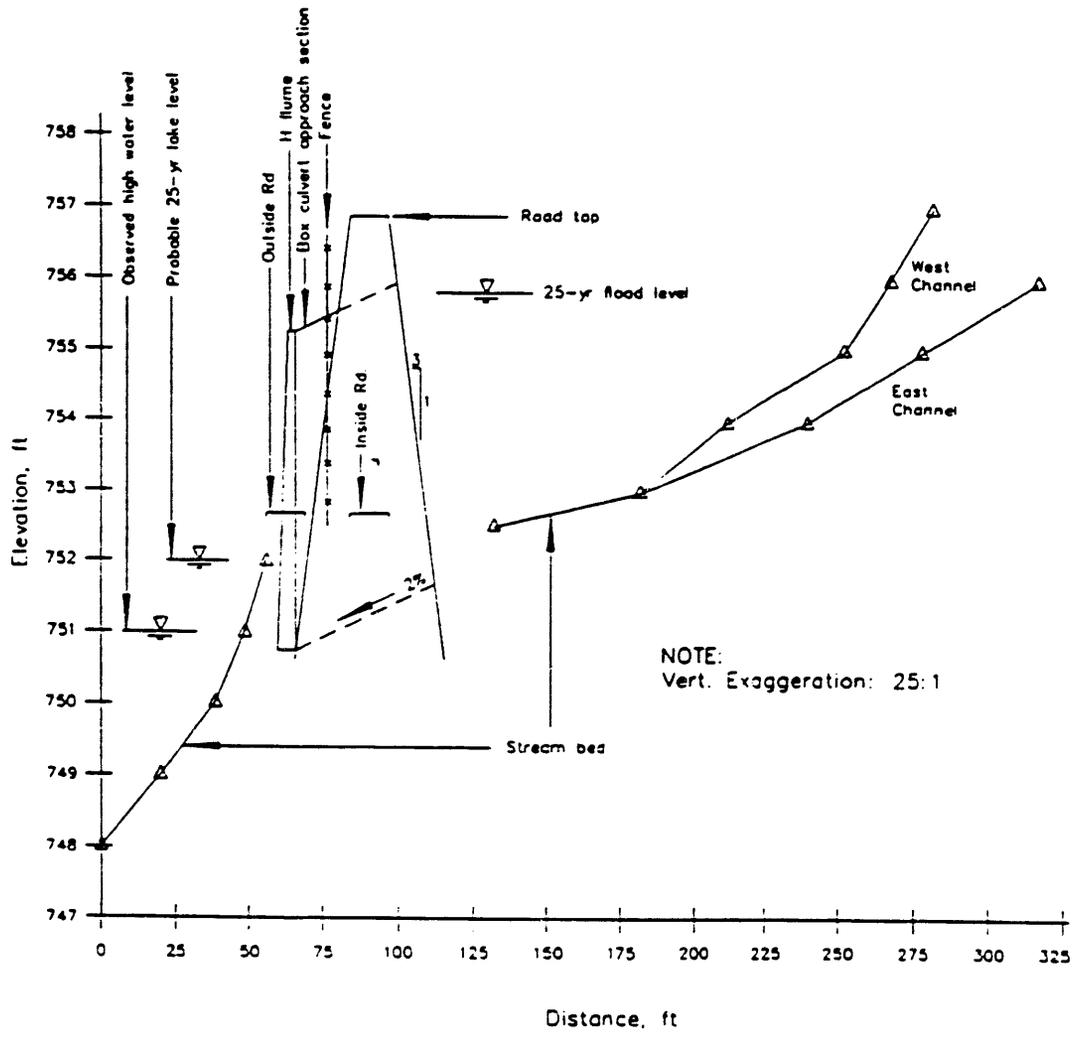

Fig. 4.4. Proposed profile view of MS 3.

as practicable. The top of the exposed portion of the approach section (Figure 4.4) shall be open to facilitate access to the water quality sampling notch.

The H flume shall be provided with a 1-on-8 sloping false floor to concentrate low flows and reduce siltation problems (Figure 4.3). The rating table shown in Appendix B is for a flume with a sloping false floor. The stilling well for the water stage measuring device shall be placed along the low flow channel side of the flume. Several techniques for measuring the depth of water in the stilling well are available, including sonic devices, gas bubblers, pressure transducers, and floats. The device chosen shall provide a high level of accuracy; for this reason a float is not acceptable. The low flow channel can be on either side of the flume. A similar low flow channel shall be provided inside the box culvert (Figure 4.3).

The low flow channel inside the box culvert shall include a small notch sized to accommodate the placement of the water quality sampling strainer. The dimensions of the notch will be dependent on the size of strainer selected, but should allow at least one inch between the bottom of the strainer and the bottom of the notch, and at least one inch from the top of the strainer and the floor elevation of the box culvert. The strainer shall be positioned in the appropriate place by using metal or durable plastic clips attached to the wall of the box culvert. The notch shall provide a smooth transition region for low flows, and shall create a pool sufficient for sample withdrawal. The notch shall be hydraulically efficient as well, so that velocities through it are sufficient to avoid trapping sediment, debris, or stagnant water.

The H flume and approach section shall be installed in strict accordance with the criteria given by the Agricultural Research Service (USDA, 1962). Installation of the flume and box culvert may be subject to the Aquatic Resource Alteration Permit requirements in which case the appropriate permit must be obtained and/or appropriate construction techniques observed. Also applicable may be the Corps of Engineers Section 404 and Tennessee Valley Authority "26 a" permit requirements, which must be fulfilled before construction of the MS 3 can be undertaken.

