DAM SAFETY GUIDEBOOK

ARIZONA EDITION

1986

INSTRUMENTATION OF DAMS
WORKSHOP ON SAFETY OF DAMS

Arizona Department of Water Resources
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A dam and its safe operation provide economic and other benefits to the shareholders and the people living downstream. However, there can be tremendous adverse consequences if the dam fails to perform as intended. As a dam owner, operating agent, administrator, regulator or designer, there are many areas of responsibilities to assure that the dam is operated in a proper and safe manner.

Five of the major elements that help assure the dam functions as intended include:

1. Operating the dam within the limits established when designed.

2. Maintaining the structure such that it can perform its function.

3. Reviewing the original design criteria and determining whether this criteria is correct for the present operation.

4. Modifying the structure or operation, if conditions have changed so that the dam can continue to be operated safely.

5. Inspecting the dam regularly for changed conditions.

Instrumentation of a dam plays a key role in each of the above five steps. Instrumentation should be thought of in two parts. The first part is a periodic visual and physical inspection of all exposed portions of the dam and appurtenant structures.

This includes the upstream, downstream, crest and abutment areas, the control or outlet works, reservoir area, surrounding natural slopes, and downstream channel areas.

All of these areas should be documented, both in writing and with photographs. As a minimum recommendation, detailed photographs of the dam should be taken every three years.

More frequent intervals may be required as dictated by field conditions. The photographs should be cataloged, dated, descriptions provided, and a duplicate set kept in a safe and separate storage area. The photos should be preserved for the life of the structure. These can be invaluable aids in determining how the dam is performing.

The first part of the instrumentation program leads to the second part. From the inspection, problem or potential problem areas are identified. The existing instrumentation data is then used to further identify the possible causes of the problem. As an alternative, instruments may be installed to obtain data to help determine what has caused the problem. From the field inspection and the analysis of the instrumentation data, possible corrective actions can be identified.

The instrumentation program includes the monitoring of the dam and its foundation for both external and internal performance. The monitoring should be conducted on a regular basis and should ideally begin when the design is initiated.

The instrument layout should be developed when the geotechnical investigation work for the dam is started. The program should continue from initial field drilling through construction and operation and until the structure is finally safely breached and abandoned.

Purpose of Instrumentation

The purpose of instrumenting a dam is to monitor the dam's performance by means of measuring its structural behavior. The objectives are:

1. To provide information for assessing the performance of the dam, to identify any indications of adverse conditions, and to assist in the evaluation of the structural safety of the dam during construction, reservoir filling and service operation.

2. To verify that the dam performs as assumed during the design.

3. To provide data that will assist future designers. Oftentimes conservative assumptions are required by designers due to the lack of knowledge on the

actual performance of dams. Instrumentation helps to provide information that will help designers make better assumptions.

4. To provide early warning of conditions that may jeopardize the safe operation of a dam.

Types of Instrumentation

Instrumentation may be classified by types of measurements made. There are five basic types of measurements, which are:

1. Movements - horizontal and vertical, which are sometimes referred to as translational. There are also rotational, relative, strain and differential movements.

2. Stresses - reactions as a result of load(s) applied to the dam and/or foundation.

3. Groundwater and pore pressure, which can be uplift pressures, groundwater, and/or seepage.

4. Temperature - atmospheric, water, concrete, and/or soil or foundation.

5. Seismic effects - accelerations or displacements.

Although the majority of these types of measurements are applicable to both concrete and embankment dams, measurements of all of these parameters is normally not required except for large (heights greater than 100 feet and storage volume of more than 50,000 acre-feet) dams. Presently none of the large dams in Arizona have seismic recorders installed.

The number and types of instruments installed on each dam is very site specific. A thorough understanding of the dam and its actual or anticipated performance is needed to develop a total instrumentation program. The program should be developed during the design and construction phases. The critical areas can be properly instrumented, and the dam can be monitored during the critical first reservoir filling period.

The measurement methods can vary from the simplest form of survey monuments for vertical movement or open standpipes for groundwater levels to very complex electronic extensometers or piezometers. The simplest form of instruments should be used that will perform the function required. If there are critical sections in a dam, duplicate or alternate methods to monitor the deflections, stresses, or water levels should be considered.

Development of Instrumentation Program

Ideally the instrumentation of a dam should be included as part of the design process. The design assumptions and parameters should be used to determine how much settlement, deflection and seepage is anticipated. From this data, instruments can be designed and specified as part of the construction contract. There will be better assurance of installing the instruments properly and in the right locations to monitor the dam's performance. This also allows for a broader selection of instruments and less expensive installation.

However, if instruments are not installed and design information is not available, then a monitoring program can still be installed. The process can often be more complex in these situations. The first step is to make a detailed inspection of the dam.

A general evaluation must be made of the structure that should be supplemented with a review of information on the operation of the dam. If drawings, plans, soil reports or previous inspection reports are available, these should be reviewed. This data may provide clues of areas of possible problems that could be verified through installation of monitoring equipment.

After analyzing data from the inspection and a review of existing information, a determination should be made of what additional information is needed. If information on the foundation or the material and/or condition of the material in the dam is required, a preliminary instrumentation program should be designed. In this manner soil boring holes could serve a dual purpose. If the drill holes are to serve a dual purpose, the specifications must be carefully developed. The use of drilling agents, such as bentonite or chemical stabilizers may have adverse impacts on the installation of piezometers. The drilling fluid can plug the soil formation that is to be monitored and create misleading information.

There are several other questions that must be answered in determining type and number of instruments to install. Some of these include:

1. What is the anticipated service life of the dam?

2. What is the cost of the instruments? These costs should include not only the initial installed cost, but also the anticipated service life and cost to take the readings. A piezometer can cost anywhere from several hundred dollars to thousands. To these costs, the price of the readout or recording device must be added.

3. What is the accuracy of the readings required? There are simple plastic gages that can be used to monitor cracks in concrete. These cost from $10 to $15 each and can measure movements in two directions to an accuracy of 0.1 mm (millimeters). There are also strain gages that can measure to 0.0001 inches, micrometer measured sets of balls that can be measured to 0.001 inch, and numerous other devices that have varying degrees of accuracy.

   Similar variations in accuracy can be specified for vertical and horizontal movements. The primary consideration should be what is the accuracy required.

4. What are the environmental conditions for the instruments? Will they be continuously immersed in water, is there a freeze-thaw problem, are there extreme temperature changes, is the area covered with snow at times, are there corrosive elements in the soil or water, are they in direct sun, or will local water runoff affect them? All of these factors should be considered when preparing specifications for the purchase and installation of the instruments.

5. Are redundant systems required to validate data or provide a back-up? If the data is critical to the safe performance of the dam, then alternate methods should be used to verify the measurements and to provide back-up.

6. How is the data validated? It is recommended that if possible the same personnel take the readings each time. The data should also be taken in the same way. Do not use a level for one set of readings, a transit the next time, and an EDM and transit the following time. Be consistent with the method, the way data is recorded, and when the data is read. Don't take crack measurement readings one day in the direct sun and the following time in the shade. The results can give misleading information.

7. Where will the control points be established? Horizontal and vertical reference points should be located in areas that will not be influenced by the loading from the dam or reservoir. They should be set in rock outcroppings that are geologically stable and at least two points should be established for reference. These references or base points should be accurately described in a field book and methods developed to reestablish these points in the event they are accidently destroyed. The measurement and reference points should be located in areas that are easily accessible and consider the weight and size of instrument required to take the readings.

8. What is the response time of the instrument? As an example, in the table below are the estimated response times for standpipe and diaphragm piezometers. The response time can vary from a few hours to many days depending on the size of the pipe, type instrument, and permeability of the soil.

Table 1

Permeability (cm./sec.)	2" Diameter Standpipe	3/8" Diameter Standpipe	Diaphragm Piezometer
10^{-3}	2 Hrs.	0	0
10^{-4}	12 Hrs.	0	0
10^{-5}	6 Days	0	0
10^{-6}	50 Days	1.6 Hrs.	0
10^{-7}	600 Days	12 Hrs.	0
10^{-8}		5 Days	0
10^{-9}		70 Days	0
10^{-10}			1 Hr.

Approximate 90% Response Time.

9. Who will collect, record, analyze and plot the data?

10. How often are the instrument readings taken? The times may vary with types of equipment, field conditions, and condition of the dam. In the first few years during and after construction, the instruments might need to be checked weekly or monthly. During the initial reservoir filling, the readings might be taken daily. After several years service, a normal cycle of twice a year might be required. Normally the instruments should be read when the reservoir is at its maximum and when it is minimum. It is also recommended that the instruments be checked if an earthquake should occur near the dam (i.e., vicinity close enough to be felt at the site) and after any major floods (i.e., when spillways have been operated).

Miscellaneous

Attached to this paper are several sketches of some of the more common methods of installing various instruments or measuring devices. There is also attached a bibliography of books and publications that provide more details on instrumentation programs. These publications cover such topics as instrumentation for concrete dams or embankment dams, calibration of distance measuring devices, and automation of collecting and recording data.

There are two tables attached that describe potential incidents (problems) which can occur with concrete or embankment dams. Many of these conditions can be identified through instrumentation. The instrumentation can provide an early warning system before a significant incident occurs.

Whenever data is collected, it should be for a specific purpose. When collected, the interpretation and analysis of the data is as important a function as the accurate recording of the data. The information should not collect dust in someone's office but should be used as an important part of the evaluation of a dam's performance.

Whenever data is collected, the atmospheric temperature, reservoir and tailwater elevations, and date should be recorded. This information plays a critical role in evaluating the data from the instruments.

Conclusions and Recommendations

1. Each dam should have instrumentation installed to evaluate the performance of the structure. Ideally the instrumentation program should be developed during the design stage and installed when the dam is constructed. This method will minimize the cost and allow for monitoring of the dam during construction and during the initial reservoir filling operation.

2. Instrumentation should be designed to be simple, serviceable, and durable. It must serve its purpose throughout the life of the structure and all costs should be considered when designing the program.

3. The instrumentation program should be continually monitored and modified as required to properly evaluate the changing condition of the dam.

4. The degree of accuracy of measurements must be determined and the total system evaluated to assure that this accuracy can be obtained.

5. Some of the simplest instruments that can provide valuable information are cameras (photographic records), thermometers, stop watches (for estimating flows), beakers (water samples), and fences or handrails (check for offsets or signs of lateral movement).

6. Even the simple, low hazard, small dam should have some instrumentation installed. The instrumentation can be simple but may provide an early warning system to identify problem areas. Some maintenance work is cheaper than losing the benefits if a dam should fail.

An instrumentation program becomes an integral part of the safe operation of any dam. It should be installed at the time of construction and should continue in operation throughout the life of the structure. The data should be analyzed, and the instrumentation program modified whenever significant changes in the structure occur. The data provides an early warning system that may help prevent a small problem from becoming a major one. The instrumentation program should be designed to be as simple as possible but provide the data required to evaluate the dam's performance.

Larry K. Lambert, P.E.
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Safety of Dams
Salt River Project

