WASHINGTON STATE
DEPARTMENT OF
ECOLOGY

DAM SAFETY GUIDELINES

Part IV:

Dam Design and Construction

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DAM DESIGN AND CONSTRUCTION GUIDELINES

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INTRODUCTION

Dams which are under the jurisdiction of the Dam Safety Office (DSO) of the Department of Ecology are defined by statute (Chapter 90.03 RCW) and rule (Chapter 173-175 WAC) to be structures which can impound more than 10 acre-feet of water. This definition is broad in scope and applies to typical dams used for: domestic, municipal and irrigation water supply, hydropower, and recreation. It also applies to other dam and storage facilities such as: stormwater detention, domestic sewage lagoons, animal waste lagoons, industrial waste storage, mine tailings storage, etc.

This wide range of project types represents a variety of operational constraints and corresponding engineering design considerations. Recognizing this diversity, these Guidelines attempt to address both normal and frequently encountered engineering issues as they affect a broad range of project types. However, *the principal focus of this document is the earthen dam of small to intermediate size, which represents the majority of the projects constructed in Washington*.

This document seeks to outline a rational engineering approach to address the majority of cases. Clearly, individual projects may pose unique problems that require specialized measures. Where engineers face such cases, it will be their responsibility to demonstrate that their approach satisfactorily addresses the pertinent engineering issues.

These Guidelines are not intended to be a comprehensive summary of current knowledge on dam design and construction as an extensive amount of information exists in hundreds of technical articles, design manuals and textbooks. The Guidelines are also not intended to be a detailed "cookbook" of procedures and requirements. Rather, the approach taken herein emphasizes a broader perspective, focusing on the larger topics of: Design Philosophy; Identification of Engineering Considerations; Discussion of Past Experience and Current Engineering Practice; and Recommendations or Requirements. Efforts have been made to include those technical references which the DSO has found to be of particular value in dam design and construction.

OVERVIEW

These guidelines on dam design and construction were developed based on current engineering practice. They also reflect a philosophy of design, herein termed the Governing Design Philosophy, which makes use of several design principles. These principles provide a framework for evaluating and establishing what design levels and design approaches are appropriate for the various elements of a project. These design principles are presented in the following sections titled *Consequence Dependent Design Levels, Balanced Protection, Redundancy, Survivability, Inspectability and Serviceability*.

GOVERNING DESIGN PHILOSOPHY

CONSEQUENCE DEPENDENT DESIGN LEVELS
It is standard practice in the civil engineering community that the degree of conservatism in design be commensurate with the intended use and the consequences of failure of a given system element. If the failure of a given system element does not pose a public safety concern, then the design level/loading is usually based on economic considerations and the effects of a disruption of operation of the system.

A contrasting situation is where the failure of a given element could pose a threat of loss of life. In these cases, the design events/loadings are typically very conservative to provide protection from the consequences of a failure. And, as the potential magnitude for loss of life and property damage resulting from a failure increases, the design levels/loadings become increasingly more stringent. This concept is termed *Consequence Dependent Design Levels*.

The physical size of a project element, its importance to project performance and the cost of replacement or repair are other characteristics which affect the choice of design levels and loadings. It is logical that a large dam represents a greater capital investment than a small dam and warrants greater protection by way of more stringent design levels.

Thus, the size and importance of a project element and the consequences of failure of that element are primary considerations in establishing minimum design levels. These issues will be discussed in the presentation of the design considerations for the various project elements throughout the guidelines.

In particular, it will be seen that design levels and requirements are markedly more stringent for those critical elements whose failure could lead to an uncontrolled release of the reservoir and pose a risk to downstream inhabitants.

BALANCED PROTECTION
A dam is comprised of numerous critical elements and, like the old chain adage, "is only as strong as the weakest link". Special attention must be given to the design and construction of the critical elements. In particular, care must be exercised to achieve a balance in the level of protection provided in the design of the various critical elements. Application of excessive design conservatism to any one element without consideration of other elements will not necessarily result in increased project safety. Thus, a balanced approach is needed during the design phase of a project to provide assurances of acceptable reliability of the entire system.

There is great value in incorporating a systems approach in the philosophy of design for the project. It is important that a conscious decision is made to examine the various design levels, and that efforts are made to strike a balance among the design levels/loadings used in design of the critical project elements. This concept is termed *Balanced Protection*.

Experience has shown that the causes of dam failures have typically been associated with three general categories of project elements. Approximately one-third of the failures have occurred in each of the three categories: spillways; outlet conduits; and the impounding barrier and foundation. These three general categories therefore comprise the primary critical elements. Failure mechanisms have typically been: overtopping by floodwaters on inadequate spillways; internal erosion along outlet conduits or through conduit joints; and internal erosion through earthen embankments and foundations or instability of impounding barriers and foundations.

Special emphasis has been given in the guidelines to providing reasonably consistent design levels and balanced protection to critical elements in these three general categories. In addition, the guidelines present information on various defense mechanisms for the failure mechanisms listed above.

REDUNDANCY

The principal of *Redundancy* has always been a common feature employed in engineering design. Redundant elements provide backup protection for the primary element or system and increase the reliability of system operation. Redundant elements are necessary design features for many of the critical project elements to achieve the high levels of reliability required in dam design and construction. Redundancy concepts are used throughout the Guidelines.

SURVIVABILITY

The minimum design levels/loading conditions for critical elements of a project are usually very stringent for situations where the valley downstream of the dam is inhabited. This is particularly true for the design levels for Inflow Design Floods and Earthquakes. Frequently, the design levels are sufficiently stringent that it is prohibitively expensive to construct a project where there would be "zero damage" to the project if the design event(s) occurred.

One solution to this problem is to employ the principle of *Survivability*; that is, to design project features which allow damage to occur to the project, provided that the structural integrity of the impounding barrier is not jeopardized and control of the reservoir is maintained. The basic premise is that the design events are so rare that it is unlikely that they will occur during the project life.
Therefore, it is often economically attractive to accept the potential for future project damages and associated repair costs in those situations where the damages would not jeopardize project safety or public safety.

Survivability concepts are applicable to the design of several elements presented in the guidelines. Most notably, survivability concepts are appropriate for design of emergency spillways and energy dissipation basins where tolerable amounts of erosion may be acceptable. Likewise, some settlement and deformation may be allowable on earthen embankments following a major earthquake.

INSPECTABILITY

The design of some project elements is governed by the need to provide a practical means for inspection and long-term monitoring. This is a constraint in addition to the usual considerations of design levels, loadings and functionality. This is often the case when man-made materials are used in the construction of the element and proper operation and safety are dependent on maintenance and/or long-term durability of the construction materials. There are a number of project elements discussed in the Guidelines whose design incorporates the principal of *Inspectability* to allow proper inspection and monitoring.

SERVICEABILITY

Most dams have expected service lives of 100 years or more. This lengthy life span and the frequently harsh service environment pose long term maintenance, repair and upgrade problems. To the extent practicable, the design should anticipate these problems and include provisions to allow refurbishing or upgrading in the future. This is the principal of *Serviceability*. It is a design consideration for such elements as hydraulic inlet/outlet works, spillways and conduits and is a design philosophy which is utilized in many sections of the Guidelines.

GUIDANCE, RECOMMENDATIONS AND REQUIREMENTS

As discussed in the introduction, this document is intended to provide a broad perspective on design philosophy, engineering design considerations, past experiences and current engineering and construction practices. As the title states the information presented here is intended primarily to provide engineering guidance. Where there is a variety of possible approaches and where a preference is warranted, recommendations are made based on past experience or current accepted practice. In a limited number of situations, requirements are identified. To avoid regulations misinterpretation between guidance, recommendations and requirements, all requirements are clearly identified by inclusion of the terms, *required* or *requirement* in the subsection heading.

CHAPTER 1 - IMPOUNDING BARRIER CLASSIFICATION

IMPOUNDING BARRIER SIZE AND RESERVOIR OPERATION CLASSIFICATION

1.1.1 OBJECTIVE:

To provide a uniform system for classifying the size of the impounding barrier and the nature of reservoir operation. These classifications will be used throughout the guidelines for determining the degree of conservatism of design and the sophistication of the methodologies to be used in analyses.

1.1.2 APPLICABILITY:

The following classification system will be used for the size/height of the impounding barrier:

SIZE CLASSIFICATION	BARRIER HYDRAULIC HEIGHT
SMALL DAM	Less Than 15 Feet
INTERMEDIATE SIZE DAM	15 Feet or Greater But Less Than 50 feet
LARGE DAM	50 Feet or Greater

The following classification system will be used to describe the basic nature of reservoir operation:

RESERVOIR OPERATION CLASSIFICATION	DETERMINING FACTOR
PERMANENT POOL OR SEASONAL POOL OPERATION	Steady State Seepage or Saturated Flow Conditions Occur in Barrier, Foundation and Abutments at or Near Normal Pool Conditions.
INTERMITTENT OPERATION	Duration of Normal High Pool Condition is Insufficient for Steady State Seepage or Saturated Flow Conditions to Develop in Barrier, Foundation and Abutments.

CHAPTER 2 - CRITICAL PROJECT ELEMENTS

2.1 DESIGN/PERFORMANCE GOALS FOR CRITICAL PROJECT ELEMENTS

Critical project elements are those elements of a project whose failure could result in dam failure and an uncontrolled release of the reservoir. Critical project elements and associated design events/loading conditions are applicable to such features as: emergency spillways (design floods); impounding barriers (static and seismic loadings); outlet conduits (conduit integrity and seepage control); and impounding barriers and foundations (seepage control).

2.1.1 APPLICATION:

This section provides an overview of the general principles and procedures used to select design/performance goals. Specific details and worksheets for application of the Decision Framework concepts are contained in *Technical Note 2* of the Dam Safety Guidelines entitled *Selection of Design/Performance Goals for Critical Project Elements*[1].

2.1.2 OBJECTIVE:

To provide a *Decision Framework* for selecting design/performance goals for critical project elements which incorporates the concepts of *Consequence Dependent Design Levels* and *Balanced Protection*.

2.1.3 DECISION FRAMEWORK PHILOSOPHY:

The decision methodologies presented in this section utilize probabilistic concepts for setting the design/performance goals to be used for design or evaluation of the various critical project elements. Probabilistic methods were chosen because they offered the capability of implementing a *Balanced Protection* approach for selecting design/performance goals across the range of engineering disciplines.

Balanced Protection - At the present time, the various engineering disciplines involved in dam design utilize methodologies and design events/loading conditions which are either deterministic, combined probabilistic-deterministic, or probabilistic in nature. This variety in methodologies has often resulted in the various elements of a project being designed to widely different standards - often affording quite dissimilar levels of protection from failure. This is in sharp contrast to the balanced protection concept. The approach advocated here is to set a common design/performance goal for the design or evaluation of each critical project element and to utilize methodologies and standard practices which assist in providing reasonably consistent levels of protection for the critical project elements.

The decision framework also utilizes the concept of *Consequent Dependent Design Levels* by setting increasingly more stringent design/performance goals as the consequences of failure become more severe.

Consequent Dependent Design Levels - It is standard practice in the civil engineering community that the degree of conservatism in design be commensurate with the intended use and the consequences of failure of a given system element. If the failure of a particular element does not pose a public safety concern, then the design level/loading condition is usually based on economic considerations and the effects of operational disruption. A contrasting case is the situation where the failure of a given element could pose a threat of loss of life. The design levels/loadings for the critical elements are typically very conservative. And, as the potential magnitude of loss of life and/or property damage resulting from a dam failure increases, the design levels/loadings become increasingly more stringent.

The consequences of a dam failure include such diverse effects as the potential for loss of life, downstream property damage, and the loss of the capital investment in the dam and the economic benefits provided by the project. Because of this diversity in potential effects, a number of indicator parameters (Table 1) have been identified to reflect the nature and severity of the consequences.

TABLE 1 - NUMERICAL RATING FORMAT FOR ADDITIVE WEIGHTING SCHEME FOR ASSESSING CONSEQUENCES OF DAM FAILURE

CONSEQUENCE CATEGORIES	CONSEQUENCE RATING POINTS	INDICATOR PARAMETER	CONSIDERATIONS
CAPITAL VALUE OF PROJECT	0 - 150	DAM HEIGHT	Capital Value of Dam
	0 - 75	PROJECT BENEFITS	Revenue Generation or Value of Reservoir Contents
POTENTIAL FOR LOSS OF LIFE	0 - 75	CATASTROPHIC INDEX	Ratio of Dam Breach Peak Discharge to 100 Year Flood
	0 - 300	POPULATION AT RISK	Population at Risk Potential for Future Development
	0 - 100	ADEQUACY OF WARNING	Likely Adequacy of Warning in Event of Dam Failure
POTENTIAL FOR PROPERTY DAMAGE	0 - 250	ITEMS DAMAGED OR SERVICES DISRUPTED	Residential and Commercial Property Roads, Bridges, Transportation Facilities Lifeline Facilities Community Services Environmental Degradation from Reservoir Contents (Tailings, Wastes, etc.)

2.1.4 TERMINOLOGY:

The following terms are used in this section and in Technical Note 2[1].

Design/Performance Goal - A goal for the performance of critical project elements which may be used in design or evaluation. It is expressed as an Annual Exceedance Probability (AEP) and is a measure of the chance of adverse behavior, or failure of a critical project element.

Annual Exceedance Probability (AEP) - The chance that a specified magnitude of some phenomenon of interest is equaled or exceeded during a given year.

Design Step - An integer value from one through eight which is used as an index for increasingly stringent design/performance goals (Figure 1).

Reliability - The likelihood of successful performance of a given project element. It may be measured on an annualized basis or for some specified time period of interest, such as the project life. Mathematically, reliability is expressed as:

RELIABILITY = 1 - PROBABILITY [*Adverse Behavior or Failure*] (1)

Design Level - In general usage, design level is a generic term used to describe the relative conservatism of a particular design event or loading condition. In many engineering applications, the actual level of protection provided by a specified design level may not be known with accuracy.

2.1.5 DECISION FRAMEWORK:

The Decision Framework is implemented through a Design Step Format (Figure 1) which utilizes eight steps where design/performance goals and corresponding design events and loading conditions become increasingly more stringent in progressing from step 1 through step 8. Design step 1 is applicable when the downstream consequences of a dam failure would be minimal and there would be no potential for loss of life. The design/performance goal at step 1 has an Annual Exceedance Probability (AEP) of 1 in 500 - one chance in 500 of being equaled or exceeded in a given year.

Design step 8 is applicable where the consequences of failure could be catastrophic with hundreds of lives at risk. In this situation, very extreme design events and loading conditions are appropriate for the extremely high levels of reliability needed to provide proper protection of public safety. Design step 8 corresponds to theoretical maximum design events and loading conditions. In those cases where a theoretical maximum does not exist for the design loading under consideration, the maximum design/performance goal is set at an AEP of 10^{-6}.

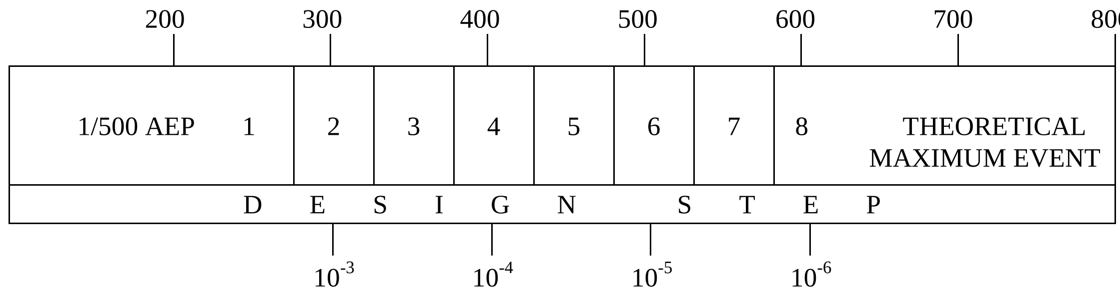
Additive-Weighting Scheme - Selection of the Design Step and corresponding design/performance goal is accomplished using an additive-weighting scheme[2,3] to incorporate all of the consequence information into a numerical format (consequence rating points) to provide guidance in decision-making. The design/performance goal is determined based on the magnitude of the consequence rating points (Figure 1).

Application of the selected design/performance goal is accomplished through a Design Step Format incorporating 8 design steps (Figure 1). The selected design step is used as an index for specific design levels, design events and design loading conditions for the various critical project elements.

The approach taken here is to use the broad spectrum of engineering design practice as a reference for setting benchmarks for design levels. While direct situational comparisons are few, there are enough similarities to provide sound guidance[1]. This approach provides a means of setting design levels which are consistent with levels of safety provided by other engineering disciplines and by existing government regulation in other engineering and product safety areas.

Often during the preliminary stages of project planning, there is a need to make a quick assessment of the Design Step. This can usually be accomplished by use of Table 2 which shows the general relationship between the Design Step and the commonly used Downstream Hazard Classification.

CUMULATIVE CONSEQUENCE RATING POINTS



DESIGN/PERFORMANCE GOAL - ANNUAL EXCEEDANCE PROBABILITY

FIGURE 1. DESIGN STEP FORMAT AND CONSEQUENCE RATING POINTS

TABLE 2 - RELATIONSHIP OF DESIGN STEP TO DOWNSTREAM HAZARD CLASSIFICATION

DOWNSTREAM HAZARD POTENTIAL	DOWNSTREAM HAZARD CLASSIFICATION	POPULATION AT RISK	ECONOMIC LOSS GENERIC DESCRIPTIONS	ENVIRONMENTAL DAMAGES	TYPICAL DESIGN STEP
LOW	3	0	Minimal. No inhabited structures. Limited agriculture development.	No deleterious materials in reservoir	1 - 2
SIGNIFICANT	2	1 to 6	Appreciable. 1 or 2 inhabited structures. Notable agriculture or work sites. Secondary highway and/or rail lines.	Limited water quality Degradation from reservoir contents and only short term consequences	3 - 4
HIGH	1C	7 to 30	Major. 3 to 10 inhabited structures. Low density suburban area with some industry and work sites. Primary highways and rail lines.		3 - 6
HIGH	1B	31-300	Extreme. 11 to 100 inhabited structures. Medium density suburban or urban area with associated industry, property and transportation features.	Severe water quality Degradation potential from reservoir contents and long term effects on aquatic and human life	4 - 8
HIGH	1A	More than 300	Extreme. More than 100 inhabited structures. Highly developed, densely populated suburban or urban area with associated industry, property, transportation and community life line features.		8

2.1.5 REFERENCES:

1. Schaefer, M.G., Selection of Design/Performance Goals for Critical Project Elements, Technical Note 2, Dam Safety Guidelines, Washington State Department of Ecology, Olympia, WA., July, 1992.

2. Hayes J.B., The Complete Problem Solver, Lawrence Erlbaum Associates Publishers, Hillsdale, N.J., 1989.

3. Keeney, R.L., Raiffa, H., Decisions With Multiple Objectives: Preferences and Value Tradeoffs, John Wiley and Sons, 1976.

2.2 BARRIER STATIC STABILITY

2.2.1 OBJECTIVE:

Minimize the likelihood of an embankment or abutment failure during construction and in service.

2.2.2 DESIGN PRACTICE:

Requirements/Minimums - All embankments must equal or exceed the minimums cited in Table 1[1] for the steady seepage case. In many instances the need for end-of-construction and sudden drawdown analyses can be dismissed by inspection. Specifically, an analysis of the end-of-construction stability is not required when the outer shells of the embankment consist of relatively pervious soils that are expected to dissipate excess pore pressures almost at the rate of their generation. A rapid drawdown analysis may be omitted if the lowlevel outlet system can only drop the pool a few feet per day when it is in an elevated state.

TABLE 1 - MINIMUM FACTORS OF SAFETY FOR EARTH AND ROCKFILL DAMS[a)]

Design Condition	Minimum Factor of Safety
End of Construction	1.3[b)]
Sudden drawdown from maximum pool	1.0[c)]
Sudden drawdown from spillway crest	1.2[c)]
Steady seepage with maximum storage pool	1.5

Notes: a) Not applicable to embankments on clay shale foundations; higher safety factors should be used for these conditions.

b) For embankments over 50 feet high on relatively weak foundation use minimum factor of safety of 1.4.

c) The safety factor should be a minimum of 1.5 when drawdown rate and pore water pressures developed from flow nets are used in stability analyses.

Current Practice - Analyses are routinely conducted with 2-dimensional slope stability models, based on limit equilibrium procedures. A satisfactory design is considered achieved when the factor of safety exceeds the generally accepted minimums listed in Table 1. The conscientious designer typically performs sensitivity studies to confirm that the design meets minimum factors of safety for reasonable variations in the magnitude of key model parameters.

Probabilistic Based Stability Approach - The DSO accepts the factor of safety methodology to justify a given embankment cross-section; it will continue to do so for the foreseeable future. None the less this method has major shortcomings. Ideally, current practice would employ static stability analyses which provided a clear measure of the actual reliability of the embankment or abutment slope; it does not. Furthermore, the factor of safety approach, as routinely applied to static slope stability evaluations, is intuitively disquieting in that there is no provision to increase or decrease the design minimum based on the downstream hazard setting. These shortcomings hamper efforts to rationally improve embankment reliability as the consequences of failure increase.

As an alternative, the DSO espouses the use of probabilistic based, capacity/demand reliability methods such as those proposed by Vanmarcke[2] and Whitman[3]. We recognize that the majority of the engineering community has serious reservations about such an approach. The principal objection to the application of probabilistic methods to embankment design is that it typically has not afforded a "meaningful" treatment of "extreme events". Here "meaningful" refers to the ability of the analysis to give a level of confidence in the likely occurrence of an "extreme event" that the designer would feel comfortable in deciding whether additional action was warranted. "Extreme events" is a term employed to describe any of a number of problems that would likely control the stability analysis but are themselves generally difficult to factor into the analysis with any confidence. This difficulty may spring from an inability to predict the position at which such an extreme event may occur even though the designer recognizes the potential occurrence of such events. Or, conversely, the engineer may not even have considered the possibility of a given type of extreme event occurring and thus not have factored this element into the analysis. The principal examples of what the DSO considers to be extreme events are:

> Missing an extensive weak seam in the foundation in the explorations,
>
> Cracks developing within the embankment that allow near reservoir head levels to develop within the downstream portion of the embankment[4,5], and
>
> Pervious features in the abutments or foundations producing large hydraulic gradients in the downstream portion of the embankment.

Such extreme events are not readily factored into the analysis by assuming that the variable of interest has a lognormal distribution about the mean.

Some of the difficulties of formulating a probabilistic model of embankment stability have been addressed by the application of fuzzy number theory. Santamarina, et. al.[6], identified the following qualitative parameters as elements in their assessment of embankment stability:

- *Qualifications of the engineer-designer,*
- *Extent and quality of the geologic assessment at the site,*
- *Quality of the available data,*
- *Quality of the design method,*
- *Completeness of the design of the structure,*
- *Importance of design errors or omissions,*
- *Contractor's prior record,*
- *Supervision during construction,*
- *Quality of field controls during construction,*
- *Importance of construction errors,*
- *Difficulties during construction,*
- *Monitoring program,*
- *In-service inspection,*
- *Malfunctions during the life of the structure, and*
- *Maintenance program.*

A few common themes standout in reviewing the preceding list.

> First and foremost, the embankment stability analysis represents an artificial distinction and is but one of a number of steps in a larger process of achieving a stable embankment. Specifically, the development of a suitable embankment section does not end with documenting by numerical analysis that the section exceeds some minimum target stability parameter. It is an on-going, "cradle to grave" endeavor spanning site investigation, design, construction and finally the

monitoring and maintenance of it in service. (This philosophy underlies Parts I through IV of the Guidelines.) Furthermore, it is our opinion that in many cases the apparent success of stability analyses is to a significant degree the result of field changes during construction of the design. During construction, it may be that weak zones are discovered and treated or, following construction, remedial action satisfactorily treated unstable areas as they manifested themselves. For whatever reason the fact that stability problems are not more common is a testament to the value of thorough explorations programs, good engineering judgement in design, and proper field monitoring during construction and over the service life of the facility. Conversely, the scarcity of embankment failures alone, in our opinion, does not constitute a ringing endorsement of current analysis methods.

Second, the engineer needs to consider the adequacy of the model itself and the data that was used to predict embankment response. There is widespread belief that the 2-dimensional, plane strain model provides a conservative estimate of the factor of safety of embankment stability. This belief is fostered largely by the fact that the analysis ignores end effects of potential failure surfaces. However, the importance of end effects decreases with increasing failure surface width. Furthermore, any conservatism in the estimation of the factor of safety arising from ignoring end effects may be lost in neglecting potential variations in mobilized soil strengths along both the width and length of the failure surface.

Finally, the designer needs to understand the sensitivity of the embankment scheme to practical constraints imposed by the contractor's ability and the ability of the operator to maintain the facility in service.

In spite of the manifold difficulties of applying probabilistic methods to dam engineering, few other areas of geotechnical practice present a greater opportunity to "know and/or control" the material properties relevant to the stability of embankments. The designer can control the phreatic line through the placement of chimney and blanket drains. The consequences of embankment cracking are mitigated through the use of properly filtered chimney drains. Soil properties can be controlled by specifying greater minimum densities, gradation limits and instituting appropriate construction control programs. These measures are "active steps" the designer can take to reduce uncertainty. "Passive measures" to reduce uncertainty can take the form of increasing the depth of foundation excavations so that zones conceivably containing weak seams are effectively banished to depths where they no longer govern the overall stability response of the embankment. There are practical limits to such measures. But, a probabilistic formulation of the problem affords the designer a rational framework to justify increasing the degree of constraint of uncertainty in key parameters related to embankment performance as the consequences of failure increase.

In probabilistic modelling of embankment stability, one can become enmeshed in arguments over the appropriateness of the assumptions as to the variability of model parameters and even over definitions of reliability. Many practicing engineers simply give up on probability methods when confronted with these issues. This is unfortunate because it is not necessary to apply rigorous probabilistic analyses to gain insight into the impact of particular steps on the overall reliability of the embankment. One's intuitive sense of the variability of a governing parameter can be employed to predict the expected range of response of the embankment with conventional limit equilibrium stability models. This affords a qualitative feel rather than a quantitative measure of the impact of a particular step or assumption on embankment reliability. None the less it serves to identify the sensitivity of the model to a given parameter. It thereby gives a relative sense of the impact of measures to constrain a given parameter's variability in the overall effort to improve the reliability of the project.

Table 2 summarizes the principal components and our qualitative sense of their overall impact on the performance of actual embankments. The Table illustrates our belief that the response of the dam and foundation is governed largely by the position of the phreatic surface, the occurrence of concentrated seeps and the potential presence of some unknown, weak layer in the foundation or abutments.

Regarding weak foundation seams, the engineer should recognize that the typical mode and frequency of field testing allow ample opportunity to miss a weak seam. This should be a sobering thought to the design engineer. Ideally, it will prompt them to incorporate "reality checks" and mitigative measures in the design that reduce the impact of unanticipated conditions particularly in the foundation phase of the work. Regarding concentrated seeps and the phreatic surface, conventional slope stability models do not readily allow considering the impact of embankment cracking that could conceivably introduce near-full reservoir, head levels into the downstream portion of the embankment. Furthermore, it is rare that we see a stability analysis that even attempts to approximate the impact of differing horizontal and vertical permeabilities on the position of the phreatic surface. Recognizing the potential variability of the phreatic surface and its crucial impact on overall embankment stability, we want to limit the "volatility" of this aspect of the problem. The magnitude of the restriction ideally should be a function of the consequences of a failure. The process of constraining the variability in the phreatic surface and mitigating the impact of embankment cracking normally involves the construction of chimney and blanket drains. The extreme value probability problems of identifying anomalous pervious seams and the extent of potential embankment cracking are circumvented. Instead, the drainage features control the position of the phreatic surface and transect cracks, dissipating high water pressures. This is a restatment of the approach advocated by Dr. DeMello in his 17th Rankine Lecture[7]. There he argued that the designer should take steps to "transform" the problem from one of dealing with extremes to one of dealing with averages, i.e., a design feature that blunts or smoothes out the effects of extreme loadings. In effect, the designer is bringing the actual dam closer to the idealized, simplified structure modelled.

The process of improving embankment reliability involves a number of steps that span the scoping of the initial exploration program to detailing the operation and maintenance plan. Beginning with the exploration program, the scope of foundation and abutments investigations should be keyed to the consequences of failure. Obviously, the problem is trivial if competent rock or otherwise suitable overburden deposits are exposed at shallow depths. Any surficial, unsuitable materials would be removed as a practical expedient. Where overburden depths make removal impractical or where the abutment contains pervious or weak seams that likewise present great difficulties in treating, judgement must be employed that is seasoned by due consideration of the consequences. If the level of uncertainty remains substantial and the consequences of failure are large, then the presumption of the existence of weak and/or pervious zones in the foundations should be made and design measures employed that provide the necessary factor of safety given the possible existence of such zones. Addressing weak zone and pervious features concerns would include such measures as:

Strength concerns

Conducting a secondary exploration program targeted at characterizing conditions within some critical depth as determined from preliminary stability assessments,

Flattening embankment slopes,

Providing a buttress for the abutments,

Overexcavating the upper phases of the foundation and/or the provision of key trenches,

TABLE 2
STATIC SLOPE STABILITY MODEL VARIANCE

MODEL PARAMETER	MODEL SENSITIVITY TO PARAMETER VARIANCE	PRINCIPAL SOURCE OF DIFFERENCES BETWEEN MODEL & ACTUAL EMBANKMENT RESPONSE	MITIGATION
Soil Unit Weights	Minimal Effect	____________	____________
Shear Strength	Minimal Effect	Friction angle determined by direct shear tests reflecting the failure plane along a horizontal surface rather than on the weakest plane. Not accounting for the reduction in friction angle for grain breakage under large confining stresses.	Normal variance considered in range of strength parameters.
	Significant Effect	<u>Embankment</u>: Strength properties determined on a degree of compaction significantly greater than that actually achieved during construction.	Specify minimum compaction level and construction control program in construction documents.
	Potential Major Effect	<u>Abutments and Foundation</u>: Failure to detect presence of a weak zone where actual strength parameters are significantly less than assumed values. Inappropriate use of peak friction angles rather than residual angles for sensitive soils.	Expand exploration program to better characterize subsurface conditions. Check for sensitive soils; i.e., those with significant different peak to residual strength ratio, design for the possible occurrence of potential weak zones.
Ground Water/Pore Pressure Regime	Potential Major Effect	<u>Embankment</u>: (Cracks) – Unanticipated cracking and hydraulic fracturing problems may introduce near reservoir pressure levels into the downstream portion of the embankment. (Inaccurate modeling of seepage) – Horizontal and vertical permeabilities that are unequal and vary throughout the embankment. This may produce significant differences in the pore pressure regime between the model and the actual embankment. <u>Abutments and Foundation</u>: Undetected pervious seams producing greater than anticipated water pressures in the downstream embankment/subgrade contact. (End of Construction Problems) – Pore Pressure response of compacted soils	Provide chimney and blanket drains to intercept and discharge seepage and constrain phreatic surfaces to the drain configuration. Expand seepage cutoff features and extend the chimney drain deeper into the abutments and foundation. Employ blanket drains over the downstream embankment/subgrade contact. Laboratory testing to determine pore pressure behavior under representative loading; where appropriate, conduct field monitoring during construction.
Trial Failure Surface (Block or Circular)	Variable	The appropriateness of a particular trial surface shape is a function of the embankment geometry and the configuration and location of any materially weaker embankment zones.	Evaluate abnormal zones as possible planes of weakness; use block segments to force trial failure surface to pass through such zones.

Increasing the minimum level of compaction, and

Providing drainage zones in buttresses at the subgrade contact to take full advantage of the buttress weight.

Pervious feature concerns

Flaring the core footprint at the abutment and foundation contact to lengthen the seepage path,

Extending cutoffs and chimney drains deeper into abutments and the foundations,

Oversizing the drains to accommodate abnormally large volumes of seepage, and

Employing an asymmetrical embankment cross-section that places the core zone as far upstream as practicable to maintain the majority of the dam in a drained state.

During construction a suitable inspection effort must be made to confirm that the project is constructed in accordance with the design. To this end the construction documents need to:

Clearly define what constitutes acceptable and conversely unacceptable subgrade conditions.

Require the inspection of the prepared foundation by a qualified engineer familiar with the principal engineering assumptions supporting the design.

This topic is also discussed in Section 7, Specifications. In short, assessing static stability involves "reality checks" to confirm that the principal design assumptions were equaled or exceeded in the field. To accomplish this, field personnel must be qualified to recognize an adverse situation.

Following construction the owner and operator must be provided with guidance to assist them in the recognition of an adverse situation, should one develop. The DSO publication Guidelines for Developing Dam Operation and Maintenance Manuals and Emergency Action Plans are available to assist in this area.

Reliability Standards - The annual probability of failure scheme outlined in the Sections on Design Floods (2.4) and Seismic Stability (2.3) is not readily applicable to slope stability problems. The principal reason the annual probability approach is not as desirable in static stability problems is in the nature of the design loading. In floods and earthquakes the protection comes in the fact that the design event the facility must survive has an extremely remote chance of occurring. However, in static slope stability problems nearly the maximum load is applied during the first filling and this load is maintained largely unabated for the remainder of the service life of the project. Therefore, the design goal is to provide a capacity (ultimate potential shear strength) greatly in excess of demand (that fraction of the shear strength that must be mobilized to just equal the driving forces).

As a practical matter it is doubtful that any recognized engineering body is going to step forward and endorse some tiered reliability standard. The liability such an endorsement would likely carry would discourage most reputable individuals. Thus, for the foreseeable future those electing to use reliability methods likely will employ a hybrid approach involving both:

Documenting that factors of safety cited in Table 1 are achieved, along with

Providing a measure of the reliability of this calculated factor of safety given a best estimate of the variability of the parameters that govern the model response.

The second step will largely be done for the designer's piece of mind. This task should reassure the engineer that the model has accounted for the normal variability typically encountered with soils and that those elements introducing the greatest uncertainty have been appropriately treated or constrained.

2.2.3 SPECIALIZED ISSUES:

Abutment Stability - Analyses tend to concentrate on the stability of the embankment. However, there are situations where one must also consider the stability of temporary abutment cut slopes. This likely occurs when the abutments consist of competent sands and silty sands and the slope of the abutments is at or steeper than the angle of repose of the soils. The act of stripping and removal of unsuitable soils from the slope and toe areas can overstress the soils and induce a slide. The problem may become acute where the slide deprives support for the upper reaches of the slope and precipitates ongoing slides up the slope. Where this situation is anticipated, due consideration should be given to staging the stripping operation so that only a small portion of the slope is unsupported at any one time. The specifications would need to identify the maximum allowable stripped swath between the top of the working surface of the fill and undisturbed ground.

Reservoir Rim Stability - The reservoir rim should be reviewed to determine whether there is the potential for slope movements into the reservoir that could generate damaging waves against the embankment or block hydraulic elements of the outlet works. No specific requirements on the nature of the investigation are set forth by the DSO. This is due primarily to the fact that reservoir rim instability has not posed a significant problem for the typical earthen dam that this guidance document covers. Significant reservoir rim stability problems have been encountered in Washington. However, the problems have been associated primarily with major reservoirs. Accordingly, the owners have had the resources and desire to conduct extensive investigation into the problem and to aggressively pursuit a fix.

It is unlikely that the smaller earthen embankment project (the target of this guidance document) would have a significant reservoir rim stability problem identified. But, assuming this is the case, it would be prudent in the majority of cases for the developer to abandon the site in favor of a less difficult one.

2.2.4 REFERENCES:

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3. Whitman, R. V., Evaluating Calculated Risk in Geotechical Engineering, Journal of Geotechnical Engineering, Vol 110, No. 2, 1984, pp. 145-188.

4. Dascal, O., Peculiar Behavior of the Manicouagan 3 Dam's Core, International Conference on Case Histories in Geotechnical Engineering, May 1984, University of Missouri-Rolla, Vol. 2, pp. 561-569.

5. Sherard, J.L., Hydraulic Fracturing in Embankment Dams, Volpe, Richard L., Kelly William E., (eds.), Seepage and Leakage from Dams and Impoundments, ASCE National Convention, Denver, CO., May 1985, pp. 115-141.

6. Santamarina, J.C., Altschaeffl, A.G., Chameau, J.L., Reliability of Slopes: Incorporating Qualitative Information, (preprint), Paper 92, Transportation Research Board, 71st Annual Meeting, January 12-16, 1992, Washington, D.C..

7. DeMello, V.F.B., Reflections on Design Decisions of Practical Significance to Embankment Dams, Geotechnique, Vol. 27, No. 3, 1977, p. 293.

General References on Conventional Slope Stability Methods

8. U.S. Bureau of Reclamation, Design Standards No. 13, Embankment Dams, Chapter 4, Static Stability Analyses.

9. Duncan, J. M., and Buchignani, A. C., An Engineering Manual for Slope Stability Studies, University of California, Berkeley, March 1975.

10. U.S. Corps of Engineers, Engineering and Design: Stability of Earth and Rockfill Dams, EM 1110-2-1902, 1970.

2.3 BARRIER SEISMIC STABILITY

2.3.1 OBJECTIVE:

The project should be configured so as to be able to experience earthquakes without releasing the reservoir contents except under an appropriately remote level of earthquake shaking.

2.3.2 APPLICABILITY AND FOCUS:

Impoundments can be broken down into two main groups: earthen embankments and concrete structures. The principal focus of the guidelines is on earthen embankments. Concrete structures, i.e., gravity, arch and roller compacted concrete impounding barriers are not discussed in any depth. This is due to the following rationale. *Concrete structures present a number of unique design problems. Generally, only specialty firms, well versed in the peculiarities posed by such structures, are qualified to formulate a suitable design. It would be misleading to imply that these guidelines could somehow substitute for the requisite experience and judgment necessary to design a suitable concrete impounding structure*.

It is also not the intention of these guidelines to present a detail methodology for the dynamic analysis of embankment dams. Instead, the focus of this section is to provide guidance on the minimum scope of analysis that the DSO considers appropriate for various seismological settings, embankment materials, and consequences of failure. In addition, the principal schemes routinely employed to address seismic stability concerns are discussed.

2.3.3 OVERVIEW:

For completeness sake, a few comments on the dynamic behavior of concrete structures are presented. Concrete structures have performed well when subjected to strong earthquakes. The most notable concrete dam failures, St. Francis and Malpasset[1] occurred as a result of weak foundation layers rather than structural problems within the dams. Historically, foundation failures have posed the principal threat to concrete structures. Dams have experienced extensive earthquake induced cracking that necessitated extensive remedial work[2]. But, the DSO is not aware of any concrete dam where a catastrophic release of the reservoir resulted from earthquake induced cracking of the structure. Where failures have occurred, the problem arose as the result of deterioration of the rock mass properties in the abutments or foundation due to the action of applied static and dynamic stresses and/or the impact of seepage. Again, these guidelines are not intended to address the difficult foundation conditions that have posed problems in the past. The assessment of the performance of rock foundations for concrete structures should be undertaken only by those well versed in the process.

Earthen embankments, the principal focus of this section, theoretically fail as the result of the dam being overtopped or the cross-section breached. The breach scenario conceivably could occur through relative displacement vertical and/or horizontal between either side of a fault that passes through the dam footprint. The DSO is not aware of any failures directly attributed to such a mechanism. Where the potential for such fault movements was anticipated in the design, embankments have been provided with wide core zones, broad non-cohesive crack filler zones and enlarged drainage features[3] to accommodate large, localized relative displacements in the embankment section.

Embankment overtopping remains the principal mechanism of earthquake induced failures. Typically the dam crest settles or drops a distance greater than the available freeboard. The overtopping flow rapidly breaches the embankment section, leading to a failure of the dam.

Seiches generated in the reservoir also potentially could overtop the embankment, presenting erosion concerns. At Hebgen Dam in a 1959 earthquake the caretaker reportedly observed a series of waves overtop the embankment. The caretaker estimated[4] that one wave overtopped the dam for a duration of 10 minutes with a maximum depth of some 3 feet. Considerable erosion damage occurred but the dam did not fail. In fact the DSO is not aware of an actual failure precipitated by a seiche. The absence of such failures in part may be responsible for the limited amount of research done in this area. Again, this issue is beyond the scope of these guidelines. If readers elect to pursue the issue, they might begin with the US Army Coastal Engineering Research Center's Shore Protection Manual[5].

2.3.4 TWO TRACKED SEISMIC ASSESSMENT APPROACH:

The behavior of embankments under earthquake loading is normally idealized as two distinct behaviors: 1) a deformation response and 2) the buildup of pore water pressures that, in extreme cases, can lead to a liquefaction condition. The Bureau of Reclamation's[6] idealization of the process is an excellent summary of the dual track analysis approach.