4.1.3.4 Water quality sampling equipment

Current sampling technology will allow automatic sampling, flow proportional sampling, and sample refrigeration of large volumes. For example, the Isco Environmental Division of Isco, Inc. manufactures a large variety of automatic wastewater samplers. Such samplers shall be reviewed, so that the equipment best suited for the specific application at MS 3 can be identified. The installation of the equipment in the field shall be complemented with structural features to provide self cleaning capability and security in accordance with DOE 6430.1 A (DOE, 1989).

4.1.4 The IWMF Access Requirements

The access road to the IWMF crosses the drainage channel at MS 3. The culvert presently under the road shall be replaced by a larger structure, capable of passing storm discharge in the 25-year design storm without overtopping the road and without drowning out the MS 3 discharge measurement equipment.

It is not the intent of this document to provide structural design requirements for the drainage structure under the access road. However, it is expected that fully loaded 18 wheel trucks will be used to transport waste to the IWMF. Therefore, the design loading to be used for the structure shall be based upon the HS 20-44 specifications developed by the American Association of State Highway and Transportation Officials (AASHTO, 1977).

A box culvert shall be used to provide the combination of discharge capacity and structural integrity required. Details concerning culvert sizes and invert elevations are given in section4.1.3.3. If the culvert is placed at the existing channel bottom grade, and assuming a 1 foot wall thickness with no earth cover, the centerline road elevation at the midpoint of the culvert will be about 756.9. Figure 4.5 shows the approximate position of the raised road. Elevations given in this section are based on the February 1990 topographic map, assumed to be precisely accurate. The map is

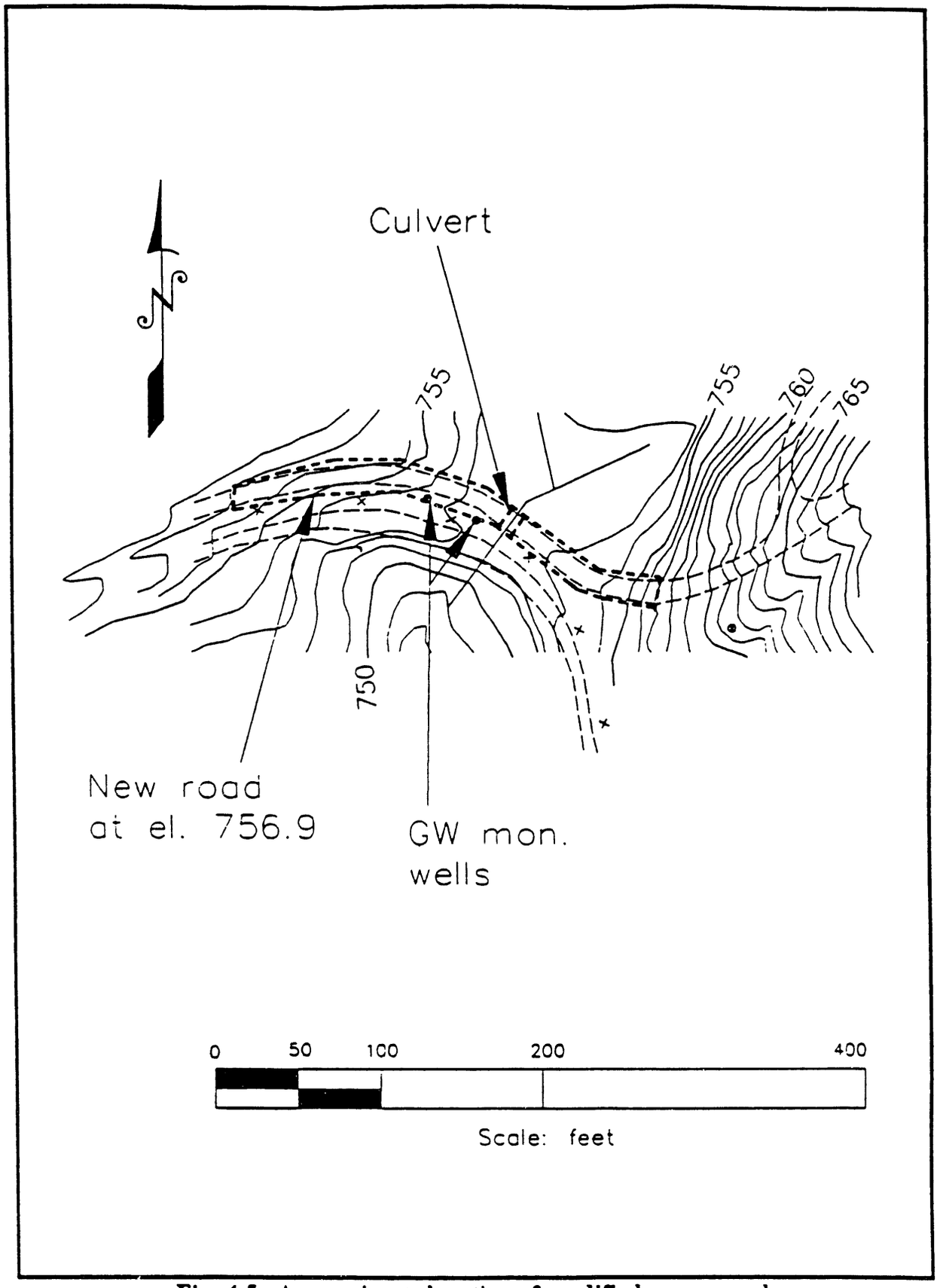

Fig. 4.5. Approximate location of modified access road.

probably accurate only to within ± 1 foot. There are two groundwater monitoring wells which may be below ground if the road embankment extends over them. Casings of these wells will have to be raised.

4.2 UNCERTAINTIES

A National Environmental Policy Act (NEPA) determination is not in place for this site. A request for categorical exclusion determination has been processed. Design is scheduled to start in the 4th quarter of FY 1991 so that construction may be initiated in the 1st quarter of FY 1992 and be complete in the 2nd quarter of FY 1992.