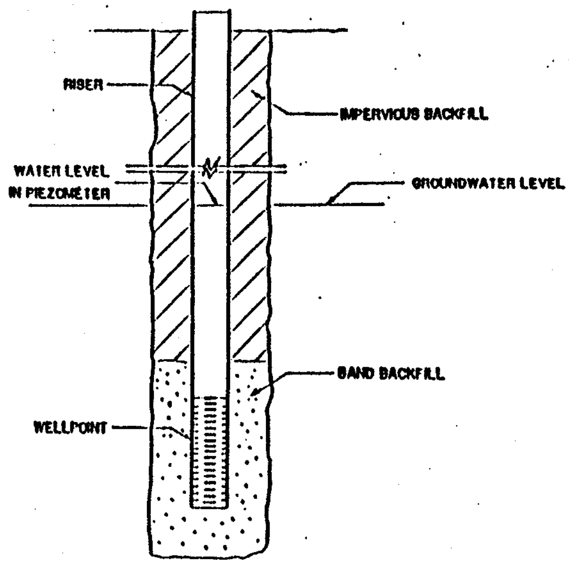

Figure 1 - Wellpoint Piezometer

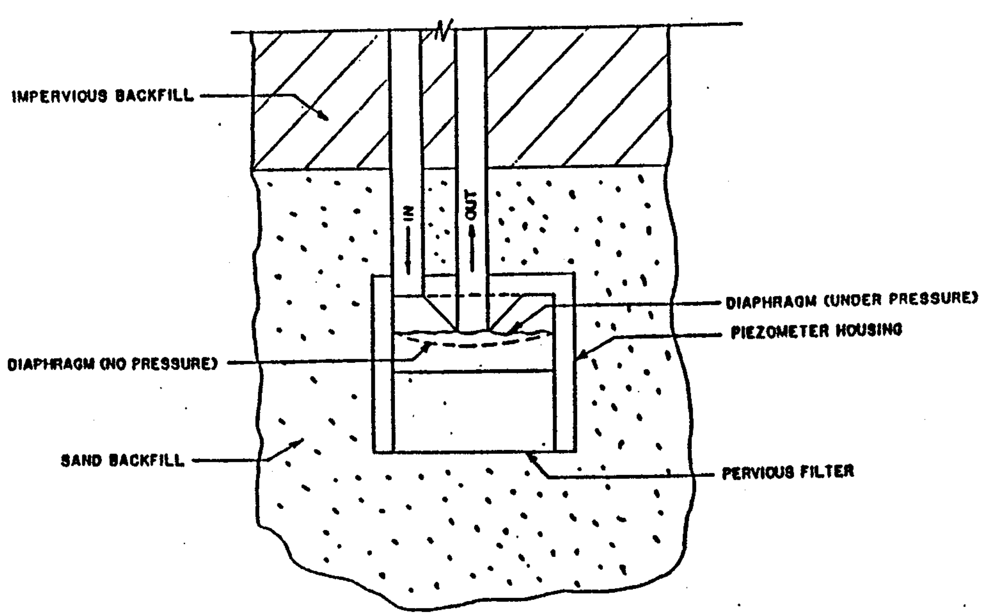

Figure 2 - Pneumatic Piezometer

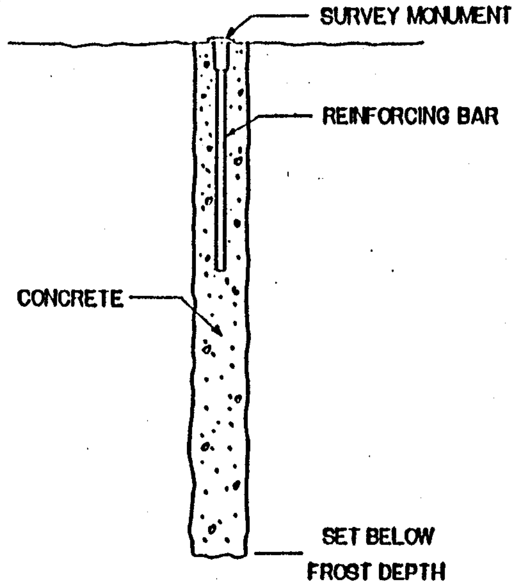

Figure 3 - Typical surface movement device

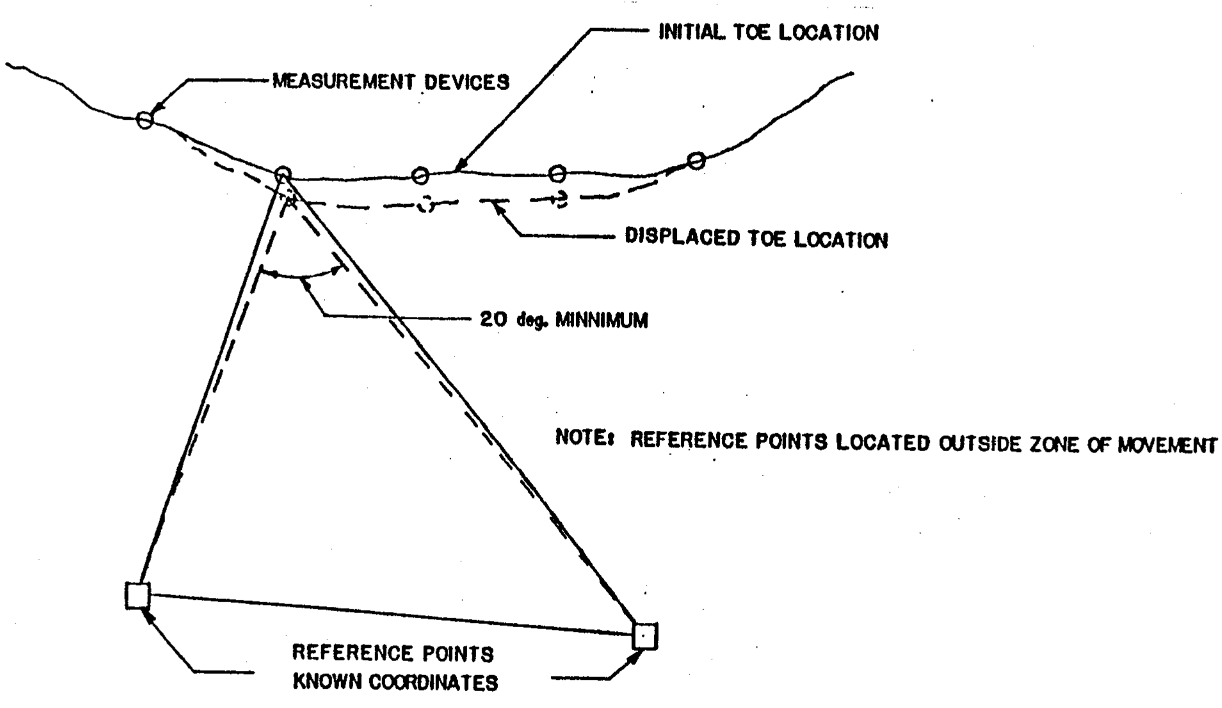

Figure 4 - Typical arrangement for movement devices

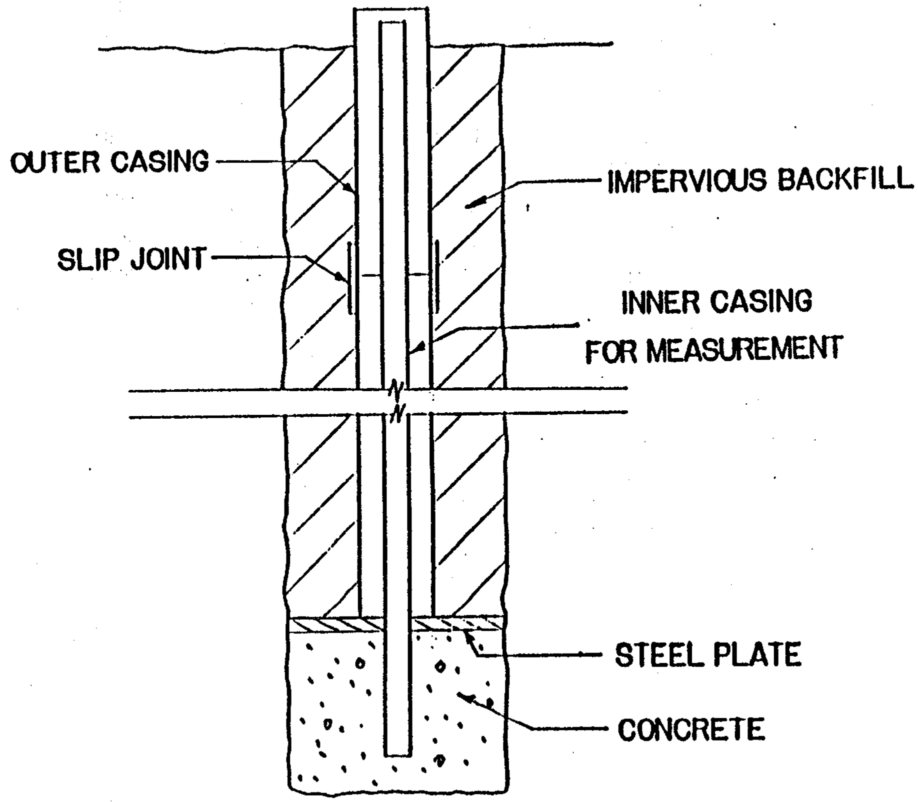

Figure 5 - Typical installation of extensometer

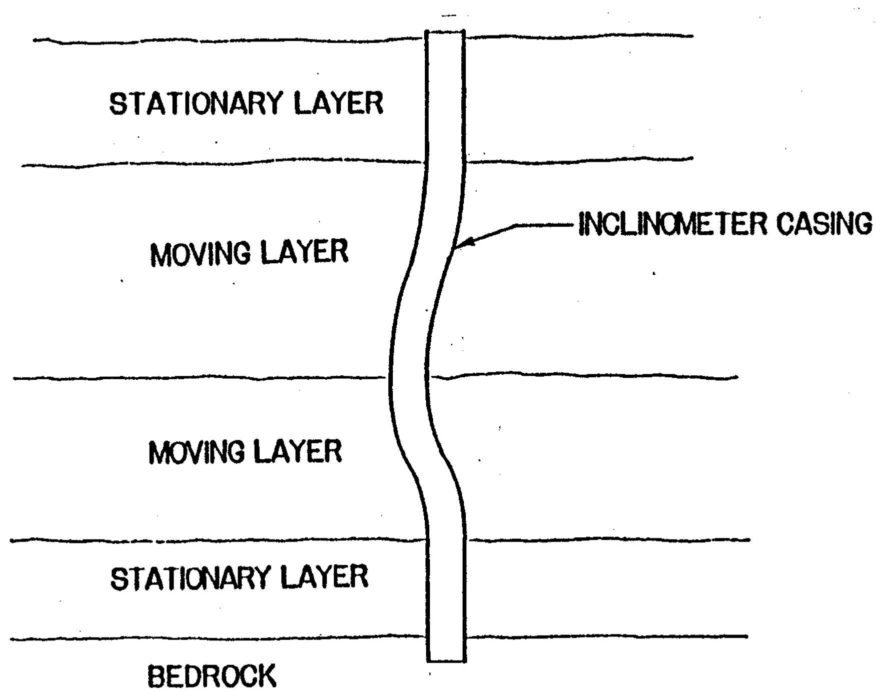

Figure 6 - Typical inclinometer installation

CHAPTER VIII

SELECTED BIBLIOGRAPHY

Published literature provides copious references with respect to instrumentation and its uses, measurements and their significance, and personnel and institutional requirements for competent and reliable monitoring systems. Hanna, included in list below, identifies approximately 1000 references related to geotechnical engineering (cross-indexed by author and subject). The USCOLD Committee on Earthquakes identifies over 300 references relevant to performance of concrete and earth and rockfill dams during earthquakes. The preponderance of literature contains technical information on specific devices, site-specific applications, conceptual or theoretical assessments, or selective performance evaluations; comprehensive references are the exception.

The general considerations provided by members of this Measurements Committee reflect extensive readings of published and unpublished documents, liaison with practitioners and researchers, and personal experience. A few "References Cited" identify detailed information on specific topics. "Reference Sources in the United States" identifies examples of entities that periodically disseminate information relevant to performance monitoring of dams. "References for Selected Reading" identifies recent publications reflecting current philosophy and general practices.

An effort has been made to minimize references to recent, more timely, material. The inclusion of references does not constitute an endorsement, nor should the exclusion be construed to indicate rejection.

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Table 4-1

DAM INSPECTION AND PERFORMANCE EVALUATION
INVENTORY OF POTENTIAL INCIDENTS
EMBANKMENT DAMS

NO.	FEATURE	DEFECT	INDICATORS	POSSIBLE CAUSES	DEGREE OF DEFICIENCY			POTENTIAL EFFECTS	POSSIBLE REMEDIAL MEASURES
					MINOR	SERIOUS	VERY SERIOUS		
4.1	CREST AND ROADWAY								
4.1.1		TRANSVERSE CRACKING.	CRACKS IN ROADWAY, SIDEWALK AND PARAPETS. DIPS AND BUMPS IN PAVEMENT SURFACES. TILTING OF POSTS AND PARAPETS. DISPLACEMENT MEASUREMENTS.	DIFFERENTIAL FOUNDATION SETTLEMENT. SHARP IRREGULARITIES IN ABUTMENT GEOMETRY. EARTHQUAKE. SHRINKAGE & DESICCATION OF CORE MATERIAL		*	IF TRANSVERSE CRACKS ARE DEEP AND EXTEND BELOW RES. WATER LEVEL.	LOSS OF FREEBOARD. EMBANKMENT CRACKING. INCREASED SEEPAGE. BREACHING AND FAILURE OF DAM	LOWER RESERVOIR. DETERMINE EXTENT OF CRACKING BY TEST PITS OR BORINGS. EXCAVATE TO DEPTH OF CRACKS AND BACKFILL WITH COMPACTED IMPERVIOUS SOIL. BENTONITE GROUTING
4.1.2		LONGITUDINAL CRACKING	CRACKS IN ROADWAY, SIDEWALKS AND PARAPETS. DIPS AND BUMPS IN PAVEMENT SURFACES. TILTING OF POSTS AND PARAPETS. DISPLACEMENT MEASUREMENTS.	RAPID RESERVOIR DRAWDOWN. EARTHQUAKE. UNEQUAL DEFORMATION OF SHELL & CORE MATERIALS. TRANSVERSE CRACKING WITHIN CORE OF DAM. SETTLEMENT OF POORLY COMPACTED ROCKFILL SHELLS.		*	IF CRACKS DAYLIGHT BELOW WATER LEVEL & EXTEND INTO CORE	EMBANKMENT CRACKING. INCREASED SEEPAGE. SLOPE FAILURE.	SEE 4.1.1
4.1.3		EXCESSIVE SETTLEMENT.	MEASUREMENTS OF SURVEY MONUMENTS OR INTERNAL SETTLEMENT. DIPS IN ROAD. DEPRESSIONS ON SLOPES	EMBANKMENT AND FOUNDATION CONSOLIDATION.		*	IF CREST SETTLES TO < 2 FT (0.6M) OF NORMAL WATER LEVEL	LOSS OF FREEBOARD. RISK OF OVERTOPPING	RAISE DAM BY PLACING ADDITIONAL COMPACTED SOIL AFTER STUDY OF FOUNDATION CONDITIONS. LOWER RESERVOIR
4.1.4		LOSS OF EMBANKMENT MATERIAL.	SUDDEN INCREASE IN SETTLEMENT AND LEAKAGE. SINKHOLES. DISCOLORATION OF SEEPAGE WATER	INTERNAL EROSION OF EMBANKMENT OR FOUNDATION. PIPING. SINKHOLES IN SOLUBLE FOUNDATION.			*	PROGRESSIVE EROSION. BREACHING AND FAILURE OF DAM	LOWER RESERVOIR. REMOVE MATERIALS FROM SINKHOLES AND REPLACE WITH COMPACTED FILL. & RESTORE DAM. SEE ALSO 4.3.1
4.1.5		SLOPE INSTABILITY.	LONGITUDINAL CRACKS WITH OR WITHOUT VERTICAL DISPLACEMENT. DISPLACEMENT MEASUREMENTS. SLIDES.	INSUFFICIENT SHEAR STRENGTH OF EMBANKMENT OR FOUNDATION MATERIALS.			*	SUCCESSIVE FAILURES MOVE UPSLOPE AS UNSTABLE SCARPS SLIDE AGAIN. POSSIBLE LOSS OF FREEBOARD AND LOSS OF EMBANKMENT BY BREACHING.	SEE 4.2.2 & 4.3.2
4.1.6		EXCESSIVE HORIZONTAL DISPLACEMENT.	MEASUREMENTS OF SURVEY MONUMENTS. LATERAL WEAVES IN WALLS OR ROADWAY.	SLIDING OF THE EMBANKMENT AND OR FOUNDATION			*	LOSS OF DAM	SEE 4.3.2

Table 4-1 (continued)

NO.	FEATURE	DEFECT	INDICATORS	POSSIBLE CAUSES	DEGREE OF DEFICIENCY			POTENTIAL EFFECTS	POSSIBLE REMEDIAL MEASURES
					MINOR	SERIOUS	VERY SERIOUS		
4.2	UPSTREAM FACE								
4.2.1		LOSS OF EMBANKMENT MATERIAL & EXCESSIVE LEAKAGE THRU DAM	BUBBLING OR VORTICES IN RESERVOIR SURFACE UNPLANNED DROP IN RESERVOIR RESERVOIR LEVEL. SINKHOLES.	INTERNAL EROSION OF EMBANKMENT OR FOUNDATION. PIPING. MAJOR CRACKING IN CONCRETE OR ASPHALT DISCOLORATION OF SEEPAGE WATER			*	INCREASING LEAKAGE. PIPING & EROSION OF DAM MATERIAL. FAILURE OF DAM.	LOWER RESERVOIR SEE 4.1.4 & 4.3.1
4.2.2		SLOPE INSTABILITY	CRACKING. RAVELLING. SLIDING. BULGING. DEPRESSINGS. SCARPS	INSUFFICIENT SHEAR STRENGTH OF EMBANKMENT OR FOUNDATION MATERIALS. EARTHQUAKE. RAPID RESERVOIR DRAWDOWN.		*		IF DEEP-SEATED, LOCAL SLIDES PROGRESSIVELY MOVING UPSLOPE; POSSIBLE LOSS OF FREEBOARD; BLOCKAGE OF INTAKES AND CONDUITS BY SLIDE DEBRIS	LOWER RESERVOIR. PLACE COMPACTED OR DUMPED FILL BUTTRESS AGAINST UPSTREAM SLOPE. REMOVE SLIDE MATERIAL & REBUILD PORTION OF DAM.
4.2.3		CRACKING	TRANSVERSE &/OR LONGITUDINAL CRACKING; DISPLACEMENT MEASUREMENT.	SEE 4.1.1 & 4.1.2		SEE 4.1.1 & 4.1.2 *	*	SEE 4.1.1	SEE 4.1.1
4.2.4		SLOPE EROSION	SLOPE EROSION; LOSS OF RIPRAP. GULLIES. MATERIAL ACCUMULATED AT TOE OF SLOPE	INSUFFICIENT PROTECTION OF FACE AGAINST WAVE ACTION OR SURFACE RUNOFF WEATHERING OF RIPRAP.	*			PROGRESSIVELY DEEPER EROSION GULLIES; LOSS OF RIPRAP & EMBANKMENT MATERIAL.	REMOVE LOOSE MATERIAL AND REPLACE WITH COMPACTED FILL. PLACE RIPRAP FOR EROSION PROTECTION, OR REPLACE ORIGINAL WITH LARGER SIZE. USE OF GABIONS.
4.2.5		CONCRETE FACING DETERIORATION	CRACKING OF SLABS; SEPARATION OF JOINTS; DAMAGE TO WATERSTOPS. TILTING OF SLABS	UNEVEN EMBANKMENT SETTLEMENT; FREEZE-THAW ACTION		FOR ROCK-FILL DAMS	FOR EARTH-FILL DAMS	LOSS OF WATER. EROSION OF EMBANKMENT MATERIALS. FAILURE OF EARTH-FILL DAM.	REPLACE DAMAGED PANELS; PLACE IMPERVIOUS MATERIALS IN LOCAL DAMAGED AREAS; CONSIDER USING SYNTHETIC MEMBRANES OR COMBINATIONS OF BITUMINOUS EMULSIONS AND POLYMERIC COMPONENTS
4.2.6		CORROSION OF STEEL REINFORCEMENT IN CONCRETE FACING	DETAILED INSPECTION	STEEL EXPOSED TO ATMOSPHERE.		*		SEE 4.2.5	REPAIR OR REPLACE DAMAGED PANELS WITH ADDITIONAL COVER & REINFORCEMENT.
4.2.7		DETERIORATION OF STEEL FACING	BUCKLING. BENDING. PUNCTURES LEAKAGE	UNEVEN EMBANKMENT SETTLEMENT; IMPACT OF LARGE ROCK OR DEBRIS		*	*	SEE 4.2.5	REPLACE DAMAGED PANELS
4.2.8		CORROSION OF STEEL FACING	VISIBLE CORROSION. LEAKAGE	STEEL EXPOSED TO ORGANIC MATTER IN RESERVOIR WATER		*		SEE 4.2.5	REPLACE DAMAGED PANELS. APPLY PROTECTIVE COATING
4.2.9		DETERIORATION OF TIMBER FACING	ROTTEN TIMBERS. LEAKAGE	ROT; FUNGI		FOR ROCK-FILL DAMS	FOR EARTH-FILL DAMS	SEE 4.2.5	REPLACE DETERIORATED TIMBERS OR PANELS. APPLY PROTECTIVE COATING.

Table 4-1 (continued)

NO.	FEATURE	DEFECT	INDICATORS	POSSIBLE CAUSES	DEGREE OF DEFICIENCY			POTENTIAL EFFECTS	POSSIBLE REMEDIAL MEASURES
					MINOR	SERIOUS	VERY SERIOUS		
4.3	DOWNSTREAM FACE AND TOE								
4.3.1		EXCESSIVE EMBANKMENT LEAKAGE	EXCESSIVE OR INCREASED FLOW FROM DRAINS. FLOW TURBIDITY. CHANGE OF FLOW COLOR. SINKHOLES	CONCENTRATED FLOW ALONG EMBANKMENT CRACKS. INTERNAL EROSION OF EMBANKMENT.			*	PROGRESSIVE INTERNAL EROSION OF EMBANKMENT AND INCREASING LEAKAGE POSSIBLE EMBANKMENT FAILURE	EMERGENCY MEASURES INCLUDE: LOWER RESERVOIR LEVEL. BULLDOZE MATERIALS INTO THE RESERVOIR FROM THE EMBANKMENT CREST. PLACE COMPACTED BUTTRESS FILL ON DOWNSTREAM SLOPE WITH PROPERLY DESIGNED FILTER AND DRAIN LAYERS. PERMANENT MEASURES INCLUDE: CONSTRUCT CUTOFF THROUGH DAM AND FOUNDATION. SUPPLEMENT GROUT CURTAIN. CONSTRUCT IMPERVIOUS UPSTREAM BLANKET WITH EMPERVIOUS CLAYEY SOIL OR SYNTHETIC MEMBRANE LINER.
			WET SPOTS. SOFT AREAS. AREAS WITH GROWTH OF HEAVY GREENER VEGETATION	SEEPAGE ALONG PERVIOUS LAYERS BYPASSING INTERNAL DRAIN SYSTEM, OR ALONG CONTACT WITH ROCK OR STRUCTURES		*	IF THERE ARE SOFT AREAS	HIGH PHREATIC LEVEL IN DOWNSTREAM SLOPE COULD LEAD TO SLOPE INSTABILITY	
			BOILS	EXCESSIVELY HIGH EXIT GRADIENT			* IF CONES BUILDING	PROGRESSIVE INTERNAL EROSION OF EMBANKMENT. POSSIBLE EMBANKMENT FAULURE	
			CHANGES IN EFFLUENT TEMPERATURE	CHANGE IN INTERNAL DRAINAGE PATTERNS:		* IF TEMP CHANGE NOT RELATED TO AMBIENT CHANGES		NEW SEEPAGE PATHS	
			SEEPAGE FROM ANIMAL BURROWS OR ALONG TREE ROOTS	CHANNELS FORMED BY DECOMPOSITION OF ORGANIC MATERIAL. LEAKAGE ALONG ANIMAL BURROWS		*		EXCESSIVE SEEPAGE AND WEAKENING OF EMBANKMENT	EXCAVATE ROOTS AND BURROWS. REPLACE WITH COMPACTED FILL. CONTROL RODENTS
4.3.2		SLOPE INSTABILITY	RAVELLING. SHALLOW SLIDES AND CRACKS. SCARPS.	INSUFFICIENT SHEAR STRENGTH OF SOIL WHEN SATURATED BY HEAVY RAINFALL		*	VERY SERIOUS IF ASSOCIATED WITH WET ZONES OR SEEPAGE AT THE DOWNSTREAM TOE	PROGRESSIVE SLOUGHING AND DEEPER SLIDES. FAILURE OF DAM.	FLATTEN DOWNSTREAM SLOPE BY CONSTRUCTING COMPACTED FILL BUTTRESS WITH ADEQUATE FILTERS AND DRAIN LAYERS INSTALL PRESSURE RELIEF WELLS THROUGH FOUNDATION MATERIALS. INSTALL DRAINS IN DOWNSTREAM PORTION OF DAM. CONSTRUCT IMPERVIOUS BLANKET ON UPSTREAM FACE.
			DEEP CRACKS AND SLIDES. BULGES AND DEPRESSIONS. SCARPS.	INSUFFICIENT SHEAR STRENGTH OF EMBANKMENT OR FOUNDATION MATERIALS EARTHQUAKE LOADING			* AS ABOVE	SUCCESSIVE SLIDES MOVE UPSLOPE AS UNSTABLE SCARPS SLIDE AGAIN. POSSIBLE LOSS OF FREEBOARD AND LOSS OF EMBANKMENT BY BREACHING	
4.3.3		SLOPE EROSION	EROSION GULLIES AT SLOPE AND ALONG EMBANKMENT-ABUTMENT CONTACTS.	INSUFFICIENT PROTECTION OF SLOPE AGAINTS SURFACE RUNOFF	*			PROGRESSIVE WEAKENING OF SLOPE AND DEEPER EROSION GULLIES	REMOVE LOOSE MATERIALS AND REPLACE WITH COMPACTED FILL SEED-MULCH SLOPE PLACE RIPRAP.
4.3.4		TRANSVERSE OR LONGITUDINAL CRACKS.	MEASUREMENTS OF SURVEY MONUMENTS OR INTERNAL DISPLACEMENT INSTR.	SEE 4.1.1 DIFFERENTIAL SETTLEMENT OF EMBANKMENT OR FOUNDATION			*	SEE 4.1.1	SEE 4.1.1
4.3.5		CLOGGED DRAINS	INCREASED PIEZOMETRIC PRESSURE	INTERNAL DISPLACEMENT OF FINE PARTICLES. EXCESSIVE VEGETATION AND ROOT GROWTH. CHEMICAL DEPOSITS. IRON BACTERIA		*		HIGH PHREATIC LEVEL IN DOWNSTREAM SLOPE POSSIBLY LEADING TO SLOPE INSTABILITY	SEE 4.3.1 INSTALL NEW DRAINAGE SYSTEM. REMOVE TREES AND ROOTS. CLEAN OUT DRAINS.