Deformation Analysis - The deformation response is the summation of the predicted displacements in the embankment and foundation that occur in the brief moments that the combined inertial and static forces exceed the shearing resistance of a surface in a segment of the embankment and/or the foundation. The static stresses alone are insufficient to maintain sliding of this mass. Normally, some reduction in shear strength properties is applied to the soil to account for the build-up of pore water pressure from the cyclic loading. The actual analysis begins with a pseudo-static assessment of the dam to determine a yield acceleration, k_{yield}. K_{yield} is the acceleration in percent of gravity that produces a factor of safety of 1 on a trial slip surface whose failure would compromise the ability of the dam to impound the reservoir. K_{yield} is then compared to k_{max}, the estimated average maximum acceleration the failure surface under consideration is predicted to experience under the design earthquake. K_{max} is a function of the maximum crest acceleration ($\ddot{u}_{max}$) and the ratio of the depth of the failure zone under consideration to that of the embankment height. The quantity $\ddot{u}_{max}$ can be determined by performing non-linear, wave propagation analyses such as the pseudo two-dimensional SHAKE[7] program or by empirical procedures such as those outlined by Makdisi and Seed[8]. Ambraseys and Sarma[9] and Makdisi and Seed[8] developed and published similar data in the form of curves relating $\ddot{u}_{max}$ to $k_{max.}$ If the analysis shows k_{yield} exceeds k_{max}, the concern over displacements is dismissed by inspection. If k_{yield} is less than k_{max}, then an empirical relationship is used to estimate the magnitude of movement. This relationship is a function principally of the ratio of k_{max} to k_{yield}, the predominant period of the embankment/foundation system and the magnitude of the earthquake.

The preceding discussion outlines the simplified deformation analysis procedure developed by Makdisi and Seed[10]. An approach incorporating probabilistic concepts has been formulated by Yegian, Marciano and Ghahraman[11]. The Corps of Engineers employs a similar analysis methodology to that of Makdisi and Seed developed by Ambraseys and Sarma[12].

The results of a deformation analysis are considered "order of magnitude"[13]. Therefore, considerable judgement is required in the determination of what constitutes satisfactory performance. Ideally, the predicted settlement would be a small fraction of the available freeboard. The selection of a suitable "small fraction" depends on the overall confidence in the characterization of the embankment and foundation soils, the reliability of the estimated peak acceleration, the extent of the crest involved in the sliding mass and the consequences of failure.

Liquefaction Analysis - The deformation model in its various formulations does not adequately predict the behavior of soils susceptible to a rapid buildup of pore water pressures under cyclic loading. This class of soils consists principally of saturated, loose to medium dense, non-plastic silts; fine, uniformly graded, sands; and mixtures of silt, sand and gravel. For these soils the dynamic loading transfers a portion of the stress in the soil structure unto the pore water phase of the saturated soil. For there to be a significant build up of pore water pressure, the drainage of the soil has to be considerably less than the rate of pore pressure rise. Where liquefaction occurs the soil generally loses a large fraction of its strength. If failure occurs, it generally happens after the earthquake where the temporarily lower soil strength properties can not resist the existing system of imposed static loads.

There are a number of methodologies employed to evaluate the potential for a liquefaction failure. They include empirical methods based on Standard Penetration blow counts, sophisticated laboratory testing and dynamic analyses such as the Cyclic Stress Ratio and steady state approaches, among others. An excellent synopsis of the various analysis procedures is presented in the National Research Council publication Liquefaction of Soils During Earthquakes[14]. Specifics on various components of conducting a dynamic analysis are discussed by Idriss and Duncan[15].

Seismic Evaluation Methodologies - The DSO does not endorse a particular method for the assessment of deformations and the potential for a liquefaction condition to occur. In fact, there are differing levels of sophistication employed in the various analysis schemes. The need for a particular degree of sophistication in a given analysis is a function of the anticipated level of probable ground motion, susceptibility of the soils to strength loss and pore water pressure build up under dynamic loadings and the downstream hazard setting.

2.3.5 ESTIMATING SITE SPECIFIC GROUND MOTIONS FOR ANALYSES:

The following outlines the general steps the engineer would take in developing design ground motions to be used in a conducting a seismic assessment. In recognition of the fact that detailed seismic studies often are unavailable at a proposed project site, a process is discussed to estimate ground motions from existing public domain, seismic risk studies.

Step 1: Estimate the character of the design ground motions at foundation level at the site.

Methodologies for Selection of Earthquakes to Include in Analysis **-** The methods of selecting earthquakes for the prediction of ground motions at foundation level can be broadly grouped into three categories: deterministic, probabilistic and hybrid (both deterministic and probabilistic components).

In the deterministic approach, all seismogenic features, believed capable of generating damaging ground motions at the project site, are evaluated for the maximum credible earthquake (MCE). This is the largest magnitude earthquake the feature is judged capable of producing. The estimated magnitude is a function of among other factors, past seismic activity on this feature or other similar features, the dimensions of the fault surface or locked segment of the plate, the type of fault movement and rates of plate convergence.

In the probabilistic approach earthquakes of lesser magnitude than the MCE in addition to the MCE are considered. The estimated frequency of occurrence of the various magnitude events is postulated and an annual probability of experiencing a given level of ground motions is calculated for the site. Idriss[16] presents an excellent description of the probabilistic seismic risk assessment process.

In the hybrid approach identifiable seismogenic features are treated in either a deterministic or probabilistic manner. Probabilistic models are used to predict the occurrence of random earthquakes, i.e., earthquakes that cannot be tied to a particular geologic structure. This would include the deep earthquakes occurring within the subducting Juan de Fuca Plate beneath Western Washington and random earthquakes in the overriding North American Plate (NAP), such as the 1872 earthquake.

Accordingly, in seismic assessments of projects in the Puget Sound area a random MCE of Magnitude 6½ within the NAP has been a common assumption. For projects situated in the North Cascades a random Magnitude 7+ crustal earthquake has been included in the principal seismic risk studies of the past decade. In the remainder of the state, a random event ranging in magnitude from 5¼ to 6¼ has routinely been assumed.

The hybrid approach in our experience is typically employed for the seismic assessment of major dams in Washington. This is because a comprehensive seismic risk analysis must deal with the possibility of a random seismic event occurring conceivably anywhere beneath a project sited in Washington State. However, the level of conservatism in seismic assessments does not place a random event directly beneath the project site. Typical practice has been to assume some minimum radius beyond which the random event is allowed to occur. This minimum radius is determined by probabilistic analyses that show the likelihood of experiencing such an event at lesser hypocentral distances to be appropriately remote.

Seismology Studies - A few of the more important sources of information on seismicity within Washington known to the DSO are discussed in Section 2.3.9.

Design or Reliability Levels - The present hybrid scheme where MCEs and random earthquakes are considered, works well for major projects with high downstream hazard settings. However, good practice is not clearly defined for the small to medium sized projects with moderate downstream hazard settings. Yet, it is this class of dams which constitute the vast majority of projects in this state. The stepped design scheme presented in Chapter 2.1 affords a rational, consistent way to address the issue of appropriate minimum design levels for this class of small to medium projects. Specifically, for projects where the calculated Performance Goal is Step 3 or less, the DSO would accept a design where the reliability under seismic loading meets or exceeds the level cited in Figure 1, Chapter 2.1. To evaluate projects for seismic stability beyond design step 3 on a purely probabilistic basis, implies seismologists have a better understanding of the magnitude, distribution and frequency of seismic events in Washington then presently exists. Where greater reliability is required, designers are forced into making decisions on qualitative rather than quantitative reasoning. This is accomplished by including greater design events, more conservative estimates of attenuation relationships, larger design spectra, and requiring increased reliability in design features.

Peak Acceleration - Ideally, the designer will use recorded earthquake time histories from earthquakes of the approximate design magnitude, recorded at similar hypocentral distances and on similar foundation conditions as that anticipated for the actual project. Furthermore, a number of representative time histories will be used to account for bias in any one record and/or to develop a design spectra that better envelopes the range of anticipated response spectra at the site. The principal source of earthquake time histories in the US is the National Geophysical Data Center (NGDC) (303) 447-2020 in Colorado. NGDC can provide records sorted on the basis of country, magnitude, free field or building records, source to site distance, and maximum acceleration and velocity. Other sources of earthquake records include the California Division of Mines and Geology Strong Motion Instrumentation Program (916) 322-3105. This program recently published recorded ground motions for the subduction events that occurred in the Cape Mendocino area in 1992. These event records are important additions to the data base for estimating ground motions associated with large magnitude interface events such as are postulated for the Washington Coast.

Unfortunately, there are insufficient acceleration time histories to cover the suite of potential events normally considered at a given site. Thus, some form of attenuation relationship is employed to determine the appropriate peak acceleration that should be used to scale a recorded or synthetic accelerogram or response spectrum for use at the project site. Various relationships are used to estimate the attentuation in earthquake ground motions between the source and the project site. An excellent summary of the various relationships is presented by Green[17]. Green notes the significant differences between the relationships developed by various researchers and recommends using a number of attenuation relationships to minimize bias conceivably introduced by a particular relationship. Crouse[18] has published an attenuation relationship for the Cascadia subduction zone that was developed based on data specific to subduction zones around the world that exhibit similarities to Washington.

In the absence of site specific earthquake studies, as a first approximation to the maximum bedrock acceleration, Plates 3 and 4 of USGS Open File Report 80-471[19] can be used. These Plates present estimates of peak acceleration on rock with a 90% probability of non-exceedance in 50 years and 250 years, respectively. The equivalent annual probabilities of exceedance are approximately 1 in 500 and 1 in 2500. Thus, these Plates provide acceleration maps corresponding to Steps 1 and 3 of the of the 8-step Design Level scheme set forth in Chapter 2.1. The DSO views these Plates as conservative in that they reflect the impact of shallow, smaller magnitude (Magnitude 5½±) earthquakes very near each node used to establish the acceleration contours. These maps reflect appropriate design levels for "peak acceleration sensitive" structures such as buildings but are conservative when used with earthen embankments. For earthen structures, the acceleration spectra and the duration of loading (number of cycles) are both key parameters in the severity of the load. The acceleration values mapped on the Plates are associated with only a few cycles of ground motion.

USGS Report 80-471 predates the discovery of evidence supporting the past occurrence of great interface earthquakes on the Cascadia Subduction Zone and the recently postulated Seattle Fault[20]. In the opinion of the DSO the principal impact to Plates 3 and 4, if Cascadia Interface events were included, would be to increase bedrock accelerations west of the 123° Longitude. The magnitude of the accelerations would be a function of the assumed maximum eastern edge of the locked sections of the interface between the Juan de Fuca and NA Plates and the frequency with which events have occurred. There is considerable controversy regarding both these items. Prior to the development of a consensus, the DSO will use accelerations on the order of 0.25g to 0.35g with larger values used for higher Design steps and for sites increasingly west of Puget Sound. The Seattle Fault, presuming it will be accepted as a seismogenic feature, at this point is probably best treated in a deterministic manner. Based on the magnitude of uplift, the Seattle Fault appears capable of generating an earthquake on the order of Magnitude 7¼.

Acceleration Response Spectra - Seed et. al[21] analyzed 104 accelerograms recorded on differing foundation conditions. Using statistical procedures they developed four "Averaged Acceleration Spectra" appropriate for soft to medium clay and sand, deep cohesionless soils, stiff soil conditions and rock. The spectra were normalized by dividing the spectra by the maximum ground acceleration. The maximum ground acceleration is equivalent to the spectral acceleration at very short periods[22]. Thus, a design acceleration spectra at a given site can be approximated by multiplying the peak acceleration value obtained from publications such as USGS Open File Report 80-471 times the appropriate normalized average acceleration spectra. This process yields an estimate of the spectral accelerations at the foundation level. It is intended to provide guidance to the designer of projects that fall within the first three design steps. Ideally, the designer will seek the services of those well versed in seismology to develop a more appropriate spectra for the site. But, in those cases where this proves impractical, the preceding discussion provides at least a ballpark approximation to the minimum ground motions that should be considered.

Step 2: Evaluate amplification of bedrock motions within the foundation and embankment section. - The acceleration spectra or design time history of ground motions developed in the previous step must be evaluated for amplification effects in any overlying foundation soils and the embankment. Ideally, several time histories that have response spectra appropriate for site conditions will be used in a dynamic analysis of the embankment. Failing this, an approximation of site amplification effects can be obtained by methods outlined by Makdisi and Seed[8]. The method yields an estimate of the maximum crest acceleration through an iterative process that considers the effects of strain dependent shear and damping moduli and acceleration spectra differences at predicted damping levels.

2.3.6 REQUIRED MINIMUM SCOPE OF SEISMIC ANALYSES:

The following, widely quoted, criteria[23] outline the scope of earthquake studies required by the DSO.

Seismic concerns dismissed by Inspection when all the following apply:

1) The dam is a well-built (densely compacted) and peak accelerations are 0.2*g* or less, or the dam is constructed of clay soils, is on clay or rock foundations and peak accelerations are 0.35*g* or less;

2) The slopes of the dam are 3 horizontal to 1 vertical or flatter;

3) The static factors of safety of the critical failure surfaces involving the crest (other than the infinite slope case) are greater than 1.5 under loading conditions expected prior to an earthquake; and

4) The freeboard at the time of the earthquake is a minimum of 2 to 3 percent of the embankment height (not less than 3 feet). Fault displacement and reservoir seiches should be considered as separate problems.

Conduct Deformation and Liquefaction Assessments when:

1) The above criteria are not met.

2.3.7 CONDITIONS REQUIRING REMEDIAL ACTION TO TREAT A POTENTIAL LIQUEFIABLE ZONE:

Where the preceding analysis predicts a soil deposit to be potentially liquefiable, generally the DSO will require the treatment or removal of these soils. The following comments cover the principal exceptions where the DSO **will not require** the removal or treatment of an identified potential liquefiable zone.

New Dams - The DSO will accept a design that leaves potentially liquefiable soils in the foundation for small dams (up to 15 feet high) where,

A failure of the dam would not result in any loss of life, and

The potentially liquefiable zone is of sufficient depth and areal extent that it is not practicable to remove or stabilize it.

In addition, the DSO will not require the removal of liquefiable materials from within the footprint of the dam under the following conditions.

The likelihood of experiencing earthquake induced ground motions capable of causing a liquefaction failure of the dike <u>while</u> the facility impounds a significant pool is smaller than the design annual exceedance probability determined as specified in Chapter 2.1, (Here, one could go to greater design steps than level 3.) and

The owner agrees in writing to accept the potential risk to the facility posed by leaving potentially liquefiable soils within the embankment footprint.

The above generally applies to new stormwater detention facilities.

Existing Dams - Where there is an identified liquefaction concern, no treatment of liquefiable zones will be required if the annual probability of failure is equal to or less than the design annual exceedance probability established on the basis of the downstream hazard, see Chapter 2.1.

In certain cases the DSO will allow a facility with an identified liquefaction concern to remain in service, but mandate severely lowered pool levels. The pool restriction must be shown to preclude a complete loss of freeboard in the event the dam is subjected to the design earthquake and undergoes crest settlement.

2.3.8 TYPICAL PRACTICE TO ADDRESS IDENTIFIED LIQUEFACTION CONCERNS:

New Projects - Where seismic stability concerns are documented, the resolution of these concerns typically has a common thread. As a practical matter, this involves one or more of the following steps:

Removing and replacing potential liquefiable soils in the foundation,

Limiting the saturated portion of the embankment and foundation by the judicious layout of drains.

Providing relatively large freeboard levels to accommodate potential embankment crest settlements.

Employing restrain devices to limit the potential movement of the embankment in the event portions of it liquefy.

Constructing wide transition zones and large filters to minimize the likelihood of fault displacement compromising the integrity of critical zones.

Existing Dams - Since the structure is already in place it is generally impractical to completely remove the zones in the embankment and/or the foundation that are judged susceptible to instability under dynamic loading. This leaves two principal avenues to address liquefaction concerns:

Construct berms to constrain in place the liquefiable zones so that their strength loss will not jeopardize the overall stability of the embankment. Seed et al.[24] present data on the undrained shear strength of soils following liquefaction that has been back-calculated from past failures. This data is of assistance in the design of such containment berms.

Provide additional drainage features to lower the phreatic surface or ideally, maintain the potentially liquefiable zones in an unsaturated condition. As a practical matter, this solution is restricted to the central and downstream portions of embankments.

There are a number of exotic solutions such as in situ densification, stone columns, cellular diaphragms and chemical stabilization. However, these specialty areas are outside the scope of this guidance document.

2.3.9 TECHNICAL RESOURCES:

The DSO will develop a technical note dealing at greater depth with seismic design. The principal focus of this technical note will be to catalogue source materials covering the various seismogenic zones in the region that conceivably could produce damaging ground motions in Washington State. This catalogue would include data on such elements as known seismogenic features, the magnitude-frequency relationships for earthquakes, focal depths, time histories considered representative of earthquakes and/or response spectra. Ideally, this would serve the designer in identifying the seismogenic zones that could conceivably produce earthquakes large enough to require inclusion in the seismic analysis of the project under consideration.

Regional seismicity studies abound. They have been performed either by or under the auspices of the Bureau of Reclamation, Corps of Engineers, the Nuclear or Hydropower communities, Federal Emergency Management Agency, state Resource agencies and as academic projects. The principal repositories for these studies are:

Corps of Engineers Library in Seattle

Department of Natural Resources Library in Olympia

Regional Office of the Federal Energy Regulatory Commission in Portland (The seismic studies done as a part of the Part 12 requirements are available for public review.)

Department of Ecology Dam Safety Office files.

The following is a brief summary of the more important studies and reports:

Overview of the Pacific Northwest - There are a number of studies that provide a general view of seismicity in the Pacific Northwest. Principal among these is the work of Perkins et al.[19] where some nine seismogenic zones are identified that either lie within or include parts of Washington or that are close enough to affect areas of the state. Data is provided on annual occurrence rates of earthquakes from magnitude 4 to 8+ for 0.6 magnitude intervals. The accompanying text briefly describes the assumptions regarding the approximate focus. Peak accelerations are predicted on rock. The anticipated peak accelerations on rock are mapped for return periods of 100, 500 and 2500 years. These correspond to annual probabilities of exceedance of nearly 1 in a 100, 500 and 2500. As previously noted, in lieu of more site-specific studies, this document provides a means of estimating acceleration levels for projects falling within the first three design steps as outlined in Chapter 2.1.

Regional Studies - The Bureau of Reclamation (BR) and the Corps of Engineers (COE) have sponsored and/or performed a number of regional seismological assessments of their projects. The COE projects generally lie west of the Cascades or along the Columbia or Snake Rivers. The COE reports are generally project specific. Conversely, the BR has a considerable number of projects in the central and eastern regions of Washington and accordingly they have sponsored more detailed regional analyses. Prime examples of these are the two seismotectonic studies done for the Northern Cascades[25] and the Walla Walla Section of the Columbia Plateau[26]. The reports identify surface features and discuss their seismic potential. Data is presented on the estimated magnitude and recurrence intervals for the range of earthquake magnitudes believed plausible for the seismic zone or feature.

The hydropower community has also performed seismological studies to support the design and/or the safety of existing facilities. Normally, these studies include a time history for the design event along with response spectra and estimates of the magnitude-frequency relationship for various earthquake magnitudes.

The State Department of Natural Resources Geologic Section produces a number of valuable documents regarding seismicity and maintains one of the most complete libraries of reference materials relating to the geology of Washington State. Their extensive collection of detailed geologic maps is of particular value.

Postulated Interface Subduction Event **-** The National Earthquake Hazards Reduction Program (NEHRP) has sponsored numerous studies[27] to investigate the question of whether recently postulated, earthquakes at the interface between tectonic plates are plausible. A convincing body of evidence has been amassed supporting the past occurrence of such events. Various approaches have been employed to estimate the magnitude, to develop a plausible time history and to predict a recurrence interval for such events.[28,29]

The Washington Public Power Supply System has sponsored extensive research into the seismicity of the Pacific Northwest. This body in conjunction with the Nuclear Regulation Commission has studied and reported on the recently postulated past occurrence of subduction earthquakes off the Pacific Northwest Coast. The Draft report[30] on the Satsop Project accepted as a plausible scenario a Moment Magnitude M_w 8¼ with the probable occurrence of some 5 events in the last 3000 years. The lineal zone for maximum energy release is believe likely to be a short distance off the Washington Coast[31]. As previously noted Crouse[18] has developed a model to evaluate the attenuation of the earthquake motions from the zone of maximum energy release that is purported to be representative of both deep intra-plate events and postulated interface events.

Regional Earthquake Catalogues **-** The Geophysics Program at the University of Washington operates a Seismology Laboratory and seismograph network across the state and in northern Oregon. A detailed catalog of seismic events detected by the network is maintained and published periodically by the Department of Natural Resources as an Information Circular of the Division of Geology and Earth Resources, e.g. Circular 84[32]. A listing can be obtained of the seismic activity proximate to a project site anywhere in the state.

In identifying the preceding key studies and repositories of data, it was not the intent of the DSO to obviate the need for a competent seismologic assessment of the project site. Instead, citing the above sources was an attempt to more widely disseminate some of the more important studies and resources that have come to the attention of the DSO through our role as regulators. As in the past major projects will continue to include input from engineering geologists and seismologist well versed on the region. This is not our target audience. Instead, the DSO hopes to alert the smaller engineering firms to the data developed on major projects so that they can include such data in their analysis, where appropriate. Where seismic concerns are of a greater magnitude, the engineer and owner will be better informed to recognize early on the need for qualified consultants specializing in seismic design.

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2.4 INFLOW DESIGN FLOOD

2.4.1 APPLICATION:

Design flood hydrographs are used in sizing the hydraulic features of spillways and in determining floodwater storage requirements for a project. The Inflow Design Flood (IDF) is the extreme loading condition used to design or evaluate the project's spillway(s). It is the largest flood that a given project is designed to safely accommodate.

2.4.2 HISTORICAL PERSPECTIVE ON FLOODS IN WASHINGTON:

Floods in Washington occur in response to a variety of conditions. Floods may be produced by major rainfall events, by combined rainfall and snowmelt events or by rapid snowmelt. Extreme floods, such as Inflow Design Floods (IDFs) are produced by either extreme rainfall or by extreme rainfall in combination with snowmelt.

The physical dimensions of a given dam and reservoir project make either flood peak discharge or a combination of flood peak and runoff volume the controlling design considerations. Thus, flood characteristics such as peak discharge, runoff volume and flood duration, as described by the flood hydrograph, need to be considered during project design.

Characteristics of extreme floods, such as an IDF, are determined by a number of meteorologic, hydrologic and hydraulic factors. In particular, the magnitude and duration of the precipitation event and its temporal and spatial distribution are dominant factors in determining the resultant flood hydrograph.

Short Duration Extreme Storms - Short duration extreme storms (1 to 6 hours duration) are warm season events in both eastern and western Washington. These "thunderstorm" events are characterized by very high intensity rainfall for an isolated period during the storm. They can produce very flashy flood hydrographs with a very large peak discharge. In eastern Washington, thunderstorm events can produce severe flooding on small watersheds and are commonly the controlling design event. Conversely, in western Washington, thunderstorm events rarely produce flooding of the magnitude produced by longer duration precipitation events. Thunderstorms in western Washington can sometimes be the controlling design event on small urbanized watersheds where much of the land area is covered by impervious surfaces.

Intermediate Duration Extreme Storms - Intermediate duration extreme storms (12 to 18 hours duration) occur generally in fall to early winter in eastern and western Washington, but can also occur in the warm season in eastern Washington. These events typically contain high rainfall intensities for a period of several hours, and produce floods which are flashy, have a large flood peak and a moderate runoff volume. Occasionally, this type of storm is the controlling design event in western Washington for projects with limited floodwater storage capability.

Long Duration Extreme Storms - Long duration extreme storms (24 to 72 hours duration) occur predominantly in the late fall and winter months in eastern and western Washington. These events typically contain moderate and uniform rainfall intensities and have a large total rainfall depth. These storms produce a flood with a sustained flood peak which is well supported by a large runoff volume. This type of storm is often the controlling design event in western Washington. It may also be the controlling design event in eastern Washington at projects with a large floodwater storage capability or at projects where the tributary watershed is very large.

Snowmelt Contribution to Flooding - Snowmelt floods are the controlling design events only on very large watersheds where the watershed size is much larger than the areal coverage of any candidate design storm. For smaller watersheds, snowmelt tends to uniformly add to the flood hydrograph produced by an extreme precipitation event and has a greater affect on increasing runoff volume than on increasing the flood peak discharge.

2.4.3 DESIGN PRACTICE - RAINFALL-RUNOFF MODELING:

Inflow design flood hydrographs are normally computed using rainfall-runoff computer models[7,8,9,10,11,13,14]. There are a number of factors which must be considered in conducting the rainfall-runoff modeling to properly evaluate the flood response of the watershed and in determining the controlling design storm/flood event for the project. Those factors can be lumped into two general categories: factors associated with the precipitation event(s) which generate the floods; and factors associated with the conditions in the watershed and at the project at the beginning of the storm/flood event. Those two categories of factors are discussed in the following sections.

2.4.3.1 Rainfall Modeling - Design Storm Considerations

Precipitation data are a primary input to the rainfall-runoff computer models. Consideration of the items discussed in section 2.4.2 above, indicates that it is usually necessary to compute flood hydrographs for several storm durations and associated storm types to determine which flood characteristics will control the design of the project. The controlling flood is labeled the Inflow Design Flood (IDF).

To compute a flood hydrograph, information is needed about the magnitude, and temporal and spatial distribution of the candidate design storms. Each of these are discussed below.

Design Storm Magnitude - The magnitude of the precipitation depth used in the design storm is dependent upon the design step selected to meet the required design/performance goal for the project. Using the 8-Step format, described in section 2.1, design storms range from a minimum of a 500 year event (Step 1) to a maximum of Probable Maximum Precipitation (PMP) at Step 8. Procedures for determining the design step and the corresponding design/performance goal are described in detail in *Technical Note 2, Selection of Design/Performance Goals for Critical Project Elements*.

As discussed previously, the storm duration (short, intermediate or long duration) which will generate the flood that controls the project design will not normally be known prior to conducting the flood analyses. Therefore, several design storm candidates, representing various durations, should be developed for use in the flood analyses. Estimation of the magnitude of the candidate design storms for the various durations is based on procedures developed by Schaefer [1,2]. These procedures are described in *Technical Note 3, Design Storm Construction*.

To summarize the design procedure, a precipitation depth is computed for each of the three candidate design storm durations. A 2 hour amount is computed for the short duration storm, a 6 hour amount is computed for the intermediate duration storm, and a 24 hour amount is computed for the long duration storm - each precipitation amount corresponding to the design step and level of design protection selected from procedures described in Technical Note 2.

If design step 8 is appropriate, the design storm magnitude corresponds to PMP which is estimated to be the theoretical maximum amount possible for a given duration. Information on computing a site specific PMP is contained in National Weather Service (NWS) Hydrometeorological Report No. 57 (HMR-43)[3], which will replace HMR-43.

The design step determined by procedures described in Technical Note 2 and the precipitation depth associated with the design/performance goal are considered to be minimum acceptable levels. Dam owners and their engineering consultants are encouraged to provide as much design protection beyond these levels as practicable.

Design Storm Temporal Distribution - The temporal characteristics of extreme storms are inherently stochastic. Pertinent storm characteristics such as seasonality of occurrence, macro storm patterns, precipitation depth-duration relationships, time of occurrence and the temporal distribution of high intensity storm segments can best be described in probabilistic terms. Probabilistic based procedures for construction of dimensionless hyetographs have been developed by Schaefer[2] and are described in Ecology publication 89-51 entitled *Characteristics of Extreme Precipitation Events in Washington State* and incorporated into *Technical Note 3, Design Storm Construction*. The probabilistic methodologies described in those reports allow the practitioner to predetermine the degree of conservatism to be applied in developing synthetic design storms. Using these methodologies, it is possible to construct a wide variety of dimensionless hyetographs. Hyetographs can be developed for typical conditions which are useful in estimating synthetic flood frequency curves. Hyetographs can also be developed which represent more severe conditions suitable for use in computation of design floods.

Design Storm Spatial Distribution - There exists only a limited amount of detailed data on the spatial characteristics of extreme storms. For this reason, probabilistic procedures to account for the decay of storm intensity with areal coverage have not been developed. The approach recommended here is to use standard deterministic depth-area duration curves. Acceptable methods are described in Schaefer[2], HMR-43/57[3] and NOAA Atlas #2[4].

Design Storm Hyetograph Construction - Recommended procedures and worksheets for construction of design storm hyetographs are contained in *Technical Note 3* of the Dam Safety Guidelines.

2.4.3.2 Runoff Modeling - Initial Conditions

As discussed previously, rainfall-runoff modeling is normally used to compute flood hydrographs. While characteristics of the design storm are dominant factors in the computation of the design flood, there are a number of other meteorologic and hydrologic considerations which are also important. To conduct flood analyses, information is needed on the conditions which are likely to exist in the tributary watershed prior to the occurrence of the design storm. The first consideration, which sets the stage for other initial conditions, is the season of occurrence of the extreme storm/design event.

Design Storm Seasonality - Initial watershed and reservoir conditions are strongly influenced by the climatic characteristics associated with the season of occurrence of the storm type being investigated. A review of observed extreme storms (Schaefer[2]) has shown that the seasonality of occurrence of extreme storms is strongly related to storm duration.

<u>Recommendations</u> - The time of year during which a candidate design storm is assumed to occur is to be chosen with consideration of the seasonality of observed extreme storms for the specified storm duration (short, intermediate, or long duration). The time of year (month) selected should represent a time for which 10% or more of extreme storms have been observed to occur and which is associated with meteorologic and hydrologic conditions conducive to generation of floods. Information in Schaefer[2] and in HMR-43/57[3] can be used to aid in selection of the month of occurrence based on the historical record.

In particular, seasonality of occurrence directly influences several initial conditions which are factors in the flood analyses. Those initial conditions include the following which are discussed below.

- Streamflow or other inflow into the reservoir
- Soil moisture conditions and associated runoff characteristics
- Snowpack water equivalent, air temperature and wind speeds needed for snowmelt computation
- Reservoir level

Initial Reservoir Inflow - Natural Streamflow - Experience has shown that meteorologic conditions antecedent to extreme storms in Washington are not significantly different from those conditions which would otherwise normally exist during a given time of year (Schaefer[2]).

Recommendations - In most design situations, initial streamflow can be taken to be that discharge which would normally be expected to occur during the time of year when the design storm is assumed to occur.

Where the design or operation of a project during flood conditions is sensitive to the magnitude of normal inflow, a more conservative initial inflow should be assumed. In this situation, a discharge with 1 in 10 chance of being exceeded during the selected month would be a reasonable assumption.

Information on monthly streamflow statistics can be obtained from publications prepared by the United States Geological Survey (USGS)[17,18,19,20]. In actual applications, most project sites do not have streamflow gages within their tributary watershed. Transposition of streamflow statistics from hydrologically similar watersheds is normally used in these cases.

Initial Reservoir Inflow - Other Sources - Some projects receive inflow from sources other than from natural streamflow. These sources of inflow are usually controlled by pumps, diversion works, etc.

Recommendations - The magnitude of inflow from other sources is to be based on one of two conditions. If the source is unregulated, the inflow magnitude should correspond to the performance goal AEP obtained from procedures in Technical Note 2. If the source of inflow is regulated, the inflow should correspond to the discharge which would be expected to occur during flood conditions. The above guidance should be conservatively interpreted for site specific conditions.

Initial Soil Moisture Conditions and Runoff Characteristics - The soil moisture conditions which exist at the time of an extreme storm significantly affect the volume of runoff which will be produced by the storm. The soil moisture conditions are, in turn, determined by the meteorologic conditions which prevail in the days and weeks prior to the occurrence of the extreme storm.

Recommendations - Soil moisture and runoff parameters chosen for use in rainfall-runoff modeling should be selected based on the typical meteorologic conditions to be expected for the time of year associated with the occurrence of the design storm. In those cases where soil moisture and runoff parameters cannot be confirmed by calibration of the model with observed rainfall-runoff data, streamflow data, or by soil moisture water budgets, conservative estimates of the parameters should be selected.

With regard to estimation of short duration thunderstorm runoff, infiltration-based runoff models have generally been found to perform best and are preferred.

In western Washington, many of the glaciated areas in Puget Sound contain deep, relatively pervious soils. The combination of moderate rainfall intensities and pervious soils results in a condition where little of the runoff occurs as overland flow. The majority of the runoff can occur as shallow subsurface flow or interflow and exhibit longer watershed response characteristics (time lags) than that for surface runoff. Where interflow is expected to be a significant contribution to runoff, computations must be conducted to explicitly account for the volume and timing of interflow. Computation procedures applicable to interflow conditions are described in Dinicola[12], HSPF[13] and COE[14].

Initial Snowpack and Snowmelt Runoff Computation - For watersheds roughly less than 1000 mi², the peak discharge of extreme floods is governed primarily by the magnitude and duration of the high intensity portion of the extreme storm. Runoff rates from snowmelt tend to be relatively uniform and, as such, tend to uniformly augment flood discharge. For these reasons, snowpack and snowmelt runoff considerations are usually more important on projects sensitive to flood volume rather than flood peak discharge.

Recommendations - The decision to include snowmelt in a flood analyses should be based on the likelihood of having a snowpack at the time that the design storm is assumed to occur. If a snowpack is likely at the time of year of the design storm, then its magnitude (water equivalent) should be based on the historical snowpacks experienced in the watershed. Specifically, snowpack must be considered when the typical duration of snow on ground (for a contiguous period) is in excess of 20% of the season of interest. For modeling purposes in watersheds with both foothills and mountainous zones, this will likely result in the use of no snowpack in the lowest zone(s) and a large snowpack in the high elevation mountainous zones.

In modeling application, increasingly conservative snowpack assumptions should be used with higher design steps, as shown in Table 1.

TABLE 1 - EXCEEDANCE PROBABILITIES FOR SELECTION OF PARAMETER VALUES FOR USE IN SNOWMELT RUNOFF COMPUTATION

METEOROLOGIC PARAMETER	D E S I G N S T E P		
	1 - 3	4	8
Snowpack Water Equivalent	1 in 2 or Average Value	1 in 5	1 in 20
Temperature and Wind Values	1 in 2 or Average Value	1 in 5	Theoretical Maximums from HMR-43 or HMR-57

Snowpack and water equivalent data may be obtained from Phillips[22], the NWS[21], and from the Soil Conservation Service (SCS)[23].

For snowmelt computations, information is needed for air temperatures, temperature lapse rates and wind speeds which accompany the design storm. Selection of appropriate values should be based on conditions which have occurred during observed extreme storms. This information may be obtained from Climatological Bulletins published by the NWS[21] and Phillips[22]. Theoretical maximum values may be obtained from NWS publication HMR-43/57[3]. As above, increasingly conservative temperature and wind values should be used with higher design steps as shown in Table 1.

Initial Reservoir Level - The reservoir level at the start of the design flood determines the storage available for accommodating floodwaters. Depending on the magnitude of storage available and the magnitude of the flood runoff volume, the initial reservoir level can be a significant factor in determining how the projects perform during floods.

Recommendations - The reservoir level which is assumed to be present at the start of the design flood should be determined based on the expected operation of the project. The selection of a starting level should be based on the time of year when the design storm is assumed to occur, the reservoir inflows expected at that time and the proposed reservoir operation scheme.

For projects with ungated spillways, it is common to assume the reservoir level to be at or above the invert of the principal spillway. In those cases where insufficient data is available to ascertain the likely initial reservoir level or where the reservoir level is highly variable, a suitably conservative estimate should be used.

2.4.5 DETERMINING THE INFLOW DESIGN FLOOD:

The IDF is selected from amongst the flood hydrographs generated by the various candidate design storms for the selected design step. The IDF is taken to be the flood hydrograph that will produce the highest reservoir level and whose flood characteristics would place the most stringent design conditions on the project.

Many of the rainfall-runoff computer models referenced previously have the capability to perform flood routing. They can be used to route flood hydrographs through the project's reservoir to determine spillway outflows and the maximum reservoir water surface elevation.

2.4.5.1 Sensitivity Analyses

Many of the parameters used in rainfall-runoff modeling are stochastic in nature and are highly variable. Yet, the parameter values assumed to be present at the onset of the design storm can significantly affect the resultant flood.

Recommendations - Sensitivity analyses should be conducted on those parameter values which are anticipated to be a source of uncertainties to determine the sensitivity of the resultant flood hydrograph. Potential sources of uncertainty would include factors such as: the temporal distribution of the design storm; soil moisture deficits and soil infiltration rates; unit hydrograph lag times; snowpack magnitudes; and initial reservoir levels. The results of any sensitivity studies should be used as a basis for final selection of the IDF.

The candidate design flood may be accepted as the Inflow Design Flood if the value of the sensitive parameter(s) used in the analyses has less than 1 chance in 10 of being exceeded during the season of interest.

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2.4A INCREMENTAL DAMAGE ANALYSIS

2.4A.1 APPLICATION:

Incremental Damage Analysis (IDA)[1,2] is an alternative method of selecting the magnitude of the Inflow Design Flood (IDF). It involves sophisticated flood inundation analyses which may allow a reduction in the design flood magnitude. It is not applicable to all projects. The methodology is applicable to those situations where a failure of the dam during an extreme flood event would not significantly increase the level of downstream flooding over that caused by the ongoing, natural flood. In this situation, there is minimal public safety benefit to be gained from constructing a larger spillway to prevent the dam from overtopping and failing - essentially all damages would have occurred from natural flooding prior to dam failure.

2.4A.2 OBJECTIVE:

To determine the magnitude of natural flooding, beyond which, little or no additional potential exists for loss of life and significant property damage due to increased flooding from reservoir waters released by failure of the dam.

2.4A.3 DETERMINING APPLICABILITY:

This methodology is generally applicable to those cases where the potential magnitude of natural flooding from the tributary watershed is similar in magnitude to an overtopping induced dam break flood. Thus, small dams on large watersheds, and those reservoirs with limited (total) storage capacity relative to the runoff capability of the watershed, are often amenable to this methodology.

As a rough measure, if ß is less than about 5, IDA methodology may be applicable.

$$ß = \frac{Q_{break}}{IDF} \tag{1}$$

where:

Q_{break}	=	Dam break peak discharge (cfs) from overtopping induced failure
IDF	=	Inflow design flood peak discharge (cfs) determined from procedures in Section 2.4

In many cases, the likelihood that this methodology will result in a reduced IDF cannot be determined in advance. The final determination can only be made by flood routing analyses to compare the incremental increase in flood levels produced by the dam failure relative to the preceding flood levels from natural flooding.

2.4A.3 ANALYSIS METHODOLOGY:

Although the dam break and downstream flood routing portions of the IDA are technically sophisticated, the basic approach to the IDA is relatively simple. The normal progression for the various steps is discussed in the following sections and described in the flowchart in Figure 1.

2.4A.3.1 Inundation Analysis for Candidate Inflow Design Flood - No Dam Failure

The analysis is started by selecting a candidate IDF as an initial starting point. The minimum acceptable magnitude for the IDF, in any situation where there is a potential for loss of life, is a flood corresponding to Design Step 3, Section 2.1, *Design/Performance Goals for Critical Project Elements*. Thus, unless there is reason to start at a larger inflow flood, a logical place to start is with a Design Step 3 inflow flood hydrograph developed in accordance with procedures described in Section 2.4, *Inflow Design Flood*.

Reservoir routing procedures, such as used in computer models HEC-1[3] and DAMBRK[4], are normally used to determine the flood hydrograph released through the spillway(s) at the dam. Hydraulic routing procedures are usually used to route the flood through the downstream valley and determine the flood levels and areal extent of inundation. This sets the base level, against which the incremental increase in flood levels caused by a dam failure are to be measured.

2.4A.3.2 Dam Break Inundation Analysis

The second step in the analysis is to determine the flood levels and areal extent of inundation produced by a failure of the dam as a result of overtopping by floodwaters. The magnitude of the inflow flood used to initiate failure of the dam should be selected sufficiently large as to produce a depth of overtopping which would likely cause failure of the dam. This often corresponds to an inflow flood in the range of 110% to 135% of the candidate IDF. Regardless of the magnitude of the inflow flood used to initiate failure, the dam should not be assumed to fail until the reservoir level is at or near its maximum level in response to the peak discharge of the inflow flood.

Sophistication of Analyses Required - Detailed inundation information is needed to properly assess the incremental damages. This usually requires that sophisticated flood routing procedures be employed in the inundation analyses. Recommended procedures for conducting dam break analyses are described in the Dam Safety Guidelines *Technical Note 1, Dam Break Inundation Analysis and Downstream Hazard Classification*[7].

<u>Requirements/Minimums</u> - There are two general cases to be considered in selecting an appropriate computer flood routing model. The first case is where an accurate analyses of the attenuation of the dam break flood is not needed and a conservative solution (no or minimal attenuation) is adequate. In this situation, a computer model such as HEC-1[3], employing a hydrologic routing procedure without attenuation would be acceptable.