The hydrologic assumptions have been consistently on the conservative side, so there is little uncertainty involved with the capacity of a 4.5 ft H flume to measure the runoff resulting from a 25-year storm. There is very limited information on the elevation of water surface in White Oak Lake, which remains an uncertain point in the installation of the flume. The design elevation of the flume invert is uncertain at this time, and will be governed by the need to minimize excavation, the desire to minimize ponding the upstream section, and the requirement to reduce submergence by White Oak Lake at the discharge point.

Water quality monitoring requirements are not well defined at this time, and will depend on the outcome of the application for categorical exclusion. Permit requirements mentioned in Section 4.1.3.3 are uncertain at this time.

4.3 INTERFACES AND INTERFACE REQUIREMENTS

This project will have to interface with other WAG 6 closure construction projects in the immediate vicinity, and the construction and operation of IWMF. There will be an interface with the ORNL Surface Water Monitoring Program and ORNL Waste Operations groups since the area immediately west of the MS 3 will be operational.

There are no requirements for electrical service or other utilities except at the monitoring station.

4.4 SPECIAL ANALYSES

A special analysis is required to determine the structural capability of the culvert to support the equipment associated with operation of IWMF. The analysis should include horizontal and vertical loads.

A hydrologic and hydraulic analysis may be required to verify the design storm and the peak runoff flow rate.

A special analysis of the culvert foundation may be required to ensure that the H flume will remain in its horizontally installed configuration throughout the life of the project.

4.5 HUMAN FACTORS

The monitoring station shall be designed to allow safe ingress and egress. Electrical outlets shall be furnished with ground fault protection devices.

4.6 MAINTENANCE

Water level monitoring and water quality sampling will be automated to minimize maintenance needs. The selected equipment is basically self-flushing. However, deposition of fine sediment and growth of molds and algae are possible. The H flume and its stilling basin as well as the water quality sampling notch will have to be inspected regularly, and cleaned out if necessary.

4.7 CRITICAL SYSTEMS

There are no critical systems associated with this project.

4.8 TESTING REQUIREMENTS

Access to all monitoring and sampling locations shall be provided.

The H flume shall be calibrated.

All test plans, testing, and test results will be documented and submitted to Environmental Restoration Division (ERD) for review. All problems and deficiencies encountered during testing will be resolved and corrective action taken in accordance with applicable QA procedures.

5.0 CONFIGURATION CONTROL PLAN

All changes to the approved design will be reviewed and approved by the Program Project Manager and ERD in accordance with the Configuration Control Plan for WAG 6.

6.0 DISTRIBUTION LIST

All organizations represented by the signatures to this document shall receive a copy of all revisions to same.

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Appendix A

DEVELOPMENT OF DESIGN STORMS AND PEAK FLOW RATES

1. TECHNICAL PAPER NO. 40

Generalized rainfall-frequency data for the Oak Ridge area are published in the United States Weather Bureau (USWB) Technical Paper Number 40 (USWB, 1961). TP-40 provides rainfall data for areas less than 400 square miles, for durations from 30 minutes to 24 hours, and for frequencies from 1 to 100 years.

The information presented in TP-40 is based upon statistical analyses of rainfall data collected in the United States. The database consists of long-term records from 200 first-order Weather Bureau stations, supplemented by less comprehensive data from an additional 5000 stations.

For durations less than 30 minutes, TP-40 suggests using ratios of shorter duration values to one-hour values. These ratios are independent of both geographic location and return period. It has been shown, however, that these ratios have a describable geographic pattern, and that they vary with return period (NOAA, 1977).

2. TECHNICAL MEMORANDUM NWS HYDRO-35

To meet the demand for better small area, short duration rainfall data, the National Oceanic and Atmospheric Administration (NOAA) published Technical Memorandum NWS HYDRO-35 (NOAA, 1977). HYDRO-35 presented values for precipitation-frequency for durations from 5 minutes to 1 hour. The database used for the revised analysis was similar to that used for TP-40, with records from 200 first-order National Weather Service (NWS) stations, and additional records from about 1900 recording rain gages.

Tables 2.1 and 2.2 summarize the precipitation data for the WAG 6 area published in TP-40 and HYDRO-35, respectively. The tables include precipitation depths for both partial duration series and annual series. The published precipitation data are based on partial duration series analyses, where all rainfall events above a base level are included in the database. These values have been converted to annual series data, which are based on one maximum event per year. The two series are approximately equal for events exceeding the 10-year frequency.

3. TECHNICAL PAPER NO. 149

While TP-40 and HYDRO-35 provide comprehensive precipitation depth-duration-frequency data, they do not define the time distribution of storm rainfall necessary to develop design storms. The Soil Conservation Service (SCS) originally developed two dimensionless 24-hour rainfall distributions for the continental United States (SCS, 1973). The Type I and II distributions were developed from the generalized precipitation depth-duration-frequency relationships published in USWB technical papers. The Type I distribution was developed for areas with maritime climates. The Type II distribution was developed for areas where the highest rates of runoff from small areas usually occur during summer thunderstorms. These distributions were developed by plotting the ratio of rainfall amount for any duration to the 24-hour amount versus duration for many different locations, and selecting a best fit curve to the data.

The Type II distribution, which should be used for the entire Tennessee Valley, is arranged so that the greatest 30-minute depth occurs near the middle of the 24-hour period. The second largest depth is assumed to occur in the next 30 minutes, and the third largest in the preceding 30 minutes. This alternation is continued for each decreasing order of magnitude, until the smallest 30-minute depths fall at the beginning and end of the 24-hour period. Since the highest rainfall intensities are

successively nested within longer duration, lower intensities, this distribution can be used to develop design storms for both small and large areas.

The SCS has since developed two additional distributions, Types IA and III. Both are modifications to the original Type I, and were developed for areas where the Type I distribution did not closely match observed storm patterns. However, the Type II distribution remains applicable to the Tennessee Valley. The Type II distribution is shown in Figure 3.1.