Table 5-1

DAM INSPECTION AND PERFORMANCE EVALUATION
INVENTORY OF POTENTIAL INCIDENTS
CONCRETE DAMS

NO.	FEATURE	DEFECT	INDICATORS	POSSIBLE CAUSES	DEGREE OF DEFICIENCY			POTENTIAL EFFECTS	POSSIBLE REMEDIAL MEASURES
					MINOR	SERIOUS	VERY SERIOUS		
1.0	CREST OF DAM								
1.1	CRACKING AND SPALLING	SHALLOW CRACKS	RANDOM PATTERN, NOT NOTICEABLE WHEN CONCRETE DRY	SURFACE SHRINKAGE, AMBIENT TEMP. CHANGES	*			NO EFFECT ON DURABILITY	NONE NECESSARY
		TRANSVERSE CRACKS EXTENDING FROM UPSTREAM TO DOWNSTREAM	EASILY DETECTED OPENING >.04" (1 MM)(IN COLDER WEATHER)	OVERLOADING OF DAM, EARTHQUAKE, FOUNDATION SETTLEMENT	IF CRACK <12" DEEP	IF CRACK >12" DEEP	IF CRACK >10' DEEP	LEAKAGE, ACCELERATED DETERIORATION OF CONCRETE, OVERSTRESSING, PENETRATION OF WEATHERING, EXTENSION OF CRACKING	
		TRANSVERSE CRACKS WITH VERTICAL OR LATERAL OFFSETS	EASILY DETECTED OPENING >.04" (1 MM) OFFSETS >.08" (2 MM)	DIFFERENTIAL SETTLEMENT OF FOUNDATION, EARTHQUAKE, OVERLOADING OF DAM BLOCKS			*	IF PROGRESSIVE INCREASE OF OFFSET, CAN LEAD TO INSTABILITY	LOWER RESERVOIR, ADDITIONAL FOUNDATION TREATMENT
		DEEP RANDOM CRACKS AND "SWELLING" OF CONCRETE	RANDOM OPEN CRACKS, CRUMBLY CONCRETE ALONG CRACKS, SILICA GEL	ALKALI-AGGREGATE REACTION			*	SEE 2.1	SEE 2.1
		CRACKS AND ABRASION IN ROADWAY CONCRETE	RANDOM SHALLOW CRACKS, CONCRETE DAMAGED NEAR JOINTS	EXCESSIVE HEAVY TRAFFIC	*			HIGHER MAINTENANCE COSTS	TRAFFIC CONTROL AND FREQUENT REPAIRS
1.2	CONTRACTION JOINTS	OPENING OF JOINTS	1) NOTICEABLE AT THE SURFACE, OPENING <.08"(2 MM)	SEASONAL TEMPERATURE CHANGE, RESERVOIR DRAWDOWN	*			NO ADVERSE EFFECT ON PERFORMANCE OF DAM	NONE REQUIRED
			2) OPENING >.10"(2.5 MM) WHEN RESERVOIR NEARLY FULL AND IN SUMMER	DEFORMATION OF FOUNDATION, EARTHQUAKE		*		LEAKAGE, IF WATER STOPS DAMAGED	JOINT GROUTING WITH WITH CEMENT,
			3) OFFSETS OR DIFFERENTIAL MOVEMENT AT JOINT	DIFFERENTIAL DEFORMATION OF FOUNDATIONS OF ADJACENT BLOCKS, EARTHQUAKE			IF > 0.10"	IF PROGRESSIVE AND CONTINUOUS CAN CAUSE INSTABILITY OF A BLOCK OF A GRAVITY OR BUTTRESS DAM	LOWER RESERVOIR, ADDITIONAL FOUNDATION TREATMENT
2.0	UPSTREAM FACE OF DAM								
2.1	CRACKING AND SPALLING	SHALLOW CRACKS	RANDOM PATTERN, NOT NOTICEABLE WHEN CONCRETE DRY	SURFACE SHRINKAGE, AMBIENT TEMPERATURE CHANGES	*			NO EFFECT ON DURABILITY	NONE NECESSARY
		VERTICAL AND DIAGONAL CRACKS	EASILY DETECTED, OPENING > .04" (1 MM)(IN COLDER WEATHER)	EXCESSIVE STRESSES, TEMPERATURE DROP IN AREAS OF RESTRAINT	IF CRACK < 5' LONG < 12" DEEP	IF CRACK > 5' LONG > 12" DEEP	IF CRACK > 20' LONG > 5' DEEP	PROGRESSIVE CRACKING INTO BODY OF DAM AND INTO GALLERIES, STRESS CONCENTRATIONS, LEAKAGE	EPOXY GROUTING TO SEAL CRACKS AND RESTORE STRENGTH OF CONCRETE
		SPALLING OF CONCRETE	LARGE PIECES OF CONCRETE DISLODGED, REINFORCING EXPOSED IN BUTTRESS DAMS	FREEZE-THAW ACTION, DIFFERENTIAL MOVEMENT AT JOINTS, STRESS CONCENTRATIONS	IF SPALLS < 6" AND FEW	IF SPALLS > 12" AND SEVERAL	IF SPALLS > 24" SEVERAL & CONC. REINF. EXPOSED	CONSEQUENCES SERIOUS FOR BUTTRESS DAMS WHERE REINFORCED CONCRETE DECK CAN DETERIORATE	PATCH-UP CONCRETE, APPLICATION OF NEW CONCRETE FACING OR SHOTCRETE IF DAMAGE IS EXTENSIVE
		DEEP RANDOM CRACKS AND "SWELLING" OF CONCRETE	RANDOM, OPEN, EXTENSIVE CRACKS, CRUMBLY CONCRETE ALONG CRACKS, SILICA GEL	ALKALI-AGREGATE REACTION			*	REDUCE USEFUL LIFE OF DAM, OR CAUSE INSTABILITY, GENERAL PROGRESSIVE PHYSICAL DETERIORATION	LOWER RESERVOIR LEVEL, RESTRICT OPERATION, EXTENSIVE RECONSTRUCTION OR DEMOLITION OF DAM
2.2	CONTRACTION JOINTS	OPENING OF JOINTS	1) NOTICEABLE AT SURFACE, OPENING <.08", (2 MM)	SEASONAL TEMPERATURE CHANGE, RESERVOIR DRAWDOWN	*			NO ADVERSE EFFECT ON PERFORMANCE OF DAM	NONE REQUIRED
			2) OPENING >.10" WHEN RESERVOIR NEARLY FULL	FOUNDATION DEFORMATION, EARTHQUAKE		*		CAN DAMAGE WATERSTOPS AND CAUSE EXCESSIVE LEAKAGE	JOINT GROUTING WITH CEMENT WHEN JOINT MAXIMUM OPEN
			3) OFFSETS AND DIFFERENTIAL MOVEMENT AT JOINT	DIFFERENTIAL FOUNDATION DEFORMATION OF ADJOINING BLOCKS, EARTHQUAKE		IF .05-.10 INCH	IF >.10"	IF PROGRESSIVE AND CONTINUOUS CAN CAUSE INSTABILITY OF A BLOCK OF GRAVITY OR BUTTRESS DAM	IMPROVE FOUNDATION RESISTANCE, STRENGTHEN DAM, LOWER RESERVOIR

Table 5-1 (continued)

NO.	FEATURE	DEFECT	INDICATORS	POSSIBLE CAUSES	DEGREE OF DEFICIENCY: MINOR	DEGREE OF DEFICIENCY: SERIOUS	DEGREE OF DEFICIENCY: VERY SERIOUS	POTENTIAL EFFECTS	POSSIBLE REMEDIAL MEASURES
3.0	DOWNSTREAM FACE OF DAM								
3.1	CRACKING AND SPALLING	SEE 2.1	SEE 2.1	SEE 2.1				SEE 2.1	SEE 2.1
3.2	CONTRACTION JOINTS	OPENING OF JOINTS	SEE 2.2	SEE 2.2				SEE 2.2	SEE 2.2
		LEAKAGE THRU JOINTS	WET SPOTS, FLOW AND LEACHATE STAINS; PARTICULARLY NEAR BASE OF DAM	DAMAGED WATERSTOPS. CRACKS OR OPEN CONSTRUCTION JOINTS CONNECTING WITH CONTRACTION JOINTS; DIFFERENTIAL MOVEMENT OF BLOCKS	IF FLOW STEADY AND <1.0GPM PER JOINT	IF FLOW PULSATING & > 20GPM PER JOINT	IF FLOW INCR'NG AND > 10GPM PER JOINT	LOSS OF WATER; LEACHING; DAMAGE DUE TO FREEZING THAWING	SEAL THE JOINT DRAIN WITH BENTONITE, GROUT CONTRACTION JOINTS WITH CEMENT
3.3	HORIZONTAL CONSTRUCTION JOINTS	OPENING AND LEAKAGE	WET SPOTS; SEEPAGE AND LEACHATE STAINS	INADEQUATE BOND; POROUS CONCRETE NEAR JOINTS	IF FLOW STEADY AND <1.0GPM PER BLOCK	IF FLOW PULSATING & > 20GPM PER BLOCK	IF FLOW INCR'NG AND >100GPM PER BLOCK	LOSS OF WATER; LEACHING; DAMAGE DUE TO FREEZING THAWING	DRILLED DRAINS TO CONTROL SEEPAGE AND GROUTING
3.4	DRAINS	EXCESSIVE OR INCREASING FLOW; MILKY WATER	INCREASING DRAIN DISCHARGE WITH STEADY RESERVOIR	INTERNAL CRACKING		IF FLOW > 50GPM AND RATE OF INCRSE >1GPM PER DAY	IF FLOW >100GPM AND RATE OF INCRSE > 5GPM PER DAY	LOSS OF WATER; LEACHING; PROGRESSIVE INTERNAL CRACKING; FREEZING THAWING DAMAGE	PLUG EXISTING DRAINS AND DRILL NEW RELIEF DRAINS
4.0	GALLERIES & SHAFTS IN DAM								
4.1	CONDITION OF CONCRETE	CRACKING; IN UPSTREAM LONGITUDINAL GALLERIES	VISIBLE CRACKS WITH NOTICEABLE SEEPAGE	THERMAL GRADIENTS BETWEEN RESERVOIR AND UPSTREAM GALLERIES	IF DRY OR FEW WET SPOTS	SEEPAGE > 5GPM THRU CRACK	SEEPAGE >20GPM AND INCR'NG	FLOODING OF GALLERY PROGRESSIVE CRACKING	SEAL CRACKS BY EPOXY INJECTION
		CRACKING; IN TRANSVERSE GALLERIES	VERTICAL CRACKS CONTINUOUS THRU WALLS, CEILING AND FLOOR	COOLING OF MASS CONCRETE AND RESTRAINT NEAR FOUNDATION	NOT CONT. OPENING <.02" (.5 MM) AND DRY	CONT. OPENING >.04" (1 MM) AND SOME SEEPAGE	CONT. PROG'NG OPENING >.08" (2 MM) LEAKAGE >10GPM	BUILDUP OF INTERNAL PORE PRESSURE; STRESS CONCENTRATIONS; RISK OF INSTABILITY	GROUTING WITH EPOXY
4.2	CONTRACTION JOINTS	OPENING OF JOINTS	SEE 2.2	SEE 2.2				SEE 2.2	SEE 2.2
4.3	DRAINS IN DAM	EXCESSIVE OR INCREASING FLOW; MILKY WATER	SEE 3.4	SEE 3.4				SEE 3.4	SEE 3.4
		PLUGGED DRAINS	FLOW IN DRAIN CHOKED BY CALCIUM AND OTHER DEPOSITS	LIME AND SILICATES LEACHED FROM CONCRETE		IF DRAINS COMPL'Y PLUGGED		BUILDUP OF INTERNAL PORE PRESSURE; OPENING OF INCIPIENT CRACKS IN DAM	REAMING OF EXISTING DRAINS; DRILLING NEW DRAINS
4.4	FOUNDATION DRAINS	EXCESSIVE OR INCREASING FLOW	INCREASE IN DRAIN DISCHARGE WITH STEADY RESERVOIR	INADEQUATE GROUT CURTAIN; PIPING OF FINES FROM FOUNDATION AND ABUTMENT STRATA		> 50GPM AND INCR'NG <1GPM PER DAY	>100GPM AND INCR'NG > 5GPM PER DAY	UNDERMINING AND WEAKENING OF FOUNDATION; HIGHER UPLIFT	GROUTING OF FOUNDATIONS AND THEN DRILLING NEW DRAINS
		MILKY WATER	SEDIMENT IN SEEPAGE WATER	PIPING OF FINES FROM FOUNDATIONS		SEDIMENT AMOUNT INCR'NG		UNDERMINING AND WEAKENING OF FOUNDATION; HIGHER UPLIFT	GROUTING OF FOUNDATIONS AND THEN DRILLING NEW DRAINS
		PLUGGED DRAINS	FLOW AND DRAIN CHOKED BY MINERAL DEPOSITS	MINERALS LEACHED FROM GROUT OR ROCKS		DRAINS PLUGGED FOR >1 YR		EXCESSIVE UPLIFT; REDUCED FACTOR OF SAFETY AGAINST SLIDING	REAMING OF EXISTING DRAINS; DRILLING NEW DRAINS
4.5	GUTTERS	NOT FUNCTIONING	OVERFLOWING	SEDIMENTATION, POOR MAINTENANCE	*			CAN IMPEDE DRAIN FUNCTION AND MONITORING OF UPLIFT METERS	PERIODIC THOROUGH CLEAN UP AND MAINTENANCE
4.6	UTILITIES	POWER FAILURE	FLOODING OF GALLERIES; NO VENTILATION	MALFUNCTION OF EQUIPMENT; SHORT CIRCUITS; POOR MAINTENANCE	IF CTRLS & EQPT ACCESSIBLE	IF ACCESS IMPOSSIBLE			EMERGENCY PUMPING TO RE-OPEN ACCESS; REPAIR AND REPLACEMENT OF FAULTY EQUIPMENT
5.0	DRAINAGE AND GROUTING(FOUNDATION) TUNNELS								
5.1	CONCRETE LINING	CRACKING	HORIZONTAL CRACKS; OPENING >.04" AND SEEPAGE FLOW	FOUNDATION OR ABUTMENT MOVEMENT; EARTHQUAKE	CRACK ≤5' LONG; DRY; OPNG ≤.08"	CRACK ≥10' LONG; FLOW > 5GPM; OPNG> .2" AND OFFSET ≥.08"	CRACK ≥20' LONG; FLOW > 20GPM; OPNG > .5" AND OFFSET ≥.2"	IF CRACK OPENING AND OFFSET PROGRESSIVE, CAN CAUSE FOUNDATION INSTABILITY	ADDITIONAL GROUTING AND DRAINAGE OF FOUNDATIONS AND ABUTMENTS

Table 5-1 (continued)