Conversely, when the attenuation of the dam break flood hydrograph is an important consideration, sophisticated hydraulic routing computation procedures, such as employed in DAMBRK[4] are necessary. In either case, flood routing must be continued to a point sufficiently far downstream that no significant threat remains to life or property from the dam failure.

2.4A.3.3 Assess the Incremental Increase in Damages and Hazards

The third step is to assess the incremental increase in damages and hazards posed by the dam break flood. Both the Federal Energy Regulatory Commission (FERC)[1] and the State of Colorado[2] have criteria where an incremental increase in flood level of 2 feet or less is not judged to pose a significant increase in hazard. In addition, the U.S. Bureau of Reclamation (USBR) have developed extensive guidelines[6] to assist in assessing the hazards posed by floodwaters. Figure 2 contains one such guideline - a hazard assessment curve for the hazard posed by various combinations of floodwater velocity and depth. The USBR has also developed hazard assessment curves for flood hazards at mobile homes and to passenger automobiles.

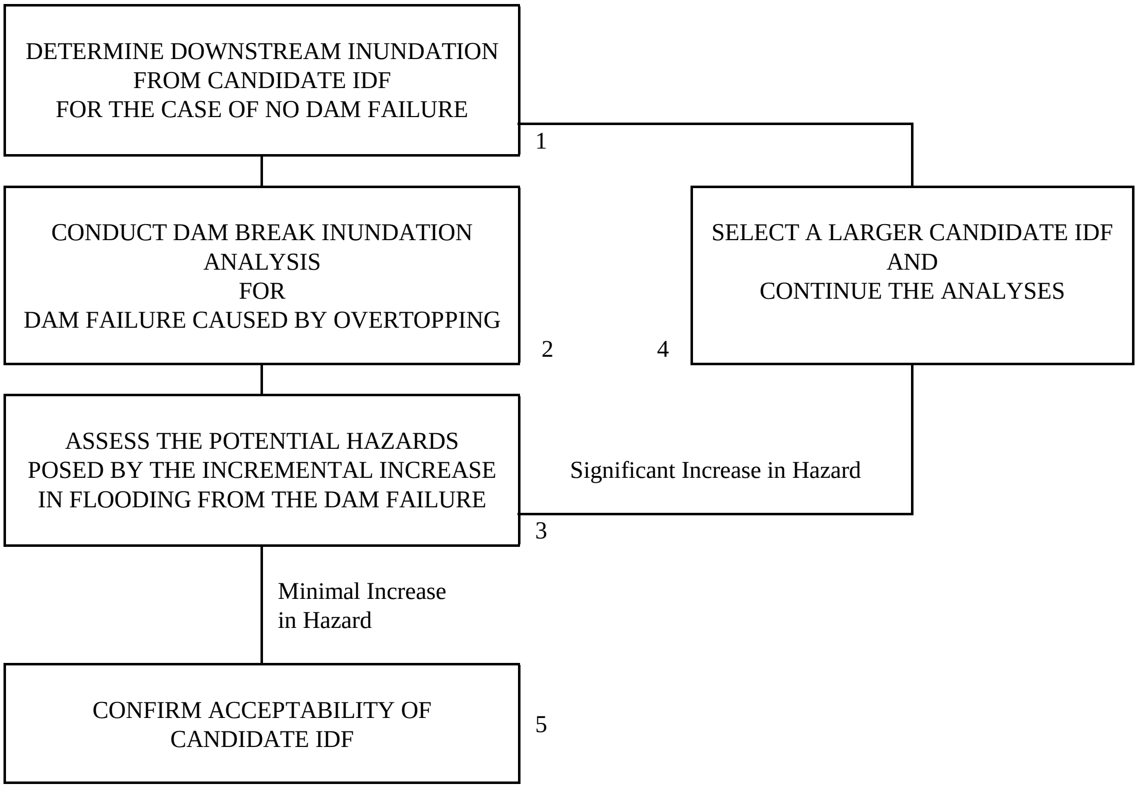


FIGURE 1 - FLOWCHART FOR CONDUCTING INCREMENTAL DAMAGE ANALYSES

Recommendations/Minimums - The general criteria for an incremental flood level increase of 2 feet or less[1,2] and the hazard assessment curves and guidelines developed by the USBR[6] represent the best information currently available. These are considered acceptable criteria for assessing the hazards from incremental increases in flooding.

If the incremental increases in flood levels along the affected reaches of the watercourse are judged to pose a significant risk to life, then a larger candidate IDF is selected and the IDA process described in the flowchart in Figure 1 is started over.

2.4A.3.4 Confirm the Validity of the Solution - Sensitivity Analyses

If the incremental increases in flooding are deemed not to be significant, the final step in the analysis is to confirm the validity of the solution. This is accomplished by conducting sensitivity analyses on those parameters which have the greatest influence on the resultant flood levels along the downstream watercourse. In many applications, the assumed breach characteristics, such as breach dimensions and elapsed time for breach development are parameters that have a significant affect. Use of computer program BREACH[8] can be used to estimate the breach characteristics and may help reduce the range of breach parameters to be evaluated in the sensitivity analyses.

The channel and overbank roughness coefficients can also have a significant influence on the incremental increases in flooding and are often included as elements of a sensitivity analyses. Additional information on sensitivity analyses are contained in Technical Note 1[7].

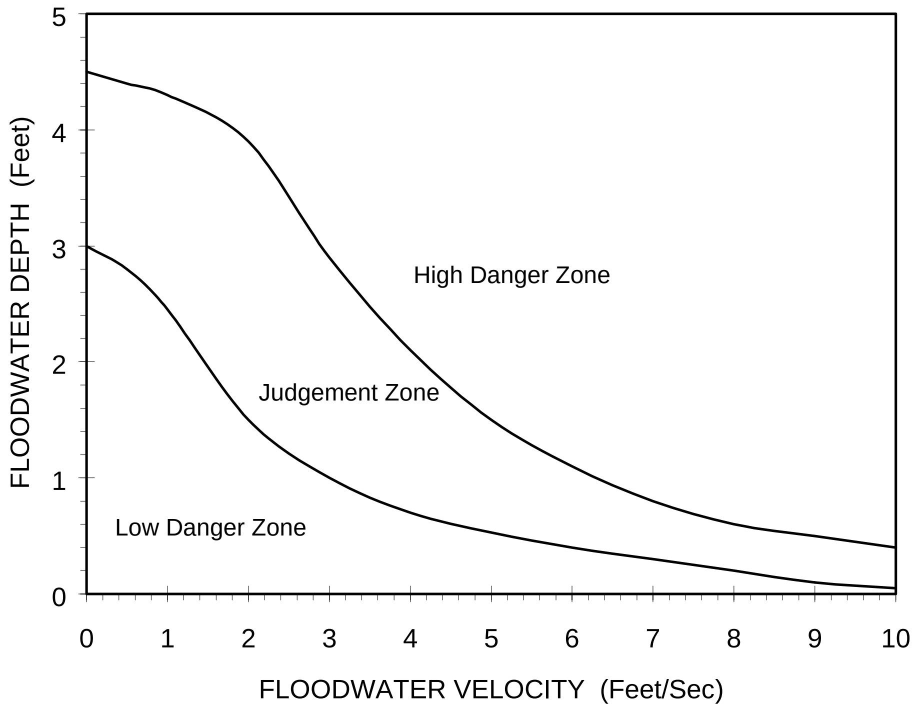


FIGURE 2 - HAZARD ASSESSMENT CURVE FOR FLOOD DANGER TO ADULTS FROM FLOOD VELOCITY AND FLOOD DEPTH

Selection of the Inflow Design Flood (IDF) - If the sensitivity analyses indicate that changes in the magnitude of assumed parameters within the range of reasonably expected values does not alter the conclusion of no significant increase in hazard, then the IDA is concluded. The IDF is selected as the candidate IDF that was used to establish the base level of flooding for the current stage of analyses.

Alternatively, if the sensitivity analyses indicate that the downstream flood levels are significantly affected by reasonable changes in breach characteristics or routing parameters, then a larger candidate IDF is selected and the analyses is restarted using more conservative breach characteristics and/or routing parameters.

Requirements/Minimums - As discussed previously, the minimum inflow design flood acceptable for use in Incremental Damage Analyses is the flood corresponding to Design Step 3, Section 2.1 *Design/Performance Goals for Critical Project Elements*. This is consistent with design applications for any situation where there is the potential for loss of life in the event of a dam failure.

2.4A.5 REFERENCES

1. Federal Energy Regulatory Commission, Engineering Guidelines for Evaluation of Hydropower Projects, Office of Hydropower Licensing, FERC 0119-2, Chapter II and Appendix IIC, April, 1991.

2. State of Colorado, Rules and Regulations for Dam Safety and Dam Construction, Rule 5.A.(5)(c), September 30, 1988.

3. US Army Corps of Engineers, HEC-1, Flood Hydrograph Package, USER's Manual, Hydrologic Engineering Center, U.S. Army Corps of Engineers, Davis, CA, September, 1981.

4. Fread, D.L., DAMBRK: The NWS-Dam Break Flood Forecasting Model, Office of Hydrology, National Weather Service, Silver Spring, MD, June 20, 1988.

5. USBR, Guidelines for Defining Inundated Areas Downstream from Bureau of Reclamation Dams, Bureau of Reclamation, Denver, CO, June, 1982.

6. USBR, Downstream Hazard Classification Guidelines, ACER Technical Memorandum No. 11, Bureau of Reclamation, Denver, CO, December, 1988.

7. Schaefer, M.G., Technical Note 1, Dam Break Inundation Analysis and Downstream Hazard Classification, Dam Safety Guidelines, Washington State Department of Ecology, Olympia, WA., July, 1992.

8. Fread, D.L., BREACH: An Erosion Model for Earthen Dam Failures, Hydrologic Research Laboratory, NOAA, NWS, June, 1987.

9. Pansic, N. and Borg, J.E., Determining Inflow Design Floods for a System Of Dams, Hydro Review, pp 60-67, September, 1992.

CHAPTER 3 - GEOTECHNICAL ISSUES

3.1 EXPLORATIONS

3.1.1 APPLICABILITY:

The focus of this document is earthen embankments. Concrete structures generally require specialized investigations that should only be done by those experienced in this particular area.

3.1.2 EXPLORATION PROGRAM OBJECTIVES:

Engineering Geology Investigation - Assess geologic hazards potentially affecting the site including: the seismic setting, reservoir rim stability, potential adverse behavior of the foundation and abutments. Provide guidance in refining the scope of the subsurface exploration program.

Subsurface Explorations - Characterized the subsurface conditions in the foundation and abutments and identify suitable sources of construction material.

3.1.3 DAM SAFETY OFFICE OBJECTIVES:

To provide guidance on the minimum level of explorations necessary to develop and support the design of an impoundment for an "average case". An "average case" would be a project site that presents no significant foundation problems. The exploration program serves primarily to identify the depth to a suitable foundation stratum and confirm that adequate quantities of acceptable borrow materials are available.

To identify public domain information sources that may be of assistance to the project proponent and their engineer. The intent **is not to supplant the services of an engineering geologist or geotechnical engineer**, but rather to help the project proponent recognize early on the scope of foundation issues that will likely have to be addressed. Ideally, they will then seek the services of such firms.

To identify some of the common difficult foundation conditions with which the designer must contend.

Ideally, all projects would employ the services of a geotechnical engineering firm. However, this is not the case for many low and medium sized projects. Instead, the general civil engineering firm of record often elects to perform these tasks itself. To encourage owners to acquire the services of geotechnical engineers, the DSO seeks to illustrate to project proponents the valuable service they provide. Ultimately, where difficult or unusual foundation problems are encountered, the DSO will require that geotechnical expertise be sought to resolve the concern.

3.1.4 MINIMUM EXPLORATION PROGRAM:

Table 1 presents a summary of the minimum elements of an exploration program that must be undertaken to support a project design submittal. This minimum level is considered appropriate for those project sites that do not present significant foundation problems.

TABLE 1 - MINIMUM REQUIRED EXPLORATION PROGRAM

EXPLORATION METHODS	EARTHEN EMBANKMENTS LOW	MEDIUM	HIGH	BORROW SITES
Backhoe Test Pits	✓	✓	✓	✓[2]
Borings	✓[1]	✓[1]	✓	✓[1,2]
Geophysical Explorations			✓[2]	✓[2]
Permeability Tests		✓[2]	✓	

NOTES:

[1] Borings appropriate where overburden has appreciable thickness

[2] Desirable on a case by case basis

3.1.5 ENGINEERING DESIGN CONSIDERATIONS:

Earthen embankments are frequently founded in part or wholly in the natural overburden. Thus, the behavior of the overburden column must be investigated to confirm that it will perform satisfactorily under the changes in stress and pore water conditions imposed by constructing the dam. A number of excellent publications covering the exploration phase of a project have been developed by the Corps[1] and Bureau[2], among others. In addition, it is important that one reviews case histories of past failures to better understand how significant geologic features were either missed, misunderstood or misjudged in their response. The 1929 work by Terzaghi[3] is a classic study in this area, among others[4,5].

Numerous excellent summaries of the geology in Washington are available. Of particular note in this area is the 1989 compendium of articles in Bulletin 78[6] published by the Department of Natural Resources in association with the state section of the Association of Engineering Geologists.

The principal problems posed by soils in Washington are as follows:

> Borrow site soils are often difficult to compact because their moisture contents are excessively wet or dry of optimum moisture content. Generally, soils in eastern Washington are on the dry side while those in western Washington are on the wet side.
>
> Glacially deposited, gap graded soils are present that are susceptible to the piping of the finer soil fraction from themselves and/or adjacent zones[7].
>
> In the western foothills of the Cascades the till ridges contain isolated, embedded, openwork gravel seams. These features pose seepage concerns and on occasion have resulted in piping failures[8].
>
> In much of southeast and south central Washington loess is present[9,10]. Loess is a weakly cemented, low density, fine grained soil deposit that potentially will collapse upon saturation.
>
> Alluvial valleys have heterogeneous foundation conditions that include buried pervious channels (stream meanders) and loose, saturated soils that are susceptible to strength loss upon earthquake shaking.

Rock foundations frequently are encountered at foundation level or at such shallow depths that the performance of the rock must be considered in the design. The following section discusses the principal rock conditions that have significant potential impacts on the structural integrity and performance of the impounding facility. These items are:

> Conglomerate and mudstone/siltstone rock foundations. These materials are highly competent in an undisturbed state but are susceptible to rapid deterioration under the combined influences of exposure to air, relieve of overburden pressure, standing water at the surface and overworking by the passage of construction traffic. On the abutments these weak rock materials are susceptible to extensive cracking as the surface is stripped of the overburden and allowed to dry. Considerable care is required to protect the material from deterioration. Typical measures to protect the formation are described in Chapter 7, Specifications.
>
> Sandstone Formations. These materials exhibit a number of problems. Typical of most rocks, the joints can pass large volumes of seepage. However, sandstones that are relatively clean and uniformly graded, medium to coarse can have significant primary permeabilities. Also, like mudstone formations, sandstone can be susceptible to rapid deterioration of the surface upon exposure to the elements. Again measures are often necessary to minimize the progressive degradation of the rock surface, such as capping the surface with gunite.
>
> Highly jointed basalt cap rock that is directly overlain by a thin veneer of fine grained plastic soils. The fine soil layer is susceptible to "blowing out" through the rock fractures under the action of seepage.
>
> Horizontally layered, Columbia River Basalt in Central and Southeast Washington. Basalt is typically encountered at or very near the ground surface in this region. The basalt was buildup in a series of flows with extended periods of time elapsing between flows. A vegetative cover often developed between successive flows that introduced impurities into the top and base of each flow. These impurities along with the extensive surface cracking arising from rapid cooling, produced a rock formation with high horizontal and vertical permeabilities along flow contacts and at shrinkage cracks, respectively. In addition, due to the layered nature of the rock formation, valley sidewalls have abrupt steps in the rock surface rather than a uniform grade. The stepped rock sidewalls pose significant rock surface preparation and sealing problems.

Explorations should not be considered as completed with the submission of the engineering reports. It is crucial that the individuals performing the initial site investigations continue to examine field conditions exposed during the course of construction. This provides an opportunity to confirm that field conditions are consistent with the understanding developed from the analysis of the boring program and that formed the basis of the design.

3.1.6 REFERENCE MATERIALS:

Table 2 summarizes the principal repositories for geologic and foundation studies covering Washington State.

TABLE 2 - GEOLOGIC INFORMATION REPOSITORIES

ITEM	SOURCE
Aerial Photographs	Department of Natural Resources (DNR) Photos and Map Sales (360) 902-1234
County Soil Surveys	Department of Agriculture Soil Conservation Service Field Office for the Appropriate County
Black and White Copies of the Soil Overlay Maps	DNR Photo and Map Sales (360) 902-1234
Text of Soil Survey	DNR Library (360) 902-1450
Geologic Maps	DNR Library
Geologic Reports General Publication Listings Statewide	
Publications of the Washington Division of Geology and Earth Resources, Washington Division of Geology and Earth Resources, December 1991	DNR Library (360) 902-1450
By Individual County	
(Summaries for 26 of 39 counties are available); e.g., Bibliography of the Geology and Mineral Resources of Jefferson County, Washington, Compiled by Manson, Connie J., Department of Natural Resources, December 1990.	DNR Library
Survey Document	
Engineering Geology in Washington, Galster, Richard (ed.), Washington Division of Geology and Earth Resources, Bulletin 78, 2 Volumes.	DNR Library
Well Logs	Regional Offices of the Department of Ecology

3.1.7 REFERENCES

1. US Army Corps of Engineers, Engineering and Design Manual - Geological Investigations, EM 1110-1-1801, Change 2, April 1978.

2. Design Standards No. 13 - Embankment Dams, Chapter 12 - Foundation and Earth Materials Investigation, U.S. Department of the Interior, Bureau of Reclamation, Denver, Colorado, October 1989.

3. Terzaghi, K., "Effect of Minor Geologic Details on the Safety of Dams", Geology and Engineering for Dams and Reservoirs, American Institute of Mining And Metallurgical Technical Publication 215, 1929, pp. 31-46.

4. Osterberg, J.O., "Failures in Exploration Programs", Dowding, C.H., ed., Site Characterization and Exploration, Proceedings - Specialty Workshop, Northwestern University, June 12-14, 1978, pp. 3-9.

5. Kiersch, G. and James, L.B., "Errors of Geologic Judgment and the Impact on Engineering Work", The Heritage of Engineering Geology: The First 100 Years, G. Kiersch, ed., Vol. 3, Geological Society of America, Boulder, Colorado, 1991, pp. 517-557.

6. Galster, Richard W. (Chairman), Engineering Geology in Washington, Washington Division of Geology and Earth Resources, Bulletin 78, Washington State Department of Natural Resources, 2 Volumes, 1989.

7. Sherard, J.L., "Sinkholes in Dams of Coarse Broadly Graded Soils", 13th ICOLD Congress, India, Vol. II, 1979, pp. 25-35.

8. Kiersch, G. and James, L.B., op. cit., p. 526.

9. Lobdell, G.T., "Hydroconsolidation Potential of Palouse Loess", ASCE Journal of the Geotechnical Engineering Division, (1981), Vol. 107, No. GT6, pp. 733-742.

10. Olson, B.O., "Loess Soils in Southeastern Washington", ASCE Journal of the Geotechnical Engineering Division, (1979), Vol. 105, No. GT6, pp. 786-791.

3.2 EMBANKMENT GEOMETRY AND ZONING

3.2.1 OBJECTIVE:

Provide an overview of the elements required for a typical earthen embankment cross-section.

3.2.2 REQUIREMENTS\MINIMUMS FOR THE EMBANKMENT CROSS-SECTION:

Table 1 presents a summary of the principal design elements of an embankment cross-section. The Table cities whether a particular element is recommended or required and identifies the appropriate section of the Guidelines where further details are provided. However, a number of embankment cross-section features are not discussed elsewhere in these guidelines. For such items, further clarification follows.

Abutment-Foundation Preparation - To minimize the potential for cracking where there are abrupt changes in slope geometry, the slopes should be laid back to a maximum slope of ¼H on 1V. Overhangs are normally removed or infilled with concrete to achieve the overall desired maximum slope. The need to regrade slopes is greatly diminished by the use of properly graded filters and drains. In the light of this, where regrading of the slope poses a particular hardship, an enlarged filter-drainage feature will be considered as an acceptable alternate to slope regrading. The designer is not required to regrade the abutment, downstream of the chimney drain that protects the core.

Where rock is exposed at the abutment and foundation, the surface should be scaled of any loose rock. Joints should be cleaned and filled with a plastic grout. Where the surface is highly jointed, consideration should be given to following the joint treatment by pouring a cover slab of concrete. The Corps[1] and Swiger[2] present excellent summaries of foundation preparation details.

A foundation grouting program is outside the scope of this discussion.

Cutoff Trench - Cutoffs should extend down to a suitable rock or low permeability soil layer. It is essential that the cutoff extend fully through any pervious overburden zones. Cedergren[3] demonstrated that partial cutoffs do not fully address exit gradient concerns. Where it is impractical to excavate in the dry to a suitable low permeability zone, soil-bentonite or cement-bentonite trenches have been successfully employed. The discussions regarding the use of cutoffs in alluvium would apply to only a small number of projects. Generally, it will be necessary to remove pervious alluvium from within the dam footprint due to seismic stability concerns. The few exceptions to removal would likely involve stormwater detention facilities where the likelihood of a liquefaction failure of the alluvium foundation at the same time the facility is impounding water would be extremely remote.

The minimum base width of cutoff trenches should be sized to allow the operation of self-propelled, heavy duty compactors. In most instances this will necessitate a minimum base width of 8 ft. The downstream side of the cutoff should be provided with a filter zone that is hydraulically connected to the drainage system. To minimize the potential for arching of the overlying embankment fill and to facilitate placement of the downstream filter, the cutoff sideslopes should be laid back on a maximum inclination of 1H on 1V.

Core Section - The DSO will accept a properly filtered core that is on the order of 25% of the head at any given elevation. This position is based on the satisfactory performance of thin core structures described by Sherard and Dunnigan[4]. Thinner cores will be considered if:

A core soil is available that can be appropriately compacted,

TABLE 1 - EMBANKMENT SECTION DESIGN ELEMENTS

ELEMENT		EARTHEN EMBANKMENTS Small	Medium	Large	GUIDELINE REFERENCE
FOUNDATION CUTOFF	Cutoff Trench	Req'd[A]	Req'd[A]	Req'd[A]	3.2
	Rock Contact Sealing	Req'd[B]	Req'd[B]	Req'd[B]	3.2
LOW LEVEL OUTLET PIPE		Req'd[C]	Req'd	Req'd	4.1
DRAINS	Chimney	Recom[D]	Req'd	Req'd	3.3
	Blanket	Recom	Req'd[E,F]	Req'd[E,F]	3.3
	Toe Drains	Recom	_____[G]	_____[G]	
FILTERS	Filter Criteria met <u>Everywhere</u>[H]	Req'd	Req'd	Req'd	3.3.A
	Pipes - Filter/Drain Diaphragm[I]	Req'd	Req'd	Req'd	3.3.B
SIDESLOPES		To be determined by engineering analysis[J]			2.2
FREEBOARD		To be determined by engineering analysis			4.6
CREST	Width	____[K]			
	Camber[L]	To be determined by engineering analysis			
EROSION PROTECTION	Upstream Face	Recom	Req'd	Req'd	3.5
	Downstream Face	Recom[M]	Req'd	Req'd	3.5
	Crest	Req'd	Req'd	Req'd	
	Downstream Groins	Recom[N]	Recom	Req'd	
INSTRUMENTATION	Reservoir Staff Gauge	Req'd	Req'd	Req'd	8
	Settlement Monuments	Recom[O]	Req'd	Req'd	8
	Piezometers	Recom	Recom	Recom	8
	Weirs/Pipe to Measure Seepage	Recom	Req'd	Req'd	8

[A] The subgrade should be stripped of any pervious surficial zone to found the cutoff on a suitable, low permeability zone.

[B] Any rock exposures should be sealed within the limits of the cutoff trench to prevent the loss of embankment materials into the foundations. Ideally, rock exposures upstream and downstream of the cutoff should be sealed and drained, respectively.

[C] For diked impoundments <10 feet high, the outlet may be omitted if it is practicable to lower the pool by syphoning.

[D] For small stormwater detention facilities, the DSO will consider eliminating the chimney drain where it can be demonstrated that neither excessive seepage is likely to develop in the abutments nor that the phreatic surface will penetrate a significant depth into the upstream surface.

[E] Where the foundations are relatively pervious, finger drains would provide an acceptable alternative to a blanket drain.

[F] The blanket drain should extend up the abutment. The extent of the abutment coverage should be a function of the magnitude and the nature of the seepage anticipated to emerge from the contact. Where isolated pervious features are indicated, separate finger drains should be considered within the blanket to safely carry off the concentrated flow and facilitate monitoring flow character.

[G] This would normally be a subelement of the blanket drain. Where the blanket drain has a large hydraulic capacity relative to the anticipated seepage flow <u>and</u> it is configured to efficiently shunt flow to one or a few points, a toe drain is not required.

[H] The zones should be internally stable in that they satisfy filter criteria for themselves. Plus, filter criteria must be satisfied at all zone contacts including the embankment interface with the foundation and abutments.

[I] Applicable to all conduits extending through or underlying the embankment cross section.

[J] For small dams, bearing on a competent foundation, constructed of a well compacted clay, the DSO will accept a design with 3H on 1V upstream and 2H on 1V downstream slopes without a supporting engineering analysis.

[K] The larger of 8 feet or the quantity $2(H)^{1/2} + 3$.

[L] Based on anticipated magnitude of crest settlement and conservatism in the level of freeboard provided.

[M] Determination based on ability to maintain a thick vegetative cover or erosion resistance of exposed soil.

[N] The need for erosion protection should be based on the volume of runoff and the erosion resistance of the groin area soils.

[O] Monuments should be provided where freeboard is a minimum and significant settlements are anticipated.

The core is suitably inclined to minimize the potential for hanging up on the shells, and

The minimum section will allow the operation of suitable heavy compaction equipment.

Where there is a concern of excessive seepage through the abutments and/or foundations and a thin core section is desired, the DSO will consider a design where the core section is flared at the base and abutment contact.

Details on the compaction of the core are presented in the specifications section of the Guidelines, Chapter 7.

Sherard and Dunnigan[4] present an excellent summary of good construction practice for zoned, earthen embankments. Their principal points are as follows: (Comments of the DSO are shown in italics.)

Cracking problems with the core are best addressed through the use of downstream filters.

Downstream filters satisfactorily address the consequences of core cracking that prompted the use of upstream sand layers as crack fillers. Consequently, the upstream sand filter may be omitted.

Clay cores have been provided with a series of progressively coarser zones (moving upstream) to prevent the migration of fines under a rapid drawdown of the pool. The low hydraulic gradients tending to erode the core into the upstream shell under such conditions are judged inadequate to cause significant migration of fines. Accordingly, it is recommended that a 6 inch minus quarried rock will serve as an adequate barrier to significant upstream loss of a clay core.

It is the authors' opinion that the central core cross-section is equally acceptable to an upstream inclined core section. *While this conclusion holds concerning cracking, the upstream, inclined core design offers significant benefits in areas of high seismicity such as in western and portions of central Washington. Here it is desirable to maintain as much of the dam section as practicable unsaturated to minimize strength loss upon cyclic loading. The excellent performance of concrete faced rockfill dams during earthquakes is a testament to the effectiveness of minimizing the saturated portion of the fill.*

Shells - The use of zoned embankments provides a practical means of making the best use of readily available materials. Typically it involves the incorporation of coarse rock fill into the embankment section that is derived from the spillway excavation. The broad range of particle sizes generally makes conventional density testing impractical. A procedural method of compaction is typically required for construction of these zones of the embankment. To determine an appropriate procedure, a test fill should be constructed to determine the appropriate number of coverages of a heavy duty, dynamic compactor for a typical lift thickness. Bertram[5] discusses the employment of a test fill for controlling the placement of coarse, broadly graded rock fill.

The shells generally contain an appreciable fraction of coarse rock. Provisions are typically made to rake out the largest rock fragments to the exterior face. This zone generally serves as a suitable erosion resistant, downstream facing.

3.2.3 REFERENCES:

1. Department of the Army, Corps of Engineers, Foundation Preparation, CW 02219, March 1981.

2. Swiger, W.F., "Preparation of Rock Foundations for Embankment Dams", in Hirschfeld, R.C., and Poulos, S.J., eds., Embankment Dam Engineering Casagrande Volume, 1973, pp. 355-364.

3. Ibid., Cedergren, H.R., "Seepage Control in Earth Dams", pp. 30-32.

4. Sherard, J.L. and Dunnigan, L.P., 1985, Filters and Leakage Control in Embankment Dams. In R.L. Volpe and W.E. Kelly (ed.), Seepage and Leakage from Dams and Impoundments. Proceeding of a Geotechnical Engineering Division symposium in Denver, Colorado, May 5, 1985. American Society of Civil Engineers. New York, N.Y., pp. 1-30.

5. Op. Cit., Hirschfeld, R.C., and Poulos, S.J., eds., Bertram, G.E., "Field Tests for Compacted Rockfill", pp. 1-20.

3.3 SEEPAGE CONTROL

3.3.1 INTRODUCTION:

Thirty-eight percent of dam failures over the period 1900 to 1975 have been attributed to piping and seepage[1]. From a balanced risk point of view, this aspect of design should be receiving the same level of scrutiny as the design flood and that of static and seismic slope stability. However, our experience in reviewing designs too often has been that embankment designs have ignored the seepage issue or have utilized unrealistic assumptions as to the magnitude of the problem and the efficiency of design features. In particular, filter criteria are routinely overlooked. Seepage volumes and phreatic surfaces are estimated from idealized homogeneous embankment sections that do not reasonably account for actual field conditions. Drain capacities are optimistically estimated without due attention paid to limiting the content and the plasticity of the minus No. 200 sieve fraction. The potential for concentrated seepage developing is not yet widely appreciated. The potential for cracks to develop in relatively impervious embankment zones, the foundation and abutments is not routinely recognized. Likewise, the use of filter-drainage diaphragms for controlling seepage along the perimeter of structures passing through embankments remains the exception, not the rule.

Satisfactorily addressing seepage concerns is not simply a matter of constructing a suitable impervious zone along with adequate drainage features. One must address filter issues, embankment zoning, foundation geometry among other factors. The generalized concerns associated with seepage are considered in this section. The Filter section deals with preventing of piping of fines from individual zones under the action of seepage. Finally, the Filter/Drain Diaphragm and Underdrains sections discuss the treatment of both concentrated and diffuse seepage.

Requirements for the inclusion of specific embankment features to address seepage concerns are presented in Sections 3.2 and 3.3A, B, and C.

3.3.2 OBJECTIVES:

Control the portion of the embankment and abutment that will become saturated.

Force significant changes in hydraulic gradient; i.e., maximum rate of head loss and/or flow direction to occur where it is most desirable.

Limit seepage volumes.

3.3.3 OPTIMIZING DESIGN:

To the extent practicable, position drains to maximize the effective stresses in the embankment and foundation.

Configure seepage outlets to facilitate measuring flow volumes and character.

3.3.4 DESIGN PHILOSOPHY:

We espouse the position taken by Dr. DeMello in his 17th Rankine lecture[2]. Dr. DeMello argued that the designer should take steps to "transform" problems from one of dealing with extremes to one of dealing with averages. This typically involves incorporating some design feature that **intercepts and blunts** or **smoothes out** the effects of extreme loadings. In effect, the designer is bringing the actual dam closer to the idealized, simplified structure he modelled.

3.3.5 DESIGN CONSIDERATIONS:

The goal of seepage control is to **prevent the development** or **mitigate the impact** of the following:

Cracks in the low permeability section of the embankment,

Cracks at the embankment contact with appurtenance works, the abutments and the foundation, and

Excessive seepage beneath or around the dam through the abutments or the foundation. (Excessive seepage here is considered that rate which leads to instability of the zone; i.e., piping, slides or boiling.)

The principal problems that arise from a failure to satisfactorily address these issues are:

Cracks potentially can expand through the erosion of their sidewalls. In extreme cases, the cracks can develop into a "pipe". If the pipe only extends partially through the embankment, it may introduce a volume of seepage into the downstream zone that induces a slide. If the pipe extends through the majority of the embankment, it may eventually "blow out" the upstream "plug" of the pipe, forming a continuous channel through the dam. The concentrated flow through the channel can then rapidly enlarge the channel and fail the dam.

Relatively stable cracks may develop that are resistant to rapid erosion of their sidewalls. These features can introduce near full reservoir pressure heads along seams which are hydraulically connected to the crack. This has prompted concern as to the overall stability of the downstream portion of the embankment.

Highly pervious zones, particularly "open work" clean gravelly seams undetected in the abutments and foundations, can pass large volumes of seepage for considerable distances with little head loss. The discharge area for these zones on the abutment typically experience shallow slides. On rare occasions, the flow is large enough to "blow out" the hillside or cause "boiling" of foundation soils.

3.3.6 ENGINEERING PRACTICE

Consideration of seepage control should be factored into all aspects of the project's investigation, design, construction and monitoring!

Exploration Phase - In this stage of the project, the goal is to identify potential "problems" the design must address. The explorations of near surface conditions should include a series of test pits. The pit sidewalls afford probably the best opportunity to view subsurface conditions prior to opening up the area during construction.

The field investigator should be alert to conditions that pose seepage and/or embankment cracking concerns. Features that potentially have a significant impact on these problems are as follows:

Seepage

"Channels" or seams of clean, uniformly graded, coarse sands to "open work" gravels.

Open jointed rock or pervious joints between rock flows, such as are typically found in the

Columbia River Basalt flows of eastern Washington.

Buried stream meanders that can pass large seepage volumes and potentially pose piping concerns. They are generally not encountered in the explorations but it is prudent to assume that broad, relatively flat valley bottoms probably contain such features.

Cracking

Steep and/or abruptly stepped rock surfaces in the abutment or foundation.

Highly compressible soil strata in the foundations.

Stress relief cracking deeper in the abutments. This is of particular concern where the abutment is a narrow, steeply sided ridge or the abutment contains a deep ravine proximate to the dam.

In most cases, it is prudent to remain skeptical that all the significant seepage problems have been revealed by the exploration program.

Design Phase - Designers should always bear in mind two thoughts. First, they should look to actively constrain the seepage regime to some desired configuration. "Actively constrain" consists of providing measures that block or intercept seepage such as cutoff trenches and blanket and chimney drains. This contrasts with the more passive approach where drainage features are provided with the expectation that they will "influence" seepage gradients at a distance. As an example of the "passive approach", a toe or horizontal blanket drain is often provided with the expectation that it will act as a "sink", whose effect propagates through the embankment pulling down the phreatic surface. This approach frequently is at odds with actual field behavior. Second, designers should bear in mind the limitations of the analyses they are using. Stated conversely, they should clearly recognize that small singularities in field performance **not accounted for in the analysis** may control the overall field response of the element modelled. For example, finite element seepage analyses are available that will reflect the impact of complex zoning with differing horizontal and vertical permeabilities. But, these models have no practical way of realistically predicting and accounting for the potential opening and widening of cracks. The designer must recognize the need to address potential cracking problems based on case histories. Once the problem of cracking is appropriately treated, finite element seepage models provide a valuable tool in predicting average pore pressures and seepage volumes associated with complex embankment zoning.

As previously noted, cracking and excessive seepage are the principal issues of concern. The design should incorporate means of minimizing the likelihood of cracks occurring. This involves treating or avoiding conditions likely to produce cracking such as:

Appropriately shaping the embankment contact with the foundation and abutments to remove abrupt changes in grade.

Avoiding symmetrical, zoned embankment cross-sections. The relatively compressible core can "hang up" on the stiffer upstream and downstream shells. A significant fraction of the overburden pressure in the core can be transferred through beam action to the stiffer shell zones. This reduces the confining stress at greater depths in the core. In extreme cases it can lead to hydraulic fracturing. It is preferable to incline the axis of the core zone.

Removing highly compressible zones from within the dam footprint. Constructing the embankment over soft, compressible zones in the foundation or abutments increases the potential for large differential strains to develop proximate to the edge of soft zones or around abrupt changes in the thickness of such zones. These large strains are often associated with cracking.

Restricting the water content of the core materials during compaction to optimum or wet of optimum moisture content. Compaction on the dry side of optimum moisture produces a more "brittle" soil structure. This structure has an increased tendency to crack in undergoing settlements and in changes in stress in service.

In addition, it should be recognized that even under the best of practice, cracks can still occur[3]. The design needs to include measures to blunt the impact of potential cracks. This consist principally of providing chimney and blanket drains. The chimney drain is an inclined zone or zones of graded material that safely conducts the seepage emerging through the low permeability section of the embankment through the dam. Where necessary, a filter zone is provided to satisfy filter criteria for the upstream zone. The chimney drain functions in two ways to blunt the effect of cracks. First, the drain restricts the seepage volume through the crack to that of the upstream element of the chimney drain. Second, fines eroded from the crack sidewalls are carried to the contact between the low permeability section of the embankment and the chimney drain. These eroded fines in test simulations of the process, form a low permeability "cake" at the contact[4]. This dramatically reduces the flow rates and allows the crack side walls to swell and pinch off the crack. The chimney drain should extend down to the surface of a foundation layer that forms a practical low permeability cutoff to significant underseepage.

To the extent practicable, measures to minimize the volume of seepage through the foundation and abutments should be employed. This typically involves sealing rock exposures at the foundation and abutment contact and construction of a cutoff through any pervious soil strata of the foundation and abutments. Typical practice in sealing rock surfaces involves using compressed air equipment to blow off any loose rock fragments and to clean out the upper portion of rock cracks. Some hand cleaning is invariably necessary. Larger cracks are then filled, generally by hand packing them with a sand-cement mixture. This is followed by brooming a sand-cement mix over the rock surface to seal fine cracks. The purpose of the foundation treatment is to prevent the piping of the embankment soils into fractures in the rock and to minimize the volume of seepage passing through cracks. Measures to reduce seepage within the foundations such as grouting are beyond the scope of these guidelines.

Major dams have been constructed where the dam is supported on relatively pervious foundations. Over the life of many of these projects elaborate systems of relief wells, upstream blankets and diaphragm walls have been added to address stability concerns. In recognition of the problems these dams have experienced, the DSO normally requires the construction of cutoffs for all dams bearing in pervious strata. However, particularly in the abutments suitable low permeability zones may not be located within distances that make it practical to establish a cutoff. Where this is the case the designer has two principal options. First, a conventional low permeability zone can be constructed back into the abutments to take advantage of the reduction in seepage forces due to the increased seepage path. Alternatively, a highly pervious zone can be constructed back into the abutments that is hydraulically connected to the internal drainage system. This scheme relies on redirecting seepage in the abutments or foundation into the embankment drains. The redirection area is selected based on its ability to withstand the anticipated seepage forces. The redirection area normally lies near the center of the embankment cross-section where the vertical and lateral confining stresses acting in the soils are greatest. This is preferable to allowing seepage under potentially relatively high exit gradients to emerge out of the abutment or foundation near the downstream toe. Here, in extreme cases the seepage forces may exceed the effective soil stresses resisting movement. In such cases, a sidehill blow out or a boiling type condition may develop in the foundations.

Horizontal blanket drains or finger drains are routinely provided to augment the chimney drain in controlling seepage. The two principal conditions warranting their use arise from the need to treat seepage emerging from the abutments or foundations. First, the chimney drain is not as effective at controlling seepage moving through the abutments as it is through the embankment cross-section. This is the case often because seepage emerging from the abutments is only partly originating from the reservoir. A significant portion of the seepage within the downstream portion of the embankment may include groundwater flow emerging from deep within the abutment. The chimney drain normally has only minimal impact on such seepage. Second, it is prudent to augment the chimney drain when the downstream portion of the dam is founded on alluvium and there is potential for significant seepage passing beneath the cutoff and feeding the alluvial foundation.

Dams frequently have lives approaching a hundred years. This lengthy service life poses long term durability issues. Although recent advances in plastics technology have greatly improved the performance of these materials, we strongly favor the use of granular materials over plastic or metal pipes as drains in the central portion of an embankment section. Ideally, metal and plastic pipes associated with seepage control elements are used in applications where they can be readily replaced and their failure would not likely have a serious impact on the overall integrity of the impounding barrier.

Construction Specifications and Construction Control Program - As noted in the exploration section, the excavations for the dam footprint particularly for seepage control measures often provide the deepest, continuous exposures of foundation and abutment conditions. The designer should take advantage of this opportunity to confirm the appropriateness of the design assumptions. To do so, in addition to describing the work, the specifications should include a description of anticipated foundation and abutment conditions. Ideally, the specifications should go on to define what constitutes unsuitable conditions. This helps field personnel recognize when unanticipated conditions are present, increases the likelihood that unsuitable materials will be removed and lessens the opportunities for the contractor to claim changed conditions. This should be reinforced in the development of the construction control plan, see Part II of the guidelines.