Table 2.1. Precipitation depth-duration-frequency data for Anderson County, Tennessee per TP-40

	Rainfall depths in inches for indicated frequency (partial duration series)					
Duration	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
5 MIN	0.44	0.56	0.67	0.74	0.81	0.89
10 MIN	0.68	0.86	1.03	1.14	1.25	1.37
15 MIN	0.86	1.08	1.30	1.44	1.58	1.73
30 MIN	1.20	1.50	1.80	2.00	2.20	2.40
60 MIN	1.50	1.90	2.20	2.50	2.80	3.00
2 HR	1.80	2.40	2.70	3.10	3.40	3.80
3 HR	2.00	2.50	3.00	3.40	3.70	4.10
6 HR	2.50	3.00	3.50	4.00	4.60	4.90
12 HR	2.90	3.60	4.20	4.70	5.40	5.80
24 HR	3.30	4.20	4.80	5.50	6.20	6.60

	Rainfall depths in inches for indicated frequency (annual series)					
Duration	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
5 MIN	0.39	0.54	0.66	0.74	0.81	0.89
10 MIN	0.60	0.83	1.02	1.14	1.25	1.37
15 MIN	0.76	1.04	1.29	1.44	1.58	1.73
30 MIN	1.06	1.44	1.78	2.00	2.20	2.40
60 MIN	1.33	1.83	2.18	2.50	2.80	3.00
2 HR	1.59	2.31	2.67	3.10	3.40	3.80
3 HR	1.77	2.40	2.97	3.40	3.70	4.10
6 HR	2.21	2.88	3.47	4.00	4.60	4.90
12 HR	2.57	3.46	4.16	4.70	5.40	5.80
24 HR	2.92	4.04	4.75	5.50	6.20	6.60

Table 2.2. Precipitation depth-duration-frequency data for Anderson County, Tennessee per HYDRO-35

	Rainfall depths in inches for indicated frequency (partial duration series)					
Duration	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
5 MIN	0.43	0.50	0.56	0.64	0.71	0.77
10 MIN	0.68	0.81	0.91	1.05	1.17	1.28
15 MIN	0.85	1.03	1.15	1.34	1.49	1.63
30 MIN	1.14	1.42	1.62	1.91	2.13	2.35
60 MIN	1.45	1.84	2.11	2.50	2.80	3.10

	Rainfall depths in inches for indicated frequency (annual series)					
Duration	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
5 MIN	0.38	0.48	0.55	0.64	0.71	0.77
10 MIN	0.60	0.78	0.90	1.05	1.17	1.28
15 MIN	0.75	0.99	1.14	1.34	1.49	1.63
30 MIN	1.01	1.37	1.61	1.91	2.13	2.35
60 MIN	1.28	1.77	2.09	2.50	2.80	3.10

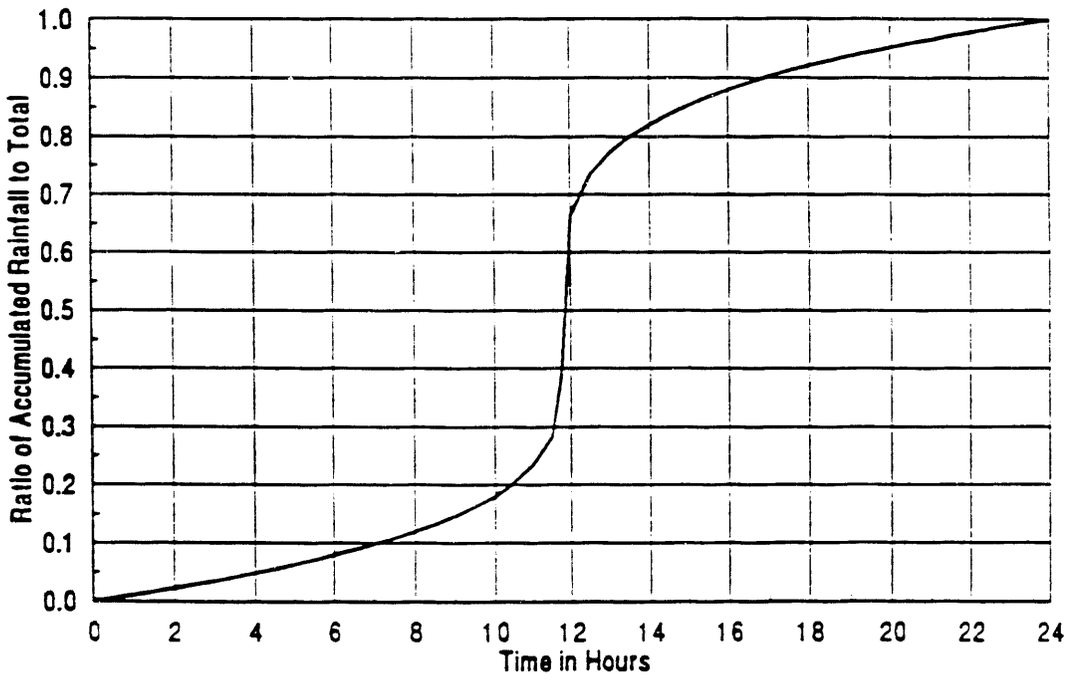


Fig. 3.1. SCS Type II rainfall distribution.

4. DESIGN STORMS

It was determined that, because of the relatively small size of the study watersheds, the duration of incremental rainfall values necessary to adequately reflect the watershed response time should be less than 2 1/2 minutes. The design storms were therefore developed in 2-minute increments for each frequency using rainfall data from HYDRO-35 (NOAA, 1977) for durations up to 1 hour and from TP-40 (USWB, 1961) for durations larger than 1 hour. For each frequency, the depth duration data were plotted on a semilog scale (with time plotted on the log axis), and a best fit curve of the form $y = a*\ln(x) + b$ computed. These curves (which plot as straight lines on the semilog graph paper) are shown on Figure 4.1.