NO.	FEATURE	DEFECT	INDICATORS	POSSIBLE CAUSES	DEGREE OF DEFICIENCY			POTENTIAL EFFECTS	POSSIBLE REMEDIAL MEASURES
					MINOR	SERIOUS	VERY SERIOUS		
5.2	DRAINS	SEE 4.3	SEE 4.3	SEE 4.3				SEE 4.3	NOTE: TUNNELS OFTEN CONNECTED TO GALLERY SYSTEM IN DAM
5.3	ROCK CONDITION IN UNLINED TUNNEL	SWELLING OR SQUEEZING ROCK: ROCK FALLS	ROCK WALL (FACE) MOVEMENT: OFF-SETS AT JOINTS	STRESS RELIEF: EARTHQUAKE: MAJOR SLIDE: FAULT MOVE-MENT		IF MOVE-MENT AT JOINTS CONT.	IF CONT. MOVE-MENT & JOINT OFFSET ≥ .2"	ABUTMENT INSTABI-LITY	RESTRICT RESERVOIR OPERATION. ADDITI-ONAL GROUTING AND DRAINAGE OF ABUT-MENT AND FOUNDATION
5.4	GUTTERS	SEE 4.5	SEE 4.5	SEE 4.5				SEE 4.5	SEE 4.5
5.5	UTILITIES	SEE 4.6	SEE 4.6	SEE 4.6				SEE 4.6	NOTE: TUNNELS OFTEN HAVE SAME UTILITIES AS GALLERIES IN DAM
6.0	ABUTMENTS AND FOUNDATION CONTACT								
6.1	DRAINAGE SYSTEM	EXCESSIVE SEE-PAGE THRU ABUTMENTS	SPRINGS AND FLOW THRU ROCK JOINTS	INADEQUATE GROUT CUT-OFF. SOLUBLE ROCK: MOVEMENT AT JOINTS. FAULTS: EARTHQUAKE		*		IF FLOW INCREASING MAY INDICATE LEACH-ING OUT AND WEAKEN-ING OF FOUNDATION - ABUTMENT	ADDITIONAL GROUTING
		PLUGGED DRILLED DRAINS	OBSTRUCTION OF FREE FLOW: MI-NERAL DEPOSITS	SEE 4.3				SEE 4.3	SEE 4.3
6.2	CONDITION OF ABUTMENTS	ROCK INSTABI-LITY.	SLIDES: JOINT MOVEMENT: LARGE LOOSE BLOCKS: SLOPE PROTECT-ION DAMAGE: LOOSE ANCHORS	DIFFERENTIAL MOVE-MENT IN ABUTMENT: EARTHQUAKE: MAJOR SLIDE		MVT. HALTED: SLOPE PROTEC-TION INTACT	PROG. MVT.: UPLIFT BUILD-UP	INSTABILITY OF DAM AND ABUTMENTS. HAZARD TO DOWN-STREAM STRUCTURES AND ACCESS TAIL-RACE BLOCKED	LOWER RESERVOIR: REINFORCE AND SUPPORT ABUTMENT: GROUTING AND DRAIN-AGE.
7.0	BUTTRESS DAMS								
7.1	FACE SLAB OR ARCH BARREL	DAMAGED CONCRETE	EXPOSED RESTEEL: LEAKAGE: CRACKED AND SWOLLEN CONCRETE	WEATHERING: SULFATE ATTACK: ALKALI-AGGRE GATE REACTION:	CRACKS TIGHT & DRY: REBARS NOT RUSTY	RUSTY REBARS: SEEPAGE > 5GPM PER BLOCK	REBAR CORROD-ED > 50% SEEPAGE >10GPM PER BLOCK	DETERIORATION AND FAILURE OF SLAB OR ARCH BARREL: LOSS OF WATER:	REM. OF DAMAGED CONC.: CLEANUP OF RUSTY STL.: ADDI-TIONAL STL.: PATCH-UP W/ EPOXY MORTAR: PART. RECONSTUCTION OF SLAB OR ARCH.
		DAMAGED CON-TRACTION JOINTS.	DAMAGED WATER-STOPS: LEAKAGE: JOINT MASTIC DAMAGED.	DIFERENTIAL MVT. OF DAM BLOCKS: DETERIO-RATION OF WATERSTOP & JOINT FILLERS W/ AGE: EARTHQUAKE.		LEAKAGE > 5GPM PER JOINT	LEAKAGE >10GPM & INCR. STEADY RSVR.	LOSS OF WATER: FREEZE-THAW DAMAGE AT JOINT	SEAL JOINT WITH FLEXIBLE GROUT: FILL JOINT DRAIN WITH ASPHALTIC MATERIAL
7.2	BUTTRESSES	VERTICAL CRACKS	CRACKS NOTICE-ABLE IN COLD SEASON: EXTEND-ING UP FROM FOUNDATION	FOUNDATION SETTLE-MENT: TEMPERATURE CHANGES IN MASS CONCRETE BUTRESSES : EARTHQUAKE	CRACKS <5' LONG TIGHT IN WINTER	CRACKS > 10' LONG DETECT-ED IN SUMMER	CRACKS > 20' LONG > 0.04" OPEN	OVERSTRESSING: INSTABILITY: PRO-GRESSIVE DETERIORA-TION OF BUTTRESS	GROUT CRACKS WITH EPOXY IN COLD WEATHER: STRENG-THEN AND BRACE BUTTRESS
		DAMAGED CONCRETE	SEE 7.1	SEE 7.1	CRACKS TIGHT. REBARS NOT RUSTY	RUSTY REBARS: LARGE SPALLS	REBARS COR-RODED > 50% LARGE SPALLS	DETERIORATION AND FAILURE OF BUTTRESS	SEE 7.1 STRENGTHEN AND BRACE BUTTRESS
7.3	FOUNDATION & UPSTREAM SLAB OR ARCH CONTACT	HIGH PIEZOME-TRIC PRESSURE AND UPLIFT	JOINTS OPEN IN FOUNDATION: EXCESSIVE SEE-PAGE AND FOUN-DATION DEFORMA-TION	HIGH UPLIFT GRAD-IENT: INADEQUATE UPLIFT RELIEF	NO FOUNDA-TION CREEP. SEEP-AGE <20 GPM PER BLOCK	SOME FOUNDA-TION MOVE-MENT. SEEP-AGE > 50 GPM PER BLOCK	CONTIN-UOUS FOUNDA-TION CREEP. SEEPAGE >100GPM PER BLK & IN-CREAS-ING	REDUCED RESISTANCE AGAINST SLIDING: INSTABILITY OF BLOCK	ADDITIONAL SHALLOW GROUTING AND THEN NEW DRILLED DRAINS TO RELIEVE UPLIFT

ARIZONA DAM OWNER AWARENESS WORKSHOP

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In order to improve future workshops, we would appreciate your taking a few minutes to complete the following questions on the Arizona Dam Owner Awareness Workshop. Your thoughts and opinions will be extremely helpful to us.

1. How would you rate the overall quality of the workshop's program? The quality was:

   (a) Excellent ______
   (b) Good ______
   (c) Fair ______
   (d) Poor ______

2. How would you rate the overall quality of speakers at the workshop? The speakers were:

   (a) Excellent ______
   (b) Good ______
   (c) Fair ______
   (d) Poor ______

3. The length of time spent on each topic was:

   (a) too short ______
   (b) too long ______
   (c) long enough to fit topics under consideration ______

   Additional comments about time spent on specific topics? ______

______

______

4. Did you particularly enjoy or find any of the topics especially interesting? ______

______

   Why was that one of particular interest? ______

______

______

RISK ASSESSMENT FOR DAMS

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RISK ASSESSMENT FOR DAMS

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ABSTRACT

Risk-based procedures for assessing appropriate safety levels for new and existing dams have been proposed for use in planning and design of dams and screening of unsafe dams. The risk assessment framework and its application to dams is presented. Approaches for estimating the various types of probabilities and consequences needed to perform a comprehensive dam risk assessment are described. Several methods for planning, screening, and design level risk assessment are summarized. The paper closes with a discussion of the advantages of, commonly stated objections to, and some conclusions related to, risk assessment for dams.

1. INTRODUCTION

1.1 Background

Interest in dam safety has grown in the past decade. According to the National Research Council (1985), reasons for this growth include: several disastrous dam failures or near failures in the United States and other countries; the classification of approximately 3,000 high-hazard dams as "unsafe" by the National Dam Inspection Program conducted by the U.S. Army Corps of Engineers; the high cost of improving these "unsafe" dams to meet current design standards; and the consideration, by many states, of more stringent regulation of privately-owned dams.

Simultaneously there has been a growing interest in the potential for using risk assessment procedures to provide a framework for addressing dam safety problems. Such procedures can provide a basis for evaluation of proposed safety improvements for new or existing structures. The purposes of this paper are to present the risk assessment approach as it is applied to dams, to compare several alternative procedures for dam risk assessment, and to discuss some of the advantages, limitations, and other issues relating to this application of risk assessment.

In this paper the term "risk" will be used to mean "the potential (probability) for the realization of unwanted consequences from impending events" (Rowe, 1977). According to this definition, the term risk has two dimensions associated with an undesirable event: the probability of its occurrence, and the magnitude of its consequences. By this definition economic damages resulting from a dam failure are considered to be a component of the consequences which may include potential life loss and environmental damage. The term "risk cost" is used to describe the expected value of economic damages on an annual basis.

1.2 Levels of Risk Assessment

Different levels of detail in the risk assessment procedures used for dam safety evaluations are appropriate at different stages in the life of a dam. As the data base for a dam grows and as the issues to be addressed change from general questions of site selection, to specific issues of the selection of design parameters, the degree of detail which can be justified in the risk assessment grows correspondingly (Bowles et al., 1978, Howell et al., 1980). Three levels of risk assessment applications to dams can be distinguished. In order of increasing detail they are the planning level, the screening level and the design level.

At the planning level it is desirable to introduce an estimate of risk cost associated with dam failure into the benefit-cost analysis as a means of including societal risk into the process of deciding to build a dam (Pate-Cornell and Togaras, 1986). At this level the estimated probabilities and consequences of dam failure are only approximate and usually will rely heavily on historical information (U.S. Water Resources Council, 1979).

The screening problem is the identification and ranking of "unsafe" dams in order of priority for expenditure of limited funds to pay for remedial action. In this context the absolute values of probability and consequence estimates are less important than a consistent procedure for estimating them so that an accurate ranking will be achieved. At the screening level site-specific conditions would typically be evaluated using reconnaissance level investigations and only approximate engineering and economic analyses. An example of the screening level is the method developed for the Federal Emergency Management Agency (FEMA) by Stanford University (McCann et al., 1985).

A design level risk assessment involves detailed questions such as the selection of design standards and choices between design alternatives for the dam and its appurtenance structures. Carefully estimated probabilities and consequences must take into account site-specific conditions based on detailed site investigations and engineering and economic analyses. In addition the sensitivity of conclusions must be investigated with respect to uncertainties in the estimates of both probabilities and consequences. The only documented procedures at the design level are by the U.S. Bureau of Reclamation (USBR, 1986). Work which contributed to these procedures includes work at Utah State University (USU) which is described by Howell et al. (1980).

1.3 Outline of Paper

This paper is divided into five sections. After this introductory section the overall framework for risk assessment and its application to dams is presented in Section 2. A description of approaches for probability and consequence estimation is provided in Section 3. The approaches described in Sections 2 and 3 are those utilized by the USU procedure (Howell et al., 1980). Several methods of risk assessment which are currently in use or have been proposed are classified by risk assessment level and are summarized in Section 4. In Section 5 the advantages of, and commonly stated objections to, risk assessment of dams are discussed. The paper is closed in Section 6 with a presentation of conclusions and other issues.

2. RISK ASSESSMENT FRAMEWORK AND ITS APPLICATION TO DAMS

2.1 General Framework

Risk assessment involves the identification, estimation and evaluation of risks associated with a natural or man-made system. The purpose of risk assessment is to evaluate whether the present margin of safety or reliability of the system is acceptable, or to select an alternative for controlling risk in terms of either the probability or consequences of system failure (see Fig. 1). Risk management comprises both the identification, estimation, and evaluation aspects of risk assessment and the implementation aspects of risk control (see Fig. 2). This paper emphasizes risk assessment although some references are made to the larger problem of risk management.

Figure 3 illustrates the relationship between the steps of risk identification, estimation, evaluation and control for environmental risk management. The implementation of risk control measures, which could be structural or nonstructural in nature, would typically introduce new risk factors which could lead to the need for a second-order risk analysis. The first two steps of risk identification and estimation are usually performed by an analyst, such as an engineer or an economist. Risk evaluation and control are usually determined by a decision maker and typically involve political judgments as to risk acceptability (e.g., how safe is safe enough?). Examples of approaches used in each of these four steps in an environmental risk management problem are given in Figure 3.

2.2 Application to Dams

Dam engineering is not an exact science. The successful design and construction of dams requires the application of judgment by highly experienced engineers, geologists, hydrologists, and others. Traditionally the approach to dam design focuses deterministic analyses on extreme events, such as the probable maximum flood (PMF) or the maximum credible earthquake (MCE), and uses conservative estimates of such properties as concrete or soil strength. Safety factors are used to evaluate the ratio of resisting to overturning moments for such failure modes as slope instability. As a result, through the practice of the traditional approach, which is based on the accumulation of many decades of dam engineering experience, an impressive safety record has been achieved.

However, the traditional approach does not attempt to explicitly quantify all significant risk factors for a dam. Nor does it explicitly determine the degree of safety which can be justified for a particular structure considering the potential consequences of a sudden release of the contents of a reservoir following dam failure. The risk assessment approach provides the framework to make such a quantitative determination.

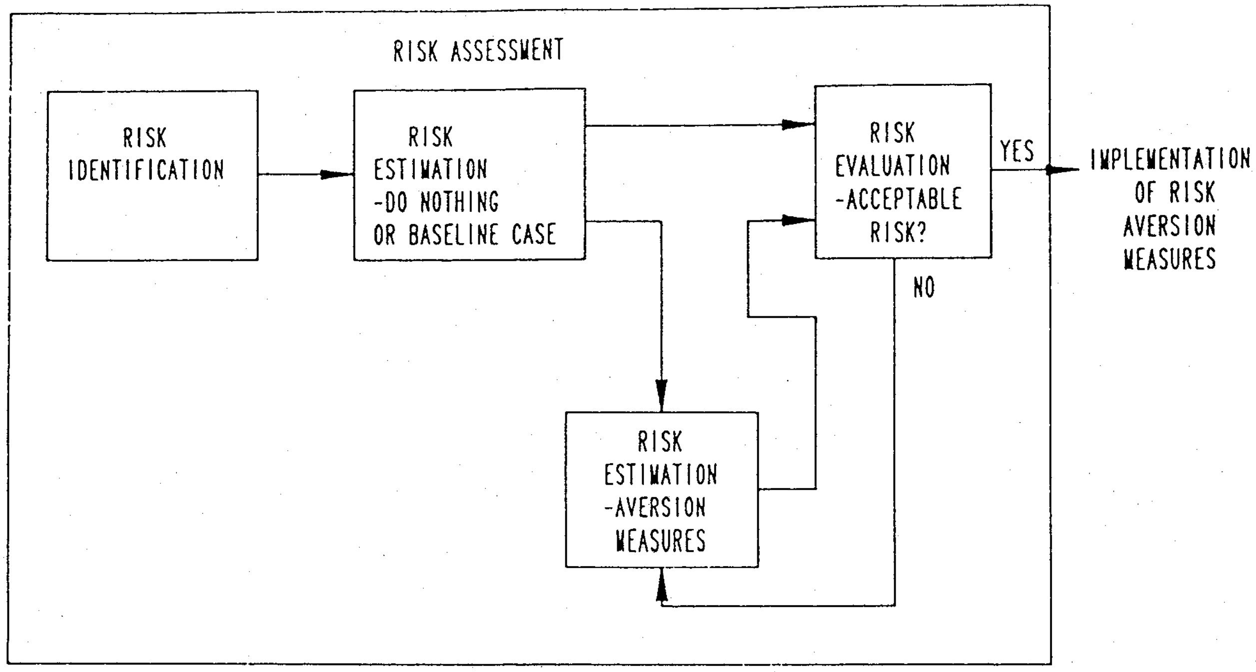

Figure 1. Risk assessment framework (after Bowles and James, 1986).

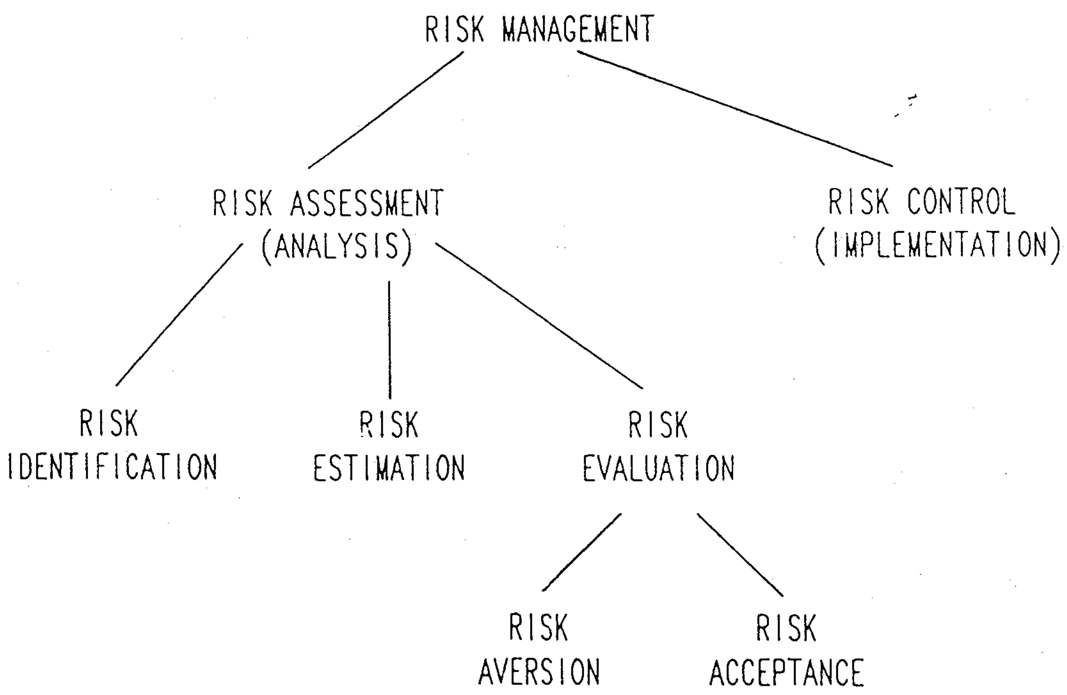

Figure 2. Components of Risk Management.

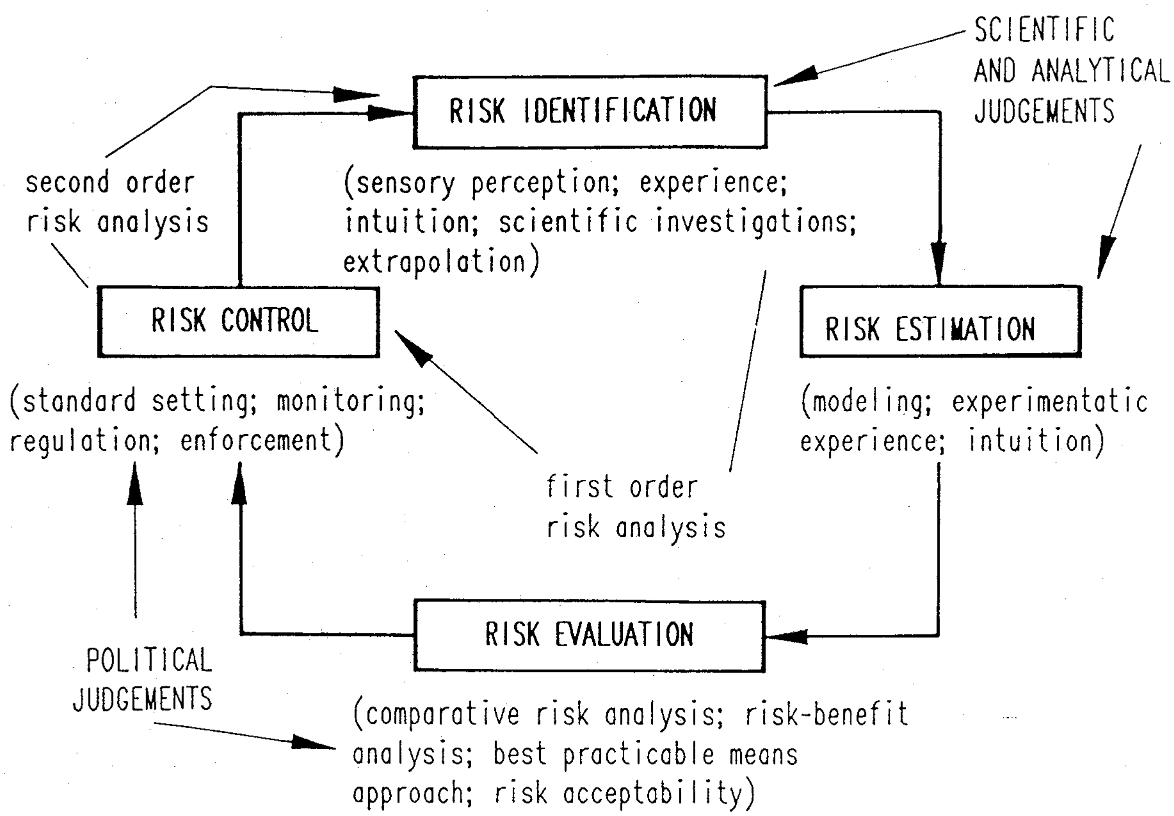

Figure 3. Environmental risk management (after O'Riordan, 1979).