3.3.7 REFERENCES:

1. Mantei, C. Leo, 1985, Seepage Control for Embankment Dams USBR Practice, In R.L. Volpe and W.E. Kelly (eds.), Seepage and Leakage from Dams and Impoundments. Proceedings of a Geotechnical Engineering Division symposium in Denver, Co., May 5, 1985, American Society of Engineers, New York, N.Y., p. 229.

2. DeMello, V.F.B., "Reflections on Design Decisions of Practical Significance to Embankment Dams", Geotechnique, Vol. 27, No. 3, 1977, p. 293.

3. Kulwahy, F.H., and Gurtowski, T.M., "Load Transfer and Hydraulic Fracturing in Zoned Dams", Journal of the Geotechnical Engineering Division, ASCE, Vol. 102, GT9, Proc. Paper 12400, 1976, pp. 963-974.

4. Sherard, J.L., Dunnigan, L.P. and Talbot, J.R., 1984, Filters for Silts and Clays. American Society of Civil Engineers. Journal of Geotechnical Engineering 110(6) June 1984, p. 707.

3.3.A FILTERS

3.3.A.1 OBJECTIVE:

Minimize the loss of soil particles, particularly the "fine" soil fraction, from the embankment and foundation.

3.3.A.2 DESIGN PHILOSOPHY:

Filters are considered as "insurance" that the various design features of which they form a sub-element, will maintain their integrity and function as intended.

3.3.A.3 OPTIMIZING DESIGN:

The filters gradation should be selected so as to provide the greatest hydraulic capacity practicable.

3.3.A.4 APPLICABILITY:

The provisions of this section apply to those cases where the filters need only have a permeability equal to or marginally greater than the base soil. Essentially, it applies in those cases where the filter is not required to materially affect the average hydraulic gradient in the adjacent layers. For example, these criteria satisfactorily define a suitable upstream filter for a chimney drain. The highly pervious chimney drain provides the necessary hydraulic capacity to discharge the volume of seepage. The filter serves only to prevent the piping of fines and to cap off any concentrated leaks. The stability of the dam would not be materially changed if the permeability of the filter match that of the adjacent upstream layer. Conversely, the following filter criteria may not be adequate for a drainage blanket that encapsulates highly pervious finger drains beneath the downstream embankment section. Here it is important to have much higher permeabilities than the base soil. This is necessary to facilitate the movement of seepage to the pervious finger drains. For broadly graded filters with a maximum allowable fines content of 5%, the filter zone may severely restrict flow to the pervious finger drains. Thus, higher hydraulic gradients will persist further downstream in the base soil and the efficiency of the finger drains is reduced.

Filter Criteria Background - The Soil Conservation Service (SCS) conducted extensive research into the issue of filter criteria in the 1980's. The results of this work were first broadly disseminated through the Journal of Geotechnical Engineering in 1984[1,2]. Additional refinements were made on the test procedure and the current methodology was first presented in 1985[3]. The work was subsequently organized into a design note[4] by the SCS. The Bureau of Reclamation essentially adopted the SCS approach in their Design Standards[5].

The Dam Safety Office uses the SCS approach in the evaluation of proposed filter designs. The SCS procedure for selecting filter criteria follows. Occasional comments are inserted to address specialized issues that were not included in the SCS Note. These comments appear in italics to delineate them as our opinion and not necessarily shared by the SCS.

SCS SOIL MECHANICS NOTE NO. 1, 210-VI, GUIDE FOR DETERMINING THE GRADATION OF SAND AND GRAVEL FILTERS, REVISED, JANUARY 1986.

I. Purpose

This note presents criteria for determining the grain-size distribution (gradation) of sand and gravel filters needed to prevent internal erosion or piping of soil in embankments or foundations of hydraulic structures.

These criteria are results of an extensive laboratory filter study carried out by the soil conservation service at the Soil Mechanics Laboratory in Lincoln, Nebraska, during the period 1980-1985. [1,2,3,6]

II. Definitions

Base Soil - Any soil through which water moves into a filter or drainage system.

d_{15}, d_{85} and d_{100} sizes - Particle sizes (mm) corresponding respectively to 15, 85 and 100 percent finer by dry weight from the gradation curve of the base soil.

D_5, D_{10}, D_{15}, D_{60}, D_{85}, and D_{100} sizes - Particle sizes (mm) corresponding to the 5, 10, 15, 60, 85, and 100 percent finer by dry weight from the gradation curve of the filter.

Gradation curve (grain-size distribution) - Plot of the distribution of particle sizes in a base soil or material used for filters or drains.

Drain - a designed pervious zone, layer, or other feature used to reduce seepage pressures and carry water.

Filter - Sand or sand and gravel having a gradation designed to prevent movement of a base soil by flowing water. Fabrics or other filter materials are not included in this note.

Fines - That portion of a soil finer than a No. 200 (0.075 mm) U.S. Standard sieve.

Soil category - One of four types of base soil materials based on the percentage finer than the No. 200 (0.075 mm) U.S. Standard sieve.

III. Basic Purpose of Filters and Drains

Filters are placed in embankment zones, foundations or other areas of hydraulic structures for two purposes:

1. To intercept cracks or openings in a base soil to prevent the erosion of soil particles by water passing through the openings. The filter is graded so that soil particles cannot pass through the filter voids. They are caught at the filter face, preventing further erosion and concentrated flow through cracks or openings.

2. To intercept seepage passing through the pores of the soil, thereby preventing the movement of soil particles at the discharge point (piping). Piping occurs when seepage gradients or pressures are high enough to produce erosive discharge velocities in the base soil. The filter zone is usually placed upstream of the discharge point where sufficient confinement prevents uplift or blow-off of the filter.

Drains consist of sand, sand and gravel, or gravel mixtures placed in embankments, foundations, and backfill of hydraulic structures, or in other locations to reduce seepage pressure. A drain's most important design feature is its capacity to reduce seepage pressures and carry collected water to a safe outlet. Drains are often used downstream of or in addition to a filter to provide outlet capacity.

IV. Permeability and Capacity

The laboratory filter study clearly demonstrated that a graded filter designed in accordance with the criteria contained in this note will seal a crack. The sealing begins whenever water runs through a crack or opening and carries soil particles to the filter face or causes filling or closing of the crack. Any subsequent flow is through the pores of the soil. Therefore, when filters are designed to intercept cracks, the permeability used to determine drain capacity is computed for saturated steady state flow through the pores of the base soil material.

Where it can be demonstrated that saturated steady state flow will not develop (i.e., dry dams having a normal drawdown within 10 days), capacity is not a necessary design requirement. Filters designed to protect against leakage and erosion in cracks are to have a thickness that compensates for the negative effects of material segregation and contamination during construction and ensures continuity (will not sustain a crack) during differential movements.

A drain of coarser materials immediately downstream of the filter or a perforated pipe in the filter is needed if seepage through the pores of the base soil material exceeds the capacity of the filter. The coarser materials must be properly graded using filter criteria in this note to prevent movement of the filter. Perforated pipes may also be used in the coarser materials to increase the capacity of the drain.

V. Determining Filter Gradation Limits

Determine filter gradation limits using the following steps:

1. Determine the gradation curve (grain-size distribution) of the base soil material. Use enough samples to define the range of grain-size for the base soil or soils and design the filter gradation based on the base soil that requires the smallest D_{15} size.

2. Proceed to step 4 if the base soil contains no gravel (material larger than No.4 sieve).

3. Prepare adjusted gradation curves for soils with particles larger than the No. 4 (4.75 mm) sieve:

   a. Obtain a correction factor by dividing 100 by the percent passing the No. 4 (4.75 mm) sieve size.

   b. Multiply the percentage passing each sieve size of the base soil smaller than No. 4 (4.75 mm) by the correction factor from step 3a.

   c. Plot these adjusted percentages to obtain a new gradation curve.

   d. Use the adjusted curve to determine the percent passing the No. 200 (0.075 mm) sieve in step 4.

4. Place the base soil in a category based on the percent passing the No. 200 (0.075 mm) sieve in accordance with Table 1.

5. Determine the maximum D_{15} size for the filter in accordance with Table 2. Note that the maximum D_{15} is not required to be smaller than 0.20 mm.

TABLE 1 - CATEGORIES OF BASE SOIL MATERIALS

Category	Percent finer than the No. 200 (0.075 mm) sieve
1	> 85
2	40-85
3	15-39
4	< 15

TABLE 2 - CRITERIA FOR FILTERS

Base soil category	Base soil description, and percent finer than No. 200 (0.075mm) sieve 1/	Filter criteria 2/
1	Fine silts and clays; more than 85% finer.	3/ $D_{15} \leq 9 \times d_{85}$
2	Sands, silts, clays, and silty and clayey sands; 40 to 85% finer.	$D_{15} \leq 0.7$ mm
3	Silty and clayey sands and gravels; 15 to 39% finer.	4,5/ $D_{15} \leq \frac{40 - A}{40 - 15} (4 \times d_{85} - 0.7\text{mm}) + 0.7\text{mm}$
4	Sands and gravels; less than 15% finer.	6/ $D_{15} \leq 4 \times d_{85}$

1/ Category designation for soil containing particles larger then 4.75 mm is determined from a gradation curve of the base soil which has been adjusted to 100% passing the No. 4 (4.75 mm) sieve.

2/ Filters are to have a maximum particle size of 3 inches (75 mm) and a maximum of 5% passing the No. 200 (0.075 mm) sieve (*as determined by wet sieving ASTM C-117-80*) with the plasticity index (PI) of the fines equal to zero. PI is determined on the material passing the No. 40 (0.425 mm) sieve in accordance with ASTM-D-4318. To ensure sufficient permeability, filters are to have a D_{15} size equal to or greater than 4 x d_{15} but no smaller than 0.1 mm.

3/ When 9 x d_{85} is less than 0.2 mm, use 0.2 mm.

4/ A = percent passing the No. 200 (0.075 mm) sieve after any regrading.

5/ When 4 x d_{85} is less than 0.7 mm, use 0.7 mm.

6/ In category 4, the d_{85} may be determined from the original gradation curve of the base soil without adjustments for particles larger than 4.75 mm.

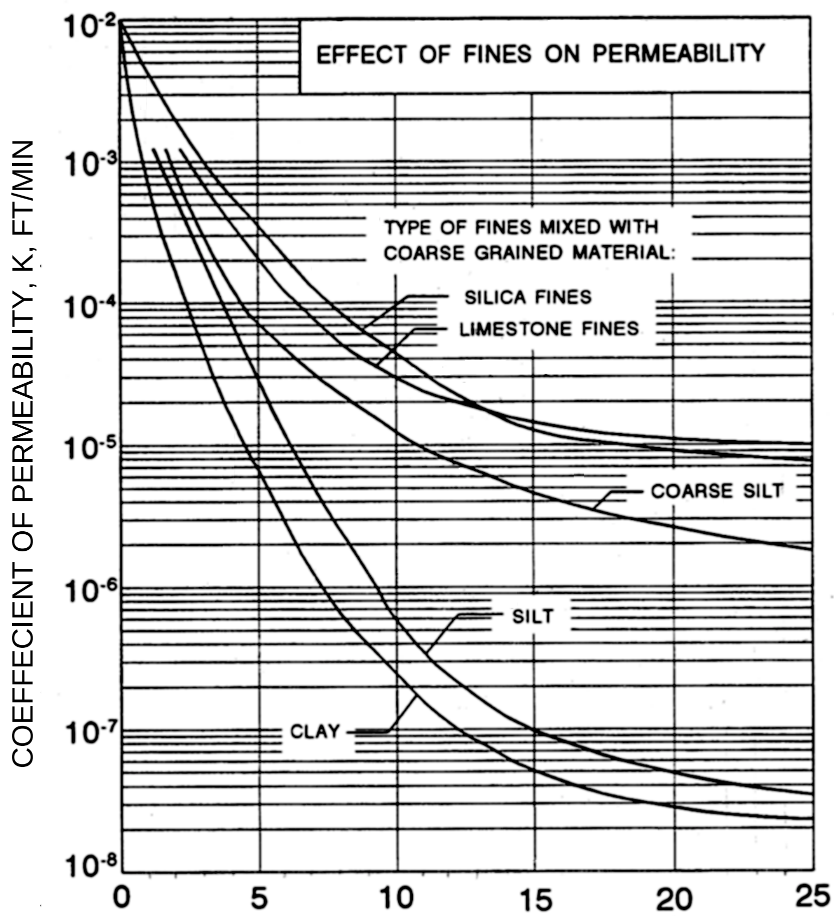
6. To ensure sufficient permeability, set the minimum D_{15} greater than or equal to 4 x d_{15} of the base soil but no less than 0.1 mm.

COMMENTARY

A principal difference between the SCS and Bureau versions of their filter design guidelines is over the issue of permeability. The Bureau identifies the criteria as being applicable to "zones or elements of drainage systems where filtering is the prime need and pore pressure or head buildup is not likely to be of any consequence"[7]. The SCS does not explicitly discuss the applicability issue with regard to hydraulic capacity. Instead, Step 6 simply requires that D_{15} be greater than 4 x d_{15} to "ensure sufficient permeability".

It must be remembered that permeability is a function of a number of factors including density, grain size distribution, particle shape and the percentage of fines. Considering the effect simply of fines, Figure 1 demonstrates that the allowance of as much as 5% (Step 7) can be large. The Figure predicts the permeability of a washed, fine to medium sand may be as much as a factor of 100 greater than a similar sand with 5% silt fines. If one were to use the approximate relationship that K is proportional to the square of the d_{15} particle diameter[8], the requirement of $D_{15} \leq 4\ d_{15}$ would as a minimum increase permeability roughly 16 times. As a practical matter, it is unlikely that D_{15}/d_{15} would be less than 10. Thus, 5% fines in the filter conceivably could reduce the filter permeability roughly to that of the foundation. If the other variables influencing filter permeability are considered, there is the potential for further reductions or increases in the filter permeability.

FIGURE 1 - EFFECTS OF 200 MINUS FRACTION ON PERMEABILITY



PERCENT BY WEIGHT PASSING NO. 200 SIEVE

SOURCE: U.S. Naval Design Manual - Soil Mechanics, Foundations and Earth Structures, NAVFAC DM-7, U.S. Naval Publications, Philadelphia, October 1971, pg. 7-8-10.

Therefore, it is appropriate in our opinion to perform permeability testing of filters, if they are expected *to be free draining relative to the base soil. This issue is discussed further in the seepage section of this chapter.*

7. Set the maximum particle size at 3 in. (75 mm) and the maximum passing the No. 200 (0.075 mm) sieve at 5 percent. The portion of the filter material passing the No. 40 (0.425 mm) sieve must have plasticity index (PI) of zero when tested in accordance with ASTM D-4318.

8. Design the filter limits within the maximum and minimum values determined in steps 5, 6, and 7. Standard gradations may be used if desired. Plot the limit values on Form SCS ENG 130 (*Grain Size Classification Sheet*) and connect all the minimum and maximum points with straight lines. To minimize segregation and related effects, filters should have relatively uniform grain-size distribution curves, without "gap grading" - sharp breaks in curvature indicating absence of certain particle sizes. This may require setting limits that reduce the broadness of filters within the maximum and minimum values determined. Sand filters with D_{90} less than about 20 mm generally do not need limitations on filter broadness to prevent segregation. For coarser filters and gravel zones that serve both as filters and drains, the ratio D_{90}/D_{10} should decrease rapidly with increasing D_{10} size. The limits in Table 3 are suggested for preventing segregation during construction of these coarser filters.

TABLE 3 - D_{10} AND D_{90} LIMITS FOR PREVENTING SEGREGATION

Minimum D_{10} (mm)	Maximum D_{90} (mm)
<0.5	20
0.5 - 1.0	25
1.0 - 2.0	30
2.0 - 5.0	40
5.0 - 10	50
10 - 50	60

9. Design filters adjacent to perforated pipe to have a D_{85} size no smaller than the perforation diameter. For critical structure drains where rapid gradient reversal (surging) is probable, it is recommended that the D_{15} size of the material surrounding the pipe be no smaller than the perforation size.

3.3.A.5 REFERENCES:

1. Sherard, J.L., Dunnigan, L.P. and Talbot, J.R., 1984, Filters for Silts and Clays. American Society of Civil Engineers. Journal of Geotechnical Engineering 110(6) June 1984: p. 701-718.

2. Sherard, J.L., Dunnigan, L.P. and Talbot, J.R., 1984, Basic Properties of Sand and Gravel Filters. American Society of Civil Engineers, Journal of Geotechnical Engineering, Vol. 110, No. 6, June 1984: p. 684-700.

3. Sherard, J.L. and Dunnigan, L.P., 1985, Filters and Leakage Control in Embankment Dams. In R.L. Volpe and W.E. Kelly (ed.), Seepage and Leakage from Dams and Impoundments. Proceeding of a Geotechnical Engineering Division symposium in Denver, Colorado, May 5, 1985. American Society of Civil Engineers. New York, N.Y., p. 1-30.

4. U.S. Dept. of Agriculture, Soil Conservation Service Engineering Division, Soil Mechanics Note No. 1 210-VI, "Guide for Determining the Gradation of Sand & Gravel Filters", revised January 1986.

5. U.S. Dept. of the Interior, Bureau of Reclamation, Engineering and Research Center, "Design Standards, No. 13, Embankment Dams", Chapter 5, Protective Filters, May 1987.

6. Talbot, J.R. and Ralston, D.C., 1985, Earth Dam Seepage Control, SCS Experience. In R.L. Volpe and W.E. Kelly (ed.), Seepage and Leakage from Dams and Impoundments. Proceedings of a Geotechnical Engineering Division Symposium in Denver, Colorado, May 5, 1985. American Society of Civil Engineers. New York, N.Y., p. 44-65.

7. Op cit., U.S. Dept. of the Interior, Section 5.7 (General Criteria).

8. Op cit., U.S. Dept. of the Interior, Section 5.8 (General Criteria).

3.3.B CONDUIT SEEPAGE CONTROL - FILTER-DRAIN DIAPHRAGMS

3.3.B.1 OBJECTIVE:
Provide a reliable, low cost, easily constructed, measure to address seepage and piping concerns along conduits.

3.3.B.2 REQUIREMENT:
All low level, outlet conduits embedded within the soil phase of the embankment or foundation shall be provided with filter-drain diaphragms.

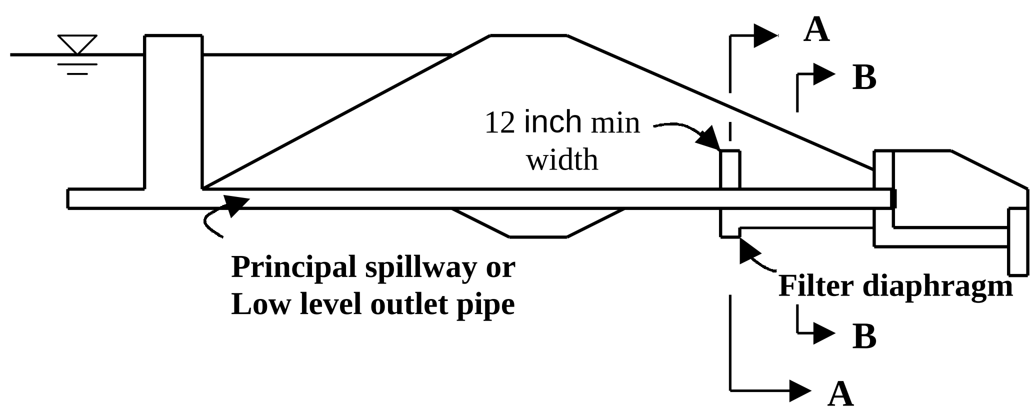
3.3.B.3 ENGINEERING CONCERNS:
Seepage along conduits pose two principal problems. The first class of problems arise from wetting of the soils. The seepage saturates a portion of the embankment around the pipe, increasing forces tending to cause movement while reducing the resistance of the soils to such movement. Typically, this seepage produces only shallow surficial slides in the immediate area of the pipe outfall. The seepage is more of a nuisance than a significant threat to the integrity of the dam. The water frequently ponds around the pipe outfall, where it fosters the growth of a thick vegetative zone. This vegetation inhibits inspection and attracts burrowing animals. Infrequently, the volume of seepage can precipitate deeper slides that require prompt attention to prevent further sliding or "piping" that could conceivably breach the impoundment.

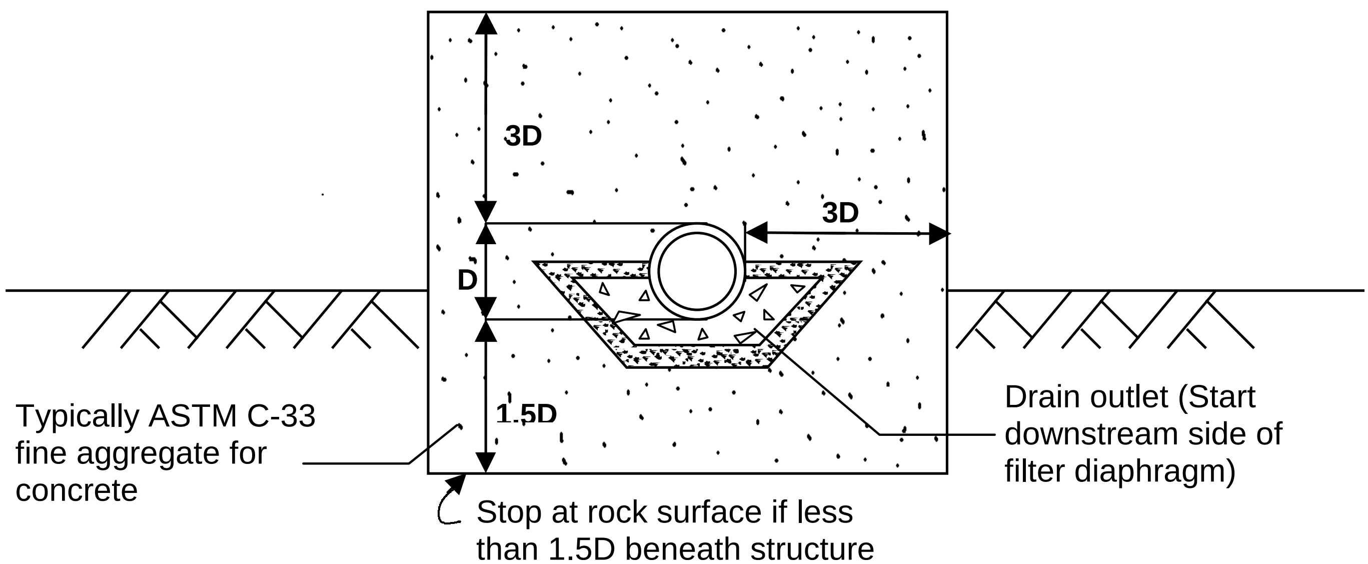
The second class of problems is associated with seepage eroding soils supporting the pipe. The loss of subgrade support for the pipe can cause joints to open or damage seals between pipe sections. This concern underlies the DSO requirement that conduits be encased in concrete when the dam height exceeds 15 feet. This policy effectively minimizes this problem as a threat to dam integrity.

3.3.B.4 DESIGN APPROACH:
Past Engineering Practice - A series of metal or concrete collars were placed at intervals around the pipe. Their purpose was to lengthen the seepage path water must follow as it moves along the exterior of the pipe. The "rule of thumb" was to provide a sufficient number of collars to increase the seepage path by 20 to 30% over the actual pipe length.[1] There has been a growing recognition that this approach was only marginally successful at addressing the problem in many cases. The poor performance of collars is believed due in a large measure to the low level of compaction achieved along the pipe. The collars limited the segments of the pipe wherein it was feasible to operate highly efficient, self-propelled compactors up to the sides of the pipe. Much of the compaction had to be accomplished with less effective, hand operated compactors that could get into narrow confined areas.

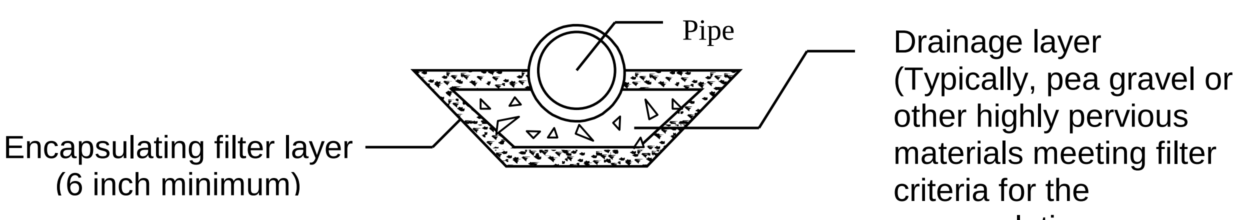
Recommended Practice - The designer should take all reasonable precautions to minimize seepage along conduits. This normally involves foregoing the cutoff collars. Instead, low permeable bedding and backfill zones are provided along the reach of the pipe passing through the low permeability zone of the dam. Where the pipe will be concrete encased, the sidewalls of the pipe encasement should be battered inward to facilitate operating heavy compaction equipment up against the encasement. Having minimized seepage, the designer then needs to provide a suitable feature to intercept any seepage that bypasses upstream control measures. In our experience this can effectively be accomplished by providing a filter-drain diaphragm.

FIGURE 1 - FILTER DIAPHRAGM SCHEME





Section A-A



Section B-B

Note: Figure prepared with minor modifications from conceptual details in Talbot and Ralston[2].

The filter-drain diaphragm scheme was developed by the Soil Conservation Service (SCS). The principle elements of the SCS scheme are shown in Figure 1. This figure is a derivative of a drawing included in an article by James Talbot and David Ralston[2] of the SCS. The diaphragm consists of a zone of sand and gravel orientated perpendicular to the pipe. The sand and gravel gradition is selected to satisfy filter criteria for the soils immediately upstream. A pervious drain is normally provided beneath the pipe from the downstream side of the diaphragm to the discharge point at the embankment toe. The pervious drain functions to both provide a pipe bedding zone and to carry off any seepage. The pervious drain normally has to be fully encapsulated with a graded filter to minimize the intrusion of fines into the drain.

Where the dam cross-section includes a chimney drain, this feature should functions as a suitable diaphragm. In such cases it is then only necessary to provide a suitable bedding layer and drain for the pipe sections downstream of the chimney drain.

The success of the diaphragm is attributed to number of factors. First, it permits the operation of efficient self-propelled compactors adjacent to the pipe. Second, in the event there is significant seepage along the pipe, the flow generally carries entrained fines plucked from the soil matrix. These fines come to rest on the upstream side of the diaphragm where they form a low permeability "cake". This "cake" then controls the rate of further seepage, dramatically reducing flow volumes.

3.3.B.5 REFERENCES:

1. Bureau of Reclamation, Design of Small Dams, 3d Edition, Department of the Interior, p. 474, 1987.

2. Talbot, J.R. and Ralston, D.C., 1985, Earth Dam Seepage Control, SCS Experience, In R.L. Volpe and W.E. Kelly (ed.), Seepage and Leakage from Dams and Impoundments, Proceedings of a Geotechnical Engineering Division Symposium in Denver, Colorado, May 5, 1985. American Society of Civil Engineers. New York, N.Y., p. 59.

3.3.C CONCRETE LINED SPILLWAY SEEPAGE CONTROL

3.3.C.1 OBJECTIVE:
Minimize the buildup of water pressures behind the concrete walls and floor of the spillway.

3.3.C.2 DESIGN APPROACH:
Four principal avenues of seepage have to be considered. These avenues are:

Reservoir seepage moving through the embankment,

Seepage from the abutment that may include a major component that is reservoir driven,

Surface runoff infiltration, and

Leakage from flow in the spillway channel.

The method of addressing these sources of seepage fall into two broad categories, blocking or restricting the passage of seepage and intercepting and draining away seepage. The following presents the typical approaches to addressing the problem.

SEEPAGE SOURCE	ENGINEERING TREATMENT
Reservoir	Minimize the volume of seepage by casting the spillway floor directly against the low permeability (core) section of the embankment. Use soils of equal or lesser permeability than that of the adjacent soils when backfilling the sidewalls of the spillway within and upstream of the limits of the core section. If practicable, extend the chimney drain up to the spillway floor and partially up the sidewalls to intercept and carry off any seepage moving along the soil-concrete contact.
Abutments	The following steps are normally considered part of the effort to treat seepage at the embankment contact with the abutments. Yet, they serve to materially reduce seepage concerns with the spillway. **Low Permeability Zone at Embankment-Abutment Contact -** If practicable, pervious features should be removed and backfilled with a lean concrete or grout. If impractical, such features should be capped with grout. **Embankment-Abutment Contact Downstream of the Core -** Intercept and divert seepage at the embankment-abutment contact. The type of drainage measures should be a function of the anticipated magnitude and pattern of seepage emerging from the abutment. Where the abutment has a low seepage potential, such as a rock with tight joints or a clay or siltstone, the chimney drain likely will suffice. When isolated pervious features are present in an otherwise relatively impervious abutment, finger drains should relieve the concentrated

	seeps. Finally, if the abutment has a complicated network of pervious features or is simply, uniformly pervious, a blanket drain should be constructed over the embankment-abutment contact.
Surface Water Infiltration	The ground surface should be contoured to facilitate runoff. A low permeable cap should be placed to minimize infiltration down the concrete-soil contact of the spillway walls. Where considerable surface runoff from adjacent areas will flow unto the dam, a paved ditch or other system should be provided to intercept and safely conduct the flow down the groin.
Flow in the Spillway	Water stops must be provided in all horizontal joints and in vertical joints to the top of the wetted section of the channel.

3.3.C.3 UNDERDRAIN REQUIREMENTS

All concrete lined chutes that serve as principal spillways shall be provided with underdrains. The upstream edge of the underdrain should lie at the downstream side of the core or in the case of a homogeneous embankment proximate to the downstream side of the crest. For a nominal cost an underdrain provides:

Uniform subgrade support for the spillway slab, and

Facilitates drainage of any seepage or leakage through deteriorating water stops or cracks in the concrete. The presence of water beneath the slab would otherwise pose frost action and/or uplift problems.

Underdrains may be omitted in the following circumstances. These circumstances are:

The spillway will be cast directly on rock. In this eventuality footing drains probably will be necessary for the sidewalls of the spillway, or

The avenues of seepage have been appropriately cutoff and the concrete chute serves as an emergency spillway that is expected to pass flows only a few times over the project life.

Where the subgrade is highly pervious, a series of isolated drains may be provided in lieu of a continuous blanket drain. The isolated drains normally are constructed adjacent to the joint between spillway slabs. Here the drain can pick up water passing through defects in the waterstop.

3.4 GEOSYNTHETICS

3.4.1 OBJECTIVE:

Provide a geosynthetic compatible with the impounded fluid or slurry that will survive anticipated strains in service, deterioration from UV exposure and vandalism over a lengthy service life.

3.4.2 ENGINEERING CONCERNS:

- *Durability in service*. The durability of the principal geomembranes types is well recognized and has not presented significant design and/or performance problems[1]. Experienced designers are generally familiar with the strengths and limitations of the various geosynthetics so that they are able to readily match an appropriate material to the expected service environment.

- *Low friction angles of elements of the "geosynthetic sandwich" or elements in direct contact with the geosynthetics*. The friction angle between a geotextile and a soil typically is on the order of 24 to 26°. This represents a reduction of as much as 25% from the soil to soil friction angle of typical sands[2]. Significantly lower friction angles have been determined for geomembranes in contact with one another, other types of geosynthetics or adjacent soil layers. Friction angles as low as 8 to 9° have been measured for HDPE geomembranes in contact with geotextiles and other HDPE geonet materials[3]. Therefore, the designer must consider the potential plane of weakness introduced into their scheme when geosynthetics are employed.

- *Tendency to use synthetics in new applications before appropriately documenting their suitability for the actual environment and service conditions; particularly compatibility problems*. For example, there is considerable product development underway to address the environmental stress cracking problems with HDPE. The industry recently introduced a Very Low Density Polyethylene (VLDPE) geomembrane that exhibited much improved resistance to environmental stress cracking. The material was marketed as a hybrid HDPE without the environmental stress cracking problem. The significant poorer performance of this material from that of HDPE in resisting degradation when exposed to a broad range of compounds was not prominently discussed in the trade literature. A considerable period of time elapsed before it became common knowledge that VLDPE was unsatisfactory for the impoundment of some wastes. Specifically, this geomembrane is inappropriate for a number of strong acids, various ketones, and alcohols, among other solutions. It is incumbent on the designers that they demonstrate the proposed liner material has been used successfully in the proposed application elsewhere. Or, if this is a new application, some form of compatibility testing should be performed on a representative sample of the waste material to document the suitability of the geosynthetic in the proposed application.

- *Basing designs on material properties from uniaxial testing that does not properly reflect the behavior of the geosynthetic in actual service*. Designs should be formulated on the basis of testing that best reflects the type of service conditions it will be required to resist. The frequently unrealistic, uniaxial test results should be used principally as a indicator of quality between two of the same type sheets. But, the design should be based on actual 2-dimensional or 3-dimensional materials testing and/or case histories.

3.4.3 DESIGN REQUIREMENTS\MINIMUMS:

Redundancy - The DSO will not accept a project design where a geosynthetic is the sole element employed to perform a "critical function". A "critical function" is defined as an element of the impounding barrier that were it to fail, there could be a catastrophic release of the reservoir. A redundant design feature is required to provide reasonable assurance of satisfactory long term performance. This redundant feature need not achieve the same level of overall performance as the geosynthetic element; it simply must prevent an uncontrolled release of the reservoir contents. For example, if an intact geomembrane lining is necessary to prevent the washout of a pervious embankment, an additional low permeability zone would have to be provided. However, if the embankment materials are themselves sufficiently impervious such that a failure of the liner would not precipitate their failure, then no further redundant elements are required to satisfy DSO concerns as to stability. Obviously, regulatory bodies other than the DSO may place additional requirements on the project to address pollution concerns.

Materials Quality Control
Geomembranes must meet or exceed the minimum specifications of the relevant sections of the National Sanitation Foundation Code 54-1991[4] [Phone (313) 769-8010] or other recognized standard.

Various bodies, including the American Society of Testing Materials (ASTM) and Task Force #25 (AASHTO-ABC-ARBTA), have developed tests for controlling geotextile quality. The DSO cites no specific materials criteria for geotextiles because quality control problems have not been an issue with them in our experience. The principal concerns with regard to the use of geotextiles in dams have been the specification of too light weight of a fabric or the wrong type of fabric. In general, a woven fabric is selected where strengthening and/or reinforcing are the sole tasks the geotextile performs. Where the geotextile functions principally to filter or provide a cushion between the subgrade and overlying geosynthetic, non-woven materials are typically specified.

Liner Installer's Qualifications - The specifications shall cite the minimum experience the contractor must have had in the installation of the particular type of liner(s) they will be responsible for placing. This normally includes a minimum square footage of successfully installed geomembrane of the type(s) proposed for the facility.

Installation Scheme - It is typical practice for the DSO to approve plans where the liner phase of the project is only outlined in conceptual detail. The approval of the plans carries the proviso that the successful prime contractor will require their lining subcontractor to submit to the DSO details of their lining scheme for review and comment. This submittal shall detail the layout scheme for the individual geomembrane panels including the manufacturer's identification number for each panel. This allows correlating test results on representative samples of the geosynthetic to the individual sheets forming the liner.

Field Seam Testing - The DSO shall be provided with details of the testing program to confirm it corresponds with accepted practice. Guidance on accepted field practice is contained in the EPA document EPA/530/SW-91/051[5]. The Geosynthetics Research Institute (GRI) at Drexel University (215) 895-2343 also publishes excellent Standards of Practice[6] for field seaming and geosynthetics materials testing. The testing program will be reviewed by the DSO to confirm that it conforms to accepted practice.

Again, the DSO is primarily concerned with the prevention of a breach of the reservoir. The Owner should recognize that other regulatory bodies may require more stringent quality assurance/quality control programs to be performed by independent, third parties when dealing with noxious materials. Our minimum scope of testing would not likely satisfy the most stringent of other regulatory bodies.

3.4.4 SPECIAL ISSUES:

Wind Damage - To prevent wind damage of the geomembrane, where ponds will be empty a significant portion of their service lives, provisions should be included to hold down the liner. Normally, this is accomplished by placing a suitable erosion resistant, zoned, soil cover. However, where vandalism is not considered a problem and the liner is resistant to ultraviolet degradation, liners have been successful anchored by draping soil filled, corrugated HDPE pipes at intervals around the interior pond sideslopes. The pipes normally are tied to anchor posts beyond the limits of the liner.

Filtration Issues in the Selection of a Geotextile - There is a growing recognition that past filter criteria have not adequately dealt with the problems posed by gap graded soils with an appreciable fines content. The prinicipal problem with these soils has been a tendency for the fines to move to the geotextile where they clog it and reduce hydraulic capacity. DeBerardino[7] presents an excellent overview of current practice in filtration design addressing this problem.

3.4.5 REFERENCES:

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2. Koerner, Robert M., Designing with Geosynthetics, 2nd Edition, Prentice Hall Publ. Co., 1990, pp. 83.

3. Seed, R.B., Mitchell, J.K., Seed, H.B., Slope Stability Failure Investigation: Landfill Unit B-19, Phase I-A, Kettleman Hills, California, Report No. UCB/GT/88-01, Department of Civil Engineering, U. of Berkeley, California, July 1988, pp. 92.

4. NSF Joint Committee on Flexible Membrane Liners, Standard 54 Flexible Membrane Liners, NSF 54-1991, Revised May 1991.

5. U. S. EPA, Technical Guidance Document: Inspection Techniques for the Fabrication of Geomembrane Field Seams, EPA/530/SW-91/051, May 1991.

6. Geosynthetic Research Institute, GRI Test Method GM-6, Standard Practice for Pressurized Air Channel Test for Dual Seamed Geomembranes, Drexel University, Philadelphia, PA, Adopted October 1989.

7. DeBerardino, Steve, "Filtration design: A look at the state-of-the-practice", Geotechnical Fabrics Report, Vol. 11, No. 8, Nov. 1993, pp. 4-10.

3.5 EMBANKMENT EROSION CONTROL AND WAVE PROTECTION

3.5.1 OBJECTIVES:

To protect the crest and slopes of earthfill dams from erosion resulting from storm runoff, wave action, or vehicular traffic. Discourage the growth of undesirable vegetation such as trees, dense brush and other deep rooted plants.

3.5.2 ENGINEERING CONCERNS:

The following problems can develop as a result of inadequate slope and/or erosion protection on earthfill dams:

> Reservoir wave action can erode the upstream face at the normal pool level, creating a beach or shelf. This "beaching" can reduce the embankment crest width. In severe cases (e.g. large waves during a major flood breaking against highly erodible embankment materials) the wave action could, over time, erode through the crest and breach the dam.
>
> Runoff from storms and/or snowmelt can result in the formation of gullies on the slopes. Such erosion can lead to gradual deterioration of the slopes. In severe cases, erosional gullies could backcut through the crest, reduce the freeboard and result in overtopping during a flood.
>
> Vehicular traffic across the dam can leave deep ruts, which collect water that saturates the crest and leads to greater rutting. Eventually, water collecting in the ruts overtops or breaches the network of ruts, discharging onto the embankment face. This concentrated discharge can form gullies on the dam slope where the soil is erodible.
>
> Trees and dense brush growing on the dam obscure conditions, hampering visual inspection. The vegetation provides attractive habitat for burrowing animals. Tree roots can create seepage paths through the embankment and allow internal erosion of the embankment to occur. Finally, uprooting of large trees during high winds can leave voids in the embankment, locally reducing the stability of the embankment.

3.5.3 DESIGN PHILOSOPHY:

In general, slope and erosion protection features are important, but not critical elements of the dam. Although it is possible for a dam to fail because of inadequate slope protection, to date there are no known cases of a dam failure by wave or runoff erosion. Thus, erosion protection features are usually designed with consideration to the costs associated with initial construction, project operation and long term maintenance.

Slope protection for the upstream face of the dam, such as riprap, is often designed using the concept of *Survivability*, i.e. to accept some damage during an extreme wind condition, provided that the wave erosion does not lead to a failure of the dam. The engineering considerations for design of slope protection and erosion control features are discussed below.

3.5.4 DESIGN PRACTICE:

Upstream Slope Protection - The upstream slope primarily needs protection from wave action. For most earthen dams, common practice is to employ a riprap blanket. Normally, the riprap extends from the crest to several feet below the normal low water line. The primary factor in the design of the riprap blanket is the maximum wave height. The maximum wave height is governed by the design wind speed, reservoir fetch, wave run up, and wave setup. The determination of the design wave height is discussed in greater detail in *Section 4.6, Reservoir Freeboard*.