The equations were used to determine 2-minute incremental rainfall values. These equations tend to underestimate HYDRO-35 rainfall values for 5-minute duration. The peak incremental rainfall values were therefore adjusted as required to match the intensity of the HYDRO-35 5-minute data (the first and second ordinates, which represent 4 minutes of rainfall, were set equal to this intensity). For each frequency, a total of 180 2-minute incremental rainfall values were determined. These values were ordered in a fashion similar to the SCS Type II distribution, with the largest rainfall ordinate followed by the second largest and preceded by the third largest, etc. The sequence of rainfall ordinates for each design storm is therefore:

179, 177, 175,..., 3, 1, 2, 4,..., 176, 178, 180.

5. SELECTION OF RUNOFF MODEL

The U.S. Army Corps of Engineers' HEC-1 model was used to determine the expected flow rates at MS 3. The model is an industry standard for developing runoff hydrographs from single rainfall events. The HEC-1 model is also very versatile in allowing the selection of a variety of available methods for calculation of the different elements of the discharge hydrograph. Discharge hydrographs for this study were developed using the SCS curve number method to determine runoff volumes, and the SCS dimensionless unit hydrograph for developing the discharge hydrograph.

6. DETERMINATION OF SUBWATERSHED HYDROLOGIC PARAMETERS

Two parameters are needed to characterize the hydrologic properties of the subwatersheds for the SCS method. These are the SCS curve number and the basin lag. The curve number depends on the surface conditions and is usually chosen from published tables. Based on a visit to the site and the previously published literature, a uniform curve number of 72 was selected for the entire watershed of MS 3.

The basin lag, the delay in time between a brief heavy rain over the watershed and the time when runoff at the watershed outlet reaches a peak (SCS 1972), is actually a weighted time of concentration. It is determined using equations given by SCS, and depends on the curve number, the average watershed slope, and the hydraulic length of the watershed. These parameters were determined for the two subwatersheds draining into MS 3. The branches draining these two subwatersheds were called the East and the West branch (see Figure 1.1 in the main body).

Average subwatershed slopes for the two branches were determined using the grid method. A 250 foot square grid was superimposed on a February, 1990 topographic map of the watershed. The maximum surface slope between each grid point and the receiving drainage channel was determined.

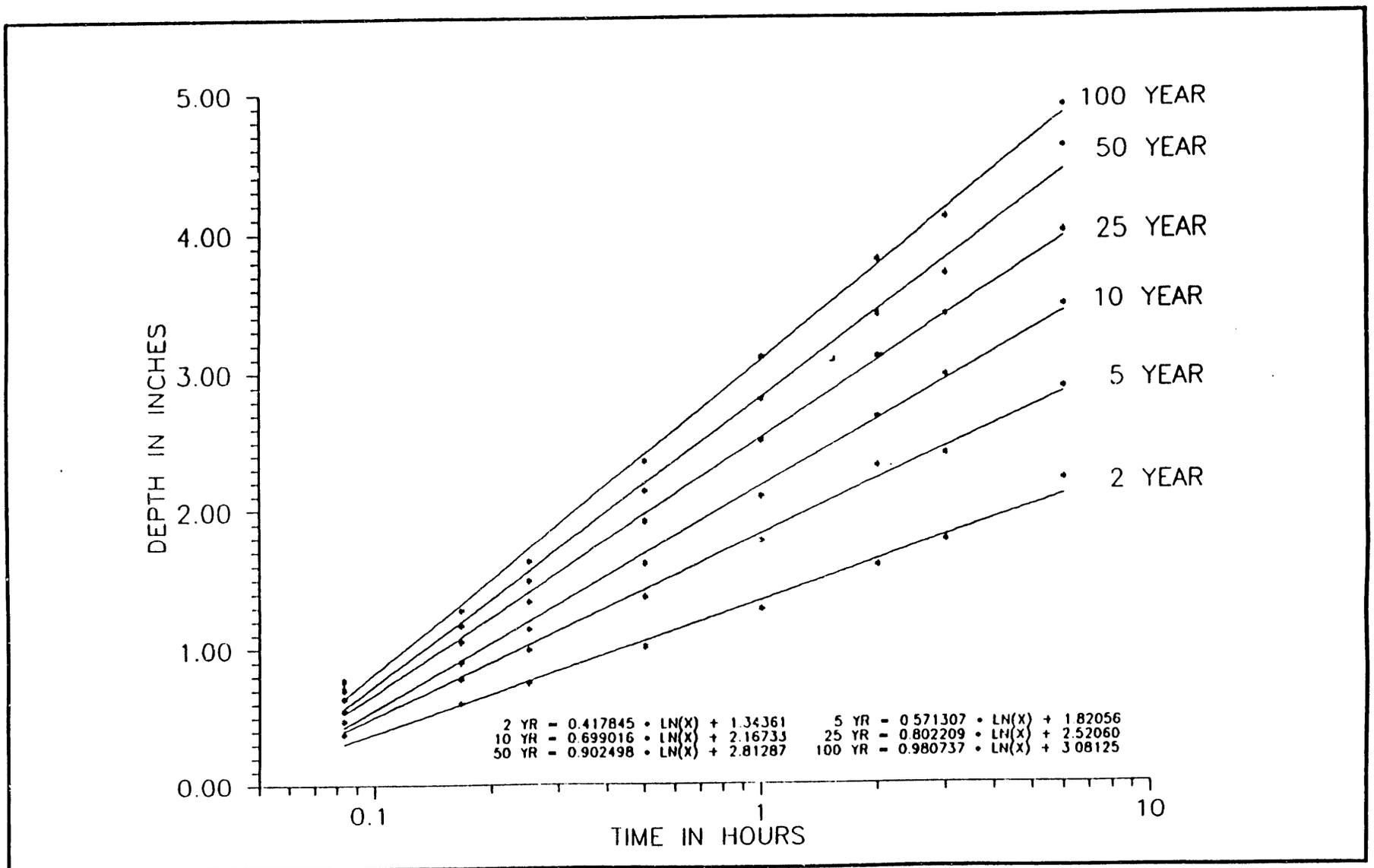


Fig. 4.1. Precipitation depth vs. duration, Anderson County, Tennessee.