The approaches to applying risk assessment to dams differ at the planning, screening and design levels. The approach appropriate at the most detailed, or design level, is presented below. This presentation follows the identification, estimation, aversion, and acceptance steps of the approach as shown in Figures 1 and 2.

2.2.1 Risk identification. Firstly a sequence of events is identified beginning with events that can initiate dam failure and ending with the consequences of the failure (see Fig. 4 and 5). Initiating events can be classified as external or internal. External events include earthquakes, floods, and upstream dam failure. Internal events include chemical changes in soil or concrete properties or latent construction defects. At low levels these events would not lead to dam failure. However, at high inflow rates a rapid rise in pool level could lead to overtopping, or a severe earthquake could result in structural deformation or liquefaction. These and other dam-foundation-spillway-reservoir system responses are failure modes which can lead to the outcome of the sudden release of the reservoir contents. The magnitude of the resulting property damage and life loss will depend on various exposure factors. These include flood routing to determine the path of the flood wave, the area of inundation, and the travel time; the time of day and season of the year; and the effectiveness of any warning systems and evacuation plans.

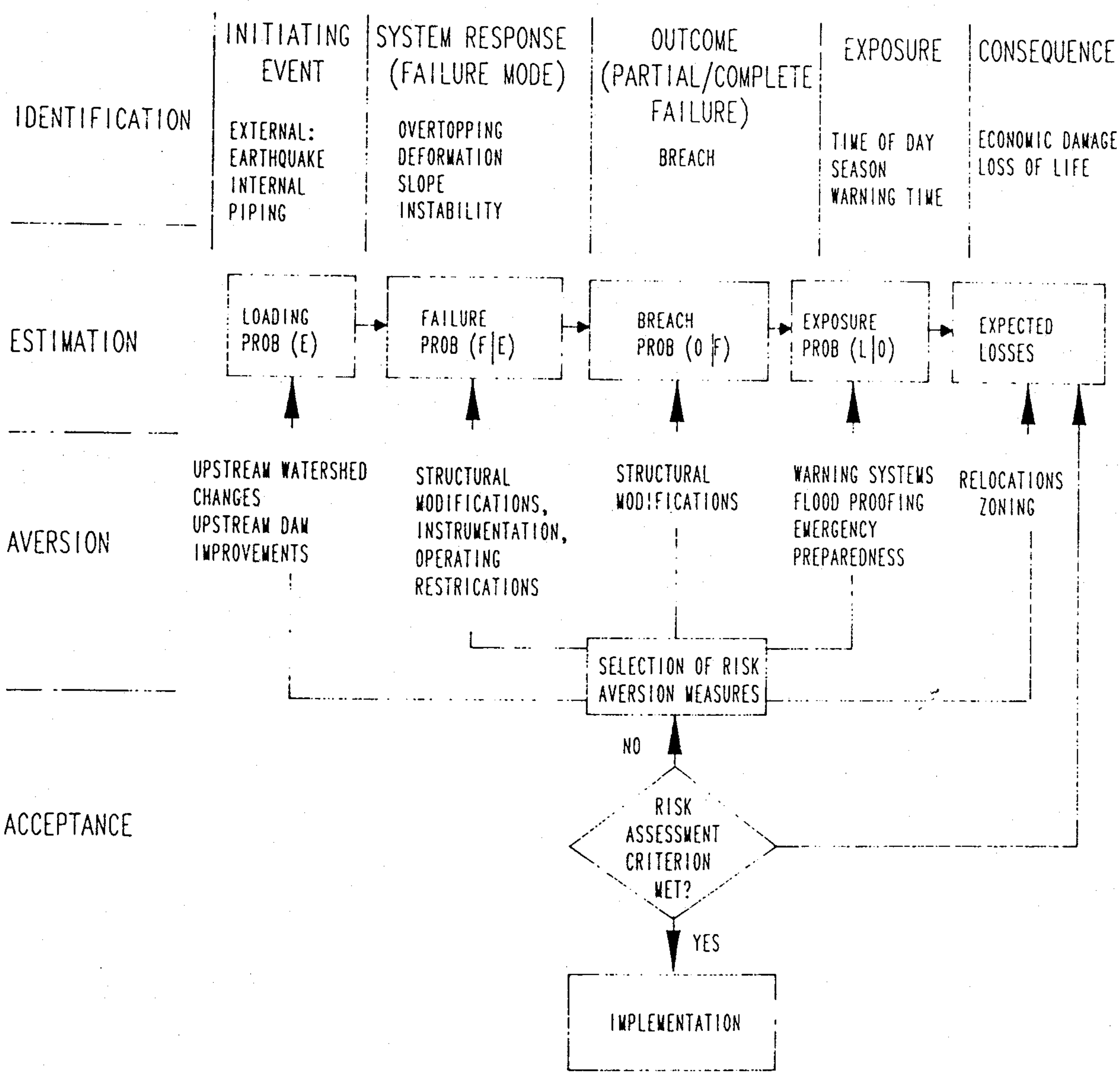

Figure 4. Risk-based method for assessing dam safety improvements (adapted from Bowles et al., 1984).

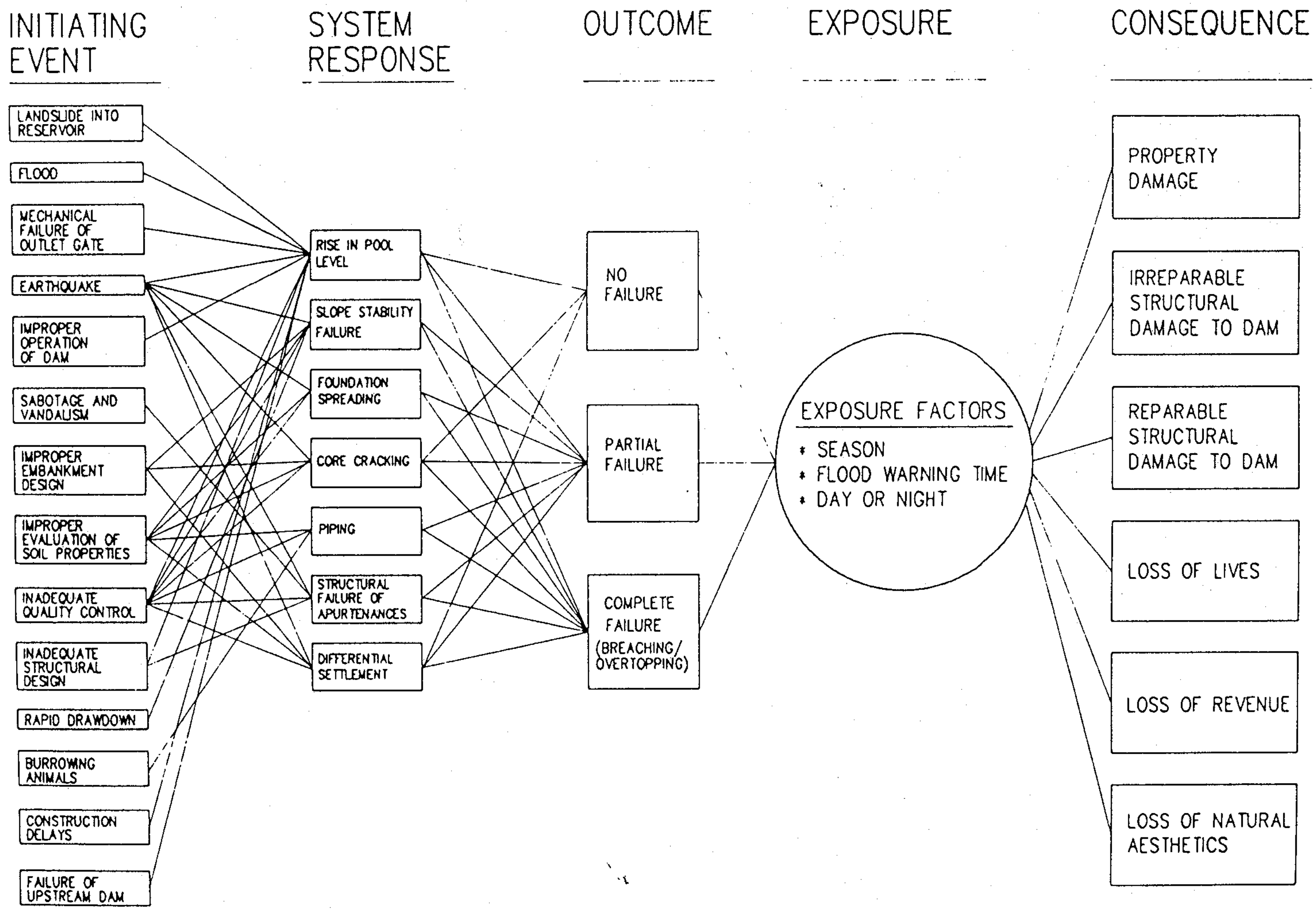

Figure 5. Event-system response-outcome-exposure-consequence diagram for an earth dam (after Howell et al., 1980).

Consequences are classified as life loss and economic loss which includes property damage, cost of dislocations, and loss of project benefits.

During the identification step, professional judgment and experience, review of available information, and site visits are used to develop a list of the types of initiating events, system responses, outcomes, exposure factors, and consequences which apply to a particular dam-foundation-spillway-reservoir system. A diagram such as that shown in Figure 5 is then constructed to describe the event-consequence sequences or initiating event-system response-outcome-exposure-consequence pathways. Using the information assembled in Figure 5 an event tree (see Fig. 6) is developed to describe each pathway associated with a particular range of magnitudes of an initiating event (e.g., a range of reservoir inflow magnitudes or a range of ground accelerations at the dam site associated with seismic activity). The event tree is the risk model for the design level risk assessment.

2.2.2 Risk estimation. The second step in the (design level) risk assessment procedure is the estimation of the probability and consequence components of risk. The types of probabilities to be estimated are shown on Figure 4 and are as follows:

. Annual probability of occurrence of loading (e.g. flood) in a range of magnitudes, E - Prob(E)

. Conditional (response) probability of dam failure by a specified failure mode (system response), F, given that loading occurs in the range, E - Prob (F/E)

. Conditional probability of the outcome, O, release of reservoir contents, given that failure mode, F occurs - Prob (O/F)

. Conditional probability of life loss (and in some cases property damage), L, for a population at risk given that the outcome O occurs - Prob (L/O)

Alternative methods for estimation of these probabilities are discussed in Section 3.1.1. All event sequences defined by the event tree risk model are considered. Estimation of economic (L_E) and life (L_L) loss and consideration of exposure factors are discussed in Section 3.2.

The partial risk cost for the ith pathway is obtained by taking the product of the four probabilities and the economic consequences as follows:

$$c_i = \text{Prob }(E)\ \text{Prob }(F/E)\ \text{Prob }(O/F)\ \text{Prob }(L/O)\ L_E \qquad (1)$$

The total risk cost is obtained by summing the partial risk costs over all N mutually exclusive pathways as follows:

$$C = \sum_{i=1}^{N} c_i \qquad (2)$$

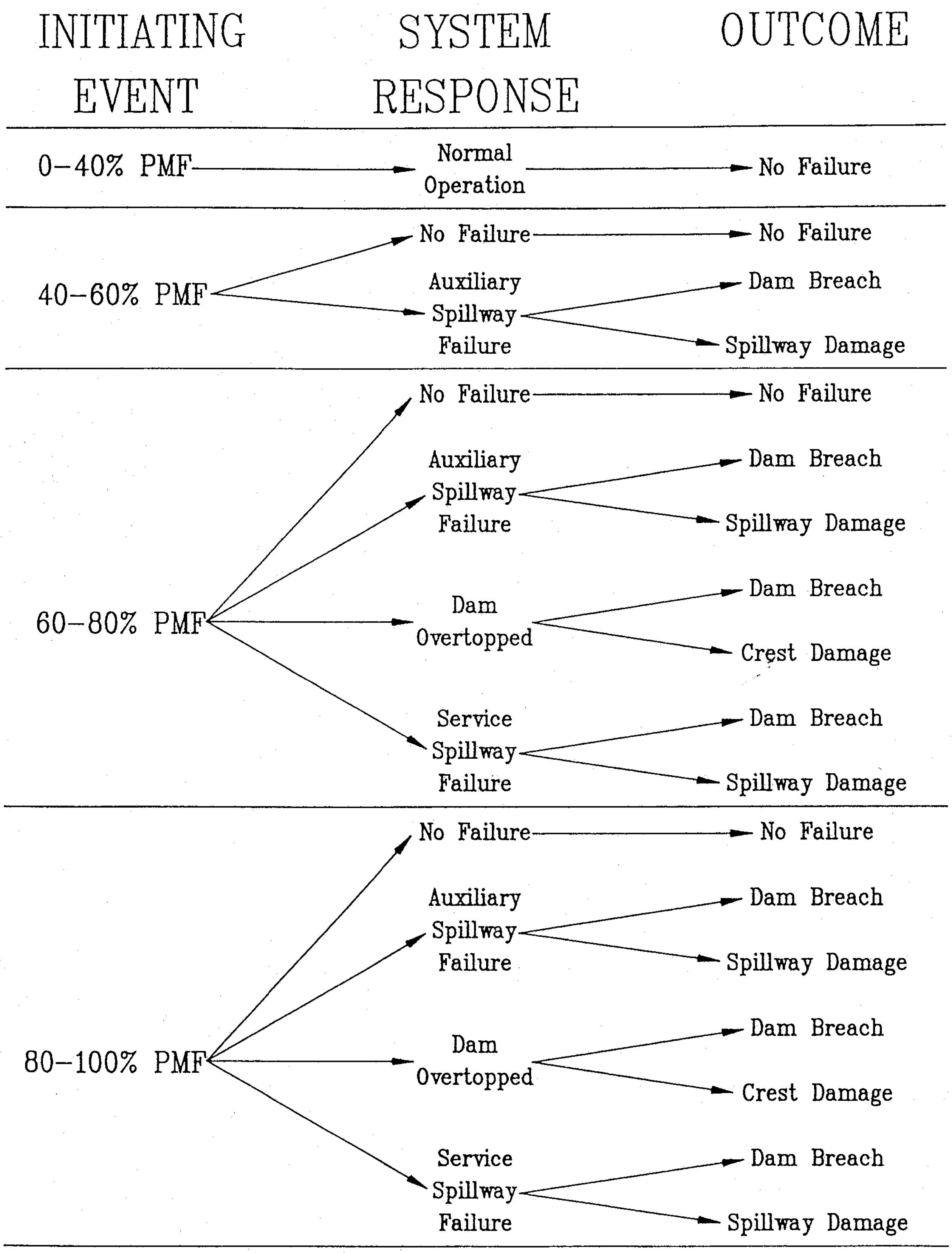

Figure 6. Event tree for hypothetical dam considering hydrologic loading only.

The probability of life loss for a population at risk due to a failure described by the i^{th} pathway is given by:

$$p_i = \text{Prob (E) Prob (F/E) Prob (O/F) Prob (L/O)} \qquad (3)$$

A histogram, $P(L_{Lj})$, of life loss can be constructed by summing the pathway probabilities associated with pathways with potential magnitudes of life loss consequences falling in several magnitude intervals:

$$P(L_{Lj}) = \quad p_i$$

in which

s = set of all pathways for which $L_{L_j} < L_L < L_{L_{j+1}}$

2.2.3 Risk aversion. The product of the second step is an estimate of the probability of failure, the risk cost, and life loss that would be associated with each failure mode, or combination of failure modes, for the baseline or do nothing alternative (see Fig. 1). If these risks are unacceptable, the analyst moves to the third step. This involves formulation and evaluation of alternatives for risk aversion, which are commonly referred to as remedial action or rehabilitation alternatives for dams. Risk aversion can be achieved by reducing the probabilities associated with a pathway or by reducing the consequences. In both cases structural or nonstructural measures may be considered. Figure 4 lists some examples of aversion measures and shows the probability or consequence that would be expected to be reduced by their implementation. An important and sometimes difficult part of the evaluation of risk aversion measures is the estimation of the reduction in the probabilities or consequences that would be expected as a result of implementing a measure. For example, what would be the reduction in Prob (F/E) for foundation failure during an earthquake if rock anchors were placed in the foundation? Or, what would be the reduction in Prob (L/O) if a flood warning system and evacuation plan were implemented?

The product of the aversion step is an estimate of the reduction in probability of failure, the risk cost, and life loss that can be attributed to the implementation of a risk aversion measure. Such reductions are used as an estimate of the benefits of the measure and hence a benefit-cost analysis can be performed.

2.2.4 Risk acceptance. The final step in the risk assessment process is the decision as to what degree of safety should be achieved, or equivalently what residual risk will be accepted. Although the analyst can supply information and recommendations for this decision, it is usually made by a decision maker such as the dam owner, operator, or regulator. The decision is especially sensitive and difficult where lives are at risk and where large investments will be required to improve safety with little or no effect on the project benefits, except of course to their expected longevity considering the reduced likelihood of dam failure.