Downstream Slope Protection - The downstream slope primarily needs protection to prevent erosion from surface runoff. Normally, a vegetative cover of native grasses will provide suitable protection against erosion. The *Seed Specification Guide*[11] provides assistance in the selection of an appropriate vegetative cover. In the more arid areas of the state, however, precipitation is insufficient to support a grass cover. In these areas, protection by a facing of cobbles or rock may be necessary. The groin areas often need additional protection, as runoff collects there. Thus, the groins may need a riprap blanket protection to prevent the formation of erosion gullies.

Crest Protection - Where vehicular (or animal) traffic across the dam crest is anticipated, an erosion resistant surfacing may be needed to minimize rutting and erosion. For dams where little or no vehicular traffic is expected, erosion protection similar to that on the downstream slope should be sufficient.

Landscaping - Trees, dense brush and other deep rooted vegetation should not be planted as landscaping or for erosion protection on dams. In some special cases (notably stormwater detention ponds), small shallow rooted trees and low growing brush are allowed as landscaping, provided they do not hinder visual inspection.

3.5.4.1 Wave Protection

Riprap Blanket - Median Stone Size - In general, dumped riprap is the erosion control measure most frequently used for protecting the upstream face of earthen dams from wave action. The size of stones needed is dependent upon the magnitude of wind generated waves, the steepness of the waves, the slope of the dam face, and the unit weight and angularity of the stones. Technical references and procedures for computing the magnitude of wind generated waves are contained in *Section 4.6, Reservoir Freeboard*.

Additional technical information on the determination of wave characteristics and required stone sizes is contained in COE[4], Cassidy[5] and Ahrens[6]. Those references were used to develop Table 1 which contains general guidance in the selection of median stone size (D_{50}) as a function of the design wave height. It should be noted that Table 1 was prepared for earthen embankments with upstream slopes of 3H:1V. The median stone size needed for slopes other than 3:1 can be determined by use of scaling factors contained in Table 2.

The values contained in Table 1 are applicable to angular rock. If rounded or sub-rounded stones are used, the median stone size should be increased by as much as 40% to offset the reduced interlocking ability of rounded stone. Finally, the riprap must be durable. The rock must be resistant to breakage upon repeated wetting and under freeze-thaw action.

TABLE 1. GUIDANCE IN SELECTION OF MEDIAN STONE SIZE (D_{50}) FOR RIPRAP LININGS ON DAMS WITH 3:1 UPSTREAM FACE

	WAVE HEIGHT (FEET)						
	0.50	1.00	2.00	3.00	4.00	5.00	6.00
MEDIAN STONE SIZE D_{50} (INCHES)	2.0	4.0	8.0	12.0	15.0	18.0	24.0

TABLE 2. SCALING OF RIPRAP SIZE FOR SLOPE OF EMBANKMENT

	SLOPE OF UPSTREAM FACE OF DAM (H:V)				
	2:1	3:1	4:1	5:1	6:1
RIPRAP SCALING FACTOR Ratio to Stone Sizes for 3:1 Slope	1.28	1.00	0.81	0.73	0.68

Riprap Blanket - Gradation - Several research investigations have been conducted to determine suitable gradations of stones comprising the riprap blanket and the developed criteria[1,2,4,5,8,9] can be used to prepare a gradation specification. However, for most small and intermediate size projects, the increased costs associated with screening the rock and confirming that a specified gradation has been obtained, precludes setting and achieving tight gradation controls.

As a practical matter, a suitable gradation can be obtained by specifying the riprap be reasonably well graded with a specific minimum stone size (D_{min}), median stone size (D_{50}) and maximum stone size (D_{max}).

Research by Lefebvre et al[9] has demonstrated the importance of achieving a gradation of stone sizes which are reasonably well graded with a minimum of fines. The following scaling factors have been found to produce an acceptable gradation[7]:

$$D_{min} = 0.25(D_{50}) \tag{1}$$

$$1.25(D_{50}) \leq D_{max} \leq 1.50(D_{50}) \tag{2}$$

Riprap Blanket - Thickness - A variety of criteria exists[1,2,3,4,5,6,8] for determining the thickness of the riprap blanket, usually as a function of the median stone size or largest stone size. Common practice is to use a thickness of 1.5 to 2 times the median stone size. As a practical consideration, the blanket thickness should be greater than the maximum stone size so that no single stone locally comprises the blanket.

In general, the blanket thickness is treated as a cure-all for a number of a considerations.

Oftentimes, rock of sufficient size is unavailable, or the desired gradation cannot be achieved economically due to limitations at the quarry. Common practice in these cases is to compensate by increasing the blanket thickness and to accept potential increased maintenance costs if damaged from extreme wind/wave action.

"Constructability" limitations governs the selection of the minimum blanket thickness where the analysis shows cobble size stones to be adequate. Here, a minimum blanket thickness on the order of 8 inches is appropriate. Lesser thicknesses, while theoretically justifiable, could result in inadequate thicknesses locally given the normal variance in blanket thickness achieved in the field.

Riprap Blanket - Filter Underlayment - Most riprap failures are due to the lack of filters, or the use of inadequate filters beneath the riprap blanket. Therefore, a filter is needed under the riprap layer to prevent erosion of the underlying soil through the voids in the riprap. The filter can consist of either a granular material, or a geotextile. The exception is the case where the underlying soil has a sufficient coarse soil fraction to meet filter criteria for the riprap.

3.5.5 REFERENCES:

WIND SPEED AND WAVE HEIGHT
Refer to Section 4-6, *Reservoir Freeboard*

RIPRAP DESIGN/SIZING
1. U.S.D.A. Soil Conservation Service, Riprap for Slope Protection Against Wave Action, Technical Release No. 69, February, 1983.

2. U.S. Army Corps of Engineers, Earth and Rock Fill Dams, General Design and Construction Considerations, Appendix C, EM 1110-2-2300, May, 1982.

3. U.S. Bureau of Reclamation, Design Standards No. 13, Embankment Dams, Chapter 7, Denver, CO, 1984.

4. U.S. Army Corps of Engineers, Shore Protection Manual, Volumes I,II,III, Coastal Engineering Research Center, Fort Belvoir, Virginia, 1984.

5. Cassidy, J.J., Riprap Design, Fundamentals of Hydraulics and Hydrology of Dam Design, Short Course - University of Missouri, Rolla, 1981.

6. Ahrens, J.P., Large Wave Tank Tests of Riprap Stability, Tech. Memorandum No. 51, U.S. Army Corps of Engineers Coastal Engineering Research Center, Ft. Belvoir, Virginia, 1975.

7. King County Surface Water Management Division, Surface Water Design Manual, King County Department of Public Works, 1990, pp4.3.7-2

8. U.S. Army Corps of Engineers, Hydraulic Design of Flood Control Channels, Design Manual EM 1110-2-1601, 1970.

9. Lefebvre, G., Rohan, K., Mahrez, B.B., and Dascal, O., Field Performance and Analysis of Steep Riprap, ASCE Journal of Geotechnical Eng., Vol 118, No. 9, 1992, pp 1431-48.

DOWNSTREAM SLOPE PROTECTION
10. U.S. Bureau of Reclamation, Design of Small Dams, Sections 6.23 and 6.24, pp 265-267, Denver, CO, Third Edition, 1987.

VEGETATIVE COVER FOR SLOPE PROTECTION TREES ON DAMS
11. Washington State Department of Agriculture, Seed Branch, Seed Specification Guide, Yakima, WA, 1980.

12. Weaver, J. D. and Tschantz, B. A., Tree Growth on Earthen Dams: A Survey of State Policy and Practice, Civil Engineering Dept., Univ. of Tennessee, Knoxville, TN, 1988.

CHAPTER 4 - HYDRAULIC ELEMENTS AND ISSUES

4.1 LOW LEVEL OUTLET CONDUITS

4.1.1 APPLICATIONS:

There are three basic applications for conduits which pass through embankment dams or are located within the footprint of dams. These include: low level outlets for release of reservoir waters to supply project water needs; outlet conduits for drop inlet or gated tower spillways; and general use pressurized conduits within the footprint of the dam principally for water or wastewater transmission.

This section primarily addresses issues related to the design and construction of low level outlet conduits. However, this section is also applicable to the general geotechnical/structural issues related to conduits for drop inlet and gated tower spillways, and other non-pressurized conduits.
The hydraulic aspects of conduit design for drop inlet and gated tower spillways are discussed in *Section 4.2 Principal Spillways*. Other issues concerning general purpose pressurized conduits are discussed in *Section 4.1A Pressurized Conduits*.

4.1.2 OBJECTIVE:

Provide a durable, low level outlet that has the necessary hydraulic capacity for project needs and that can be readily inspected and renovated.

4.1.3 ENGINEERING CONCERNS:

Outlet conduits through earthen embankments must perform under relatively unique and severe service conditions. Conduits may be exposed to high seepage gradients along their perimeter. They experience significant variation in the level of stresses along their length due to differing heights of overlying embankments. The stress state generally changes over the life of the pipe as portions of the embankment becomes saturated under steady-state or seasonally varying seepage conditions. The pipe is also subjected to transverse and longitudinal stresses arising from the relatively large horizontal and vertical strains of the encapsulating soil as it consolidates. Subgrade support may vary significantly and induce large localized stresses in the pipe as it is forced to "bridge over" softer subgrade areas.

Because of these harsh service conditions, conduit related problems have, on occasion, occurred at dams in the U.S. In particular, approximately one sixth of all dam failures can be attributed to problems with conduits. The failures generally are of two types. The first class of problems are those where the integrity of the pipe is lost either by joint failures or through breaches in the pipe wall. The second class of problems are those caused by excessive seepage along the pipe perimeter where the seepage waters may originate from the reservoir, or from leakage out of the conduit.

Given the above background information, it is readily seen that the design of an outlet conduit for an earthen embankment must address a variety of hydraulic, geotechnical and structural concerns.

Hydraulic Concerns - The principal hydraulic design concerns for low level outlet conduits are:

> Positioning it sufficiently low in the reservoir that the major portion of the reservoir storage volume can be evacuated, but not so low as to be affected by sedimentation buildup[1,2] over the life of the project.
>
> Providing sufficient discharge capacity to meet project demands and anticipated future needs.

Providing adequate hydraulic capacity to drawdown the reservoir in some reasonable period of time for emergency purposes or for inspection and repair of project elements which are normally submerged.

Including features to reduce the potential for slug flow and hydraulic transients in the pipe.

Designing the layout of valves to allow for conduit inspection and repairs, and providing redundant shut-off capabilities to prevent an uncontrolled release of reservoir waters if a system component were to fail.

Geotechnical/Structural Concerns - The principal geotechnical/structural design considerations for **all** conduits which pass through earthen embankments include:

Maintaining the integrity of the individual lengths of pipe and the joints between pipe sections in undergoing the strains produced by long term static and dynamic loadings.

Minimizing the likelihood of overstressing the pipe during installation and, after completion of the facility, under normal service conditions.

Minimizing seepage along the perimeter of the pipe.

Improving the durability of the conduit considering corrosion, abrasion and free-thaw action.

Providing a means to facilitate inspection of the pipe.

Incorporating features in the design to allow for future renovation of the pipe.

4.1.3 DESIGN PHILOSOPHY:

Outlet conduits are considered critical project elements and design features are necessary which address the relatively unique and severe service condition of conduits within dams. The concepts of *Redundancy, Inspectability, Serviceability, and Consequent Dependent Design Features* are all pertinent to the design and construction of conduits.

Redundancy - Redundant design features are appropriate in several areas. There is a need for redundant valves to allow for conduit inspection and repair, and to provide for emergency shutoff. Redundant features are also needed for those elements which have experienced problems in the past. These include defense mechanisms such as reinforcing pipe joints, increasing pipe wall thickness and minimizing seepage and piping potentials.

Inspectability\Performance Monitoring - Conduits are susceptible to corrosion, abrasion and long term deterioration. It is important to include features in the design which allow for inspection and monitoring of the condition of the conduit. In the past, inspection capabilities were very limited. Now, detailed visual inspection can be accomplished with remote video units. Accommodation of the above considerations suggests specifying a minimum pipe diameter and selecting favorable locations for valves and gates. Use of seepage interception features such as filter-drain diaphragms also allow the measurement of seepage and long term monitoring for any adverse changes.

Serviceability - It is reasonable to expect that the useful life of the dam may far exceed the service life of the conduit. Provisions are needed in the design to allow for future renovation. This is normally accomplished by providing oversized conduits, straight alignments and locating valves where they can be readily removed or at least will not obstruct the future sleeving of the conduit.

Consequent Dependent Design Features - The design scheme for outlet conduits should become increasingly more conservative as the consequences of failure become more severe. Ideally, it would be desirable to utilize the Design Steps/Levels (Section 2.1) employed in other areas of design. However, quantitative procedures are not currently available to assess the level of safety afforded by various design schemes for conduits. This suggests a qualitative rather than quantitative approach. Therefore, the design philosophy used here is to incrementally increase the defense mechanisms and the general level of conservatism as the consequences of a conduit/dam failure become more severe. This approach considers such factors as: the consequences of failure and the downstream hazard classification; the size of the dam; and whether the facility will have a permanent pool, or, like some flood control dams, only have a significant reservoir depth during and immediately following a flood event.

The Design Philosophy described above has been used to develop the design minimums and recommended design practices described in *Section 4.1.5 Design Practices* and listed in Tables 3 and 4.

4.1.4 PAST EXPERIENCES WITH INADEQUATE CONDUIT DESIGN PRACTICES:

Utility Conduit Design Practice - A significant number of the plans that have been received by our office over the years have included unacceptable conduit design schemes. Utility line practices have often been applied to the design of conduits within dams. Specifically: relatively free-draining bedding materials have been called out for the full length of the conduit; filter criteria have not been satisfied for the surrounding soils; and hay bales have been specified to be placed atop the pipe through the maximum embankment section to reduce pipe stresses through "arch action". While these design practices have often performed satisfactorily for utilities, they are **inappropriate** for conduits in dams. It is imperative that the designer take steps to preclude piping of embankment and foundation soils. Failure to do so could result in the formation of voids within the embankment. Should the voids progressively extend upstream, eventually a blow out of the embankment will occur. Alternatively, if the void grows vertically, a sink hole can develop. If only the fines are lost from within portions of the soil matrix, voids may not be produced, but there could be a significant increase in permeability and associated seepage volumes. This increase in seepage may cause slides to occur in the downstream face where the flow emerges. While placing hay bales in deep utility trenches may be a practical means of reducing the class of pipe necessary, in dams this practice could lead to hydraulic fracturing locally that could have disastrous consequences.

Rigid Seepage Cutoff Collars - Some standard textbooks still recommend rigid cutoffs along the pipe to limit seepage. However, experience has shown that rigid seepage cutoffs may create as many problems (stress concentrations on the pipe, inability to compact around the collars) as they are purported to solve. It should be remembered that the principal concern that prompted the original use of seepage cutoff collars was to minimize seepage along the conduit and to also prevent piping of materials in this confined area where it is difficult to achieve the desired level of compaction.

With current design and construction practices, there are measures available to minimize seepage (Table 2), which in combination with filter-drain diaphragms (*Section 3.3B, Conduit Seepage Control - Filter-Drain Diaphragms*) address both the seepage and piping issues. In most cases, they offer a relatively low cost, simple to construct, improved alternative to the rigid cutoff collar. The many advantages of this design have allowed it to rapidly supplant the traditional, but less effective rigid cutoff collar.

4.1.5 DESIGN PRACTICE:

The following sections present some commonly used design schemes and mitigative measures to address the wide range of engineering concerns described in the previous sections.

4.1.5.1 Hydraulic Capacity

The primary consideration is that the low level outlet conduit has sufficient hydraulic capacity to meet current project demands and anticipated future needs. The second consideration is that provisions are incorporated to allow for the timely drawdown of the reservoir in response to emergencies and for purposes of inspection and repairs of project elements which are normally submerged. For small dams and reservoirs, this may be satisfactorily accomplished by pumps or siphons. However, for large reservoirs, the low level outlet works, in combination with other in-place hydraulic structures are the only practical means of lowering the reservoir. Therefore, the determination of the hydraulic capacity of the low level outlet conduit is usually governed by this requirement rather than on the water use needs.

Recommendations - There are no hard and fast rules for setting the period of time during which the low level outlet should be capable of drawing down the reservoir. Periods of several days for small projects to several weeks for large projects appear reasonable. Whatever time span is selected, it is important to factor in the expected quantity of reservoir inflow in addition to the reservoir storage volume. The final determination is left to the project engineer to be based on site specific project considerations. Additional guidance on this subject can be obtained from US Bureau of Reclamation (USBR) publications[3,4].

4.1.5.2 Vents for Pressurized Conduits

Under high reservoir heads, large negative pressures can develop immediately downstream of partially open upstream gates and valves and lead to cavitation damage. Also, partial vacuum pressures can occur in long conduits when the conduits are being dewatered if the upstream valve is closed. Under adverse circumstances this can lead to vacuum buckling of the pipe.

Requirements - An atmospheric vent is required immediately downstream of the upstream gate or valve on pressurized low level outlet conduits at intermediate and large dams to minimize the effects of cavitation and/or vacuum buckling. Information on vent sizing is contained in SCS[6] and US Army Corps of Engineers (COE) design manuals[5] on outlet works.

4.1.5.3 Vents for Non-Pressurized Conduits

Slug flow and "make and break" siphon action can occur in outlet conduits leading from drop inlet spillways and other intake structures. These hydraulic transients can result in large scale vibration and pressure surges in the conduit and surging and poorly distributed flow in the energy stilling basin or receiving channel.

Requirements - An atmospheric vent is needed at the entrance of the conduit to stabilize the flow and preclude the occurrence of siphon action and slug flow. *See also Section 4.2.4.2 Principal Spillways*.

4.1.5.4 Valving for Pressurized Conduits

In most cases, regulation of reservoir releases for pressurized conduits is controlled by a valve at the downstream end of the conduit. Provisions are needed to allow draining of the conduit for inspection and for shut-off in the event of a conduit related emergency.

Requirements - All pressurized conduits must have an upstream valve or other means of effecting a shut-off.

4.1.5.5 Structural Issues

A variety of structural concerns arise as a result of either foundation conditions, the non-uniform loading conditions from the overlying embankment or from construction loads. Table 1 identifies a number of common problems and measures that can be taken to mitigate the problems.

TABLE 1 - STRUCTURAL ISSUES FOR CONDUITS

STRUCTURAL PROBLEM	MITIGATION MEASURES
Uneven subgrade support and/or pyramid shaped overburden load generating differential horizontal and vertical strains in the soil encapsulating the pipe	Where practicable, lower pipe grade to bear uniformly on rock. If impractical, provide a structural fill pad beneath pipe footprint Provide a concrete cradle or full concrete encasement of the pipe
Obtaining a dense, low permeability zone along the pipe	Overbuild the working surface of the fill, excavate a trench to accommodate the pipe and a concrete cradle zone, or ideally full pipe encasement
Tendency for all strains along the pipe to be relieved at the joints	Use concrete cradle or full concrete encasement so pipe acts monolithically rather than as an assemblage of rigid lengths where horizontal and vertical strains are relieved at the joints Provide reinforcing at pipe joint
Adequate pipe strength and integrity for lengthy service life (About 50 dams in Washington are 80 years or older)	Specifying heavier gauge or thicker walled pipe Providing for future relining or sleeving by specifying oversized pipe and maintaining straight alignments
Overstressing of the pipe during installation	Installing the pipe in a trench Concrete encasement Controlling construction traffic near the conduit Considering potential construction loads when specifying the class of pipe

4.1.5.6 Service/Durability Issues - Corrosion/Abrasion

Corrosion may result from naturally occurring waters or as a result of action of wastewater(s) at water quality projects or mine waste runoff at mining projects. Abrasion usually occurs as a result of sediment which is entrained in the water. Studies have also shown that clear water will abrade concrete at velocities in excess of about 25 ft/sec. Whatever the cause, corrosion and abrasion reduce the service life of the conduit.

Mitigative Measures - Use of plastic pipe in conjunction with concrete encasement or use of precast reinforced concrete pipe are effective means of combating corrosion. In some applications, use of heavy gauge pipe and/or asphalt or concrete linings are also effective against both corrosion and abrasion.

Regardless of the preventative measures taken, monitoring of the conduit condition is necessary to confirm the continued safe performance.

4.1.5.7 Seepage Issues

A major cause of conduit related problems is seepage moving along the perimeter of the conduit. A discussion of this issue is contained in *Section 3.3B Conduit Seepage Control - Filter-Drain Diaphragms* and several technical articles referenced in that section. A number of measures commonly used to mitigate the seepage concerns are described in Table 2.

TABLE 2 - SEEPAGE ISSUES ALONG CONDUITS

SEEPAGE PROBLEMS	MITIGATION MEASURES
Potential For Excess Seepage along the Outside Perimeter of the Pipe	Employ a low permeability soil as a backfill material Take extra care to achieve a low in-place permeability by including a specification which fully describes the requirements and performance goals for compaction adjacent to conduits. Also provide full time inspection of this work Confirm that filter criteria is satisfied between the backfill and the adjacent soils Batter the sides of concrete encasement of pipes to facilitate the operation of compaction equipment next to the encased pipe Overbuild the working surface of the fill, excavate a neatline trench to accommodate the pipe and fill the trench with concrete to fully encase the pipe Provide for the controlled interception and discharge of seepage by constructing a filter-drain drainage diaphragm for the pipe, as described in Section 3.3B.4

4.1.5.8 Summary of Requirements and Recommended Practices

A summary of the design minimums, general requirements and recommended design practices presented in the previous sections for the more common engineering concerns are listed in Tables 3 and 4. In preparing these tables it is also recognized that some latitude in the design is necessary to reasonably address the large variations in the size and scope of projects constructed in Washington. The goal in these guidelines is to present a technically sound design philosophy with a reasonably consistent level of design reliability between projects with roughly similar downstream hazard classifications.

TABLE 3
DESIGN MINIMUMS FOR NON-PRESSURIZED CONDUITS FOR EARTHEN DAMS WITH LOW DOWNSTREAM HAZARD CLASSIFICATIONS
[Design Step Levels 1 and 2]

ITEM - ISSUE	REQUIRED MINIMUM/DESIGN PRACTICE
Minimum Pipe Size[1]	12 inch diameter for concrete encased pipe, otherwise 15 inch diameter Provisions must be available to pass the normal reservoir inflow during periods of high runoff while still pulling the reservoir down within a span of a few days to weeks for inspection, repairs or emergency purposes. This discharge capacity may be obtained from use of the low level outlet and/or from other permanent or temporary hydraulic systems
Pipe Gauge or Wall Thickness	Adequate to account for anticipated construction and service loads, abrasion, long term durability and non-uniform foundation support
Pipe Joints	Rubber gasketed joints are required, except for welded pipes For corrugated metal pipe, widest available bolted connectors are required
Concrete Encasement	Required for pressurized conduits in small dams and for all intermediate and large dams[2]
Upstream Control Valve to Regulate Water Releases[3]	Required
Atmospheric Vent for Low Level Outlet	Required on all conduits with the exception of pressurized conduits in small dams
Filter-Drainage Diaphragm	Required
Low Permeability Pipe Bedding Zone	All bedding material upstream of filter-drainage diaphragm must have a permeability less than or equal to that of the surrounding material and must satisfy filter criteria for all adjacent materials

[1] Use straight alignment whenever practicable to facilitate future sleeving of the pipe

[2] Pipe cradle scheme considered for non-pressurized pipes in stormwater detention facilities with temporary pools

[3] Not required on conduits for drop inlet, culvert spillways or conduits where inflow is regulated by intake structures

TABLE 4
DESIGN MINIMUMS FOR CONDUITS FOR EARTHEN DAMS
WITH HIGH OR SIGNIFICANT DOWNSTREAM HAZARD CLASSIFICATIONS
[Design Step Levels 3 and Greater]

ITEM - ISSUE	PERMANENT OR SEASONAL POOL			TEMPORARY POOL/INTERMITTENT RESERVOIR OPERATION		
	SMALL DAM	INTERMEDIATE DAM	LARGE DAM	SMALL DAM	INTERMEDIATE DAM	LARGE DAM
Minimum Pipe Diameter[1,2,3]	12"	12"	12"	12"	12"	12"
Complete Concrete Encasement of Pipe[4]	Required[5]	Required	Required	Required[5]	Required[5]	Required[5]
Upstream Shutoff or Control Valve[6]	Required	Required	Required	Required[7]	Required[7]	Required[7]
Atmospheric Vent for Low Level Outlet	Required	Required	Required	Required	Required	Required
Low Permeability Foundation and Backfill	All earthen materials upstream of filter-drainage diaphragm must have a permeability less than or equal to that of the surrounding material and must satisfy filter criteria for all adjacent materials			All earthen materials upstream of filter-drainage diaphragm must have a permeability less than or equal to that of the surrounding material and must satisfy filter criteria for all adjacent materials		
Filter-Drainage Diaphragm	Required	Required[8]	Required[8]	Required	Required[8]	Required[8]

[1] Use straight alignment whenever practicable to facilitate future sleeving of the pipe

[2] Outlet should be sized to be able to pass the normal reservoir inflow during the high runoff period while still capable of pulling the reservoir down within a span of a few weeks

[3] Pipe gauge or wall thickness adequate to account for abrasion, long term durability and other site-specific concerns

[4] Minimum of 6 inches of reinforced concrete for encasement of pipe section

[5] Pipe cradle in combination with precast reinforced concrete pipe may be used where the design can be justified on the basis of favorable site conditions

[6] Not required for conduits on drop inlet spillways and for conduits where the inflow is regulated by intake structures

[7] May not be required for stormwater detention and other flood control projects

[8] The chimney drain zone (*Section 3.2 Embankment Geometry and Zoning*) generally satisfies this requirement

4.1.6 REFERENCES

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4. Bureau of Reclamation, Design of Small Dams, Chapter 10, Outlet Works, US Dept. of Interior, Denver CO, pp 435-489, 1987.

5. US Army Corps of Engineers, Hydraulic Design of Reservoir Outlet Works, EM 1110-2-1602, Washington DC, October 1980.

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4.1A PRESSURIZED CONDUITS

4.1A.1 APPLICATION:

This section applies to the design and construction of pressurized conduits which pass through or are routed within, beneath, or along the embankment or the exterior embankment(s) in the case of a multi-celled impoundment.

4.1A.2 OBJECTIVE:

Provide a highly reliable system where miss-operation or failure of a system component will not lead to a dam failure and catastrophic release of the reservoir.

4.1A.3 ENGINEERING CONCERNS:

The principal Dam Safety concerns are:

- Minimizing the potential for seepage problems along the pipe perimeter,
- Containing the pressurized flow in the event of a failure of the pipe wall or joint so that the release does not sluice away the adjacent embankment,
- Configuring the conduit intake and/or outfall so that the flow does not scour the subgrade or, in the case of lined impoundments, pull up or abrade the geomembrane,
- Protecting the pipe from possible over-stressing during construction, and
- Accommodating stresses and pressure surges in pipe joints from vibrations induced by pumping equipment and operation of the system valves.

4.1A.4 DESIGN PHILOSOPHY:

Pressurized conduits should be kept off embankments to the extent practicable. Where they must penetrate through the embankment, that section of the pipe within the embankment footprint must be encased in concrete. Exceptions are discussed in the following section.

4.1A.6 DESIGN PRACTICE:

The design features for pressurized conduits are similar to those used in the design of outlet conduits.

4.1A.6.1 Concrete Encasement Requirements

To the extent practicable, pressurized conduits should be routed outside of the embankment toe. Where the pipe is within the embankment footprint, it shall be encased in concrete. There are a few limited exceptions to this requirement.

Conduits - The requirement for concrete encasement may be waived if the engineer can show that: 1) the downstream hazard setting is low and there would be no potential for loss of life in the event of a dam failure, 2) although a significant portion of the embankment could be sluiced away, an uncontrolled release of the reservoir contents is unlikely and 3) the owner acknowledges acceptance of the increase risked (albeit small) of adverse performance of this element.

Mine tailings discharge lines - Tailings discharge lines are normally placed along the interior sideslopes of the dam section in impoundments retaining industrial process slurries. Generally, the slurry system discharges from a number of points along the interior face. A beach of tails forms immediately down gradient of the discharge line. Erosion protection is unnecessary in most cases around the discharge points. However, in some instances it is necessary to route a section of the slurry supply lines on or over the dam crest. Where it is necessary to frequently move these lines, it is generally impractical to provide concrete or some other form of permanent encasement for the piping. The DSO has accepted schemes where "critical pipe runs", those segments of pipe located within or upon the exterior dikes, were sleeved by placing these lines inside of larger, jointed, corrugated metal pipes. In the event of a pipe or pipe joint failure, the sleeving would contain the flow and minimize the erosion damage to the areas immediately around the end sections of the sleeve.

4.1A.6.2 Pipe Settlement and Vibration Considerations

Pumping equipment is frequently placed on a structural pad proximate to the point the conduit emerges from the embankment. Consideration must be given to the potential effects on the pipe of static and dynamic movement of the pump and its foundation pad. The particular sections of concern to the overall stability of the reservoir would be those sections of pipe within the embankment and the first few sections projecting downstream beyond the dam toe. The design of the pipe should incorporate measures to allow it to accommodate the stresses and movements induced by potential settlement of the pump foundation pad. Likewise, the foundation pad and pipe should be designed so that they do not significantly respond to the predominant periods of the dynamic forces generated by running the pump.

The hydrodynamics of pressurized flow causes both static and dynamic loads to be placed on piping, particularly where there are abrupt changes in direction. Pipe anchors and thrust blocks may be required to resist the forces generated by changes in momentum of the flow.

4.1A.6.3 Recommendations on Valving

It is standard practice to provide some form of shut-off valve upstream of the pump to allow the pump to be removed and serviced without draining the reservoir. It is also desirable to have a means of draining the pipeline downstream of the pump. These items may be omitted where other elements in the system will accomplish these functions.

4.2 PRINCIPAL OR SERVICE SPILLWAY

4.2.1 OBJECTIVE:

Provide control of reservoir levels for reservoir inflows ranging from normal inflows to moderate flood flows.

4.2.2 COMMON TYPES OF PRINCIPAL SPILLWAYS:

There are a wide variety of hydraulic structures which are used as principal spillways. The more common spillway types are listed in Table 1 and technical design information can be obtained from the design manuals and technical articles in *Section 4.2.5 References*.

TABLE 1 COMMON TYPES OF PRINCIPAL SPILLWAYS

GENERAL APPLICABILITY OF SPILLWAY TYPE				
SPILLWAY TYPE	SMALL DAM	INTERMEDIATE AND LARGE DAMS	OFF-CHANNEL STORAGE FACILITIES	REGIONAL STORMWATER DETENTION PONDS
Chute	✓	✓	✓	✓
Chute with Bascule Gate		✓		
Chute with OGEE Weir		✓		✓
Chute with Stoplog Entrance	✓			
Chute with Gated Entrance		✓		
Culvert	✓		✓	✓
Drop Inlet	✓		✓	✓
Gated Tower	✓	✓	✓	✓
Labyrinth	✓	✓		✓
Morning Glory		✓		
Open Channel	✓			
Side Channel		✓		

4.2.3 ENGINEERING CONCERNS:

Given the wide variety of spillway types listed above, there is a similar wide range of engineering concerns. Some of the concerns are specific to the spillway type. However, the following list of engineering concerns is generally applicable to most spillway types and may aid in the selection of a spillway type for a particular application. Each of these engineering concerns are discussed in the following sections.

- Long Term Durability
- Desired Mode of Reservoir Operation
- Flexibility to Change Reservoir Operational Mode
- Expected Frequency of Site Visits by Operator
- Ease of Maintenance, Inspection and Future Repair
- Positive Control of Discharge
- Resistance to Debris Blockage
- Need for Atmospheric Venting or Aeration
- Potential for Ice Damage
- Energy Dissipation at Spillway Outlet
- Fish Passage Considerations

4.2.3.1 Long Term Durability
The principal spillway generally operates frequently and thus is exposed to repeated hydraulic loadings. These forces can cause abrasion, corrosion, vibration, etc., which can accelerate wear, scour and general deterioration. Particular care should be exercised when selecting a spillway type and construction material(s) to provide for durability.

Requirements/Minimums - Concrete mix design and design of reinforced concrete elements should be developed with specific attention to durability issues as outlined in *Chapter 5 Structural Elements and Issues*.

4.2.3.2 Desired Mode of Reservoir Operation
The manner in which the reservoir is to be operated is a primary consideration in the selection of a spillway type and size. Particular attention should be given to matching the spillway discharge characteristics to the range of anticipated inflows.

4.2.3.3 Flexibility to Change Reservoir Operational Mode
During the life of a project, there sometimes arises the need to change the way in which the reservoir is being operated. This situation commonly occurs on small on-stream reservoirs where insufficient streamflow data is available during the design stage to properly size the spillway. This is also true for flood control structures in urban areas where the streamflow characteristics are changing with increased development. In these types of situations, discharge features should be built into the spillway headworks to allow the flexibility to change operation as dictated by the needs of the project.

4.2.3.4 Expected Frequency of Site Visits by Operator
The availability of a project operator and the frequency of site visits influences the selection of the spillway type. In those cases where a full time operator or highly reliable remote operation cannot be provided, the spillway must be ungated and self regulating. This is often the case on small and intermediate sized projects. This situation also means that more simplified and conservative reservoir operation schemes are appropriate and that greater design conservatism should be applied to issues such as debris protection.

4.2.3.5 Ease of Maintenance, Inspection and Future Repairs
All man-made hydraulic elements deteriorate with aging. It is important that elements of the spillway and appurtenances be constructed in such a manner which will allow for easy maintenance and inspection. Likewise, the useful life of most dams far exceeds the life expectancy of the man-made hydraulic structures. Consideration should be given during design to provide features which will allow for rehabilitation at some future time.

4.2.3.6 Positive Control of Discharge
For most spillway types, there is one location, the discharge control point, which determines the relationship between reservoir stage and discharge. There are other spillway types such as drop inlet spillways, culvert spillways, and chute spillways with rapidly converging sidewalls where there may be a shift in control location with increase in reservoir stage. This shift in control can be accompanied by surging and/or slug flow which can cause increased hydraulic loadings. This often produces vibration of the hydraulic structure and surging of flow in the outlet channel. These situations should be avoided or minimized to the extent practicable.

Whenever possible, measures should be taken during design to provide for positive control of the discharge by establishing one discharge control point. For those spillway types where there are multiple discharge control points, sufficient analyses should be conducted during design to properly assess the spillway hydraulics. Design features should be employed to provide a smooth transition as control shifts from one control point to the other.

4.2.3.7 Resistance to Debris Blockage

Floating debris often accompanies moderate and extreme flood flows. Trashracks and/or log booms are necessary features to minimize debris blockage and the associated reduction of discharge capacity of the principal spillway. The discussion of debris protection and control measures is contained in *Section 4.5 Debris Protection for Hydraulic Structures*.

4.2.3.8 Need for Atmospheric Venting or Aeration

Venting of outlet conduits for culvert spillways, drop inlet spillways and morning glory spillways is usually needed to preclude the occurrence of slug flow. Likewise, aeration of the nappe just below the crest on drop inlet and morning glory spillways is needed to stabilize the flow pattern. For spillways on small projects, the venting requirements can be reasonably estimated using information developed by the SCS[14]. For large morning glory spillways, computer and physical model studies are normally used to determine the air requirements.

For high dams with steep conveyance chutes, flow velocities often exceed 60 feet/second. In these cases, cavitation may occur downstream of small surface irregularities[29]. Extra care is needed during construction to produce a smooth surface free of irregularities, particularly at joints. Alternatively, aeration of the flow has been found to be effective at minimizing cavitation damage. Discussions by Pinto et al.[26] and Rutschmann and Volkart[27] contain design information for entraining air into the high velocity flow.

4.2.3.9 Potential for Ice Damage

There is the potential for forces produced by ice to damage structures which are subject to severe freezing conditions. In general, structures which are located within the geometry of the impounding barrier are reasonably protected by the barrier. By contrast, exposed structures such as free standing towers are subject to the full force produced by ice pressures and ice movement. Ice forces[24,25] must be considered in the design of these structures.

4.2.3.10 Energy Dissipation at Spillway Outlet

During moderate to extreme floods, flow conditions are usually supercritical at the terminus of the spillway conveyance section (chute, conduit, etc.). The discharge is characterized as having high velocities and severe soil erosion capability. For these reasons, measures must be taken to dissipate the excess energy and control the flow before returning it to the receiving stream. Measures must be taken to assure that any erosion which occurs during the design flood does not jeopardize the integrity of the spillway or the impounding barrier. Use of survivability design concepts may be helpful in achieving economy in the construction of Energy Stilling Basins.

Requirements/Minimums:

Energy dissipation and erosion control measures must be provided at the terminus of the conveyance section of the principal spillway. Information on energy dissipation structures is contained in *Section 4.4 Energy Stilling Basins and Erosion Protection*.

4.2.3.11 Fish Passage Considerations

Information on fish passage requirements can be obtained by contacting representatives of the Departments of Fisheries and Wildlife.

4.2.4 EXPERIENCE/CURRENT ENGINEERING PRACTICE FOR VARIOUS SPILLWAY ELEMENTS

The following topics address issues which are frequently encountered during design and reflect engineering design practice on principal spillways:

4.2.4.1 Conveyance Channels/Chutes

Channel Freeboard - Freeboard is needed in spillway conveyance channels to accommodate wave action, air entrainment[17], splash and to provide for uncertainties in estimating the surface water profile under supercritical flow conditions. Experience has shown that a reasonable value of freeboard[20] can be estimated from:

$$\text{Channel Freeboard (ft)} = 2.0 + .025\,V\,[y]^{.333} \tag{1}$$

where: V = Velocity of flow (ft/sec) at a given location
y = Depth of flow (ft) at a given location

Convergence and Divergence of Channel Sidewalls - Wherever possible, chute sidewalls should be designed symmetrical to the channel centerline to minimize unevenly distributed flow, cross-waves, standing waves and splash. Experience has shown[20] that the maximum angular convergence or divergence (θ) of the sidewall with respect to the channel centerline is governed by:

$$\mathrm{Tan}(\theta) \leq \frac{1}{3\mathrm{F}} \tag{2}$$

where: $\boldsymbol{F}$ = Froude number, and

$$F = \frac{V}{\sqrt{gy}} \tag{3}$$

where: V = Velocity (ft/sec) at the given location
g = Gravitational acceleration (32.2 ft/sec^2)
y = Depth of flow (ft) at the given location

Contraction - Expansion Joints - Contraction-expansion joints must be provided on spillway chutes to maintain floor alignment while allowing for floor slab movement. The joint must be supported by a corbel-like pad.

Waterstops - Experience has shown that waterstops are an essential design detail for concrete spillway chutes. They serve to minimize the flow of water through contraction-expansion joints which could allow uplift pressures to develop beneath the floor slab. When used at construction joints, waterstops act to minimize the contact of oxygenated waters from reaching the reinforcing steel. This tends to prolong the useable life of the spillway.

Underdrains - Drains are required beneath contraction-expansion joints to prevent joint leakage from saturating the subgrade beneath the spillway slab and/or producing uplift pressures. It is common practice to use drains in combination with waterstops as design features of concrete chute spillways. See *Section 3.3C Concrete Lined Spillway Seepage Control* for further information.

4.2.4.2 Conveyance Conduits - Non-Pressurized Under Normal Conditions

Full Pipe Flow - Full pipe flow, which occurs as a result of a transfer of the discharge control point, will not be allowed unless it can be designed to occur in a manner with minimal surging of the flow and resultant hydraulic transient pressures.

Conduit Design - Part Full Pipe Flow - To avoid pressurized flow and problems posed by slug flow, the conduit should be designed with sufficient slope to restrict the maximum depth (Ymax) to less than that representing 70 percent of full pipe flow by area. For circular sections with diameter D, this represents:

$$Y_{max} \leq .66D \qquad (4)$$

A positive flow control device such as an eyebrow[18] or fixed sluice gate at the entrance to circular conduit will normally be needed to stabilize the flow hydraulics. The conduit must be vented to the atmosphere, with the air vent placed immediately downstream of the eyebrow[18].

4.2.4.3 Drop Inlet Spillways

Connection of Riser to Outlet Conduit - On small sized projects, drop inlet spillways are often constructed from sections of reinforced concrete pipe (RCP) or corrugated metal pipe (CMP). In these cases, the vertical riser section must be suitably founded on a structural pad of reinforced concrete. The connection between the riser and outlet conduit must be encased by reinforced concrete and this encasement should also tie into the concrete foundation pad. The inlet structure must be shown to be stable under the effects of buoyancy.

Riser Floor - Particular care should be given to the construction of the floor of the vertical riser. This area is subjected to high velocity, very turbulent flow conditions which accelerate erosion of the concrete. Additional wearing surface should be provided by overbuilding this section. See *Chapter 5 Structural Elements and Issues* for a discussion of concrete durability.