The arithmetic mean of the grid point slopes for each subwatershed was calculated, and used as its average slope.

Hydraulic length of a watershed is the maximum length that water would travel between the divide and the outlet. The same topographic map of WAG 6 was used to determine the hydraulic lengths of the two subwatersheds. Based on these parameters, the lag times for the East and the West branches were found to be 0.22 and 0.15 hours, respectively. A summary of the hydrologic characteristics of the subwatersheds is shown in Table 6.1.

7. ASSUMPTIONS FOR GENERATED DATA

Two conditions have been assumed for the purpose of generation of runoff hydrographs. These are referred to as the "present condition" and the "future condition." The present condition refers to the relatively near future, when the Tumulus pads will be filled and capped, the IWMF will be in the process of filling, and no other alterations will be done to the surface conditions of the watershed. The future condition refers to the time when the IWMF is completely filled and capped. Areas presently capped with an impermeable flexible membrane liner may or may not be capped with an engineered clay cover in the future.

7.1 THE IMPERMEABLE CAPS

There are two areas within the East branch subwatershed that have been capped with flexible membrane liners, constituting a minute portion of the subwatershed area. For the present condition, it has been assumed that the surface cover of these areas will remain unchanged. Visual inspection of the installed french drains and ditches indicated that in case of a very heavy rain, the ditches will probably run overbank and the water will drain into the East branch as surface runoff. Therefore, these areas have been treated in the HEC-1 model as impermeable surfaces. For the future condition, if the liners are replaced with an engineered multi-layer clay cap, they will drain more slowly, and will not result in any increase in peak runoff hydrograph flow rate.

7.2 THE TUMULUS PADS

As mentioned above, the Tumulus pads have been assumed to be filled and capped for the present condition. Therefore, their response to the runoff hydrograph will be similar to the rest of the watershed surface, and no special treatment of that portion of the watershed will be needed.

Table 6.1. Summary of Hydrologic Characteristics for Monitoring Station 3 Watershed

Parameter	West Branch	East Branch
Drainage Area (square miles)	0.03085	0.02440
SCS Curve Number	72	72
SCS Lag Time (hours)	0.15	0.22
Average Watershed Slope	0.11	0.13
Hydraulic Length (feet)	1260	2350

7.3 THE IWMF

For the present condition, it has been assumed that at any one time a maximum of two concrete pads (out of six) of the IWMF will be open to storm rainfall. One of these two pads will be constructed and awaiting wastes, and the second will be the operating pad, or the pad that is being filled. It has been assumed that any pad which is filled to capacity will be immediately capped, and can be treated as a permeable watershed surface for drainage purposes.

7.4 OTHER DISTURBANCES TO THE SURFACE

As part of operation of WAG 6 Closure, other portions of the watershed surface may undergo changes. At the present time, there is no definite record of what activities are planned for the entire WAG 6 Closure. It is, however, assumed that if any disturbance is done to the surface, it will soon be covered by an engineered cap. The capped surface(s) will act similar to the presently vegetated surface of the watershed and no special treatment would be needed for them.

8. RESULTS OF HYDROLOGIC MODELING

8.1 PRESENT CONDITION

A summary of the peak flow rates expected for various storm frequencies for the present condition is shown in Table 8.1. The peak flow rate resulting from the 25-year 6-hour design storm is 71.4 cfs. Therefore, in any given year, there is a 4% chance that the runoff flow rate will equal or exceed 71.4 cfs. A flow measuring device designed to accurately measure a peak flow rate of 71.4 cfs would miss 4% of the peak flows in an extended period of observation. The volume of runoff associated with flows in excess of 71.4 cfs would likely be much less than 4% of the total storm volume, because of the expected short duration of peak runoff from the relatively small watershed. While there are insufficient reliable flow duration data for this watershed, the annual percentage of time for which flows are expected to exceed the 25-year design storm peak is probably much less than 0.1.

Table 8.1. Summary of peak flow rates for 6-hr storms.

Recurrence Interval (yrs)	Peak Flow Rate (cfs)
2	15.1
5	19.4
10	52.0
25	71.4
50	88.2

8.2 FUTURE CONDITION

The future condition of the MS 3 watershed, as described previously, assumes complete filling and capping of the IWMF, and possible replacement of the flexible membrane liners with engineered covers. These operations will replace an impermeable portion of the watershed surface with a new surface consisting of at least 18 inches of top soil, with established vegetation. Such a layer can store up to 6 inches of water, whereas the 25-yr 6-hr storm depth is less than 4 inches. The capping activity will therefore result in infiltration of storm water into the soil, storage of large depths of water in the top soil, and a considerable delay in the drainage of storm water into the stream. The net effect of these processes is a drop in the peak runoff flow rate. Therefore, for a given rainfall, the watershed in its future condition will effect lower surface runoff volumes and discharges.

The peak design discharge for which reliable flow measurements will be required will remain 71.4 cfs. Requirements regarding suitable flow measuring devices are given in Section 4 in the main body.