3. ESTIMATION PROCEDURES

3.1 Probability Estimation

3.1.1 General approaches. There are three general approaches to probability estimation: historical/empirical, judgmental, and analytical. The historical/empirical approach utilizes historical frequencies of events as probability estimates. The larger the available sample size used the better the estimates are expected to be. Thus a long record of flows or seismicity can be expected to yield better (i.e., less uncertain) estimates of extreme floods or earthquakes than a short record. Similarly a large data base of dam failures, categorized according to such factors as the type, size, age, and location of the dam, can be expected to provide better estimates of the probability of failure due to a specified failure mode than would a small data base. However, the available data base for dam failures is usually small for a particular category of dam, especially large dams, and in any case will provide only an estimate of probability of failure for "average" conditions rather than for the specific conditions of the dam which is under evaluation.

In order to incorporate additional information about the study dam obtained from field work, laboratory testing, engineering analyses and expert judgment, the historical/empirical probability estimates are treated as initial estimates which can be updated judgmentally using the additional information. The Bayesian approach to estimation provides a formal method for updating historical probability estimates using "judgmental" information, as follows:

$$\text{Prob}''(F/X) = \frac{\text{Prob}'(F)L(X/F)}{\text{Prob}'(F)L(X/F) + \text{Prob}'(F)L(X/F)} \tag{4}$$

in which

Prob"(F/X) = updated estimate of probability of failure given additional information X about the dam

Prob'(F) = prior estimate of probability of failure

Prob'(F) = prior estimate of probability of no failure

L(X/F) = likelihood of observing information X given that the dam were to fail

L(X/F) = likelihood of observing information X given that the dam were not to fail

McCann et al. (1985) provide values for the likelihood functions, L, for several dam failure modes although it is not clear what is the basis for the suggested values.

The third method of probability estimation is the analytical method in which models are used to transform probability distributions of loads and strength parameters into an estimate of the probability of failure. For example, the probabilistic slope stability method of Sharp et al.

(1981) can be used to estimate the probability of slope failure due to the spatial variability in shear strength in a zoned embankment. Unfortunately the analytical approach usually requires additional expensive field work in order to estimate distributions for strength parameters. Also, site specific information is often lacking and realistic models necessary for estimating failure probabilities for such phenomena as piping are not available. At this time the analytical approach is generally considered to be an approach which needs additional research and development before it is ready for use in practical dam risk assessments. Therefore, the historical/empirical approach with judgmental updating where appropriate is most commonly used.

The four types of probabilities to be estimated in a dam risk assessment were listed in Section 2.2.2 as Prob (E), Prob (F/E), Prob (O/F) and Prob (L/O). Estimation of the first three probability types is briefly discussed below and estimation of the fourth type is discussed in Section 3.2.

3.1.2 Probability of initiating events, Prob (E). Most commonly, three types of loading conditions are considered for a dam risk assessment: static reservoir load, hydrologic (flood) load and seismic load. Other loading conditions may be identified for a particular structure but only these three will be considered here.

Static loading can lead to various failure modes such as piping or embankment instability. The probability of reservoir loading (i.e., reservoir stage) being in a range that could lead to such failure modes can be evaluated from information on the operating policies for the dam.

Hydrologic loading is evaluated in terms of peak inflow design rates. Where measured flow data are available, a flood frequency analysis can be performed to estimate the probability of occurrence of more frequent events. Depending on the length of record it might be reasonable to extend such a frequency curve to events with return periods of 100 or 200 years (see for example Bulletin No. 17B, U.S. Department of Interior, 1982). However, the probable maximum flood (PMF) is associated with much rarer events and its return period cannot be determined with confidence. Since a probability must be estimated for flow ranges up through the PMF in order to perform a quantitative risk assessment it is customary to make an arbitrary probability assignment for the estimated PMF. Typically this would be in the range of 10^{-4} to 10^{-6} per year. USBR (1986) suggests an approach whereby the 5% and 95% confidence limits on the PMF are assigned probabilities of 10^{-6} and 10^{-4}, respectively. The frequency curve is extended to the plotted point corresponding to the PMF, and thus a probability of peak reservoir inflow being in any flow range up to the PMF can be obtained. Typically in a risk assessment the dividing points between flow ranges are on critical flows associated with thresholds for failure modes of the existing dam or its rehabilitation alternatives, or with changes in the downstream stage-damage relationship.

The estimation of probabilities of seismic load ranges has certain similarities to the case of estimating the probabilities of hydrologic load ranges. Frequency analysis is performed on available historical seismic data and then extensions are made to the maximum credible earthquake (MCE). All known faults (and random locations if appropriate) which could affect the dam site after attenuation is taken into account must be considered and the probability of accelerations at the dam being in various ranges are estimated. As with the case of hydrologic loading, acceleration ranges are based on critical values of acceleration associated with thresholds for failure modes of the existing dam or its rehabilitation alternatives.

3.1.3 Conditional system response probability, Prob (F/E). For each loading type the event tree constructed during the risk identification step (see Fig. 6) specifies failure modes or system responses which could occur during initiating events with magnitudes in each loading range. In some cases such as overtopping of an embankment dam this relationship between Prob (F/E) and E may approximate a step function (see Fig. 7a). In other cases, such as a seismically induced failure mode in a region of low seismicity, the relationship may be a smooth curve which does not reach a probability of 1.0 at the maximum acceleration at the site.

Response probability relationships are based on engineering analysis, experience, and judgment. Research in the form of scale model studies, numerical modeling, and evaluation of historical dam failures is needed in order to improve our current estimates of these relationships. Uncertainty in estimates of the threshold at which Prob (F/E) significantly departs from zero, or the steepness of the response

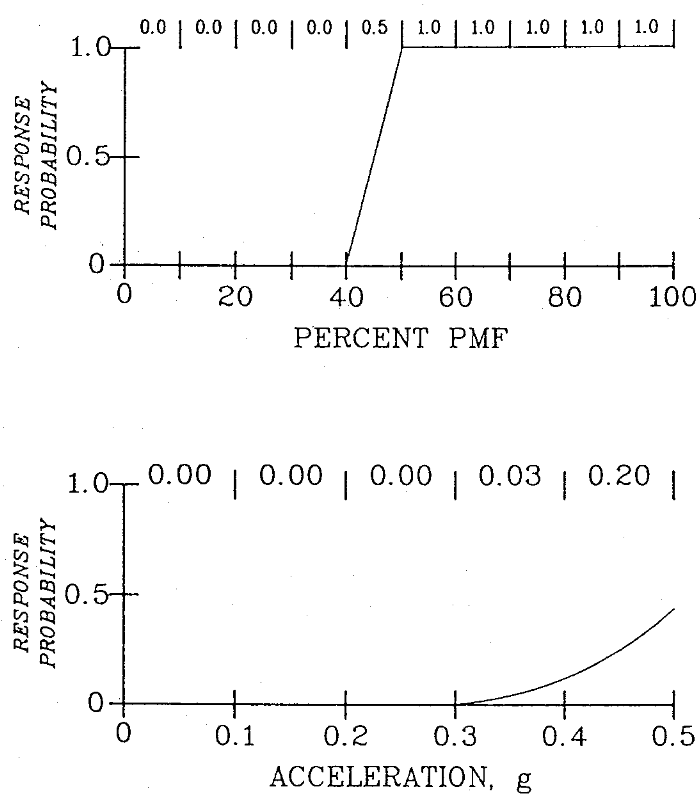

Figure 7. Examples of system response probability relationships.

probability curve, leads to wider confidence limits on estimated probabilities of dam failure and risk costs.

3.1.4 Outcome probabilities, Prob (O/F). The outcome of a failure mode may be a partial or complete failure of the structure. A complete failure implies a breach with release of the reservoir contents and a partial failure implies damage to the structure but no catastrophic release of the reservoir contents.

For a seismically induced failure mode the type of outcome would typically be dependent on the level of the reservoir at the time of the failure. Thus an evaluation of the reservoir operating policy can lead to an estimate of Prob (O/F). For a hydrologically or statically induced failure mode the outcome probability may be set equal to 1.0 since the probability of reservoir level is usually accounted for in Prob(E) or it can be used to account for other factors such as the duration of an overtopping event which could determine the difference between a partial and complete failure.

3.2 Consequence Estimation

3.2.1 Types of consequences. The consequences of a catastrophic dam failure can be grouped into various categories such as economic, life loss, environmental, social, etc. In this paper consequence estimation for only the first two categories will be discussed. This discussion includes reference to estimation of exposure probabilities, Prob (L/O).

3.2.2 Economic damages. The quantification of economic damages requires identification of the inundated areas for each dam failure mode. The inundated areas are obtained from dam break modeling and flood routing. Land use characteristics and business activities which would be affected by flooding and public facilities and service operations which may be interrupted by flooding must be identified. Potential losses which could result from dam failure, include property damages, incomes foregone, and project benefits which are foregone because of the failure.

Damages that will occur without dam failure are defined as baseline damages (see Fig. 8). They include operational damages resulting from normal spillway operations or overtopping prior to or without structural failure. Baseline damages precede dam failure and are not properly attributable to the uncontrolled release of reservoir storage. As such, they must be deducted from the total estimate of flood damages in order to determine those that are directly related to structural failure. These damages are not preventable by dam safety modification, except by providing additional storage. They would occur regardless of the changes made to spillways or outlet works.

Damage estimates for a given failure mode must be made for alternative loading conditions (e.g., flow range as a percentage of the PMF). These damages are estimated for agricultural, instream, residential, public and industrial losses incident to dam failure. Stage-damage ratio curves have been developed by the Federal Insurance Administration (FIA) and used for residential structures. The U.S. Army Corps of Engineers has developed stage-damage ratios which can be used for

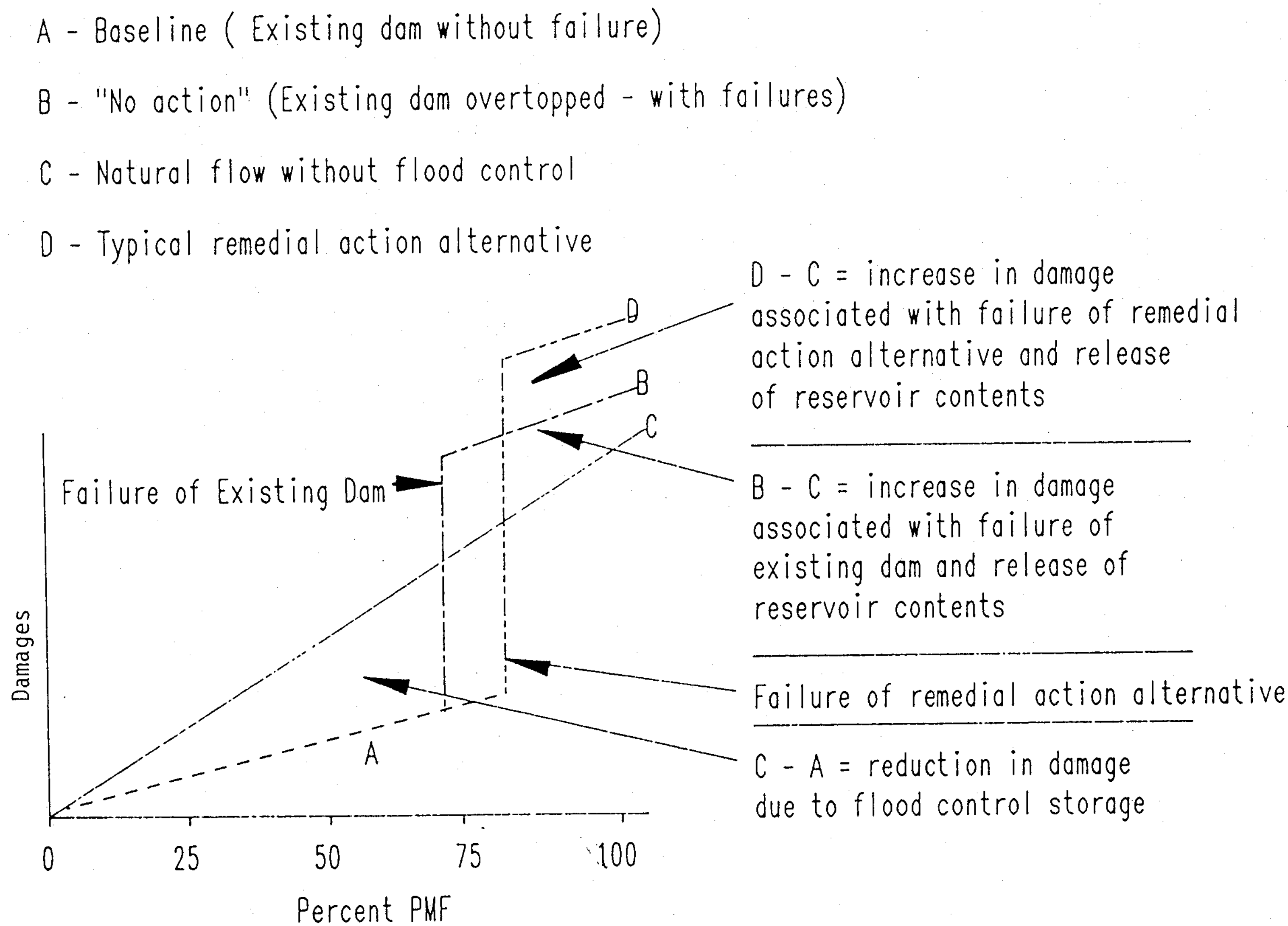

Figure 8. Example of determination of differential damages (adapted from USBR, 1986).

commercial and industrial structures and for public facilities. The losses to residential areas from flood inundation include damages to dwellings, costs of displacement during the time dwellings are repaired or rebuilt, and damages to personal property which can be estimated from local insurance records. Dwelling values can be obtained for broad residential districts from local records.

Business losses can be estimated for two categories of loss: capital losses and employment and income losses. The former involves the losses of equipment, structures and contents. Values are taken from industrial indices for various standard industrial code (SIC) industry classifications. Employment losses involve the jobs and associated incomes that are lost due to flooding. Current population reports and census reports are also of value in estimating commercial as well as residential conditions.

The cost of repairing or replacing public facilities and services are assumed as a proxy for damage estimates. For example, bridges in the flood path may be destroyed with the result that traffic will be forced to take longer routes until they are replaced.

Agricultural losses are divided into capital losses, crop losses and losses of the productive capacity of land. Capital losses are damages to equipment and structures. Crop losses include foregone income from crop operations which are partially or totally interrupted by flooding. The losses are estimated by crop type, average productive capacity of land bases, and normalized prices for the crops involved. Input costs, such as fertilizer and seed costs, should be included in the estimated damages since their services in production are partially or totally lost due to destruction of the crop. Most of the analyses related to these loss estimates are similar to the economic benefit evaluations of standard water development projects and loss estimation procedures and farm enterprise budgets for use in the U.S. Such procedures are available from the Division of Planning Technical Services of the USBR. If any of the inundated areas are expected to experience severe erosion or flood debris damage, the productivity of the land is diminished. Lost value of the land should then be estimated and included in the damages.

The amount of warning time and emergency evacuation procedures affect the amount of property damage. Given sufficient time, people can generally evacuate with some amount of property, including equipment, personal cars and other high value but mobile items. Livestock may also be moved to higher ground or away from areas where they may be isolated by flood. These actions can reduce damages considerably. Therefore, an assessment of any currently installed or proposed emergency warning system and subsequent warning times and the emergency evacuation procedures needs to be completed in order to apply reduction factors (exposure probabilities) to the damage estimates, particularly capital and personal property estimates. Figure 9 shows graphically a probabilistic analysis of a monitoring system and warning system.

In addition to property damages, loss of reservoir benefits must be estimated and included in the damage calculations. These losses are included for the period of time that reservoir storage is lost or reduced.

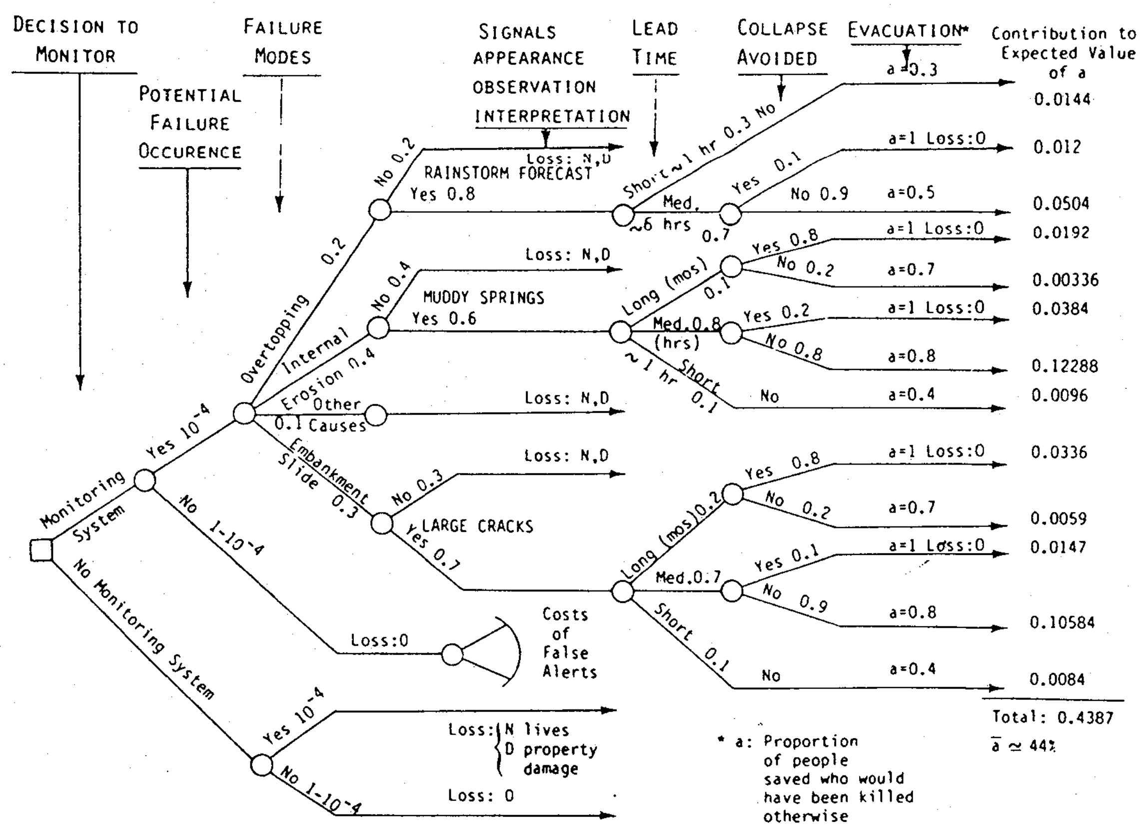

Figure 9. Probabilistic analysis of a monitoring and warning system (after Paté-Cornell and Tagaras, 1986).