Outlet Conduit - The outlet conduit for a drop inlet spillway should include an atmospheric vent and eyebrow as described above in section 4.2.4.2. Technical information on the hydraulic design of outlet conduits is contained in SCS[14], Schaefer[18] and USBR[20].

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4.3 EMERGENCY SPILLWAYS

4.3.1 OBJECTIVE:

Passage of moderate to extreme flood flows and control of the reservoir level to an elevation below the dam crest.

4.3.2 DESIGN PHILOSOPHY:

Emergency spillways are normally designed to operate only in response to moderate or extreme floods, perhaps once or twice during the life of the project. Because of this infrequent operation, economy in design and construction can sometimes be accomplished for small and intermediate size dams by utilizing the concept of *Survivability*. That is, erosional damage can often be tolerated provided the damage does not jeopardize the structural integrity of the impounding barrier or allow an uncontrolled release of the reservoir waters. This may be accomplished by locating the emergency spillway in an area with erosional resistant materials or by providing sufficient structural features to limit erosional damage. In all cases, sufficient measures must be taken to preclude the uncontrolled release of reservoir waters through a breached spillway.

4.3.3 COMMON TYPES OF EMERGENCY SPILLWAYS:

Emergency spillways are intended to pass large flood flows. Those spillway types which have large discharge capacity and can be constructed with reasonable economy are usually selected. The more common emergency spillway types and their applicability are described in Table 1.

TABLE 1. COMMON TYPES OF EMERGENCY SPILLWAYS

GENERAL APPLICABILITY OF SPILLWAY TYPE				
SPILLWAY TYPE	**SMALL DAM**	**INTERMEDIATE AND LARGE DAMS**	**OFF-CHANNEL STORAGE FACILITIES**	**REGIONAL STORMWATER DETENTION PONDS**
Chute	✓	✓	✓	✓
Gated Chute		✓		
Gabion Lined Chute	✓		✓	✓
Culvert			✓	
Drop Inlet	✓		✓	✓
Labyrinth	✓	✓		✓
Open Channel	✓	✓	✓	✓
Side Channel		✓		

Spillway types which do not require human intervention for operation provide increased likelihood of proper performance during unusual flood conditions. These constraints often make unregulated, wide, open channel and chute spillways the preferred choices. Conversely, large projects designed for flood control typically employ gated spillways to provide greater regulation of the flood releases.

4.3.4 ENGINEERING CONCERNS:

An emergency spillway is a critical project element whose proper operation during extreme flood conditions is essential to prevent overtopping of the impounding barrier. The passage of extreme flood flows and the conditions associated with extreme floods pose several general engineering concerns and design issues which must be addressed. These conditions, concerns and issues include:

- Frequency of Operation

- Operator Intervention
- Resistance to Debris Blockage
- Positive Control of Discharge
- Limiting Erosional Damage
- Duration of Operation
- Need for Atmospheric Venting or Aeration of the Flow

Technical design information on these and related topics can be obtained from the design manuals and technical articles in the *References, Sections 4.2.5, 4.3.6*. Likewise, related topics are discussed in *Section 4.2 Principal and Service Spillways*.

4.3.4.1 Frequency of Operation

The selection of the frequency of operation of the emergency spillway is at the discretion of the design engineer. Common practice is to design the principal spillway to accommodate floods with a magnitude up to the 50 year or 100 year recurrence level or more. The emergency spillway would then be utilized to pass floodwaters in excess of the capacity of the principal spillway. This approach usually results in the emergency spillway operating only once or twice in the life of the project. This allows the concept of survivability to be used in designing erosional control features for the emergency spillway.

4.3.4.2 Operator Intervention

In general, spillways which do not require operator intervention (unregulated spillways) provide a greater margin of safety and predictability for proper operation during extreme flood conditions. In those cases where gated emergency spillways are utilized, proper procedures and decision points must be clearly defined in the Operation and Maintenance (O & M) Manuals.

Requirements/Minimums - Dam tenders must periodically review procedures and receive training as needed to attain a very high degree of reliability for operation of the project under extreme conditions. Dam tender(s) responsible for operation of gated spillways must be on-site during flood conditions. Remote, telemetered operation of any gates must have backup by on-site operators during extreme flood conditions.

4.3.4.3 Resistance to Debris Blockage

Floating debris often accompanies moderate and extreme flood flows particularly from heavily forested, steep, mountainous areas. Features of the spillway and approach area should be incorporated to allow passage of floating debris, or debris control features such as log booms should be utilized. The discussion of debris protection and control measures is contained in *Section 4.5 Debris Protection for Hydraulic Structures*.

4.3.4.4 Positive Control of Discharge

Measures need to be taken during the design process to provide for positive control of the discharge by establishing either one discharge control point or to provide a smooth transition between discharge control points. See Section 4.2.3.6 for further information.

4.3.4.5 Limiting Erosional Damage

Headward erosion along the spillway outlet channel is the primary mechanism which causes erosional breaching of unlined spillways[2]. Headward erosion normally initiates at channel locations where the channel bedslope flattens or changes abruptly such as at the spillway outfall.

As previously discussed in Section 4.2.2 Design Philosophy, sufficient measures must be taken to preclude erosional breaching of the emergency spillway. On small and intermediate sized dams, this can often be accomplished using erosion protection measures such as: vegetative linings; riprap or gabion linings; grouted riprap; and buried concrete erosion cutoff walls.

On large dams where large spillway discharges produce very high velocities, the only practical ways to limit erosion, particularly headward erosion, are to locate the spillway on erosion resistant bedrock or to construct channel linings from roller compacted concrete[16] or conventional reinforced concrete.

4.3.4.6 Duration of Operation

If survivability design concepts are used, the elapsed time that the spillway passes flow can be an important consideration. If the design event is a short duration thunderstorm event, then the spillway may only be subjected to a limited period of high energy erosional forces. In contrast, if the design event is a winter long-duration storm, the spillway may be subjected to a protracted period of erosion. The duration and intensity of erosional forces should be considered in the design of erosion protection features which limit but do not eliminate erosional damage.

4.4.3.7 Need for Atmospheric Venting or Aeration of the Flow

Venting requirements for culvert spillways, drop inlet spillways and morning glory spillways are discussed in Section 4.2.3.8. Issues regarding cavitation and aeration of high velocity flow are also discussed in that section.

4.3.5 EXPERIENCE/CURRENT ENGINEERING PRACTICE FOR VARIOUS SPILLWAY FEATURES

The following topics address issues which are frequently encountered during design and reflect common engineering design practice for various features of emergency spillways at small and intermediate size dams. Related topics are also discussed in *Section 4.2 Principal and Service Spillways*.

4.3.5.1 Vegetation Lined Open Channel Spillways

Grass lined emergency spillways are often practical solutions for small dams where climatic conditions will support a good grass cover. Wide, shallow, open channels with moderate slopes are usually selected to limit velocities and erosion damage. Information from the SCS[1,2] and Chow[11] provide guidance on the design of grass lined spillways.

4.3.5.2 Riprap Lined Open Channel Spillways

Experience has shown that loose riprap has very limited applicability on channel bedslopes steeper than about 20H:1V. Model tests conducted by the U.S. Bureau of Reclamation[10], Colorado State University[4] and Abt and Johnson[12] also indicate that loose riprap has low reliability in protecting embankments from overtopping flows when unit discharges are large. The combination of steep slopes, high energy, unevenly distributed flow, variability and segregation of riprap gradation during placement combine to restrict usage of loose riprap to more moderate channel slopes, velocities and unit discharges.

Recommendations - Use of loose riprap for channel protection should generally be restricted to slopes less than about 5 percent. Conventional riprap sizing criteria [13,14,15] should be applied conservatively. In addition, riprap should be augmented by the use of headward erosion cutoff walls to restrict unraveling of the riprap lining.

4.3.5.3 Gabion Lined Chute Spillways

Gabion lined chute spillways are sometimes used on small dams where the valley is narrow and the emergency spillway is placed on the embankment. This is often the case on small flood control dams in urban areas.

Requirements/Minimums - The typical mode of failure of gabions is by a loss of subgrade support. This occurs when velocities are sufficiently high to cause movement of the rockfill in the baskets, which in turn causes the baskets to deform. This exposes the underlying filter and subgrade to erosion. To preclude this occurrence, current design practice recommended by gabion manufacturers for gabion construction in high velocity areas is to underlay the gabion baskets with a non-woven geotextile material. The geotextile is then underlain with a sand and gravel filter which meets filter criteria for both the subgrade and the gabion rockfill. See also *Section 3.3A Filters*. Current literature on gabions[4] indicates maximum permissible velocities as shown I n Table 2.

TABLE 2 PERMISSIBLE VELOCITIES FOR GABIONS[4]

GABION BASKET THICKNESS	PERMISSIBLE VELOCITY
6 Inches	11 ft/sec
9 Inches	13 ft/sec
12 Inches	15 ft/sec
18 Inches	17 ft/sec
36 Inches	21 ft/sec

Additional buried gabions should be constructed at the spillway crest and toe of the slope to provide anchorage and act as shear keys.

4.3.5.4 Headward Erosion Cutoff Walls

Headward erosion cutoff walls can be effective at limiting the upstream migration of erosional damage. Ideally, the wall should penetrate to an erosion resistant layer of the foundation. The wall should also extend sufficiently far into the channel sidewalls to preclude an erosional end-run of the wall. In addition, the wall should be designed with a geometry which provides overturning resistance if the wall is partially undermined. This is usually accomplished by constructing the walls in the shape of a chevron, with the apex pointed upstream, or by adding counterforts to the wall.

Recommendations - A series of cutoff walls constructed from concrete, gabions or grouted riprap can be used in conjunction with grass linings or loose riprap to retard the erosional rate and provide adequate time for passing the design flood at small and intermediate dams.

Also, in these situations, it is often desirable to employ a channel layout where the deepest waters, highest velocities and greatest erosional potential occurs in those areas of the channel furthest from the dam. This can be accomplished by utilizing a minor cross-slope to the channel to place the deeper sections of the channel furthest from the dam. Care must also be taken in this arrangement to evaluate the effect of the increased erosional potential at the deepened section.

4.3.5.5 Culvert Spillways for Off-Channel Storage Facilities

Culvert spillways are often an economical solution for discharging excess floodwaters from off-channel storage reservoirs. The culverts should be sized to accommodate the design storm event in combination with the maximum diverted or pumped inflow.

4.3.6 REFERENCES

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13. U.S. Army Corps of Engineers, Shore Protection Manual, Volume II, Coastal Engineering Research Center, Fort Belvoir, Virginia, 1973, pp7-205.

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ASCE Journal of Hydraulic Engineering, Vol. 118, No.5, May 1990.

4.4 ENERGY STILLING BASINS

4.4.1 OBJECTIVE:
Provide a means for dissipating the energy and controlling the velocity of discharge exiting from spillways and outlet works to prevent damaging erosion and scour of the dam embankment, foundation and/or streambed.

4.4.2 APPLICABILITY:
Energy stilling basins are required for outlet works and principal spillways. Stilling basins may take the form of a simple riprap lined basin for outlet works required to pass small discharges, or may be elaborate reinforced concrete structures for principal spillways with large, high velocity discharges.

4.4.3 ENGINEERING CONCERNS:
Releases of large discharges from outlet works and spillways are usually characterized as having high energy levels. The discharge velocity is typically much greater than the velocity the receiving stream has previously experienced under natural conditions. Unless adequate energy dissipation is provided, severe erosion and scour of the foundation and/or natural streambed can result. Such erosion can progress in the upstream direction (headward erosion) and undermine and/or erode the spillway outfall. Unchecked headward erosion could jeopardize the integrity and stability of the dam.

The need for, and degree of, energy dissipation and/or erosion protection generally depends on several factors, including:

- The erosion resistance of the foundation or streambed material
- The velocity and magnitude of the discharge, particularly the unit discharge
- The frequency of occurrence and duration of both normal discharges and flood flows
- The type of spillway or outlet
- Environmental concerns (e.g. the need for fish passage and preservation of fish habitat)

4.4.4 DESIGN PHILOSOPHY:
Energy stilling basins are considered critical project elements for those situations where their failure could lead to a failure of the dam. Two design approaches are in common practice. The first approach sizes the stilling basin based on the maximum potential discharge the stilling basin would ever be required to accommodate. The second approach utilizes *Survivability* concepts to size the stilling basin and erosion protection features to achieve acceptable performance under extreme flood discharges. Both approaches are briefly described below.

Size of Energy Stilling Basin Based on Maximum Potential Discharge - In this approach, the size of the basin, and it's energy dissipation and downstream erosion control features are designed to withstand the maximum anticipated discharges with minimal erosional damage. Common applications for this approach are for hydraulic structures such as:

Outlet Works **-** For outlet works, the maximum discharge is governed by the reservoir head, conduit size and conduit valving arrangements. The maximum discharge is commonly used as the design discharge for sizing the stilling basin.

Principal Spillways on Large Dams with Erodible Foundations **-** At large dams with erodible foundations, the combination of large unit discharges and high velocities warrant a very conservative approach. In these cases, the discharge used to size and design the energy stilling basin is based on the maximum discharge from the spillway in response to the Inflow Design Flood (IDF).

Size of Energy Stilling Basin Based on Survivability Concepts - Conversely, in this approach it is allowable to accept substantial erosional damage during an extreme event, provided the integrity or stability of the dam is not jeopardized. The basic premise is to provide defense mechanisms, such as oversized stilling basin end walls and side walls, to preclude undermining and catastrophic erosion which could jeopardize the stilling basin and the dam in the event of an extreme flood event. This approach is often used at small and intermediate size dams where project owners are amenable to accepting a small risk of some potential future erosion damage and repair costs in exchange for savings in initial construction cost. Common applications of this approach include:

Principal Spillways on Small and Intermediate Size Dams - The limited duration and smaller magnitude of flood flows at small and intermediate size dams often allow the use of survivability concepts to be effectively employed. A design discharge corresponding to the 100 year flood peak discharge is commonly used for sizing the stilling basin and energy dissipation features.

Principal Spillways on Large Dams with Erosion Resistant Foundations - The presence of erosion resistant foundations may preclude the potential for headward erosion to undermine the stilling basin and essentially eliminate concerns for erosion to jeopardize the dam. In these cases, survivability concepts are directly applicable and there is discretion in setting the discharge for design of the energy stilling basin. Common practice is to use a discharge in the range of the 500 year recurrence flood for sizing the stilling basin.

4.4.5 DESIGN PRACTICE:

The hydraulic procedures used in design of energy stilling basins are well detailed in numerous textbooks[4,7] technical references[1,3,4,5,6,7] and design manuals[2,8,9]. The purpose of the following sections is to provide some background reference information and to highlight some important design considerations.

4.4.5.1 Common Types of Energy Stilling Basins

A multitude of energy stilling basins and dissipators have been developed for spillways and outlets over the years. The selection of a specific type of dissipator is left to the designer. In general, the stilling basins can be grouped into the following categories:

ENERGY STILLING BASINS FOR CHUTE SPILLWAYS

- USBR Types I, II and III - Hydraulic Jump Type Basins[1,2]
- Saint Anthony Falls (SAF) Stilling Basin[2,4]
- USBR Type IX - Baffled Chute Spillway[1,2]
- Contra Costa Basin (for small dams)[3,8]

ENERGY STILLING BASINS FOR CONDUIT SPILLWAYS OR OUTLETS

- USBR Type VI - Impact Type Basin[1,2,9]
- Contra Costa Basin[3,8]
- SCS Plunge Pool[5]
- USBR Plunge Basin[1]

ENERGY STILLING BASINS FOR HIGH OVERFLOW SPILLWAYS

- USBR Type VII - Submerged Bucket[1]
- Flip Bucket Energy Dissipator[1]

4.4.5.2 Headward Erosion Protection

Headward erosion protection features are necessary to prevent undermining and other erosional damage of the stilling basin. These features are considered critical project elements and must be designed to withstand the maximum design flood discharge for spillways, or the maximum discharge capacity for outlet works. This is normally accomplished by the construction of oversized erosion cutoff walls which extend beneath, and outward from, the end of the stilling basin. The cutoff walls should be extended sufficiently far beneath the stilling basin end sill to either penetrate an erosion resistant material, or be extended a conservative distance below any anticipated scour depth which could result from passage of the maximum design flood discharge.

4.4.5.3 Erosion Protection for Outlet Channels

Erosion protection in the form of riprap, grouted riprap, or gabions is normally needed downstream of the stilling basin to protect the natural streambed from erosion and scour. Survivability concepts are often used in selecting and designing the protection in this area - provided adequate headward erosion protection for the stilling basin has been incorporated in the design. A number of references[8,10,11] on riprap are available to assist in the selection of the riprap size, gradation and lining thickness (see also *Section 3.5 Embankment Erosion Control and Wave Protection*).

4.4.5.4 Tailwater Conditions

Analysis of the tailwater conditions for the stilling basin is essential to the proper design of many types of stilling basins. A tailwater discharge rating curve depicting the relationship between discharge and the corresponding water surface elevation in the receiving stream is necessary to evaluate the performance of the basin and in determining if/when "sweepout" of the basin will occur. Information from the analysis of stilling basin exit velocities and depths is also used in evaluating the necessary penetration of headward erosion cutoff walls and in sizing riprap or other channel erosion protection measures.

4.4.6 REFERENCES

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4.5 DEBRIS PROTECTION FOR HYDRAULIC STRUCTURES

4.5.1 OBJECTIVES:

Prevent floating debris in the reservoir from accumulating and blocking the spillway entrance. Prevent submerged debris from blocking and/or entering low level outlet works.

4.5.2 ENGINEERING CONCERNS:

Spillway - Natural watersheds that are heavily forested, and urbanized watersheds can produce a significant quantity of floating debris, especially during floods. This debris can accumulate in the spillway entrance or near the discharge control section and reduce the spillway discharge capacity. As a result, the reservoir freeboard would be reduced, and in extreme cases, the dam could be overtopped.

Outlet Works - Submerged logs and debris can clog the intake or the outlet works, reducing the discharge capacity. In addition, debris could enter the conduit and damage control gates and valves, or hamper normal operation.

4.5.3 DESIGN PHILOSOPHY:

Spillways - For spillways, debris protection is a critical element of design. Failure of the debris protection could lead to an accumulation of debris, reduction of spillway discharge capacity, and overtopping and failure of the dam. Thus, trashracks, debris barriers and log booms should be designed conservatively, to minimize the possibility of spillway blockage.

Outlet Works - For outlet works, debris protection is an important, but non-critical element of design. Failure of debris protection features for outlet works could result in damage or reduce the discharge capacity, but would not lead to a dam failure. Outlet works debris protection should be designed with serviceability in mind by making provisions to allow clearing of the debris as needed.

4.5.4 DESIGN PRACTICE:

4.5.4.1 Spillways

Debris protection features for spillways would include such items as floating log booms or fixed in-place debris barriers. The choice of debris protection is dependent on the type of spillway used. Drop inlet (morning glory) spillways and conduit spillways are very susceptible to debris blockage. For these spillways, cage type trashracks are typically installed at the spillway entrance. The spacing between the trashrack bars is normally sized so that smaller organic debris such as leaves and small branches can pass through, while larger debris is retained on the rack. In order to prevent the larger debris from choking off the spillway flow, the trashrack usually has a surface area several times greater than the area of the spillway entrance.

Open channel spillways, (e.g chute and overflow spillways) are less susceptible to debris blockage. For these spillways, floating booms are normally used. Floating booms are often constructed of treated timber logs or hollow steel pipe sections with the ends welded shut, which are chained together and anchored to the shore upstream from the spillway.

In some cases, debris barriers can be an economical alternative to log-booms. These structures serve the same purpose as log-booms and typically consist of a series of posts or piles driven into the spillway approach channel to snag floating debris, particularly large logs and downed trees, before they enter the spillway.

Recommendations/Requirements/Minimums - In those cases where floating debris is anticipated to be a problem during extreme floods, measures must be taken to minimize spillway blockage and the associated reduction in discharge capacity. Standard methods of minimizing debris problems include:

Log Booms **-** Log booms should be located sufficiently far upstream of the spillway entrance to be in a reservoir location where the approach velocity is relatively low. This will reduce the forces on the boom anchors and reduce energy losses in the approach channel. Ideally, approach velocities under design flood conditions should be less than 1 ft. per second. Alternatively, measures must be taken to provide assurance of the effectiveness of the boom during flood conditions. Log booms should also be constructed with sufficient freedom of movement to be capable of floating freely up to the maximum design reservoir level.

Deepened Approach Channel **-** The approach channel/area to the emergency spillway should be sufficiently deep that floating debris does not get hung up on the channel bed and partially block the approach to the spillway.

Mechanical Removal **-** In those cases where control or passage of the debris cannot be accomplished with a high degree of reliability, mechanical methods of removal may be used. Prior arrangements should be made to have equipment available during extreme flood conditions which is capable of physical removal or control of the debris.

Over-Design of Spillway Capacity and/or Freeboard **-** The spillway discharge capacity may be over-designed relative to that needed for accommodating the design flood to account for partial debris blockage. In addition, the dam crest elevation can be raised to give greater floodwater storage capacity to account for the reduction in discharge capacity from partial debris blockage.

Trashracks **-** Tightly spaced bars on trashracks, resembling grating, are easily clogged by small debris. Reduced discharge capacity on culvert spillways and drop inlet spillways from debris collection on grating type trashracks have been responsible for several dam failures in the U.S.

To minimize this potential problem, trashracks should be designed with a surface area of from 3 to 5 times that of the entrance area which it is protecting. Bar spacing on the trashrack should be as large as practicable, subject to the constraint that whatever passes through the trashrack must freely pass through the conveyance conduit or channel. For projects located near developed areas, bar spacings must also not be so large as to pose an attractive nuisance and be a threat to the safety of children who could fall through the openings.

Debris barriers **-** Debris barriers should be constructed with adequate cross-sectional area for debris collection and be placed sufficiently far upstream of the spillway mouth to minimize restriction of the flow.

4.5.4.2 Outlet Works

The design of a trashrack at the entrance to the conduit for the outlet works is dependent on several factors, including: the size of the conduit; the likely magnitude of debris, both quantity and dimensions;

the type of control device; the need for excluding small debris from the outflow; and the needs of the water users.

Recommendations:

Trashracks - Trashracks for outlet works should be oversized to provide for a debris collection area of 1.5 to 5 times that of the intake opening, dependent upon the magnitude of the expected debris problem and the ease with which debris can be cleared. Bar spacing for trashracks for outlet works can be determined in the same manner as that discussed above for spillways.

4.5.5 PAST EXPERIENCE:

Timber log booms tend to become waterlogged with age and partially sink, allowing debris to pass over the top of the boom. Thus, timber booms must be replaced several times during the life of the facility. Alternative materials for log booms would include using sections of aluminum pipe or thin wall steel pipe, with the ends welded shut to allow the sections to float.

Off channel diked impoundments, such as sewage lagoons, city reservoirs and waterski ponds, where minimal debris can enter the reservoir, often require little or no debris protection measures.

4.5.6 REFERENCES

TRASHRACKS FOR OUTLET WORKS

1. U.S. Bureau of Reclamation, Design of Small Dams, Sections 10.9 and 10.15, Denver, CO, Third Edition, 1987.

DEBRIS BARRIERS

2. Washington State, Dept. of Transportation, Highway Hydraulic Manual, Section 3.06.00, M23-03(HB)

4.6 RESERVOIR FREEBOARD

4.6.1 APPLICATION:

The magnitude of reservoir freeboard is used to establish the elevation of the dam crest relative to the normal or maximum operating levels of the reservoir.

4.6.2 OBJECTIVES:

Provide a buffer for protection against dam overtopping by reservoir waters for those cases where overtopping could produce serious damage or erosional failure of the embankment.

Reservoir freeboard is evaluated at two reservoir conditions - normal pool condition and maximum pool during passage of the inflow design flood. The more stringent design considerations at these two reservoir conditions are used to establish the elevation of the dam crest. Freeboard at these two conditions is defined as:

Normal Freeboard - the vertical distance between the normal storage elevation (normal high pool level) and the dam crest elevation.

Minimum Freeboard - the vertical distance between the maximum reservoir water surface elevation attained during passage of the inflow design flood and the dam crest elevation.

4.6.3 DESIGN PHILOSOPHY:

Earthen Dams - It is standard practice for reservoir freeboard to be determined in a conservative manner. However, while erosional damage has occurred at earthen dams due to intermittent overtopping by wave action, there are no known cases where wave erosion has resulted in dam failure. For this reason, the determination of reservoir freeboard is considered an important but not critical element of design. Therefore, it has been standard practice that conservative design conditions, but not necessarily theoretical maximum design conditions, are used to determine the magnitude of freeboard needed.

Concrete Dams - In most cases, concrete dams can withstand significant overtopping by either waves or floodwaters without damage. Exceptions are when the abutments or foundation are comprised of materials which are susceptible to erosional damage and when erosion could jeopardize the stability or integrity of the structure or allow release of the reservoir contents. Standard practice has been to use conservative procedures and judgement in the determination of reservoir freeboard for concrete dams.

Engineering Judgement - It will be seen later that there are many factors to be considered in selecting the reservoir freeboard and that some factors are not easily quantified. Thus, considerable engineering judgement is necessary when evaluating the various factors and in arriving at a final determination/selection of the reservoir freeboard.

4.6.4 ENGINEERING DESIGN CONSIDERATIONS:

Determination of the magnitude of reservoir freeboard should be based on a number of considerations, any one of which, or combinations of which, may govern in a particular application. There are four general categories of considerations to be evaluated in selecting the amount of reservoir freeboard. Each of these considerations are listed below and discussed in the following sections.

- Wind/wave action for the cases when the reservoir is at normal pool condition and when the reservoir is at its maximum level during passage of the inflow design flood

- Embankment settlement

- Uncertainties in design or uncertainties associated with project operation

- Exposure to geologic hazards

4.6.4.1 Wind/Wave Action

In most cases, the magnitude of wind generated waves is the dominant factor in determining reservoir freeboard. The magnitude of wind generated waves is, in turn, primarily determined by the magnitude and direction of the extreme winds which can affect the site. A further complication is that site specific topographic and meteorologic characteristics greatly influence the behavior of the winds at a given site. Site specific characteristics include such features as: the orientation of the upstream face of the dam relative to the likely direction of extreme winds; the project setting (sheltering or funneling of winds by surface topography); and the seasonality of the extreme winds relative to the likely reservoir water level at the time of extreme wind. These complications not withstanding, standard procedures for estimating the effects of wind/wave action and required freeboard are discussed in the following sections.

Extreme Wind Characteristics - As mentioned previously, the magnitude and direction of extreme sustained winds are dominant factors in assessing wind/wave action on reservoirs. Unfortunately, most data available on extreme winds are for instantaneous maxima. It is common knowledge that wind gusts and other short duration wind bursts are larger than sustained winds. Thus, procedures are needed to estimate sustained wind speeds from available data sources.

Very little site specific extreme wind magnitude-frequency-duration data[10,11] exists for locations in Washington. Most information currently available was obtained from a small number of sites[8,9,10,11,12,13] and reflects generalized characteristics for both magnitude-frequency-duration and predominant direction for observed extreme winds.

Analyses of wind magnitude-duration characteristics for Washington[10] indicates that sustained wind speeds can be estimated based on the magnitude of recorded instantaneous wind speeds. The one minute sustained wind speed can be estimated based on the instantaneous value[9] by:

$$W_1 = (W_{inst} - 10) \qquad (1)$$

where: W_1 = One Minute Sustained Wind Speed (MPH)
W_{inst} = Instantaneous Wind Speed (MPH), at 25 Feet above the Ground

Average sustained wind speeds for durations greater than 1 minute can be obtained as a ratio of W_1 based on Figure 1[9].

Likewise, limited data is available on the directions of observed extreme winds[10,11]. Figure 2 depicts the published data for selected sites. It can be seen that most extreme winds have a southern or western component which usually reflects the source as being winter storms originating off the Pacific Ocean. Nonetheless, in most applications, local information will be needed to supplement the data shown in Figure 2.

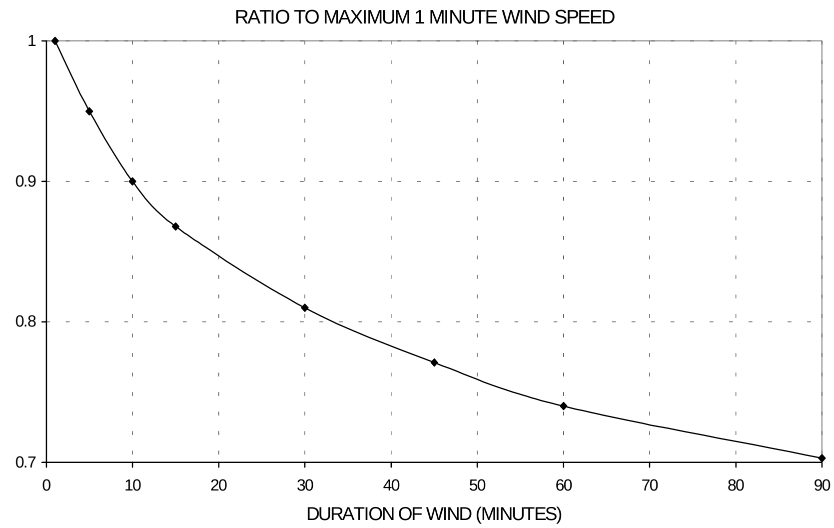

FIGURE 1 - GENERALIZED WIND MAGNITUDE-DURATION CHARACTERISTICS FOR SITES IN WASHINGTON

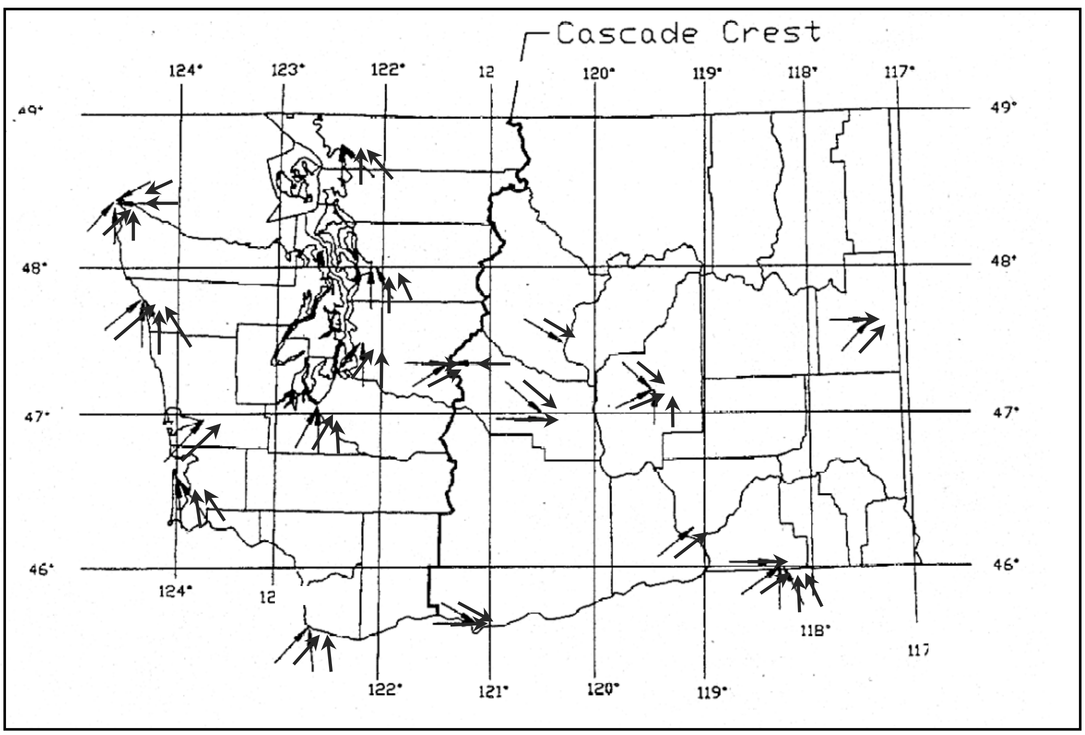

FIGURE 2 - DIRECTION OF EXTREME WINDS AT SELECTED WASHINGTON SITES

Design Wind Speed - As previously discussed, the computation of freeboard is an important, but not critical, element of design and thus there is some latitude in selecting the design wind speed. A review of pertinent literature[1,4,5,6,12,13] indicates that a variety of procedures have commonly been used for selecting design wind speeds.

Standard practice is to use a conservative wind speed for determining normal freeboard and a lesser wind speed for determining minimum freeboard. Recommendations in the literature for selecting a design wind speed range from using the observed historical maximum wind[4,5] to using a 100 MPH wind speed[6] at the normal pool condition.

Values cited in the literature for evaluating minimum freeboard range from a generic 40 MPH[5] to 50 MPH[6] wind. The use of a smaller wind speed for computing minimum freeboard is reasonable considering that the maximum reservoir level produced by the inflow design flood will generally occur at some time after the peak of the causative storm, and concurrent winds, have passed the reservoir site. In addition, high winds do not necessarily occur in combination with an extreme precipitation event.

Recommendations - Design Wind Speed - When local data on the magnitude and direction of historical extreme winds are available, it should be examined first and incorporated in the decision process for determining the design wind speed. As guidance, the historical maximum wind speed or a wind speed with an annual exceedance probability of 0.01 may be used in assessing the requirements for normal freeboard. Additional guidance on selecting design wind speeds can be obtained by examining engineering codes[12,13] for use at other important structures comparable to dams. For evaluating minimum freeboard, the mean value of observed annual extreme winds or a wind speed with an annual exceedance probability of 0.50 may be used.

In addition, it should be recognized that sustained winds, not instantaneous maximum winds are needed for analyses of wind/wave action. The wind duration chosen should be sufficient to fully develop the reservoir waves and should represent a period of time over which wave action could pose an erosional concern. In most cases for reservoirs with a fetch less than 1 mile, a duration of 20 to 30 minutes is appropriate. Information on computing the time required to fully develop reservoir waves is contained in COE report[1]. Equation 1 and Figure 1 data may be used to estimate the value of the sustained wind for a chosen duration.

When local data is not available, Table 1 values may be used to assess the requirements for reservoir freeboard. These values represent generalized 20 minute sustained wind speeds with an annual exceedance probability of 0.01. They are somewhat more conservative than wind speeds used in the Uniform Building Code[12,13] and exceed most historical maxima. The values shown for minimum freeboard represent generalized 20 minute sustained wind speeds with an annual exceedance probability of 0.50. Again, caution should be used in areas where valleys and other local topographic features can concentrate winds.

Design Practice - Wind/Wave Action - Computation of the freeboard required to contain wind/wave action is accomplished by evaluating three wind/wave conditions - wind setup, the significant wave height and wave runup. These wind/wave conditions are primarily determined by the magnitude of the design wind speed and the effective fetch and are defined below:

Wind Setup the vertical rise in the reservoir level above stillwater level (measured at the dam face) caused by wind stresses acting on the surface of the water.

Significant Wave Height the average height of the one-third highest waves of a given wave group. It should be noted that waves are not uniform in height but rather occur over a range of heights in a given wave group.

TABLE 1 - SUGGESTED DESIGN WIND SPEEDS FOR AREAS OF LIMITED DATA

GENERALIZED DESIGN WIND SPEEDS			
FREEBOARD CONDITION	**WASHINGTON LOCATION**		
	COASTAL AREAS	WESTERN WASHINGTON	EASTERN WASHINGTON
NORMAL FREEBOARD	70 MPH	60 MPH	50 MPH
MINIMUM FREEBOARD	50 MPH	30 MPH	30 MPH

Wave Runup the vertical height above stillwater level that waves reach upon impacting the dam face.

Effective Fetch the unobstructed horizontal distance measured along the reservoir surface towards the dam face over which winds can generate waves or wind setup. More detailed information on computing effective fetch can be obtained in COE reports[1,2,3,6].

Design Wind Speed the average wind speed, sustained over some specified duration, which is used in computing wind setup, significant wave height and wave runup.

Figure 3 depicts the general relationship between reservoir freeboard, significant wave height, wind setup and wave runup as it applies to dams for use in freeboard analyses for wind generated waves.

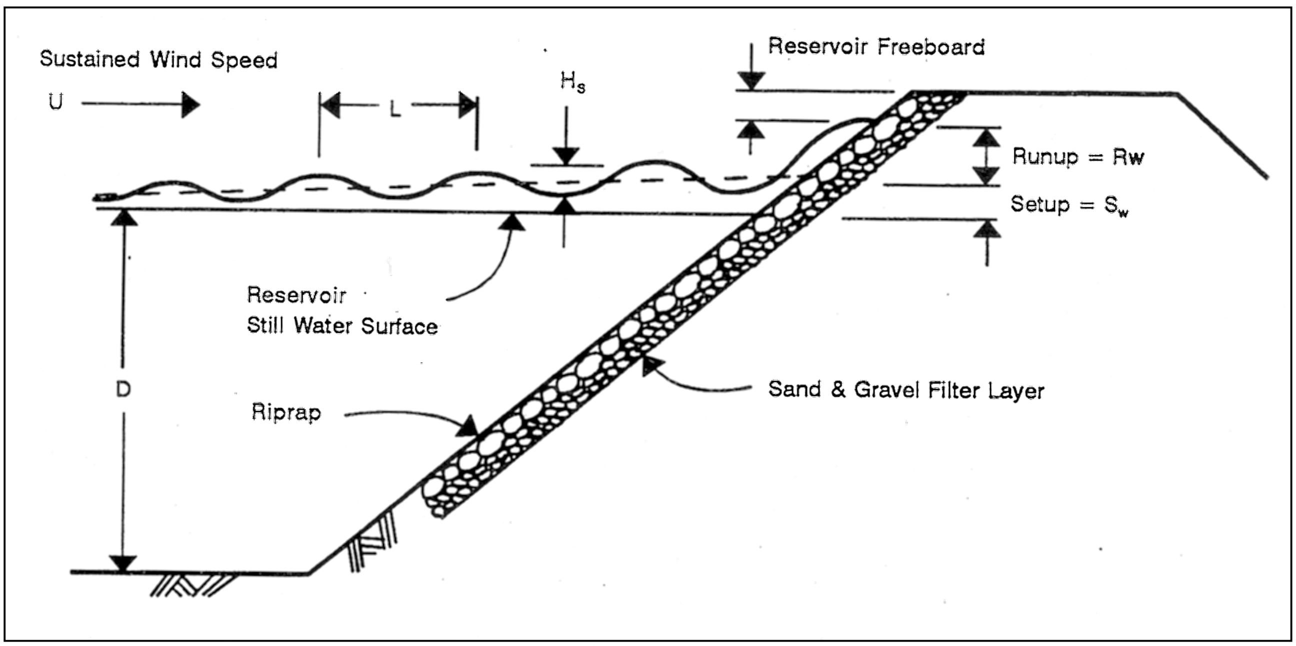

FIGURE 3 - DEPICTION OF RESERVOIR FREEBOARD AND WIND/WAVE EFFECTS

Referring to Figure 3, the freeboard needed to contain wind/wave action is computed as:

$$F_w = S_w + R_w \tag{2}$$

where:

F_w = Freeboard needed to contain wind/wave action (Feet)
S_w = Wind Setup (Feet)
R_w = Wave Runup (Feet)

Wind Setup - Computation of wind setup[4] is depicted in equation 3. In general, wind setup is small relative to the magnitude of the significant wave height - except for large reservoirs.

$$S_w = \frac{U^2 F}{1400D} \tag{3}$$

where:

S_w = Wind Setup (Feet)
F = Effective Fetch (Miles)
U = Design Wind Speed (MPH) at 25 Feet above the Reservoir Surface
D = Average Reservoir Depth (Feet)

Wind Generated Waves - The magnitude of waves which can be generated by wind is generally not a significant consideration for small reservoirs. However, at large reservoirs with long effective fetches, large amounts of freeboard may be needed to contain wave action. Table 2 has been prepared based on U.S. Army Corps of Engineers (COE) report ETL 1110-2-8[1] which can be used to estimate the significant wave height (H_s) generated by sustained winds.

TABLE 2. ESTIMATION OF WIND GENERATED WAVES

SIGNIFICANT WAVE HEIGHT (FEET)							
SUSTAINED WIND SPEED	EFFECTIVE FETCH (MILES)						
	.05	.10	.25	.50	1	2	5
20 MPH	0.20	0.30	0.45	0.60	0.85	1.20	1.75
30 MPH	0.35	0.45	0.65	0.90	1.30	1.75	2.70
40 MPH	0.45	0.60	0.90	1.25	1.75	2.45	3.70
50 MPH	0.55	0.75	1.15	1.60	2.20	3.00	4.70
60 MPH	0.65	0.90	1.40	1.90	2.70	3.60	5.70
70 MPH	0.75	1.15	1.60	2.25	3.20	4.30	6.70
80 MPH	0.90	1.20	1.85	2.60	3.70	5.00	7.80
90 MPH	1.00	1.35	2.10	3.00	4.20	4.70	8.90
100 MPH	1.15	1.50	2.40	3.40	4.70	6.40	10.00

Estimation of the significant wave height may be made based on Table 2 values or on procedures contained in COE reports[1,2,3].