8.3 PASSAGE OF THE 24-HOUR STORM

The flow rate of 71.4 cfs as found for the 25-year 6-hour storm is the peak runoff rate from the watershed surface. Because there will be some ponding of water upstream of MS 3, this flow is likely to be attenuated. However, due to uncertainties associated with the final installation, such attenuation was ignored in specifying the MS 3 requirements. For the 25-year 24-hour storm, the peak runoff flow rate is 98.9 cfs. However, based on the available elevation-storage capacity upstream of MS 3, the routed hydrograph will have a peak flow rate of 70.0 cfs, which is within the capacity of a 4.5 ft H flume.

MARTIN MARIETTA ENERGY SYSTEMS, INC.

OAK RIDGE, TN - PADUCAH, KY

ENGINEERING TRANSMITTAL

	7/8/91
PROCUREMENT BY*	FIELD WORK BY: ECE, Inc.
PROJECT TITLE: WAG 6 Functional Requirements Document (FRD)	PROJECT NO. / E.S.O. (W.O.) NO.: P-KX885-03
JOB TITLE: WAG 6 Monitoring Station 3 Upgrade	BUILDING / PLANT: X-10

ISSUE FOR: [X] Info/Record [ ] Procurement [ ] Comment [ ] Approval [ ] Approved [ ] CFC [ ] Design Comp.

Return Comments To | By (Date)

	FUNCTION	NAME	BLDG.	Mail Stop	Xmtl	Dwg. Red/Full	B/M	D/S Spec	*
PROJECT MANAGEMENT	Project Engineer	D. D. Van Hoesen	1000	6340	1				3
DESIGN	Civil Site and Waste	W. E. Manrod III	K-1550-E	7233	1				
	Management Design	A. N. Wylie	K-1550-I	7233	1				1
		P. J. Atwater	K-1550-E	7233	1				
SPECIAL									
OTHER	Data Center								
	Retain PWF								
	Engineering Services								
				TOTALS	4				4

LIST OF ATTACHMENTS & INFORMATION

[ ] RC [X] NoRC

* Functional Requirements for the Waste Area Grouping 6 Monitoring Station 3 Upgrade

RECEIVING INSPECTION REQUIRED

[ ] (1) Routine [ ] (2) Special

Refer Technical Questions

To:

Phone:

*If company, list responsible group, e.g., Engineering, Maintenance, others. Explain above.

Return originals to

[ ] Add disclaimer and substitute clause.

Originated By	Section (Group)	Division (Department)	Principal Engineer	Project Engr (Manager)
S. B. Jones		W. E. Manrod III		D. D. Van Hoesen

Appendix B
Rating Table For H Flume
4.5 Feet Deep (USDA, 1962)

Rating tables for H flume[1] (Flume 4.5 Feet Deep)