If the structure is to be repaired, then these losses are included for a partial rather than project life basis, but the repair costs are included in the estimates of losses.

3.2.3 Potential life loss. The importance of considering the potential impact of a dam failure on downstream populations cannot be over emphasized. This issue is discussed in Section 5.2. In order to estimate potential life loss the population at risk must be identified from inundation maps. Often there will be more than one population at risk (e.g., recreational use below dam, ranchers below dam, downstream town) where populations are distinguished by location (and warning time) and exposure characteristics (e.g., sparse such as farmers, dense as in a town, seasonal as for campers, residential or business with different day and night time characteristics).

Each population at risk must be evaluated separately with respect to the effectiveness of a warning system, the length of warning time considering the travel time for the flood wave, and the effectiveness of evacuation plans. An evaluation of historical dam failures by USBR (1986) indicates a very significant reduction in exposure probability for smaller communities with adequate warning time over similar communities without adequate warning time. USBR (1986) found an exposure probability of 0.0002 for situations in which the warning time exceeded 1.5 hours. Estimates of exposure probabilities for large metropolitan areas, or for other special situations, can be improved by using evacuation modeling (Jaske, 1985). Differences in exposure probabilities associated with different failure modes must be considered. For example, if failure occurred as the result of an earthquake, warning time would likely be less than a failure resulting from hydrologic causes for which inflow forecasts might provide forewarning of exceptionally large inflows.

3.3 Sensitivity Analyses

Traditionally, estimates of loads on material strengths in engineering design are made on a conservative basis. In a risk based approach additional estimates are made to describe a range of estimates of each load or probability or consequence. With this "internal" estimate approach the effects of uncertainty in the estimation of the inputs to a dam safety decision can be explored and documented through sensitivity analysis. Where the recommended decision is sensitive to uncertain factors additional efforts to reduce the degree of uncertainty can be considered. Alternatively, if the level of uncertainty cannot be economically reduced the conservative design approaches can be used to achieve acceptable safety standards. Since uncertainty exists in all the inputs (e.g., probability and consequences) to a risk assessment the use of sensitivity analyses is an important part of a comprehensive dam risk assessment.

4. BRIEF REVIEW OF METHODS

4.1 Scope of Review

Methods compared are grouped into three categories based level of risk assessment. The traditional or subjective engineering design approach which does not explicitly quantify risk or assess its acceptability is excluded from this comparison. Neither does the comparison include generic evaluation approaches such as surrogate worth trade-off (Haimes and Hall, 1984), ELECTRE (Gershon and Duckstein, 1983), or multi-attribute utility analysis (Keeney and Raiffa, 1976). A recent evaluation for the U.S. Army Corps of Engineers (University of Southern California, 1985) includes the traditional engineering design and generic evaluation approaches with an apparent emphasis on considering hydrologic loading.

4.2 Planning Level Methods

At the planning level an estimate of risk cost associated with dam failure can be introduced into the benefit-cost analysis which is used to provide the economic justification for a decision to build a dam. Baecher et al. (1979, 1980) describe a method for estimating the risk cost at the planning stage based on an average (or default) historical/empirical failure rate of 10^{-4} per year for large dams. In a recent paper Pate-Cornell and Togaras (1986) presented three case studies of planning level risk assessments and extended the procedure to the case of sequential dam failure. The level of detail used in these planning level methods is consistent with the level of effort which is expended on other planning studies and the availability of site specific information. Therefore, this level would not be suitable for comparing the risks associated with specific design alternatives for a proposed structure or for remedial action alternatives at an existing structure.

4.3 Screening Level Methods

The screening problem is the identification and ranking of "unsafe" dams in order of priority for expenditure of limited funds to pay for remedial action. By its very nature the screening problem implies that several, and perhaps many, dams must be assessed. Therefore, the level of detail required for analysis must be limited, and consistent procedures will be important to achieve an accurate ranking. Screening level methods are divided into index or qualitative methods and quantitative methods.

4.3.1 Index (qualitative) methods. Both the U.S. Army Corps of Engineers and the U.S. Bureau of Reclamation have developed index methods for ranking existing dams in terms of a subjective measure of "risk." Hagen's (1982) method, developed for the Corps of Engineers, defines a "relative risk index," R as the sum of an "overtopping failure score," O and a "structural failure score," S.

These scores are determined subjectively based on site inspection, review of design and construction records, and other available information. The maximum or worst case index value would be 250 and a high score would lead to assignment of a high priority for remedial action to reduce the risk of dam failure.

The Bureau of Reclamation index method was developed as part of their Safety Evaluation of Existing Dams (SEED) (USBR, 1980) program. The evaluation process includes a site inspection and a review of available information on dam design, construction, and operation. A detailed evaluation report is prepared and a numerical measure of the dam's condition and damage potential, referred to as a site rating (SR) is assigned. The SR is obtained by summing the scores assigned to four elements relating to the conditions of the dam (age, general condition, seepage problems, and structural behavior measurements) and four elements describing the damage potential (capacity, hydraulic height, hazard potential/hydrologic adequacy, and seismic zone). Each element is scored on a scale of 0 to 9 with 0 being the most favorable value.

During the evaluation a list of recommendations for upgrading the dam is made and their significance is represented by a weighting system (USBR, 1980). The sum of the weights for all recommendations is added to the SR to give a "SEED" value which is used to rank the dam with respect to other dams.

4.3.2 Quantitative methods. Quantitative procedures for screening level risk assessment of dams have been developed by Stanford University for the Federal Emergency Management Agency (McCann et al., 1985) and by MIT (Vanmarcke and Bohnenblust, 1982). The framework for both procedures is similar to that described in Section 2.2. Each is summarized below.

The Stanford procedure is divided into three parts. The first part involves identification of the expected losses from dam failure and utilizes the information which is usually required for a Phase I inspection of the National Dam Safety Inspection Program. It involves collection of data, estimation of the probability of failure of the dam due to various initiating events, dam breach modeling and flood routing, and estimation of direct economic losses.

In the second part of the procedure the expected loss due to dam failure is estimated from the failure probability and economic loss estimates in the first part. Under the third part the cost and degree of safety improvement for each rehabilitation alternative is considered. This information is presented on a cost-effectiveness basis in order to rank a portfolio of unsafe dams for the expenditure of limited remedial action funds. Since ranking is the objective of the Stanford procedure it emphasizes standardized methods of analysis which can be expected to lead to reasonably consistent and reproducible results. Typically these methods are simplifications of what would normally be done at the design stage.

The MIT Screening Procedure is similar to the Stanford approach. It is illustrated by a case study in which 16 Vermont dams are ranked for remedial action using information available in inspection reports from the National Dam Inspection Program. The updating of historical frequency/empirical probability estimates of dam failure using information contained in the inspection reports using a Bayesian procedure is illustrated.

4.4 Design Level Methods

A framework for design level methods of risk assessment for dams was proposed by Bowles et al. (1978) at Utah State University (USU) and was detailed by Howell et al. (1980). The USU method is presented in Sections 2 and 3 of this paper. A related approach was developed by the U.S. Bureau of Reclamation (USBR) and has most recently been described by the USBR (1986). The design level approach has been in use by the Engineering and Research Center of the USBR for several years and by the writer in conjunction with risk-based assessments of remedial action alternatives at several dams.

The USBR approach is divided into two major phases: a hazard assessment and a risk cost analysis of rehabilitation alternatives for existing dam or design alternatives for new dam. In the first phase the consequences of dam failure are evaluated in order to establish the minimum acceptable level of protection for an existing structure. For most dams this minimum protection level will be determined largely by the consideration of potential life loss assuming that an effective warning system and emergency action plan has been adopted. For a new structure the maximum loading conditions of the PMF for the inflow design flood and the MCE for seismic loading would normally be used.

Once the minimum acceptable level of (protection) loading before failure has been selected, the risk cost analysis is performed to determine whether or not higher levels of protection can be economically justified. The risk assessment follows the basic steps described in Sections 2.2 and 3 of this paper and is illustrated by a hypothetical example in USBR (1986).

Although the overall framework for the USU and USBR approach resembles that of the Stanford or MIT screening approaches the methods of analysis and probability and consequence estimation are more detailed for the USU and USBR methods and correspond to those normally used at the design stage of dams. A crucial factor in the successful performance of design level risk assessments is the incorporation of professional judgment in the identification of rehabilitation alternatives, identification and evaluation of failure modes and outcomes, the subjective estimation and updating of probabilities, the performance of meaningful sensitivity studies, and the interpretation of risk assessment results. Therefore, it is essential that qualified and experienced professionals, who would normally be responsible for dam design, should also be responsible for design level dam risk assessments. Individuals are are experienced in risk assessment can serve best as consultants to the responsible engineers and should preferably have dam engineering experience of their own in order to be able to effectively perform risk assessments for dams.

5. ADVANTAGES AND COMMON OBJECTIONS

5.1 Advantages

Proponents of quantitative risk assessment of dams argue that it provides a comprehensive framework for the evaluation and presentation of dam design or remedial action alternatives. It requires the consideration of the interaction between initiating events, failure modes, possible outcomes, exposure conditions and consequences over the full range of magnitudes of each. Therefore, it aids in the identification of which factors (e.g., loading types, failure modes, and exposure factors), or ranges of factors, contribute most to the total risk associated with dam failures, and which options are most promising or cost effective for achieving risk reduction.

Presentation of the results of a quantitative risk assessment provides a concise and systematic display of the economic and noneconomic consequences of each alternative which is considered. The open presentation of all alternatives requires a high degree of objectivity in both the presentation and the supporting analyses. The effects of uncertainty in both the analyses and the exercise of professional judgment can be displayed in a manner which allows for understanding of the degree of technical confidence in the risk assessment. In fact the rational framework of risk assessment leads to clear documentation of the information necessary for a good appreciation by lay people of the significance of complex technical issues. It, therefore, facilitates constructive debate among opposing parties in the decision making process. The information obtained from a risk assessment is an input to the decision making process, and not the decision itself.

The decision maker can readily compare risk of failure at one dam with that at another, or with the risk associated with other types of public works projects. Also he can compare estimated risks with acceptable risk criteria and can consider such quantitative measures as estimates of risk reduction, residual risk, benefit cost ratios, cost effectiveness or cost to save a life. Such measures can provide a useful input to decision making on dam safety which is often a very emotional and politically sensitive issue. As the information base available to the analyst grows, a risk assessment can readily be updated to include revised estimates of probabilities and consequences, changes in uncertainty bounds, new insights into the likely performance of the structure, or new alternatives to be evaluated.

5.2 Common Objections

Frequently those unfamiliar with, or unconvinced about, the merits of the risk assessment approach to the evaluation of dam safety will focus attention on the limitations of the approach. Also, on occasion their objections are based on misconceptions about the approach.

An example of the latter is the statement that "risk assessment diminishes the role of engineering judgment." This statement has been made by some highly experienced engineers who fear that risk assessment is an attempt to replace professional experience and judgment with automatic decision rules which imply a level of knowledge of dam structure performance which they know is currently beyond our reach. The truth is that there can be no meaningful risk assessment of a dam without the involvement of highly competent and experienced dam designers, geological engineers, hydrologists, and others. A risk assessment can only be as good as the input provided by these professionals. The risk assessment approach requires the quantification of the judgments of these professionals and this can only be effectively achieved through involving them in the entire risk assessment process. The risk analyst can help to manage the collection and synthesis of information provided by dam engineering professionals, but he can never substitute for the role of experienced professionals. Under the risk assessment approach the role of such professionals is expanded rather than diminished, since a much wider range of loadings and other conditions is considered than would traditionally be the case under the traditional "worst case" approach to design.

In a similar vein, it is sometimes pointed out that "deterministic approaches to analysis and design are time-tested and that no competent professional can be expected to replace them with new probabilistic methods until these have been similarly proven." It is unlikely that anyone would disagree with this statement. However, it should be pointed out that an important basis for the inputs to a comprehensive risk assessment is the deterministic analyses of slope stability, dynamic structural stability, dam break, flood routing, seepage, etc. Where new analytical probabilistic methods are available they should be run in parallel with the deterministic approaches in order to establish a basis for their evaluation for potential future use.

Other commonly expressed objections to the risk assessment approach focus on its present limitations. These include the fact that probability and statistics are not broadly understood and are viewed with suspicion by many. This objection is not a fundamental limitation of the approach, but rather a handicap to its rapid acceptance which must be addressed by those who pioneer its use. In the long term it presents a challenge to educators to better prepare the public to understand and deal rationally with risk which impacts virtually every aspect of our lives. Another objection is that engineers, geologists and other professionals are generally inexperienced at making subjective probability estimates. This is certainly a valid concern and one that the pioneers in this field are having to address. Sensitivity analysis should be used to assess the relative importance of errors in subjectively updated or estimated probability estimates.

Others express concern that the use of risk assessment implies acknowledgment that there is "some chance of a dam failure occurring." However, this fact is unchanged whether or not a risk assessment is performed. It would seem preferable to systematically identify and examine all "reasonably probable" failure mechanisms and to invest wisely in safety improvement measures rather than deceive ourselves that the chance of dam failure is mysteriously reduced if we do not acknowledge that such a possibility exists.

Currently, certain causes of dam failure, such as inadequate grouting of an abutment, or piping of an embankment cannot be adequately assessed with analytical tools and therefore the basis for their inclusion in a risk assessment is highly subjective. However, this is not an inadequacy of risk assessment per se, but rather of the current state-of-the-art in dam engineering.

Another topic which is highly sensitive is the incorporation of life loss potential into a risk assessment. Dam failure frequently results in the loss of human life. No one would dispute the undesirability of this anymore than they would dispute the undesirability of people losing their lives from any other accidental cause. However, risk is unavoidable. It is only the degree of risk which we can control, and then we are subject to limitations of how much safety we can afford to purchase. In the case of dam safety the lives threatened are not necessarily those of the direct beneficiaries of the presence of the dam and reservoir, and therefore they are usually considered to be exposed to the risk of dam failure involuntarily.

At one extreme of this sensitive topic are those who would advocate placing an economic value on life and thereby integrating life loss into an overall economic assessment of the consequences of dam failure. At the other extreme are those who would require that the risk of life loss from dam failure should be reduced to zero, which of course, is an impossible requirement, although it can be approached as the investment of funds to save a life become "infinitely" large. Obviously as individuals and as a society we cannot afford such levels of safety. However, the simple inclusion of the value of life into an economic evaluation appears to be equally unattractive, and even immoral. On the other hand, to ignore the issue implies a zero value of life which is obviously an unacceptable alternative also.

Risk assessment does not require its users to take either extreme position, or any other position on this issue. But as with the idea of accepting some, albeit very small probability of dam failure, it does enable us to openly address the issue in such a way that high levels of human safety should be achieved with cost effectiveness. If the potential life loss from a dam failure is significant, then a hazard evaluation would suggest the use of probable maximum flood and maximum credible earthquake design criteria. In other cases the most significant reductions in the probability of life loss often can be made through the nonstructural approach of a monitoring system, emergency warning system, and evacuation plan. Such systems are typically quite inexpensive to design, install, and operate relative to the cost of structural measures. If these systems can be used to reduce the probability of life loss to very small levels, then the main basis for a decision on selection between structural alternatives for rehabilitation can be economic.

6. CONCLUSIONS

None of the limitations to the risk assessment approach which have been identified to date appear to warrant dismissal of the approach. Further research is needed in order to develop improved approaches for dealing with them, but meanwhile application of risk assessment to dams can provide valuable insights into the choices available to the decision maker. Such research can be effectively conducted through collaboration between universities and experienced practicing dam engineers in the public and private sector. Joint case studies on actual dams can be an effective means for developing improvement in the risk assessment approach.

The need for providing engineers and other professionals with better backgrounds in probability and statistics has already been identified. For the next generation of professionals this need can be addressed through the undergraduate and graduate curricula, but for the present generation of professionals, workshops and short courses will be needed to familiarize these individuals with the utility of risk-based approaches.

Related to the topic of education is the challenge of communicating the results and conclusions of a risk assessment to the lay public and to decision makers. In this regard, carefully prepared graphics and tabular presentations combined with the use of analogies to more familiar business and insurance situations can be very effective. Even when provided with quantitative risk estimates, individuals are highly influenced by their perceptions of the risk even if these are at variance with the estimates obtained from a formal risk assessment performed by highly experienced professionals. An excellent example of this is in the nuclear field. Just what the role of perceived risk should be in the field of dam safety decision making needs to be defined.

Risk assessment should be recognized as a part of the overall cycle of risk management referred to in Section 2.1 of this paper. The cost effectiveness of nonstructural measures for improving and "managing" dam safety should be remembered (see Fig. 4). Such measures include operator training for detection of problems with a dam, frequent safety inspections by qualified professionals, steps to limit growth of downstream development in the potential inundation area, and implementation and updating of an emergency action plan.

Successful application of risk assessment and risk management approaches to evaluating dam safety requires a combination of the skills of experienced dam design professionals and risk assessment consultants. At this time pioneering work has been completed by USBR or is currently underway, involving the writers, on the application of design level risk assessment approach to evaluating remedial action alternatives on several dams. It is a field with both technical and communications challenges but when properly used it can be a valuable tool, particularly for those responsible for deciding remedial action at existing dams. Risk assessment can be expected to lead to improved dam safety and more effective use of limited funds available for rehabilitating existing dams.

ACKNOWLEDGMENTS

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DAM

SAFETY

GUIDEBOOK

ARIZONA EDITION

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CHAPTER 1

INTRODUCTION

I. Purpose

The purpose of this guidebook is to promote dam safety and to inform you, the dam owner, of your responsibility to safely operate, maintain and repair your dam. This guidebook is intended to be nontechnical yet provide a basis for discussion between dam owners and state dam safety agencies. By the nature of the topic, some technical terms must be used. It is believed that the basic dam safety terminology contained in the glossary (Chapter 2) will assist you in understanding your dam and the impoundment behind it. The responsibilities and liabilities of dam ownership, the role of governmental agencies and emergency response will also be discussed.

To help you evaluate the condition of your dam, a dam safety checklist is included. An operation plan form and guidelines for emergency action plan preparation are also provided for you to use in meeting your responsibilities as a dam owner. This guidebook does not override state specific requirements. All state standards must be followed.

II. Federal Roles in Dam Safety

Several government agencies play active roles in dam safety.