Wave Runup - The amount of wave runup (Figure 3) is dependent on the wave height, the steepness of the wave (significant wave height (H_s) relative to wave length (L), the ratio of (H_s/L)) and upon the slope and roughness of the dam face. Wave runup (R_w) can be estimated by:

$$R_w = C_r(H_s) \tag{4}$$

where: C_r = Coefficient of Wave Runup

Estimates of the coefficient of wave runup were obtained from information contained in COE report[1] and are displayed in Table 3. Additional information on wave runup can be found in COE reports[1,2] and Saville[7].

TABLE 3. VALUES OF THE COEFFICIENT OF WAVE RUNUP

COEFFICIENT OF WAVE RUNUP (C_r)							
DAM FACE	SLOPE OF UPSTREAM FACE OF DAM (H:V)						
	Vertical	1.5:1	2:1	3:1	4:1	6:1	10:1
Concrete Facing	1.7	2.5	2.2	1.8	1.3	0.8	0.5
Grass Lined	---	2.3	2.0	1.6	1.2	0.8	0.5
Riprap	---	1.4	1.3	1.2	1.0	0.7	0.4

4.6.4.2 Embankment Settlement

The amount of reservoir freeboard may need to be increased to allow for consolidation/settlement of the embankment and/or foundation following construction. It is standard practice to either overbuild the embankment to allow for settlement or to monitor settlement and raise the dam to its design elevation after settlement has occurred.

Embankment settlement due to earthquake loadings is also possible and may be a significant design consideration in seismically active areas (see Section 2.3).

4.6.4.3 Uncertainties in Project Design or Operation

Freeboard is also used to provide a factor of safety against uncertainties in design or uncertainties associated with project operation. A number of considerations fall into this category and may be grouped according to usage for assessing normal or minimum freeboard.

Uncertainty Considerations at Normal Pool Condition - The following issues/items should be considered in addition to wind/wave action in determining normal freeboard:

- Site specific operational constraints
- Likelihood of elevated reservoir levels due to misoperation or failure of hydraulic features of the project
- Erodibility of the embankment from wave action and/or wave overtopping of the dam

Uncertainty Considerations at Maximum Pool Condition - The following issues/items should be considered in addition to wind/wave action in determining minimum freeboard during passage of the inflow design flood:

- Potential for partial debris blockage of the spillway(s)
- Uncertainties in the computed magnitude of the inflow design flood
- Spillways which have discharge characteristics (such as some drop inlet spillways) where increases in reservoir levels do not produce significant increases in spillway discharge
- Site specific operational constraints during extreme flood events
- Likelihood of elevated reservoir levels due to misoperation or failure of hydraulic features

during an extreme flood event
- Erodibility of the embankment from wave action and/or wave overtopping of the dam

4.6.4.4 Geologic Hazards

The geologic setting in Washington poses some unusual considerations which may need to be assessed during the planning and design stages of the project. While geologic hazards are not design considerations at most sites, they can be significant concerns at some projects. The following list identifies those concerns which could influence the magnitude of freeboard required at a given project.

- Waves produced by landslides into the reservoir
- Seiches produced by earthquake motions
- Mudflows and other hazards associated with volcanos

4.6.5 DESIGN MINIMUMS

As indicated in the previous discussion, there are numerous factors to be considered in selecting freeboard. Table 4 has been prepared to provide initial guidance in selecting freeboard. These values are to be considered as minimums and are not intended to supersede larger values obtained from thorough analyses of wind/wave action and engineering judgement used in assessing all pertinent factors.

TABLE 4. DESIGN MINIMUMS IN SELECTING RESERVOIR FREEBOARD

RESERVOIR FREEBOARD (FEET)	SMALL DAM	INTERMEDIATE DAM	LARGE DAM
NORMAL FREEBOARD	2.00	3.50	5.00
MINIMUM FREEBOARD	0.50	0.75	1.00

4.6.6 REFERENCES

1. U.S. Army Corps of Engineers, Computation of Freeboard Allowances for Waves in Reservoirs, ETL 1110-2-8, Office of the Chief of Engineers, Wash. D.C., 1966.

2. U.S. Army Corps of Engineers, Shore Protection Manual, Volumes I,II,III, Coastal Engineering Research Center, Fort Belvoir, Virginia, 1973.

3. Smith, J.M., U.S. Army Corps of Engineers, Wind-Wave Generation on Restricted Fetches, Coastal Engineering Research Center, Waterways Experiment Station, Vicksburg, MS., 1991.

4. Johnson, S.H., and Anderson, D.C., Environmental Protection Agency, Freeboard Determination and Management in Hazardous Waste Surface Impoundments, Hazardous Waste Engineering Research Center, Cincinnati OH, Sept, 1988.

5. Cassidy, J.J., Reservoir Freeboard Design, Fundamental Hydraulics and Hydrology of Dam Design, Short Course - University of Missouri, Rolla, 1981.

6. U.S. Bureau of Reclamation, Design of Small Dams, Third Edition, 1987, Denver, CO, pp255-258.

7. Saville Jr., T., An Approximation of the Wave Runup Frequency Distribution, Proceedings 8th Conference on Coastal Engineering, Mexico City, 1962.

8. Thom, H.C.S., New Distribution of Extreme Winds in the United States, ASCE Journal of the Structural Division, July 1968.

9. Simiu,E., Changery, M.J., and Filliben, J.J., Extreme Wind Speeds at 129 Stations in the Contiguous United States, U.S. Department of Commerce, National Bureau of Standards, March, 1973.

10. U.S. Army Corps of Engineers, Seattle District, Unpublished Data on the Magnitude-Frequency-Duration of Observed Winds at Selected Stations in Washington, 1987.

11. Washington State University, Agricultural Extension Service, Washington Climate, Multiple Volume Series by County.

12. International Conference of Building Officials, Uniform Building Code, 1988.

13. ASCE, Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, July, 1990.

CHAPTER 5 - STRUCTURAL ELEMENTS AND ISSUES

5.1 OBJECTIVE:

To insure that concrete meets minimum durability levels and performs as intended for the life of the project.

5.2 APPLICABILITY:

This discussion deals primarily with spillway slabs and walls, chutes, energy dissipaters, and other miscellaneous project appurtenances constructed of concrete. It is not intended to serve as a guide for impounding barriers built using mass or roller-compacted concrete, even though many of the same recommendations would be applicable.

5.3 DESIGN PHILOSOPHY:

Combining proper materials and good construction techniques usually add little extra cost to a project. The "payback" in increased design life and/or lowered maintenance costs, generally well outweighs any expenditures associated with the added attention to design and construction control.

5.4 RECOMMENDED DESIGN PRACTICE:

Concrete structures for dams are subject to an extreme environment that can greatly shorten their service life. Specifically, concrete will be exposed to: hydraulic pressures, wet-dry and freeze-thaw cycles, abrasion and erosion from sediment-laden water, and static and seismic loadings. Repairing deteriorated concrete structures frequently approaches efforts similar in scope to original construction, with all of the attendant costs, dangers, and environmental consequences. The difficulties associated with repairing concrete argue strongly for undertaking every cost-effective, practical step that will improve the overall durability and service life of concrete structures.

Given the potential for an extended life of some 100 years or more, it is prudent to assume that it will be necessary to retrofit the structure at some future time. Accordingly, measures to minimize the difficulties and scope of the work necessary to restore the structure should be incorporated into the design.

The following four areas in the DSO's experience are too often inadequately addressed:

- Proper concrete mix design.
- Good construction practice.
- Sufficient concrete cover on reinforcing steel.
- Provision of control joints to limit cracking.

Concrete Mix Design Recommendations - One of the major threats to concrete in a damp or wet environment is the deterioration resulting from cycles of freezing and thawing of the moist concrete. Saturated concrete forms ice within the pore structure when it is cooled to the point of freezing. Since water expands upon freezing, hydraulic pressures build up in the pore structure. If there is no relief for these pressures or the strength of the concrete is low, the concrete begins a slow deterioration from microcracking. This increases the permeability of the concrete locally which hastens the deterioration process as the volume of water available for freezing is increased. The surficial concrete becomes weak and drummy and the net structural section is decreased. Eventually this process can significantly weaken the structure. Usually, a more immediate effect is to degrade cover concrete over reinforcing steel to the point where corrosion of the reinforcing becomes the most serious consequence. Rust and spalling over the bars both decreases the amount of steel and lessens bond, reducing the effectiveness of the reinforcing.

The principal method of minimizing freeze-thaw damage is the utilization of an air entraining agent in the concrete. Secondary measures include:

> **Using the lowest practical water/cement ratio (w/c ratio) -** A superplasticizer is normally employed to reduce water in the mix, while still providing the desirable level of "workability" of the mix.
>
> **Selecting a cement content based on both *strength and durability concerns*.**
>
> **Reducing the permeability of the concrete -** Increased cement content, proper concrete consolidation by vibration, with a suitably graded aggregate, limit water saturation of concrete. This in turn limits the mass of concrete affected by freeze-thaw action.

In some instances, designers have limited the cement content because of their concerns of heat buildup from cement hydration and attendant temperature-induced cracking. However, such concerns are primarily associated with mass concrete pours. This is not normally an issue with the wall/slab type structures discussed here, except during "hot weather," as defined in ACI 318-89[1].

AIR-ENTRAINMENT[a]

Nominal Max. Aggregate Size (inches)	Air Content (%)	
	Severe Exposure	**Moderate Exposure**
3/8	7½	6
1/2	7	5½
3/4	6	5
1	6	4½
1	5½	4½
2	5	4
3	4½	3½

[a] Reference 3, Table 5-4

MAXIMUM WATER-CEMENT RATIOS[a]

Exposure Condition	Max. W/C Ratio (by Weight)
Concrete usually inundated or Dry	0.50
Freeze and Thaw Zone Concrete Frequently Moist	0.45

[a] Excerpts from ACI 318-89, Table 4.1.2

CEMENT CONTENT[a]

Exposure (Based on # of Freeze/Thaw Cycles)	Cement Content, lb. per cu. yd.[b]
Severe[c] Freeze-Thaw	564
Moderated[d] Exposure	517
Little or No Exposure	470

a Does not apply to lean concrete fills

b Reference 3, Chapter 7, Proportioning normal weight concrete.

c Severe exposure is "one where, in a cold climate the concrete may be in almost continuous contact with moisture prior to freezing..", Reference 1.,

d Moderate exposure is one where, in a cold climate, the concrete will be only occasionally exposed to moisture prior to freezing..", Reference 1.

Good Construction Practices - It should be recognized that a suitably designed concrete mix in the truck can be enhanced or compromised by construction practices in placing it and how it is treated as it hydrates. Ideally, construction practices will be geared to: enhancing surface curing, minimizing the bleeding of mix water to surfaces and thereby increasing the w/c ratio in this zone, and maximizing concrete density.

- Proper curing of concrete is primarily a matter of facilitating the process of hydrating the surficial zone of concrete. Specifically, the goal is to assure that sufficient water remains of the concrete mix water to complete the hydration process. This normally is accomplished by either periodically flooding the surface to replace water lost through evaporation or sealing the surface to minimize evaporative losses. Where a high degree of hydration is achieved, the concrete exhibits a reduced permeability, increased erosion resistance and a lower susceptibility to the leaching of the cementious elements from the cement paste. One common failure the DSO has observed that potentially lessens the degree of concrete hydration is the use of wooden forms that have not been sufficiently prewetted. The wood tends to absorb water from the surface layer of the concrete, removing water necessary for proper hydration of the cement.

- Steel troweling densifies slab surfaces and renders them less permeable. Proper consolidation of internal concrete accomplishes the same thing, increasing the density and lessening permeability of the interior concrete. Removal of surface irregularities by troweling will also improve concrete erosion and abrasion resistance.

5.5 REQUIRED DESIGN PRACTICE:

Required Minimum Concrete Cover - Research has shown that carbon dioxide in the atmosphere reacts with the surficial zone of concrete, replacing calcium hydroxide and tobermorite gel ($3CaO \cdot 2SiO_2 \cdot 3H_2O$) with calcium carbonate. This reduces the pH of the affected concrete significantly. The pH change transforms the affected concrete from an environment that inhibits corrosion of the reinforcing to an environment that is potentially corrosive. The degree of corrosion of the reinforcing is a function of the availability of oxygen and the quantity of absorbed or capillary water around the steel. Key to minimizing corrosion of the reinforcing is achieving a dense, low-permeability, concrete cover. This restricts the penetration depth and the concentrations of those elements driving the corrosion process beneath the concrete surface. As one would expect, research shows that proper concrete mix design and construction practices are crucial to achieving dense, low permeability concrete. Specifically, research on factors influencing carbonation depth showed an increase in carbonation depth for weak concretes, low cement content mixes and in environments with relatively high carbon dioxide levels, i.e. urban settings. Where particularly aggressive conditions are anticipated, the designer may wish to consider the use of epoxy-coated reinforcing to prolong the service life.

The preceding discussion focused on concrete cover as a deterrent to carbonation and associated corrosion of the reinforcing. A number of designers routinely increase cover depths over ACI minimums in this harsh environment. This is done, among other reasons, to provide a greater service life by increasing the tolerable loss of cover before the reinforcing is affected. Furthermore, it facilitates the task of placing a concrete overlay. Thus, the recommendations that follow are **minimums**. Increasing the minimum cover will likely improve life cycle costs for concrete elements of the project.

CONCRETE COVER FOR STEEL REINFORCEMENT[1]

Location	Minimum Cover, Inches
Concrete cast against and permanently exposed to earth (or backfill)	3
Concrete exposed to earth or weather	2
Concrete not exposed to weather or in contact with ground	1½

[1] Reference 1, 7.7 - Concrete Cover for Reinforcement

Joints (General Discussion) - Concrete undergoes many volumetric changes during its life. Shrinkage occurs as the result of the loss of moisture, carbonation, and cooling-related contraction. Expansion occurs as the result of creep, the absorption of moisture and temperature increases. Cracks are the result of these volumetric changes that take place in the presence of restraint. The concrete is not free to expand or contract and stresses, generally tensile, develop in excess of the strength of the concrete.

In the concrete construction covered in this chapter (principally walls and slabs), the designer usually employs a single joint to accommodate expansion and contraction of concrete members. While a rational analysis is possible for joint spacing, in practice calculated joint spacings are routinely reduced to account for non-uniform subgrade restraint and the potential for high thermal gradients. For most walls and slabs, joint spacing is on the order of 15 to 40 feet. Again, smaller spacings are used when relatively large thermal gradients are anticipated and on the estimated degree of slab restraint.

Joint Requirements - Where control joints are necessary and they shall be submerged continuously or intermittently, they shall include a waterstop.

5.6 REFERENCES

1. "Building Code Requirements for Reinforced Concrete, ACI 318-89, and Commentary, ACI 318R-89," American Concrete Institute, Detroit, MI, 1989

2. Concrete Manual, 8th Edition, U.S. Dept. of Interior, Bureau of Reclamation, Denver, Colo., 1988

3. Design and Control of Concrete Mixtures, 13th Edition; Kosmatka, Steven and Panarese, William; Portland Cement Association, Skokie, Il, 1988

CHAPTER 6 - ELECTRICAL/MECHANICAL ELEMENTS AND ISSUES

6.1 OBJECTIVE:

To provide reliability levels commensurate with the design levels cited for Critical Project Elements, Section 2.1, in those electrical and mechanical elements required to function properly to prevent a failure of the impoundment under extreme loadings.

6.2 DESIGN PHILOSOPHY:

Individual pieces of electrical and mechanical equipment are not designed for levels of reliability approaching that cited in Chapter 2.1. To achieve the desired level of functioning, it is necessary to take a systems approach similar to that used in the nuclear or airline industry. There, redundant systems are provided to accomplish a particular task. By providing enough backup systems, the likelihood of all the backups for a critical system not functioning in a given instance can be made sufficiently remote.

This systems approach requires that the main system and redundant elements meet certain constraints. These include the following:

- A suitable maintenance and testing program must be implemented so that the primary system and each of the redundant elements can reasonably be presumed to remain functional.

- The failure of one actuating system should not "lock up" the equipment preventing redundant systems from operating it.

- Independent power sources, delivery systems and controls should be provided to actuate a critical mechanism.

- Instrumentation should be provided to give accurate information on the functioning of critical systems.

- An emergency action plan that provides for the use of redundant systems must be developed, and the staff periodically tested in implementing it in realistic, simulated emergencies.

6.3 ENGINEERING PRACTICE:

Good engineering practice involves considering, among other things, the following:

Operation & Control

- Protecting the controls for the system from unauthorized operation by vandals.

- Identifying a responsible, trained party to operate and maintain the system.

- Recognizing the potential for misoperation of the equipment and, to the extent practicable, incorporate measures to minimize the adverse consequences of misuse. As examples, closing rates for valves should be restricted to rates that will not generate large hydraulic transients or where flashboards are used, their tripping should not unduly ramp the river.

- Ideally, where conditions are approaching failure, the equipment will have a "fail safe" mode of operation that requires no intervention on the part of the operators. For example, on a project with a gated spillway, should the pool level encroach upon the minimum level of freeboard and contrary to the operating scheme, the gates remain closed, the gate(s) will automatically be operated according to some predetermined rule curve.

Design

- To the extent practical in design, the modes of failure should be constrained to maximize "running time" and provide some limited but residual functioning or capacity. The first category would include such items as an early warning system to provide timely notice of an abnormal operating condition. In the latter case, should the system fail, ideally it will continue to have some remaining capacity. For example, with run-of-the-river hydroelectric projects, all of the river flow may be diverted into the powerhouse forebay and then out through the turbines. If the plant intertie to the power grid is lost, the electrical load on the turbine is removed. Ideally, the turbines have been designed to continue to pass flows in a "no load" operating mode. If this capacity is not provided, there will be significant waves generated in the forebay associated with the sudden shutdown of the flow through the turbines. In the short term, these waves can pose an overtopping concern[1]. If the reduction in flow through the turbines is not accommodated by increasing flows at the diversion structure, the low spot in the system will eventually be overtopped with potentially disastrous consequences.

- The design should account for the extreme loadings that may occur in operation. The impact of vibrations and hydraulic transients acting on gates and piping during the opening and closing of these elements should be considered in design.

- A locking system should be provided to maintain a particular operational configuration where a number of operational states are possible. For example, movable ring gates on morning glory spillways should not rely solely on hydraulic pressure to maintain them at a given stage for a protracted period due to the concern for bleeding off of hydraulic pressure.

Periodic Testing

- It should be recognize that there may be problems in operation not anticipated in the design. The system should be tested under as realistic of emergency conditions as practicable to confirm the system performs as intended. For instance in testing spillway gates, the gates should be "bounced" with the pool at an elevated level.

- Separate tests should be conducted on the main and backup systems, not individual elements of the various systems. Failure to test an auxiliary system as a whole may fail to show a "weak link" common to all systems. For instance, all actuating mechanisms may require hydraulic fluid from a common reservoir. Should the hydraulic system leak, insufficient fluid may be available to operate the backup system.

6.4 REFERENCES

1. Region X Interagency Hazard Mitigation Team, Interagency Flood Hazard Mitigation Report, In Response to the July 26, 1986 Disaster Declaration, State of Washington, FEMA-769-DR, November 10, 1986, pg. 3.

CHAPTER 7 - CONSTRUCTION SPECIFICATIONS

7.1 OBJECTIVES:

- Identify the goal of the work,
- Describe the assumed, existing conditions (primarily associated with foundation work),
- Provide for testing to control quality,
- Outline a means of dealing with unanticipated or changed conditions and a program for correcting deficiencies in construction,
- Protect completed elements of the project, and
- Assure completion of tasks in a timely manner.

7.2 ENGINEERING CONCERNS:

Dams pose specialized engineering problems. One must contend with significant seepage forces, long term settlements, harsh service conditions and long service lives. Moreover, it often is not enough that one specifies the use of quality materials. The specifier must consider the changes that are likely to occur in the material resulting from construction practices and normal service conditions. Ideally, the specifications anticipate problems and include provisions to minimize or mitigate their impact. The principal elements of this process are:

Requiring qualified contractors - A significant portion of the work associated with dams is outside the scope of normal construction practice. Dewatering of large excavations is often required along with treating problem soils. Extensive subgrade work is often necessary, involving the shaping of rock surfaces and slush grouting of fractures. Much of the concrete work involves specialized surface treatments and curing appropriate for harsh service conditions. Geosynthetics are being used increasingly in dam construction. Ideally, the specifications include provisions requiring contractors' bids to document past satisfactory completion of similar work.

Treating and controlling groundwater during construction - Springs are often encountered within the footprint of the dam. The treatment of springs varies depending on where they occur within the dam footprint. **The principal design consideration common to all is that the "fix" not allow the movement of soils into or out of, such features**. Within the core section, springs should be treated both to prevent the loss of fines and so as not to adversely affect compaction of the adjacent core materials and minimize seepage. Downstream of the core, seeps should be tied into the drainage system.

The construction specifications should require the contractor to maintain the working surface of the fill free of standing water so that fill compaction can be accomplished in the dry. The specifications typically note the need to treat wet areas and control groundwater. However, the specific manner of treatment and/or control is left up to the contractor. To confirm that the work will be accomplished in a suitable manner, the specifications should require the contractor to submit a dewatering plan to the engineer for review and comment.

Controlling material properties - The specifications should describe the minimum character to be achieved in the "final product". For example, in subgrade preparation the type of soils anticipated as a suitable bearing surface in the foundation and on the abutments should be described; conversely, what constitutes unsuitable soils should be identified. Acceptable density or consistency levels of subgrade soils should be cited. The goal is to assure that field conditions, crucial to the performance of the structure, meet or exceed the assumptions of the designer.

In compacting moisture sensitive, fine grained soils it is generally impractical to adjust the moisture content more than about 2 percent after it is deposited on the working surface of the fill. Where a highly impervious, uniform clay liner is required, the specifications normally require that moisture conditioning of the fill be accomplished in the borrow pit. This illustrates the need for the specifier to be aware of the practical constraints faced by the contractor. To the extent practicable, the specifications need to anticipate difficulties in construction and, in limited cases, require special measures of the contractor.

Controlling construction practices - Properly timing the phases of work on dams can be crucial. Much of the construction must be scheduled around the weather. Usually, the earthwork has to be accomplished in the dry season. Projects are often constructed within a river channel. Where the period of construction of these projects will extend through the runoff season, the site must be maintained in a state to survive the runoff from a potential flood event. This is a particular concern in earthen embankments if the dam section has not reached a sufficient height to have a functional spillway. The lowlevel outlet pipes are often sized to accommodate the runoff from an unusual storm, predicated on having achieved a minimum embankment height. The designer assumed that the flow could be accommodated through a combination of storage and lowlevel outlet capacity. This approach is reasonable provided there is some means of assuring that the contractor has constructed the embankment to the necessary elevation. The specifications need to provide the field engineer with the tools to require the contractor to undertake and complete crucial phases of the project in a timely manner, consistent with foreseeable weather and surface water flow constraints. The principal element in this is the requirement that the contractor work up a schedule which clearly identifies a practical construction timeline that duly considers the preceding factors. Ideally, the contractor is made responsible for accomplishing the minimum work necessary for the project to survive a winter shutdown rather than the owner's field engineer. The basic thrust of the above comments is to put requirements in the contract documents that put the onus on the contractor to achieve project milestones. The engineer should not be left in the position of having to cajole the contractor into accomplishing crucial aspects of the project in a timely manner.

Dam projects frequently involve esoteric construction problems. Where the plans call for an unusual task, the specifications should require that the contractor detail the proposed scheme to accomplish the work. It is crucial that the scheme is received with sufficient lead time for the engineer to confirm that the approach is practical.

Where specialty work is necessary, or adverse conditions are expected to complicate normal tasks, it is prudent to have the contractor demonstrate the suitability of his proposed construction scheme under realistic field conditions. For instance, say the purpose is to construct a low permeability soil lining by blending bentonite into the subgrade. The amount of bentonite along with the mixing and compaction process are generally left up to the contractor with the proviso that the completed liner exhibit a permeability less than some specified figure. The contractor, prior to starting the actual liner, should be required to complete a test section that reasonably simulates actual construction practice on the project. This would include utilizing the blending equipment (generally a rotovator), a number of different bentonite mixes at differing compacted densities with the type of compactor that will be use for the actual liner, and finally testing the process on both representative embankment slopes and bottom sections. Permeability tests should then be conducted to demonstrate the appropriateness of a particular mix and construction procedure.

As previously noted, a key element of the specifications is a description of the product you are trying to achieve. Conversely, it is important often to describe what is to be avoided or minimized. Gravelly clays derived from in place, highly weathered basalt, are often employed for a core material in Washington. The gravel fraction is generally angular and has a coarse gradation. When properly handled the soil forms a highly impermeable, competent core. However, segregation problems have occurred on occasion with this material. The problems generally occur due to improper handling or working of it on the fill surface. Ideally, the fill should be dumped from scrapers so that there is minimal need for further movement of the material prior to compaction. When the soil is dumped in piles from trucks, as is often done, the piles must be spread to form a uniform lift. The spreading operation must be done with care to minimize segregation. In spreading, there is a tendency for the coarser gravels and the occasional cobble to be "raked" out of the pile and deposited with other coarse fragments together in a "pocket" where the tractor stops. The overall gradation of the soils within these pockets may not have sufficient fines to fill the interstices between the larger fragments. The presence of these features can lead to increased seepage and, on rare occasions, to piping problems.

Segregation can also occur when the contractor does not bring up the working surface of the fill uniformly. Normally, this occurs where the embankment has a number of different zones and the contractor continues to place coarse shell materials during wet weather, that is not conducive to the placement of the moisture sensitive core materials. Two problems arise. First, inadequate compaction is generally achieved near the face of the temporary slope at the zone contact. Second, there is a tendency for the larger soil particles to roll down this slope and form a significantly coarser zone at the chimney drain contact and at the upstream shell/core contact. This potentially increases seepage and piping problems. Ideally, such problems are anticipated by the designer, based on the characterization of the borrow site by the geotechnical investigation. Where such problems are expected, the specifications need to clearly state what is acceptable and unacceptable in the compacted fill. There should be a maximum particle size limitation and a proviso that the coarser materials should not be nested but instead be well distributed throughout the fill. Second, the specifications should provide for the excavation of test trenches periodically to confirm that the desired soil structure is achieved. The method of best achieving this soil structure should be left up to the contractor. He can choose to employ a grizzly or other means to accomplish the required product.

Protecting completed elements of the work - Once an element has been completed satisfactorily, it does not necessarily follow that it remains in an acceptable condition. A suitably compacted soil can be changed to an unacceptable material through frost action, subsequent overworking of the area under construction traffic or even allowing a fine grained soil to dry out and crack. Thus, the specifications need to require that the work be protected. This is normally accomplished by requiring that the contractor remove or rework soils exhibiting specified deficiencies.

A problem routinely encountered in suitably founding the dam is the transition of hard tills and mudstones into a slurry under the combined action of stress release, air slaking, and surface water flows. The contractor exposes competent foundation materials which within the span of a few minutes turns to a muck. In extreme cases this deterioration of the foundation has led to the unnecessary overexcavation of tens of thousands of yards on a single project. Eventually, a practical scheme is worked out to overcome the problem. This generally involves: intercepting all surface flows and piping them around the excavation area, excavating a large element of the dam footprint down to within roughly a foot of final grade, dropping the ground water level in the foundations locally, and then, in rapid succession, excavating the last foot of disturbed material, high pressure air cleaning of the surface followed immediately by the capping of the area with embankment fill. This final phase of excavation can be facilitated by welding a flat plate across the teeth of the backhoe. The flat plate minimizes the depth of disturbed material which has to be removed in the subsequent air cleaning operation.

This rapid breakdown of the subgrade poses difficult construction problems for such features as narrow, concrete diaphragm cutoff walls. The wall is excavated and a considerable period of time elapses before the reinforcing steel and water stops are placed and concrete poured. During this period, the base of the trench is being gradually filled by loose materials sloughing from the excavation walls, and ground water is flowing down the base of the trench. This flow tends to wash away the fines from the material at the base of the trench, leaving a relatively pervious sandy gravel that may be contiguous along the entire trench base. Obviously, the presence of such a relatively pervious zone immediately beneath the cutoff wall largely defeats the purpose of the wall. The problem is reduced somewhat by mucking out the trench immediately prior to placing panels of pre-tied reinforcing panels. However, it is still usually necessary to use an air driven suction pipe to clear the residual muck immediately ahead of the concrete placement. The above details were provided to illustrate the fact that the specifier needs to understand the difficulties faced by the contractor in the field. If a particularly difficult construction feature is required, then the specifications need to clearly outline what constitutes an acceptable condition prior to the placement of concrete. In this instance, the specifications should require that concrete be poured on a suitable low permeable till stripped of unsuitable zones.

Embankment fill is susceptible to being overworked by the action of construction traffic. Often the construction equipment is working in a confined space and access to and from the dam is concentrated in a few areas. The specifications should provide for the removal of soft areas, "pumped" by traffic action.

Contamination of filter zones is a concern. To minimize this problem, it is prudent for the specifications to require that "crossing areas" be identified for construction traffic. These crossing areas are typically protected by placing a geotextile cap and, if appropriate, a thin wearing course for the equipment.

The issue of keeping frozen soils from incorporation into the working surface of the fill should be addressed in the specifications. Again, the actual means of accomplishing the goal should be left to the contractor. Typically, where the frost problem is not too severe, the contractor can "blanket" the last compacted lift of the shift with a loose lift. This loose layer acts as an insulator. Depending on the degree of frost penetration, the layer is either incorporated into the fill or removed as unsuitable. In colder weather, the construction season may be prolonged if the filling operation is double shifted. Filling can proceed as long as the time for excavation, placement and burial is insufficient for the soil to freeze.

Pipe designs are generally predicated on the normal stresses under service conditions. However, the stresses associated with placing the pipe and the operation of traffic over the pipe prior to placing adequate cover can be greater than allowable loads. The specifications need to clearly state the minimum cure time for concrete encasement to harden and the minimum cover over the pipe prior to the operation of construction traffic.

7.3 COMMENTARY:

Some of the failures to deal with the special problems posed in dam construction spring from a natural desire not to "reinvent the wheel". Rather than draft a new specification to cover an aspect of the construction, writers frequently "cut and paste" provisions from other projects. Increasingly, specifications do not explicitly describe the elements of the work. Instead, the work is required to comply with the provisions of the Standard Specifications published by the Washington State Department of Transportation (DOT) in conjunction with the Washington Chapter of the American Public Works Association. This document provides reasonable provisions governing the construction of roadways and utilities. However, it was not intended to cover the peculiar problems posed in constructing embankments to permanently retain bodies of water. The specifications need to be appropriately modified if they are to be used for dam construction.

For example, Section 2-03.3(14), Method C of the Standard Specifications is increasingly cited as a compaction specification. This requires that the embankment fill be compacted to a minimum of 95 percent of the maximum density determined by DOT Test Procedure 606 (non-granular soils) or 609 (granular soils). Test Procedure 609 roughly corresponds to ASTM Procedure D 698 (Standard Proctor); Test Procedure 606 is something done only by the DOT. First, the soils frequently used in constructing the shells of dams fall into the granular soil category and one would have to go to the DOT to develop a compaction curve. Second, as noted in the compaction subsection of Chapter 3.2, compaction often must be a minimum of 95 percent of the Modified Proctor Maximum Density. The DOT does not currently have an equivalent standard. Finally, the DOT specifications require that field tests of the degree of compaction be accomplished with a Washington State Densometer or nuclear gauge. Washington Densometers are not widely used, even by the DOT at present, and many test labs do not have a nuclear gauge. Presumably, the specifications requiring Test Procedure 606 and use of the a particular densometer are frequently simply ignored during construction. It would be preferable to specify generally accepted standard procedures for the contractor.

CHAPTER 8 - OPERATIONAL AND MONITORING ISSUES

8.1 OBJECTIVES:

Identify operational issues that need to be addressed to ensure long term safety of the dam and outline the minimum scope of instrumentation and monitoring.

8.2 APPLICABILITY:

This discussion is intended to cover the great majority of small to intermediate size earthen embankments that typically have few operational elements and receive relatively infrequent visits by the dam tender. Much of the basic discussion would apply to larger projects. But, the redundancy in discharge facilities, stricter operational constraints and the additional equipment and manpower typically available at larger projects generally *allow and necessitate* considerably more elaborate operational and monitoring schemes. This complexity and the site-specific character of these larger facility operation plans place them outside the scope of the general discussions here.

The instrumentation discussion covers the elements necessary to confirm that the structure is performing in general accordance with the designers' assumptions. The need for further instrumentation to investigate an adverse change in the behavior of the impounding structure and/or the foundation, is outside the scope of this discussion.

8.3 ENGINEERING CONCERNS:

Operation - Improper operation and maintenance of the reservoir can impair the safety of the dam and in extreme cases, lead to failure. Project information and operating instructions, critical to the performance of the dam, may be forgotten or lost in the transfer of ownership or the changing of dam tenders, if not included in an operation and maintenance plan.

Monitoring - Instrumentation and the monitoring of that instrumentation, can provide timely notice of an adverse change in the state of the impoundment. In particular, changes in seepage character and/or volume, abnormal settlement patterns, slope movements, etc., frequently are symptoms of deterioration in the embankment and foundations. Instrumentation and the *dutiful recording and analysis* of the generated data are crucial to the timely identification of significant trends in the performance of the impounding barrier.

8.4 DISCUSSION:

Overview - General guidance in the development of operation plans is presented in Ecology publications "Guidelines for Developing Dam Operation and Maintenance Manuals" and Chapters 7 and 9 of Part III of the Guidelines, "An Owner's Guidance Manual". These documents also provide guidance on the selection of instrumentation and the drafting of monitoring programs.

Integrating the Operation, Monitoring and Emergency Action Plans - It is important to recognize the relationship between the Operation and Maintenance Program, the Monitoring Program and the Emergency Action Plan. Together they represent a continuum of responses to various event scenarios at the project. Key to the development of these plans is the identification of the foreseeable range of events that could beset the facility and the "scripting" of appropriate responses.

8.5 OPERATION PLAN:

The hallmarks of a good operation plan are:

- A responsible individual is named to carry out the plan and provisions are included requiring this individual to document routine compliance with the scheme.

- The inclusion of a comprehensive list of the normal and reasonably foreseeable extreme operational conditions the facility could be subjected to, and appropriate "scripts" for managing the project through the various scenarios.

- The monitoring program and emergency action plan, if necessary, are integrated into the provisions of the operation plan. Data is required from the monitoring program in carrying out the scripts of the operation plan. An "alert phase" is included in the operational scheme providing a transition period as an unusual event unfolds prior to imminent failure.

- To the extent practicable, the facility is designed to survive extreme loading conditions without the need for intervention by the dam tender. Ideally, the scripted response is built into the design and requires no action by the operator. For example, an ungated, emergency spillway is preferable to a gated unit that would require some "action" to obtain maximum capacity.

- Where action by the tender is required, it is crucial that the conditions precipitating that action are triggered early in the course of an event. This allows greater time to "ramp up" changes in facility operation and reduces the risk to the dam tender in accomplishing required operations under high pool or other extreme conditions.

- Actions in the script the dam tender must follow are tied to clear benchmarks. For external problems such as floods, predetermined pool levels and rates of reservoir rise should initiate action. For earthquakes, the inspection and precautionary drawdown of the pool should be dictated by the occurrence of a seismic event larger than some specified magnitude within a specified region centered on the project. For internal structural problems such as piping, excessive seepage or embankment slides, the operation plan should require lowering of the pool and provide guidance on whether conditions warrant implementing the emergency action plan.

The following discusses specific issues and elements in an operation plan.

- **Stoplog Operation**: Stoplogs are frequently used in spillways for regulation of the reservoir level, or to increase the storage capacity of the reservoir. Normally, the dam size, reservoir capacity, and spillway discharge are designed to provide the maximum possible storage while retaining the ability to pass the Inflow Design Flood (IDF). However, in some cases, stop logs can be used to temporarily raise the water level during times when the occurrence of the IDF is least likely and removed to provide flood storage when the IDF is more likely. The seasonal placement and removal of the stop ogs must be described in the Operation and Maintenance (O&M) Plan and approved by the Dam Safety Office.

- **Access**: In designing gate or valve controls for outlets, spillways or other critical operating equipment, care should be taken to provide for safe access under both normal and emergency conditions. This would entail placing critical outlet or spillway controls in an easily accessible location such as the dam crest, or in a location well above the normal reservoir level or flood level.

- **Vandalism**: Vandalism is often a problem at facilities. Valves are opened and often damaged. Debris is jammed into the housing around valve stems and into the entrances or mouths of pipes. Monitoring devices are smashed. To the extent practicable, operating and monitoring equipment should be placed inside protective devices. Furthermore, the protective devices should be configured to make vandalism as difficult as practicable. This would include: making them unobtrusive, placing them flush or below surrounding grade and restricting vehicle access. Finally, the designer must assume that valves conceivably could be prevented from operating or conversely, opened to their maximum rating. This potential for misoperation should be considered in the design and factored into the operational plan.

- **Backup Operation**: Electrically controlled gates and valves for outlets and spillways should be provided with backup, manual controls. The backup controls should be easily accessible during a major flood or other emergency situation.

- **Operation Instructions**: An operation plan should provide clear, complete, step-by-step instructions for operating all mechanisms associated with a dam. Instructions on the general operation of the reservoir, including any inflow and outlet ditches, should be given. The instructions should cite the maximum pool levels to be allowed at different times of the year, permissible outlet releases, and operation of the outlet to limit or prevent spillway flow.

8.6 MONITORING PROGRAM

General - Because of the wide variety of dams within the purview of the Dam Safety Office, it is impractical to develop an all-encompassing plan, covering every project. However, there are general issues that typically must be addressed at each project. The remainder of this section discusses these typical elements of a monitoring program. *Section 8.7* summarizes the specific requirements germane to small, intermediate and large dams.

Instrumentation - As in the operation plan, the engineer should incorporate features in the dam cross-section during design to minimize the need for instrumentation. The designer is encouraged (in some cases required) to actively constrain the variability of crucial elements influencing embankment performance. First, this serves to improve the overall reliability of the structure and second, it obviates the need for sophisticated monitoring with all the attendant problems with data analysis, equipment failures, misreadings and the cost of running the program.

Where instrumentation is necessary, the design should facilitate the use of the simplest type of instrumentation. For example, one of the principal factors affecting embankment stability is the position of the phreatic surface in the embankment cross-section. Various types of observation wells have been employed in the past to confirm the designer's assumptions as to the position of the phreatic surface. Considerable controversy and money has been spent to explain anomalous observation well readings. The inclusion of chimney and blanket drains in the design (espoused in Chapter 3.2) provides a positive means of controlling the position of the phreatic surface within the embankment. The other parameters of interest in monitoring groundwater in the embankment are the volume and character of the seepage. Collector drains embedded within the chimney drains and outlet pipes or free draining gravel zones within the blanket drain, afford the opportunity to easily monitor seepage volumes and recover representative water samples. These water samples can be collected in clear jars and a visual assessment made as to whether entrained fines are present that would indicate "piping" is occurring. In the case of measuring seepage, a simple v-notch weir or bucket and stop watch system can be employed to determine the volume of flow. Further refinements in flow monitoring can be achieved by providing a number of outlets for the drains in the abutments and at intervals along the valley floor. Such a scheme provides the ability to evaluate differential seepage across the dam footprint at a nominal cost.

We see three key elements in conducting a successful monitoring program:

- An operation plan that requires the taking of monitoring data and the dutiful carrying out of the observation program,

- The *timely* evaluation of that data for significant changes or adverse trends in anticipated behavior, and

- A script in the operation plan to follow when the monitoring suggests significant changes are occurring.

It should be noted that many of the adverse changes in the dam occur at a gradual rate. Eventually, the deterioration passes some trigger level and further degradation may occur at a precipitous rate. The gradual deterioration phase may be so lengthy that the dam tender is not aware of the adverse trend. Often, initial concerns as to seepage and slumps are relieved or forgotten when the situation continues without appearing to dramatically worsen; familiarity with a problem breeds complacency.

Photographs - One of the most effective tools at maintaining a perspective on the state of the dam is the periodic photographing of the embankment and downstream toe. Ideally, a set of photographs would be taken each year. The individual sets of photographs should be compared with one another. The principal item of interest to be looking for are changes in the type and area of vegetative cover. This gives an early indication of any changes in seepage conditions taking place within the embankment and foundations. More dramatic changes such as slumps and piping features would most likely follow sometime after the vegetative changes. Ideally, the changes in vegetation would have initiated an appropriate response. However, in the event these changes are missed, more dramatic changes such as surficial creeping of the embankment, piping or other problems likely would be picked up in a comparison of photographs and an appropriate response triggered.