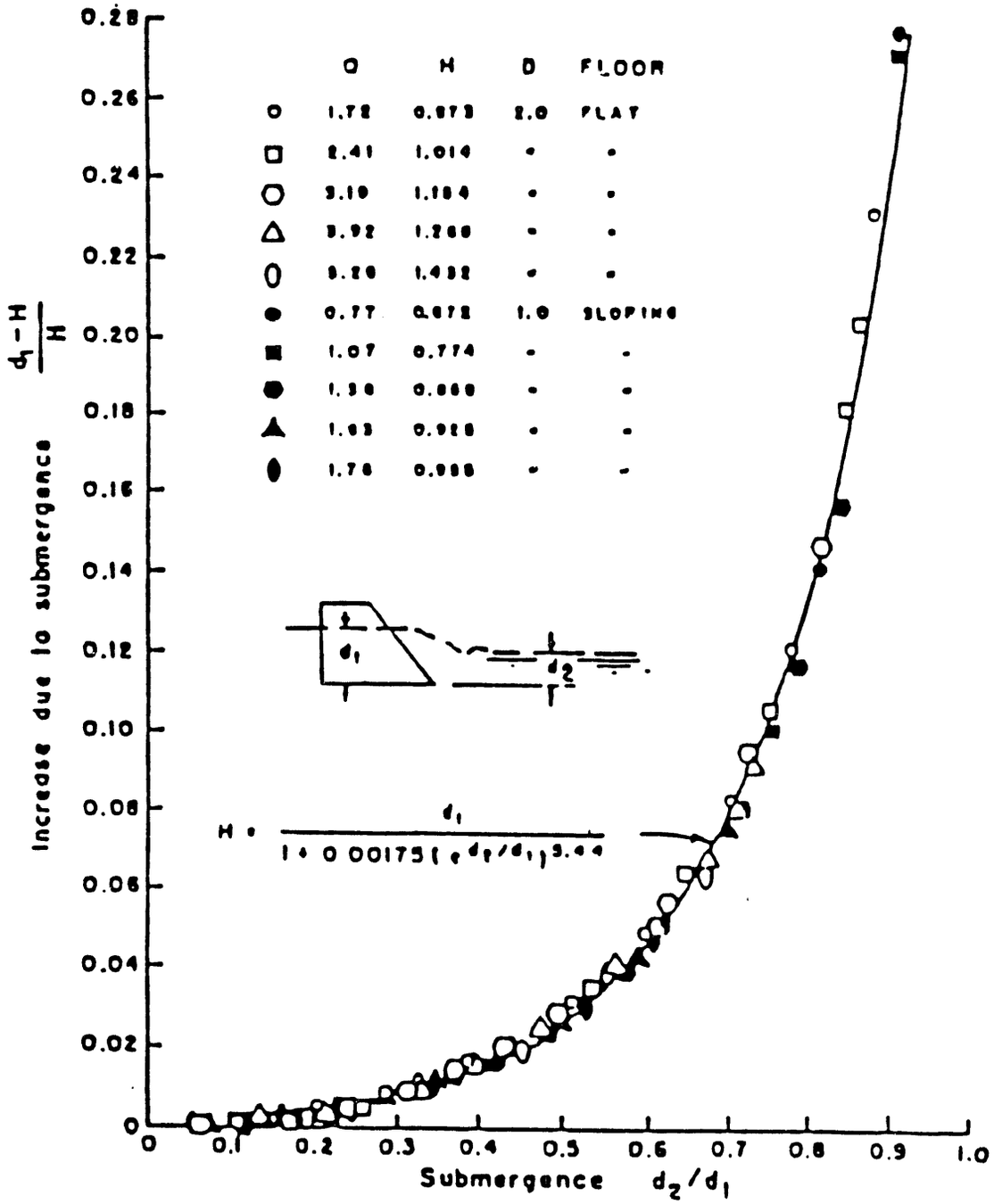
Head (feet)	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0	0	Trace	0.0031	0.0066	0.0106	0.0154	0.0208	0.0269	0.0337	0.0413
0.1	.0496	0.0578	.0666	.0758	.0855	0.959	.1067	.1180	.1298	.1420
0.2	.155	.168	.182	.196	.211	.226	.242	.259	.276	.293
0.3	.311	.330	.349	.368	.388	.409	.430	.452	.474	.497
0.4	.520	.544	.569	.594	.620	.646	.673	.700	.728	.756
0.5	.785	.815	.845	.876	.907	.939	.972	1.005	1.039	1.073
0.6	1.11	1.14	1.18	1.122	1.25	1.29	1.33	1.38	1.41	1.45
0.7	1.49	1.53	1.58	1.62	1.66	1.71	1.75	1.80	1.84	1.89
0.8	1.94	1.99	2.04	2.09	2.14	2.19	2.24	2.29	2.35	2.40
0.9	2.45	2.51	2.56	2.62	2.68	2.74	2.79	2.85	2.91	2.98
1.0	3.04	3.10	3.16	3.22	3.29	3.35	3.42	3.49	3.55	3.62
1.1	3.69	3.76	3.83	3.90	3.97	4.04	4.12	4.19	4.27	4.34
1.2	4.42	4.50	4.58	4.65	4.73	4.81	4.89	4.98	5.06	5.14
1.3	5.22	5.31	5.39	5.48	5.57	5.66	5.74	5.83	5.92	6.02
1.4	6.11	6.20	6.29	6.39	6.48	6.58	6.68	6.77	6.87	6.97
1.5	7.07	7.17	7.27	7.37	7.48	7.59	7.69	7.80	7.90	8.01
1.6	8.12	8.23	8.34	8.45	8.56	8.68	8.79	8.90	9.02	9.14
1.7	9.25	9.37	9.49	9.61	9.73	9.85	9.98	10.10	10.22	10.35
1.8	10.5	10.6	10.7	10.8	11.0	11.1	11.2	11.4	11.5	11.6
1.9	11.8	11.9	12.0	12.2	12.3	12.5	12.6	12.8	12.9	13.0
2.0	13.2	13.3	13.5	13.6	13.7	13.9	14.1	14.2	14.4	14.5
2.1	14.7	14.8	15.0	15.2	15.3	15.5	15.6	15.8	15.9	16.1
2.2	16.3	16.4	16.6	16.8	16.9	17.1	17.3	17.4	17.6	17.8
2.3	18.0	18.1	18.3	18.5	18.7	18.8	19.0	19.2	19.4	19.6
2.4	19.7	19.9	20.1	20.3	20.5	20.7	20.9	21.0	21.2	21.4
2.5	21.6	21.8	22.0	22.2	22.4	22.6	22.8	23.0	23.2	23.4
2.6	23.6	23.8	24.0	24.2	24.4	24.6	24.9	25.1	25.3	25.5
2.7	25.7	25.9	26.1	26.4	26.6	26.8	27.0	27.2	27.4	27.7
2.8	27.9	28.1	28.4	28.6	28.8	29.0	29.3	29.5	29.7	30.0
2.9	30.2	30.4	30.7	30.9	31.2	31.4	31.7	31.9	32.2	32.4
3.0	32.7	32.9	33.2	33.4	33.7	33.9	34.2	34.4	34.7	35.0
3.1	35.2	35.5	35.8	36.0	36.3	36.6	36.8	37.1	37.4	37.7
3.2	37.9	38.2	38.5	38.8	39.0	39.3	39.6	39.9	40.2	40.5
3.3	40.8	41.2	41.3	41.6	41.9	42.2	42.5	42.8	43.1	43.4
3.4	43.7	44.0	44.3	44.6	44.9	45.2	45.5	45.8	46.1	46.4
3.5	46.8	47.1	47.4	47.7	48.0	48.3	48.6	49.0	49.3	49.6
3.6	49.9	50.3	50.6	50.9	51.2	51.6	51.9	52.2	52.6	52.9
3.7	53.2	53.6	53.9	54.3	54.6	54.9	55.3	55.6	56.0	56.3
3.8	56.7	57.0	57.4	57.7	58.1	58.4	58.8	59.2	59.5	59.9
3.9	60.2	60.6	61.0	61.3	61.7	62.1	62.4	62.8	63.2	63.6
4.0	63.9	64.3	64.7	65.1	65.4	65.8	66.2	66.6	67.0	67.4
4.1	67.8	68.2	68.5	68.9	69.3	69.7	70.1	70.5	70.9	71.3
4.2	71.7	72.1	72.5	72.9	73.3	73.8	74.2	74.6	75.0	75.4
4.3	75.8	76.2	76.6	77.1	77.5	77.9	78.3	78.8	79.2	79.6
4.4	80.0	80.5	80.9	81.3	81.8	82.2	82.6	83.1	83.5	84.0

[1]Rating derived from tests made by the Soil Conservation Service at the Hydraulic Laboratory of the National Bureau of Standards using 1-on-8 sloping false floor.

Appendix C

Effect Of Submergence On Calibration of H Flumes

(Gwinn and Parsons, 1976)



Effect of Submergence on Calibration of H Flumes

END

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01/22/92