Log Book - Along with a photographic record, a log book of observations and monitoring data should be maintained. The log book should note significant events, cite normal maintenance and operational tasks performed, and record and plot any observations and monitoring data on the performance of the structure and appurtenances. Abnormal performance of the structure should trigger an appropriate response in the operation plan. It should be recognized that normal and abnormal performance probably will not be known until after the structure has been put in service and steady state conditions have developed in the foundations and embankment. For the majority of small dams under consideration here, this condition probably will be achieved within a year or so of first filling. At this point, the monitoring program developed by the designer should be reviewed to confirm that it appropriately covers the range of actual observed conditions. In addition, the designer's initial monitoring program should have included an increased frequency of readings to obtain a clear picture of embankment performance fluctuations in response to various loadings. After obtaining a history of readings, the data should be reviewed. Specifically, monitoring data should be evaluated to determine: 1) what is a normal reading and what should constitute a significant abnormal reading, 2) the minimum frequency of readings necessary to model a particular aspect of dam performance, and 3) the best means of plotting data and how the data should be used in the operation plan.

8.7 REQUIRED MINIMUM OPERATION AND INSTRUMENTATION PROGRAMS:

All projects shall have an Operation and Maintenance Plan as per WAC 173-175-210. Where the operation plan allows the seasonal use of stoplogs, the scheme for stoplog use must be approved by the Dam Safety Office.

Table 1 summarizes the minimum instrumentation program for impoundments.

TABLE 1 - EMBANKMENT INSTRUMENTATION

DAM SIZE CLASSIFICATION	MINIMUM INSTRUMENTATION ELEMENTS
SMALL DAM (Less Than 15 Feet High)	• Reservoir staff gage.
INTERMEDIATE SIZE DAM (15 Feet to 50 feet in height)	• Reservoir staff gage. • Underdrain outlet volume measurement scheme. • Simple settlement/displacement monuments in the dam crest referenced to a fixed point on either abutment.
LARGE DAM (50 Feet or Greater in height)	• Reservoir staff gage. • Underdrain outlet volume measurement scheme. • Simple settlement/displacement monuments in the dam crest referenced to a fixed point on either abutment.

8.8 REFERENCES:

1. U.S. Bureau of Reclamation, Training Aids for Dam Safety, Module: How to organize an Operation and Maintenance Program, U.S. Government Printing Office, Denver, CO., 1990.

APPENDIX A
GLOSSARY

Abutment - That contact location at either end and beneath the flanks of a dam where the artificial barrier joins or faces against the natural earth or rock foundation material upon which the dam is constructed. The left and right abutments of dams are defined with the observer viewing the dam looking in the downstream direction.

Appurtenant Works - Structures such as outlet works and associated gates and valves; water conveyance structures such as spillways channels, fish ladders, tunnels, pipelines or penstocks; powerhouse sections; and navigation locks, either in the dam or separate therefrom.

Axis of dam - The vertical plane or curved surface, chosen by a designer, appearing as a line, in plan or in cross-section, to which the horizontal dimensions of the dam are referenced.

Baffle block - A block, usually of concrete, constructed in a channel or stilling basin to dissipate the energy of water flowing at high velocity.

Base thickness - Also referred to as base width. The maximum thickness or width of the dam measured horizontally between upstream and downstream faces and normal to the axis of the dam, but excluding projections for outlets, or other appurtenant structures.

Bedrock - The consolidated body of natural solid mineral matter which underlies the overburden soils.

Berm - A nearly horizontal step in the sloping profile of an embankment dam.

Borrow area - The area from which material for an embankment is excavated.

Breach - An eroded opening through a dam which drains the reservoir. A controlled breach is a constructed opening. An uncontrolled breach is an unintentional opening which allows uncontrolled discharge from the reservoir.

Channel - A general term for any natural or artificial facility for conveying water.

Compaction - Mechanical action which increases the density by reducing the voids in a material.

Conduit - A closed channel to convey water through, around, or under a dam.

Construction joint - The interface between two successive placings or pours of concrete where bond, and not permanent separation, is intended.

Core - A zone of low permeability material in an embankment dam. The core is sometimes referred to as central core, inclined core, puddle clay core, rolled clay core, or impervious zone.

Crest Length - The total horizontal distance measured along the axis of the dam, at the elevation of the top of the dam, between abutments or ends of the dam. Where applicable, this includes the spillway, powerhouse sections and navigation locks, where they form a continuous part of the impounding structure.

Crest thickness (top width) - The thickness or width of a dam at the level of the top of dam (excluding corbels or parapets). In general, the term thickness is used for gravity and arch dams, and width is used for other dams.

Critical Project Element - An element of a project whose failure could result in the uncontrolled release of the reservoir.

Cross section - An elevation view of a dam formed by passing a plane through the dam perpendicular to the axis.

Cutoff trench - A foundation excavation later to be backfilled with material so as to limit seepage beneath a dam.

Dam - Any artificial barrier and/or any controlling works, together with appurtenant works that can or does impound or divert water.

a. Arch dam. A concrete or masonry dam which is curved upstream so as to transmit the major part of the water load to the abutments.

b. Cofferdam. A temporary structure enclosing all or part of the construction area so that construction can proceed in the dry. A diversion cofferdam diverts a stream into a pipe, channel, tunnel, or other watercourse.

c. Crib dam. A gravity dam built up of boxes, crossed timbers or gabions, filled with earth or rock.

d. Diversion dam. A dam built to divert water from a waterway or stream into a different watercourse.

e. Earth dam. An embankment dam in which more than 50% of the total volume is formed of compacted earth material generally smaller than 3-inch size.

f. Embankment dam. Any dam constructed of excavated natural materials or of industrial waste materials.

g. Gravity dam. A dam constructed of concrete and/or masonry which relies on its weight and internal strength for stability.

h. Hydraulic fill dam. An earth dam constructed of materials, often dredged, which are conveyed and placed by suspension in flowing water.

i. Industrial waste dam. An embankment dam, usually built in stages, to create storage for the disposal of waste products from an industrial process. The waste products are conveyed as fine material suspended in water to the reservoir impounded by the embankment. The embankment may be built of conventional materials but sometimes incorporates suitable waste products.

j. Masonry dam. Any dam constructed mainly of stone, brick, or concrete blocks jointed with mortar. A dam having only a masonry facing should not be referred to as a masonry dam.

k. Mine tailings dam. An industrial waste dam in which the waste materials come from mining operations or mineral processing.

l. Regulating dam. A dam impounding a reservoir from which water is released to regulate the flow downstream.

m. Rockfill dam. An embankment dam in which more than 50% of the total volume is comprised of compacted or dumped cobbles, boulders, rock fragments, or quarried rock generally larger than 3-inch size.

n. Roller compacted concrete dam. A concrete gravity dam constructed by the use of a dry mix concrete transported by conventional construction equipment and compacted by rolling, usually with vibratory rollers.

o. Saddle dam (or dike). A subsidiary dam of any type constructed across a saddle or low point on the perimeter of a reservoir.

p. Tailings dam. See mine tailings dam.

<u>Dam failure</u> - The uncontrolled release of impounded water. It is recognized that there are lesser degrees of failure and that any malfunction or abnormality outside the design assumptions and parameters which adversely affect a dam's primary function of impounding water is properly considered a failure. They are, however, normally amenable to corrective action.

<u>Dam Height</u> - The effective hydraulic height of a dam as measured by the vertical distance from the natural bed of the stream or watercourse at the downstream toe of the impounding barrier to the maximum storage elevation. If the dam is not across a stream or watercourse, the height is measured from the lowest elevation of the outside limit of the impounding barrier to the maximum storage elevation.

<u>Design Step Level</u> - An integer value between one and ten used to designate increasingly stringent design loadings and conditions for design of critical project elements.

<u>Dike</u> - See saddle dam.

<u>Diversion channel, canal, or tunnel</u> - A waterway used to divert water from its natural course. The term is generally applied to a temporary arrangement, e.g. to by-pass water around a dam site during construction. "Channel" is normally used instead of "canal" when the waterway is short.

<u>Downstream Hazard Classification</u> - A rating to describe the potential for loss of human life and/or property damage if the dam were to fail and release the reservoir onto downstream areas. Downstream hazard classifications of 3, 2 and 1C, 1B, 1A correspond to low, significant and high downstream hazard classes respectively.

<u>Drain, blanket</u> - A layer of pervious material placed to facilitate drainage of the foundation and/or embankment.

<u>Drain, chimney</u> - A vertical or inclined layer of pervious material in an embankment to facilitate and control drainage of the embankment fill.

Drain, toe - A system of pipe and/or pervious material along the downstream toe of a dam used to collect seepage from the foundation and embankment and convey it to a free outlet.

Drainage area - The area which drains to a particular point on a river or stream.

Drawdown - The difference between a water level and a lower water level in a reservoir within a particular time. Used as a verb, it is the lowering of the water surface.

Earthquake - A sudden motion or trembling in the earth caused by the abrupt release of accumulated stress along a fault.

Emergency Action Plan (EAP) - A plan of action to be taken to reduce the potential for property damage and loss of life in an area affected by a dam failure.

Emergency Condition - A situation where life and property are at imminent risk and actions are needed within minutes or hours to initiate corrective actions and/or warn the public.]

Emergency Spillway - Any secondary spillway which is designed to be operated very infrequently and possibly in anticipation of some degree of structural damage or erosion to the spillway during operation.

Energy dissipator - A device constructed in a waterway to reduce the kinetic energy of fast flowing water.

Epicenter - The point on the earth's surface located vertically above the point of origin of an earthquake.

Exigency Condition - A situation where the impounding structure is significantly underdesigned according to generally accepted engineering standards or is in a deteriorated condition and life and property are clearly at risk. Although present conditions do not pose an imminent threat, if adverse conditions were to occur, the situation could quickly become an emergency.

Fault - A fracture or fracture zone in the earth crust along which there has been displacement of the two sides relative to one another.

Fault, Active - A fault which, because of its present tectonic setting, can undergo movement from time to time in the immediate geologic future.

Fault, Capable - An active fault that is judged capable of producing macro-earthquakes and exhibits one or more of the following characteristics:

a. Movement at or near the ground surface at least once within the past 35,000 years.

b. Macroseismicity (3.5 magnitude Richter or greater) instrumentally determined with records of sufficient precision to demonstrate a direct relationship with the fault.

c. A structural relationship to a capable fault such that movement on one fault could be reasonably expected to cause movement on the other.

d. Established patterns of microseismicity which define a fault, with historic macroseismicity that can reasonably be associated with the fault.

Fetch - The straight line distance across a body of water subject to wind forces. The fetch is one of the factors used in calculating wave heights in a reservoir.

Filter (filter zone) - One or more layers of granular material graded (either naturally or by selection) so as to allow seepage through or within the layers while preventing the migration of material from adjacent zones.

Flashboards - Structural members of timber, concrete, or steel placed in channels or on the crest of a spillway to raise the reservoir water level but that may be quickly removed in the event of a flood.

Flip bucket - An energy dissipator located at the downstream end of a spillway and shaped so that water flowing at a high velocity is deflected upwards in a trajectory away from the foundation of the spillway.

Flood - A temporary rise in water levels resulting in inundation of areas not normally covered by water. May be expressed in terms of probability of exceedance per year such as one percent chance flood or expressed as a fraction of the probable maximum flood or other reference flood.

Floodplain - An area adjoining a body of water or natural stream that has been or may be covered by floodwater.

Freeboard - The vertical distance between the dam crest elevation and some reservoir level of interest.

Fuse Plug Spillway - A form of auxiliary spillway consisting of a low embankment designed to be overtopped and washed away during an exceptionally large flood.

Gate - A movable, watertight barrier for the control of water in a waterway.

a. Bascule gate. See flap gate.

b. Bulkhead gate. A gate used either for temporary closure of a channel or conduit before dewatering it for inspection or maintenance or for closure against flowing water when the head difference is small, e.g., for diversion tunnel closure.

c. Crest gate (spillway gate). A gate on the crest of a spillway to control the discharge or reservoir water level.

d. Drum gate. A type of spillway gate consisting of a long hollow drum. The drum may be held in its raised position by the water pressure in a flotation chamber beneath the drum.

e. Emergency gate. A standby or auxiliary gate used when the normal means of water control is not available. Sometimes referred to as guard gate.

f. Fixed wheel gate (fixed roller gate) (fixed axle gate). A gate having wheels or rollers mounted on the end posts of the gate. The wheels bear against rails fixed in side grooves or gate guides.

g. Flap gate. A gate hinged along one edge, usually either the top or bottom edge. Examples of bottom-hinged flap gates are tilting gates and fish belly gates so called from their shape in cross section.

h. Flood gate. A gate to control flood release from a reservoir.

i. Outlet gate. A gate controlling the flow of water through a reservoir outlet.

j. Radial gate (Tainter gate). A gate with a curved upstream plate and radial arms hinged to piers or other supporting structure.

k. Regulating gate (regulating valve). A gate or valve that operates under full pressure flow conditions to regulate the rate of discharge.

l. Roller drum gate. See drum gate.

m. Roller gate (stoney gate). A gate for large openings that bears on a train of rollers in each gate guide.

n. Skimmer gate. A gate at the spillway crest whose prime purpose is to control the release of debris and logs with a limited amount of water. It is usually a bottom hinged flap or Bascule gate.

o. Slide gate (sluice gate). A gate that can be opened or closed by sliding in supporting guides.

Gate chamber (valve chamber) - A room from which a gate or valve can be operated, or sometimes in which the gate is located.

Geotextiles - Any fabric or textile when used as an engineering material in conjunction with soil, foundations or rock. Geotextiles have the following uses: drainage, filtration, separation of materials, reinforcement, moisture barriers, and erosion protection.

Groin - The area along the contact (or intersection) of the face of a dam with the abutments.

Grout - A fluidized material that is injected into soil, rock, concrete, or other construction material to seal openings and to lower the permeability and/or provide additional structural strength. There are four major types of grouting materials: chemical; cement; clay; and bitumen.

Grout curtain - One or more zones, usually thin, in the foundation into which grout is injected to reduce seepage under or around a dam.

Hydraulic Height - The vertical difference between the maximum design water level and the lowest point in the original streambed.

Hydrograph - A graphical representation of discharge, stage, or other hydraulic property with respect to time for a particular location on a watercourse.

Hydrograph, Breach or Dam Failure - A flood hydrograph resulting from a dam breach.

Hydrograph, Unit - A hydrograph with a volume of one inch of runoff resulting from a storm of a specified duration and areal distribution. Hydrographs from other storms of the same duration and distribution are assumed to have the same time base but with ordinates of flow in proportion to the runoff volumes.

Hypocenter - The point or focus within the earth which is the center of an earthquake and the origin of its elastic waves.

Impounding Barrier - The structural element of the dam that has the primary purpose of impounding or diverting water. It may be constructed of natural and/or man-made materials.

Incident - The occurrence of any dam-related event where problems or conditions arise which may have posed a threat to the safety or integrity of the project or which may have posed a threat of loss of life or which resulted in loss of life.

Inflow Design Flood (IDF) - The reservoir inflow flood hydrograph used for sizing the spillways and for determining freeboard. It represents the largest flood that a given project is designed to safely accommodate.

Instrumentation - An arrangement of devices installed into or near dams (i.e., piezometers, inclinometer, strain gages, measurement points, etc.) which provide for measurements that can be used to evaluate the structural behavior and performance parameters of the structure.

Intake - Any structure in a reservoir, dam or river through which water can be discharged.

Inundation map - A map delineating the area that would be flooded by a particular flood event.

Liquefaction - A condition whereby soil undergoes continued deformation at a constant low residual stress or with low residual resistance, due to the buildup and maintenance of high pore water pressures, which reduces the effective confining pressure to a very low value. Pore pressure buildup leading to liquefaction may be due either to static or cyclic stress applications and the possibility of its occurrence will depend on the void ratio or relative density of a cohesionless soil and the confining pressure.

Logboom - A chain of logs, drums, or pontoons secured end to end and floating on the surface of a reservoir so as to divert floating debris, trash, and logs.

Maximum design water level - The maximum water elevation including the flood surcharge, that a dam is designed to withstand.

Maximum flood control level - The highest elevation of the flood control storage.

Maximum Storage Elevation - The maximum attainable water surface elevation of the reservoir pool that could occur during extreme operating conditions. This elevation normally corresponds to the crest elevation of the dam.

Minimum operating level - The lowest level to which the reservoir is drawn down under normal operating conditions.

Normal Pool Height - The vertical distance between the lowest point of the upstream toe of the impounding barrier and the normal storage elevation.

Normal Storage Elevation - The normal maximum operating pool level in a reservoir. Where the principal spillway is ungated, the normal storage elevation is usually established by the level of the spillway crest.

Observation well - A hole used to observe the groundwater surface at atmospheric pressure within soil or rock.

100 Year Floodplain - The area inundated during the passage of a flood with a peak discharge having a one percent chance of being equalled or exceeded in any given year at a specified location on a watercourse.

Outlet - A conduit and/or channel structure for the controlled release of the contents normally impounded by a dam and reservoir.

Parapet wall - A solid wall built along the top of a dam (upstream or downstream edge) used for ornamentation, for safety of vehicles and pedestrians, or to prevent overtopping caused by wave runup.

Penstock - A pressurized pipeline or shaft between the reservoir and hydraulic machinery.

Periodic Inspection - A detailed inspection of the dam and appurtenant works conducted on regular intervals and includes, as necessary, associated engineering analyses to confirm the continued safe operation of the project.

Phreatic surface - The free surface of water seeping at atmospheric pressure through soil or rock.

Piezometer - An instrument used for measuring fluid pressure (air or water) within soil, rock, or concrete.

Piping - The progressive development of internal erosion within a soil mass by seepage.

Plunge pool - A natural or artificially created pool that dissipates the energy of free falling water.

Population At Risk - The number of people who may be present in areas downstream of a dam and could be at risk in the event of a dam failure.

Principal Spillway (or Service Spillway) - A spillway designed to provide continuous or frequent releases from a reservoir, without significant damage to either the dam or its appurtenant structures.

Probable Maximum Flood (PMF) - The most severe flood that is considered reasonable possible at a site as a result of meteorologic and hydrologic conditions.

Probability - The likelihood of an event occurring.

Probable Maximum Precipitation (PMP) - Theoretically, the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location.

Reservoir - Any basin that contains or will contain the water impounded by a dam.

Reservoir Routing - The procedures used to determine the attenuating effect of reservoir storage on a flood as it passes through a reservoir.

Reservoir surface area - The area covered by a reservoir when filled to a specified level.

Riprap - A layer of large uncoursed stone, precast blocks, bags of cement or other suitable material, generally placed on the upstream slopes of an embankment or along a watercourse as protection against wave action, erosion or scour. Riprap is usually placed by dumping or other mechanical methods and in some cases is hand placed. It consists of pieces of relatively large size as distinguished from a gravel blanket.

Risk - The relationship between the consequences resulting from an adverse event and its probability of occurrence.

Risk assessment - As applied to dam safety, the process of identifying the likelihood and consequences of dam failure to provide the basis for informed decisions on a course of action.

Rule Curve - The rules and procedures used to regulate reservoir levels and project operation for various reservoir inflows and for both normal and unusual seasonal conditions.

Safety Evaluation Flood(SEF) - The largest flood for which the safety of a dam and appurtenant structure is to be evaluated.

Slope - Inclination from the horizontal. Sometimes referred to as batter when measured from vertical.

Spillway - A channel structure and/or conduit for the safe release of surplus water or floodwater.

Spillway channel - An open channel or closed conduit conveying water from the spillway inlet downstream.

Spillway chute - A steeply sloping spillway channel that conveys discharges at super-critical velocities.

Spillway crest - The lowest level at which water can flow over or through the spillway.

Spillway, Shaft - A vertical or inclined shaft into which water spills and then is conveyed through, under, or around a dam by means of a conduit or tunnel. If the upper part of the shaft is splayed out and terminates in a circular horizontal weir, it is termed a bellmouth or morning glory spillway.

Stilling basin - A basin constructed to dissipate the energy of rapidly flowing water, e.g., from a spillway or outlet, and to protect the riverbed from erosion.

Stoplogs - Large logs, timbers, or steel beams placed on top of each other with their ends held in guides on each side of a channel or conduit so as to provide a cheaper or more easily handled means of temporary closure than a bulkhead gate.

Storage - The retention of water or delay of runoff either by planned operation, as in a reservoir, or by temporary filling of overflow areas, as in the progression of a flood wave through a natural stream channel. Definitions of specific types of storage in reservoirs are:

a. Dead storage. The storage that lies below the invert of the lowest outlet and that, therefore, cannot readily be withdrawn from the reservoir.

b. Inactive storage. The storage volume of a reservoir between the crest of the invert of the lowest outlet and the minimum operating level.

c. Active storage. The volume of the reservoir that is available for some use such as power generation, irrigation, flood control, water supply, etc. The bottom elevation is the minimum operating level.

d. Live storage. The sum of the active and the inactive storage.

e. Reservoir capacity. The sum of the dead and live storage of the reservoir.

f. Flood surcharge. The storage volume between the top of the active storage and the design water level.

<u>Structural Height</u> - The vertical distance between the lowest point of the excavated foundation to the top of the dam.

<u>Surficial Inspection</u> - A visual inspection conducted to identify obvious defects or changed conditions.

<u>Sweepout</u> - The hydraulic condition in a hydraulic jump energy stilling basin where there is insufficient tailwater depth to offset the hydrodynamic forces of the incoming discharge. The hydraulic jump is "swept" out of the basin and the downstream receiving stream is subjected to the full erosional forces of the high velocity discharge.

<u>Thrust block</u> - A massive block of concrete built to withstand a thrust or pull.

<u>Toe of dam</u> - The junction of the face of a dam with the ground surface. For concrete dams, see heel.

<u>Trashrack</u> - A device located at an intake to prevent floating or submerged debris from entering the intake.

<u>Tunnel</u> - A long underground excavation with two or more openings to the surface, usually having a uniform cross section used for access, conveying flows, etc.

<u>Valve</u> - A device fitted to a pipeline or orifice in which the closure member is either rotated or moved transversely or longitudinally in the waterway so as to control or stop the flow.

<u>Watershed divide</u> - The divide or boundary between catchment areas (or drainage areas).

<u>Wave runup</u> - The vertical height above the stillwater level to which water from a specific wave will run up the face of a structure or embankment.

<u>Weir</u> - A notch of regular form through which water flows.

a. Weir, broad-crested. An overflow structure on which the nappe is supported for an appreciable length in the direction of flow.

b. Weir, measuring. A device for measuring the rate of flow of water. It generally consists of a rectangular, trapezoidal, triangular, or other shaped notch, located in a vertical, thin plate over which water flows. The height of water above the weir crest is used to determine the rate of flow.

c. Weir, ogee. A reverse curve, shaped like an elongated letter "S." The downstream faces of overflow spillways are often made to this shape.

Wind setup - The vertical rise in the stillwater level at the face of a structure or embankment caused by wind stresses on the surface of the water.

SOURCE:

Interagency Committee on Dam Safety, Task Group on Glossary of Terms, Glossary of Terms for Dam Safety, Federal Emergency Management Agency, February 1988.

APPENDIX B

SUMMARY OF REQUIREMENTS

GEOTECHNICAL ISSUES
MINIMUM REQUIRED EXPLORATION PROGRAM

EXPLORATION METHODS	EARTHEN EMBANKMENT SMALL	INTERMEDIATE	LARGE	BORROW SITES
BACKHOE TEST PITS	✓	✓	✓	✓[2]
BORINGS	✓[1]	✓[1]	✓	✓[1,2]
GEOPHYSICAL EXPLORATIONS			✓[2]	✓[2]
PERMEABILITY TESTS		✓[2]	✓	

[1] Borings appropriate where overburden has appreciable thickness
[2] Desirable on a case by case basis

EMBANKMENT GEOMETRY AND ZONING

ELEMENT		EARTHEN EMBANKMENTS SMALL	INTERMEDIATE	LARGE	GUIDELINE REFERENCE
FOUNDATION CUTOFF	Cutoff Trench	Req'd[A]	Req'd[A]	Req'd[A]	3.2
	Rock Contact Sealing	Req'd[B]	Req'd[B]	Req'd[B]	3.2
LOW LEVEL OUTLET PIPE		Req'd[C]	Req'd	Req'd	4.1
DRAINS	Chimney	Recom[D]	Req'd	Req'd	3.3
	Blanket	Recom	Req'd[E,F]	Req'd[E,F]	3.3
	Toe Drains	Recom	_____[G]	_____[G]	
FILTERS	Filter Criteria met <u>Everywhere</u>[H]	Req'd	Req'd	Req'd	3.3.A
		Req'd	Req'd	Req'd	3.3.B
SIDESLOPES		To be determined by engineering analysis[J]			2.2
FREEBOARD		To be determined by engineering analysis			4.6
CREST		_____[K] To be determined by engineering analysis			
EROSION PROTECTION	Upstream Face	Recom	Req'd	Req'd	3.5
	Downstream Face	Recom[M]	Req'd	Req'd	3.5
	Crest	Req'd	Req'd	Req'd	
	Downstream Groins	Recom[N]	Recom	Req'd	
INSTRUMENTATION	Reservoir Staff Gauge	Req'd	Req'd	Req'd	8
	Settlement Monuments	Recom[O]	Req'd	Req'd	8
	Piezometers	Recom	Recom	Recom	8
	Weirs/Pipe to Measure Seepage	Recom[M]	Recom	Req'd	8

[A] The subgrade should be stripped of any pervious surficial zone to found the cutoff on a suitable, low permeability zone

[B] Any rock exposures should be sealed within the limits of the cutoff trench to prevent the loss of embankment materials into the foundations. Ideally, rock exposures upstream and downstream of the cutoff should be sealed and drained, respectively.

[C] For diked impoundments <10 feet high, the outlet may be omitted if it is practicable to lower the pool by siphoning.

[D] For small stormwater detention facilities, the DSO will consider eliminating the chimney drain where it can be demonstrated that neither excessive seepage is likely to develop in the abutments nor that the phreatic surface will penetrate a significant depth into the upstream surface.

[E] Where the foundations are relatively pervious, finger drains would provide an acceptable alternative to a blanket drain.

[F] The blanket drain should extend up the abutment. The extent of the abutment coverage should be a function of the magnitude and the nature of the seepage anticipated to emerge from the contact. Where isolated pervious features are indicated, separate finger drains should be considered within the blanket to safely carry off the concentrated flow and facilitate monitoring flow character.

[G] This would normally be a subelement of the blanket drain. Where the blanket drain has a large hydraulic capacity relative to the anticipated seepage flow <u>and</u> it is configured to efficiently shunt flow to one or a few points, a toe drain is not required.

[H] The zones should be internally stable in that they satisfy filter criteria for themselves. Plus, filter criteria must be satisfied at all zone contacts including the embankment interface with the foundation and abutments.

[I] Applicable to all conduits extending through or underlying the embankment cross-section.

[J] For small dams, bearing on a competent foundation, constructed of a well compacted clay, the DSO will accept a design with 3H on 1V upstream and 2H on 1 V downstream slopes without a supporting engineering analysis.

[K] The larger of 8 feet or the quantity $2(H)^{1/2} + 3$.

[L] Based on anticipated magnitude of crest settlement and conservatism in the level of freeboard provided.

[M] Determination based on ability to maintain a thick vegetative cover or erosion resistance of exposed soil.

[N] The need for erosion protection should be based on the volume of runoff and the erosion resistance of the groin area soils.

[O] Monuments should be provided where freeboard is a minimum and significant settlements are anticipated.

FILTERS

CATEGORIES OF BASE SOIL MATERIALS

Category	Percent finer than the No. 200
1	>85
2	40-85
3	15-39
4	<15

CRITERIA FOR FILTERS

Base soil category	Base soil description, and percent finer than No. 200 (0.075mm) sieve 1/	Filter criteria 2/
1	Fine silts and clays; more than 85% finer.	3/ $D15 \leq 9 \times d_{85}$
2	Sands, silts, clays, and silty and clayey sands; 40 to 85% finer.	$D_{15} \leq 0.7$ mm
3	Silty and clayey sands and gravels; 15 to 39% finer.	4,5/ $D_{15} \leq \frac{40-A}{40-15}(4 \times d_{85} - 0.7\text{mm}) + 0.7\text{mm}$
4	Sands and gravels; less than 15% finer.	6/ $D_{15} \leq 4 \times d_{85}$

1/ Category designation for soil containing particles larger than 4.75 mm is determined from a gradation curve of the base soil which has been adjusted to 100% passing the No. 4 (4.75 mm) sieve.

2/ Filters are to have a maximum particle size of 3 inches (75 mm) and a maximum of 5% passing the No. 200 (0.075 mm) sieve *(as determined by wet sieving ASTM C-117-80)* with the plasticity index (PI) of the fines equal to zero. PI is determined on the material passing the No. 40 (0.425 mm) sieve in accordance with ASTM-D-4318. To ensure sufficient permeability, filters are to have a D_{15} size equal to or great than 4 x d_{15} but no smaller than 0.1 mm.

3/ When 9 x d_{85} is less than 0.2 mm, use 0.2 mm.

4/ A = percent passing the No. 200 (0.075 mm) sieve after any regrading.

5/ When 4 x d_{85} is less than 0.7 mm, use 0.7 mm.

6/ In category 4, the d_{85} may e determined from the original gradation curve of the base soil without adjustments for particles larger than 4.75 mm.

D10 and D90 LIMITS FOR PREVENTING SEGREGATION

Minimum D (mm)	Maximum D90 (mm)
<0.5	20
05.-1.0	25
1.0-2.0	30
2.0-5.0	40
5.0-10	50
10-50	60

GEOSYNTHETICS

Redundancy - The DSO will not accept a project design where a geosynthetic is the sole element employed to perform a "critical function". A "critical function" is defined as an element of the impounding barrier that were it to fail, there could be a catastrophic release of the reservoir. A redundant design feature is required to provide reasonable assurance of satisfactory long term performance. This redundant feature need not achieve the same level of overall performance as the geosynthetic element; it simply must prevent an uncontrolled release of the reservoir contents.

Materials Quality Control - Geomembranes must satisfy the minimum specifications of a recognized geosynthetics material standard.

Liner Installer's Qualifications - The specifications shall cite the minimum experience the contractor shall have had with the particular type of liner(s) they propose to install.

Field Seam Testing - The DSO shall be provided with details of the testing program. Guidance on accepted field practice is contained in EPA/530/SW-91-051.

HYDRAULIC ISSUES

LOW LEVEL CONDUITS

Conduit seepage control with filter-drain diaphragms - All low level, outlet conduits embedded within the soil phase of the embankment or foundation shall be provided with filter-drain diaphragms.

Vents - An atmospheric vent is needed near the entrance of the low level conduit to stabilize the flow and preclude the occurrence of siphon action and slug flow.

DESIGN MINIMUMS FOR NON-PRESSURIZED CONDUITS
FOR EARTHEN DAMS WITH LOW DOWNSTREAM HAZARD CLASSIFICATIONS
(Design Step Levels 1 and 2)

ITEM - ISSUE	REQUIRED MINIMUM/DESIGN PRACTICE
Minimum Pipe Size[1]	12 inch diameter for concrete encased pipe, otherwise 15 inch diameter Provisions must be available to pass the normal reservoir inflow during periods of high runoff while still pulling the reservoir down within a span of a few days to weeks for inspection, repairs or emergency purposes. This discharge capacity may be obtained from use of the low level outlet and/or from other permanent or temporary hydraulic systems
Pipe Gauge or Wall Thickness	Adequate considering anticipated construction and service loads, abrasion, service life and varying subgrade support
Pipe Joints	Rubber gasketed joints are required, except for welded pipes For corrugated metal pipe, widest available bolted connectors are required.
Concrete Encasement	Required for pressurized conduits din small dams and for all conduits in intermediate and large dams[2]
Upstream Control Valve to Regulate Water Releases[3]	Required
Atmospheric Vent for Low Level Outlet	Required on all conduits with the exception of pressurized conduits in small dams
Filter-Drainage Diaphragm	Required
Low Permeability Pipe Bedding Zone	Bedding soil upstream of the filter-drainage diaphragm must be of equal or lower permeability than that of the adjacent soil and must satisfy filter criteria for all surrounding soils.

1 Use straight alignment whenever practicable to facilitate future sleeving of the pipe

2 Pipe cradle scheme considered for non-pressurized pipes in stormwater detention facilities with temporary pools

3 Not required on conduits for drop inlet, culvert spillways or conduits where inflow is regulated by intake structures

DESIGN MINIMUMS FOR CONDUITS FOR EARTHEN DAMS
WITH HIGH OR SIGNIFICANT DOWNSTREAM HAZARD CLASSIFICATIONS
(Design Step Levels 3 and Greater)

ITEM - ISSUE	PERMANENT OR SEASONAL POOL			TEMPORARY POOL/INTERMITTENT RESERVOIR OPERATION		
	SMALL DAM	INTERMEDIATE DAM	LARGE DAM	SMALL DAM	INTERMEDIATE DAM	LARGE DAM
Minimum pipe Diameter[1,2,3]	12"	12"	12"	12"	12"	12"
Complete Concrete Encasement of Pipe[4]	Required[5]	Required	Required	Required[5]	Required[5]	Required[5]
Upstream Shutoff or Control Valve[6]	Required	Required	Required	Required[7]	Required[7]	Required[7]
Atmospheric Vent for Low Level Outlet	Required	Required	Required	Required	Required	Required
Low Permeability Foundation and Backfill	All earthen materials upstream of filter-drainage diaphragm must have a permeability less than or equal to that of the surrounding material and must satisfy filter criteria for all adjacent materials.			All earthen materials upstream of filter-drainage diaphragm must have a permeability less than or equal to that of the surrounding material and must satisfy filter criteria for all adjacent materials.		
Filter-Drainage Diaphragm	Required	Required[8]	Required[8]	Required	Required[8]	Required[8]

1 Use straight alignment whenever practicable to facilitate future sleeving of the pipe

2 Outlet should be sized to be able to pass the normal reservoir inflow during the high runoff period while still capable of pulling the reservoir down within a span of a few weeks

3 Pipe gauge or wall thickness adequate to account for abrasion, long term durability and other site-specific concerns

4 Minimum of 6 inches of reinforced concrete for encasement of pipe section

5 Pipe cradle in combination with precast reinforced concrete pipe may be used where the design can be justified on the basis of favorable site conditions

6 Not required for conduits on drop inlet spillways and for conduits where the inflow is regulated by intake structures

7 May not be required for stormwater detention and other flood control projects

8 The chimney drain zone (*Section 3.2 Embankment Geometry and Zoning*) generally satisfies this requirement

PRESSURIZED CONDUITS

Concrete Encasement - To the extent practicable, pressurized conduits should be routed outside of the embankment toe. Where the pipe is within the embankment footprint, it shall be encased in concrete. The principal exceptions to encasement are:

> **Conduits -** The requirement for concrete encasement may be waived if the engineer can show that: 1) the downstream hazard setting is low, 2) although a significant portion of the embankment could be sluiced away, an uncontrolled release of the reservoir contents is unlikely and 3) the owner acknowledges acceptance of the increase risked of problems.
>
> **Mine tailings discharge lines -** Often sections of the slurry lines are routed over the dam crest. Where it is necessary to frequently move these lines, it is generally impractical to provide concrete or other forms of encasement for the piping. The DSO has accepted schemes where "critical pipe runs", those segments of pipe located within or upon the exterior dikes, were sleeved by running these lines inside of larger, jointed, corrugated metal pipes.

Vents - An atmospheric vent is required immediately downstream of the upstream gate of valve on pressurized low level outlet conduits at intermediate and large dams to minimize the effects of cavitation and/or vacuum buckling.

Valving - Pressurized pipes require valves or other means of shut-off at the upstream end.

PRINCIPAL OR SERVICE SPILLWAY

Concrete lined spillway underdrains - All concrete lined chutes that serve as principal spillways shall be provided with underdrains. Underdrains may be omitted in the following circumstances:

> The spillway will be cast directly on rock. In this eventuality footing drains probably will be necessary for the sidewalls of the spillway, or
>
> The avenues of seepage have been properly cutoff and the concrete chute serves as an emergency spillway that passes flows only a few times over the project life.

Atmospheric Venting or Aeration - Venting of outlet conduits for culvert spillways, drop inlet spillways and morning glory spillways is usually needed to preclude the occurrence of slug flow. Likewise, aeration of the nappe just below the crest on drop inlet and morning glory spillways is needed to stabilize the flow pattern.

Energy Dissipation at Spillway Outlet - During large floods, flow conditions are usually supercritical at the terminus of the spillway conveyance section (chute, conduit, etc.). The discharge is characterized as having high velocities and severe soil erosion capability. Measures must be taken to dissipate the excess energy and control the flow before returning it to the receiving stream. It is anticipated that returning flows to the receiving channel still will precipitate some degree of erosion under extreme flood flows. Measures must be taken to assure that this erosion will not jeopardize the integrity of the spillway or the impounding barrier.

Contraction & Expansion Joints for Spillway Chutes - Contraction-expansion joints must be provided to maintain floor alignment while allowing for floor slab movement. The joint must be supported by a corbel-like pad.

Underdrains & Waterstops - Drains are required beneath contraction-expansion joints to prevent joint leakage from saturating the subgrade beneath the spillway slab and/or producing uplift pressures. To minimize the magnitude of seepage entering the underdrain it is necessary to provide waterstops.

EMERGENCY SPILLWAYS

Resistance to Debris Blockage - Floating debris often accompanies moderate and extreme flood flows particularly from heavily forested, steep, mountainous areas. Features of the spillway and approach area should be incorporated to allow passage of floating debris, or debris control features such as log booms should be utilized.

Positive Control of Discharge - Spillway design must provide for positive control of the discharge by establishing either one discharge control point or, where multiple discharge control points are necessary, to provide a smooth transition between control points.
Atmospheric Venting or Aeration of the Flow - Venting is required for culvert spillways, drop inlet spillways and morning glory spillways.

DEBRIS PROTECTION FOR HYDRAULIC STRUCTURES

Trashracks - Tightly spaced bars on trashracks are easily clogged by small debris. Trashracks should be designed with a surface area of from 3 to 5 times that of the entrance area which it is protecting. Bar spacing on the trashrack should be as large as practicable, subject to the constraint that whatever passes through the trashrack must freely pass through the conveyance conduit or channel. For projects located near developed areas, bar spacings must also not be so large as to pose an attractive nuisance and be a threat to the safety of children who could fall through the openings.

RESERVOIR FREEBOARD

Table values represent minimums, they should be adjusted as necessary to account for wind/wave action and other pertinent factors.

DESIGN MINIMUMS IN SELECTING RESERVOIR FREEBOARD

RESERVOIR FREEBOARD (FEET)	SMALL DAM	INTERMEDIATE DAM	LARGE DAM
NORMAL FREEBOARD	2.00	3.50	5.00
MINIMUM FREEBOARD	0.50	0.75	1.00

STRUCTURAL ELEMENTS AND ISSUES

Concrete Cover - ACI minimum cover depths are routinely increased at hydraulic structures for durability concerns from the exposure to repeated, we-dry and freeze-thaw cycles.

CONCRETE COVER FOR STEEL REINFORCEMENT

Location	Minimum Cover, Inches
Concrete cast against and permanently exposed to earth (or backfill)	3
Concrete exposed to earth or weather	2
Concrete not exposed to weather or in contact with ground	1½

Joints and Waterstops - While a rational analysis is possible for joint spacing, in practice calculated joint spacings are routinely reduced to account for non-uniform subgrade restraint and the potential for high thermal gradients. For most walls and slabs, joint spacing is on the order of 15 to 40 feet. Smaller spacings are used when relatively large thermal gradients are anticipated and on the estimated degree of slab restraint. Where control joints are necessary and they shall be submerged continuously or intermittently, they shall include a waterstop.

OPERATION AND MAINTENANCE PLANS

All projects shall have an Operation and Maintenance Plan as per WAC 173-175-210. Where the operation plan allows the seasonal use of stop logs, the scheme for stop log use must be approved by the Dam Safety office.

Remote, telemetered operation of any gates must have back up by on-site operators during extreme flood conditions.

INSTRUMENTATION PROGRAMS

EMBANKMENT INSTRUMENTATION

DAM SIZE CLASSIFICATION	MINIMUM INSTRUMENTATION ELEMENTS
SMALL DAM (Less Than 15 Feet High)	• Reservoir staff gage.
INTERMEDIATE SIZE DAM (15 Feet to 50 Feet in Height)	• Reservoir staff gage. • Underdrain outlet volume measurement scheme. • Simple settlement/displacement monuments in the dam crest referenced to a fix points on either abutment.
LARGE DAM (50 Feet or Greater in Height)	• Reservoir staff gage. • Underdrain outlet volume measurement scheme. • Simple settlement/displacement monuments in the dam crest referenced to a fix points on either abutment.