The Federal Emergency Management Agency (FEMA) develops and maintains policy guidelines for dam safety. FEMA develops programs for preparedness planning and emergency response. They also coordinate federal dam safety programs and encourage nonfederal programs to reduce the public hazard of unsafe dams. This **Dam Safety Guidebook** is one of FEMA's methods of fulfilling this charge.

The Federal Energy Regulatory Commission (FERC) supervises the dam safety program of the Federal Power Act. They issue rules and regulations to ensure licensed projects are adequately constructed, operated and maintained to protect life, health and property. FERC's jurisdiction includes dams at hydroelectric projects on navigable streams or on federally owned land; projects using surplus water or waterpower from federally owned dams; or dams affecting the interest of interstate or foreign commerce.

The Department of the Army, Corps of Engineers, is authorized by the Federal Water Pollution Control Act of 1972 and the River and Harbor Act of 1899 to issue permits for the filling in of the nation's waterways. Under the National Dam Safety Act of 1972, the Corps, working with individual states, inventoried 68,153 dams and inspected 8,818.

Five agencies within the Department of Agriculture are involved with nonfederal dams. These include the Agricultural Stabilization and Conservation Service (ASCS), the Farmer's Home Administration (FMHA), the Forest Service, the Rural

Electrification Administration (REA) and the Soil Conservation Service (SCS). Technical engineering responsibility for dams is assigned to the Soil Conservation Service. The other agencies are involved with funding of projects.

The U.S Department of the Interior Office of Surface Mining (OSM) provides support to state regulatory agencies in dam inspection and monitoring as it relates to surface mining. The Department's Bureau of Reclamation has a multipurpose water development function which includes providing water for irrigation, hydroelectric power industry, and recreation. This is done with dams, canals and pipelines.

III. State Roles in Dam Safety

The role of individual states in dam safety varies from state to state. States do have regulatory authority over dam safety, and require the owner to be responsible in the design and construction, operation, maintenance and repair of the dam. Details are contained in Chapter 3.

IV. Scope

This guidebook will introduce the dam owner to dam safety items and dam terminology. Stimulated by this guidebook, the dam owner is encouraged to seek, when necessary, professional legal, regulatory and engineering assistance. This guidebook cannot replace the site specific study and evaluation of a dam by the conscientious owner and professional consultants.

CHAPTER 2

GLOSSARY

I. Definitions

This chapter provides basic terminology used in this guidebook. It is not a complete list but is meant to introduce some basic terminology. Other terminology may be obtained from dam safety books (see reference at end of chapter).

Abutment - That part of the valley side or concrete walls against which the dam is constructed. See Page 2-4. Right and left abutments are those on respective sides of an observer when viewed looking downstream.

Alterations - Such changes in the design of the dam as may directly affect the integrity of the dam and thereby affect the safety of persons, property or natural resources.

Appurtenant Works - The structures or machinery auxiliary to dams which are built to operate and maintain dams; such as outlet works, spillway, powerhouse, tunnels, etc.

Berm - A horizontal step or bench in the sloping profile of an embankment dam. See Pages 2-4 and 2-5.

Breach - A break, gap or opening (failure) in a dam which releases impoundment water. See Page 2-5.

Core - A zone of material of low permeability in an embankment dam. See Page 2-5.

Dam - A barrier built for impounding or diverting the flow of water.

Dike (Levee) - An embankment, usually applied to embankments or structures built to protect land from flooding. See Page 2-5.

Drain, Layer or Blanket - A layer of pervious material in a dam to facilitate drainage. Includes toe drain, weephole and chimney drain. See Page 2-4.

Drawdown - The resultant lowering of water surface level due to release of water from the impoundment.

Embankment - Fill material, usually earth or rock, placed with sloping sides. See also Embankment Dam and Page 2-5.

Embankment Dam (Fill Dam) - Any dam constructed of excavated natural materials or of industrial waste materials. See Page 2-5.

Homogeneous Earth Fill Dam - An embankment constructed of similar earth material throughout, except for possible inclusion of drains. Used to differentiate from a zoned earth fill dam.

Zoned Embankment Dam - An embankment dam is composed of zones of selected materials having different degrees of porosity, permeability and density.

Emergency Action Plan - A predetermined plan of action to be taken to reduce the potential for property damage and loss of lives.

Engineer - A licensed or registered engineer in a given state; offers experience and expertise in the design and inspection of dams.

Failure - An incident resulting in the uncontrolled release of water from a dam.

Freeboard - The vertical distance between a stated water level and the top of a dam. See Page 2-4.

Gate or Valve - In general, a device in which a leaf or member is moved across the waterway to control or stop the flow.

Impoundment - Water or wastewater held back by a dam.

Instrumentation - Permanent devices which are installed in/near a dam to allow monitoring of the dam and impoundment. These devices may include a staff gage (to measure impoundment levels), piezometers, observation wells, settlement or alignment points, rain gage, etc. See Page 2-6.

Maintenance - The upkeep necessary for efficient operation of dams and their appurtenant works. It involves labor and materials, but is not to be confused with alterations or repairs.

Operator - The owner, or an agent or employee of the owner.

Outlet - An opening through which water can freely discharge for a particular purpose from an impoundment. See Page 2-4.

Owner - Any person who owns, leases, controls, operates, maintains or manages a dam or impoundment.

Phreatic Surface - The upper surface of saturation in an embankment. See Page 2-4.

Piping - The progressive development of internal erosion by seepage, appearing downstream as a hole or seam discharging water that contains soil particles.

Plunge Pool - A natural or sometimes artificially created pool that dissipates the energy of free-falling water. The pool is located at a safe distance downstream of the structure from which water is being released. See Stilling Basin and Page 2-4.

Repair - To essentially restore a dam to its approved design condition.

Riprap - A layer of large stones, broken rock or precast blocks placed in a random fashion on the upstream slope of an embankment dam, on a reservoir shore, or on the sides of a channel as a protection against wave and ice action. See Page 2-4.

Silt/Sediment - Soil particles and debris in an impoundment.

Slump Area - A portion of earth embankment which moves downslope, sometimes suddenly, often with cracks developing. See Page 2-5.

Spillway System - A structure over or through which flows are discharged. If the flow is controlled by gates, it is considered a controlled spillway; if the elevation of the spillway crest is the only control, it is considered an uncontrolled spillway. See Pages 2-4 and 2-5.

> **Emergency Spillway** - A secondary spillway designed to operate only during exceptionally large floods.
>
> **Principal Spillway** - The main spillway for normal and flood flows.

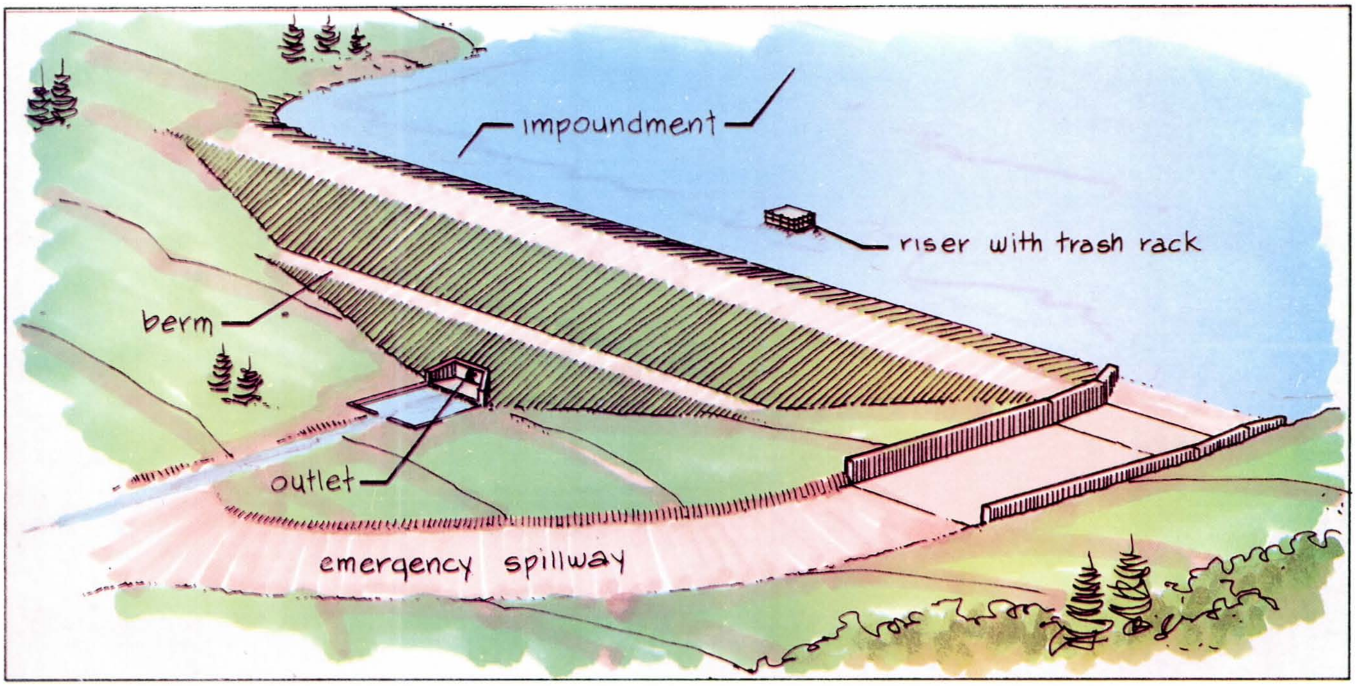
Stilling Basin - A basin constructed to dissipate the energy of fast-flowing water, eg. from a spillway or bottom outlet, and to protect the river bed from erosion. See Plunge Pool. See Page 2-4.

Stoplogs - Logs or timbers, steel or concrete beams placed on top of each other with their ends held in guides on each side of a channel or conduit.

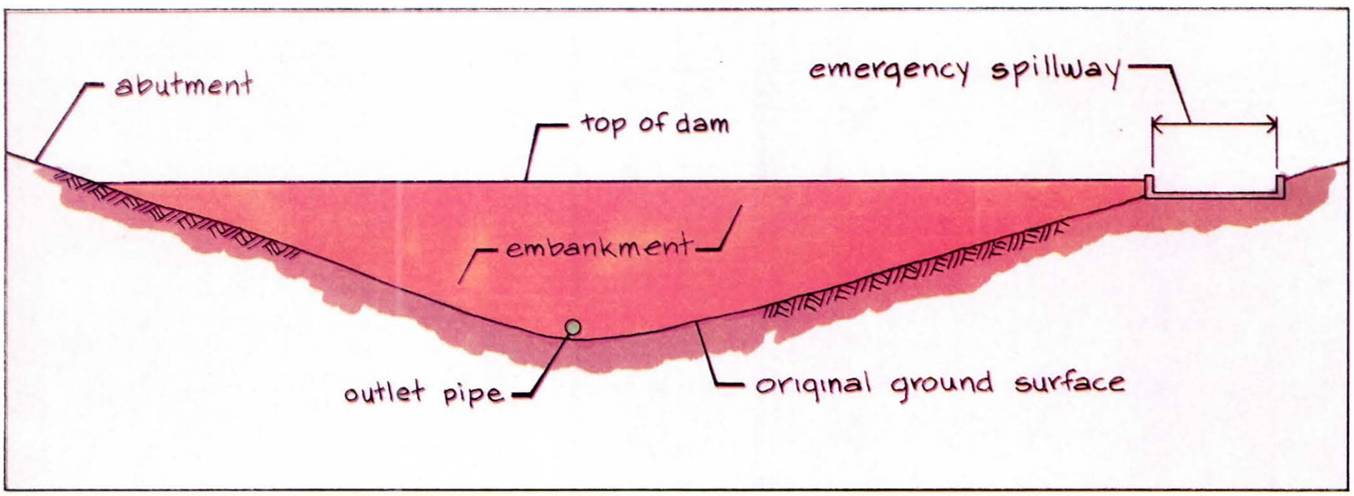
Toe of Embankment - The junction of the face of the dam with the ground surface.

Trash Rack - A structure of metal or concrete bars located in the waterway at an intake to prevent the entry of floating or submerged debris.

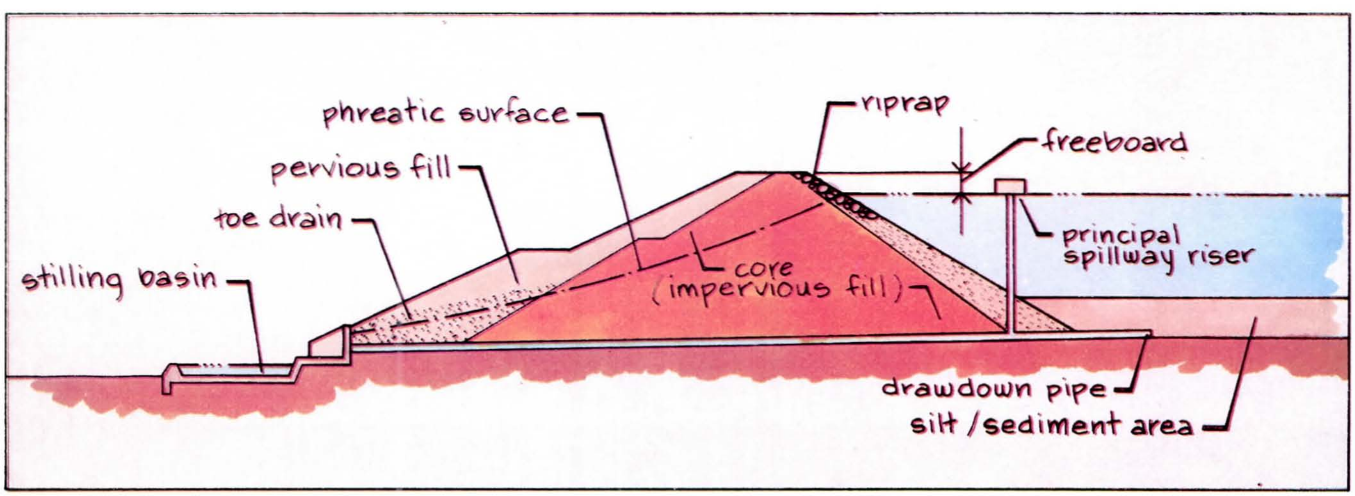
Other technical terms are explained in the text.



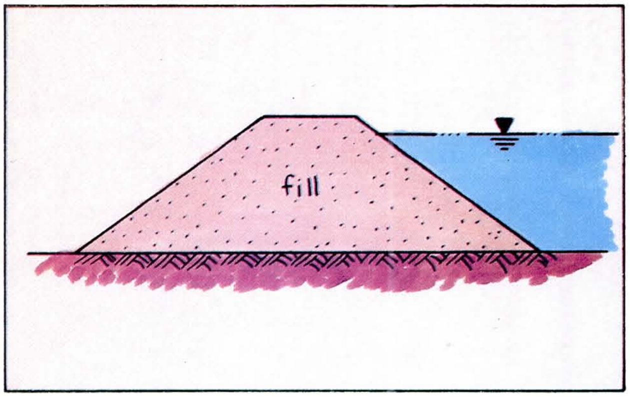
embankment dam



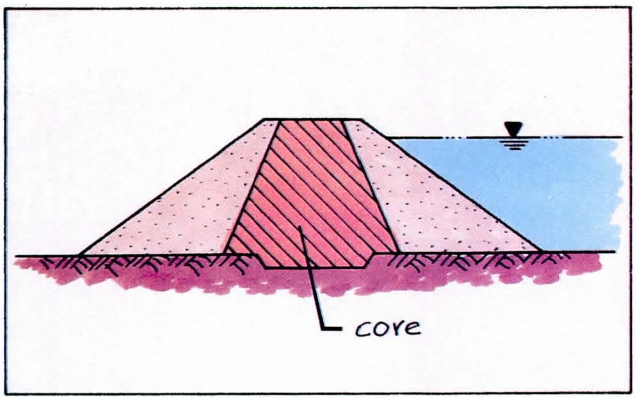
profile along crest dam



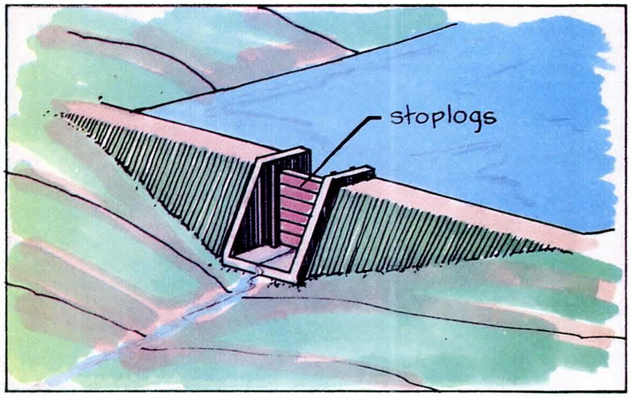
section thru dam



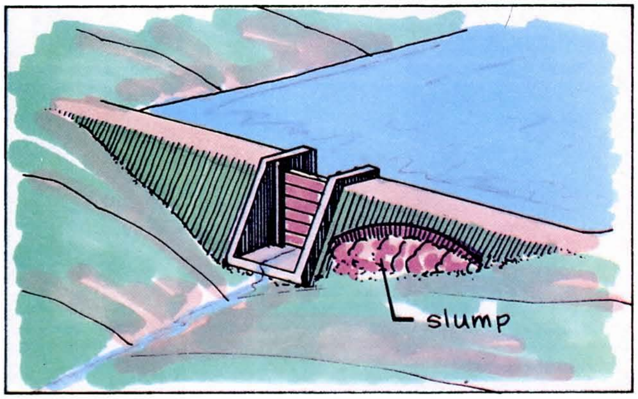
homogeneous earth fill dam



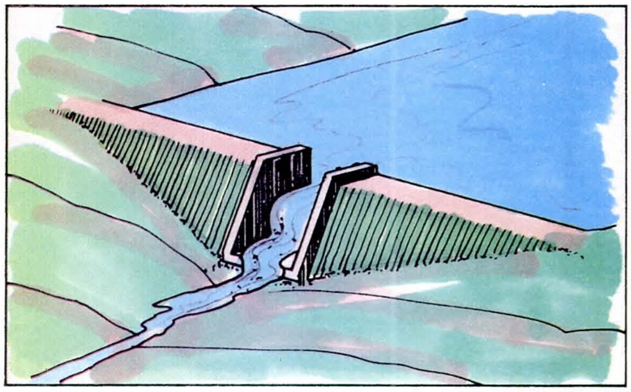
zoned embankment dam



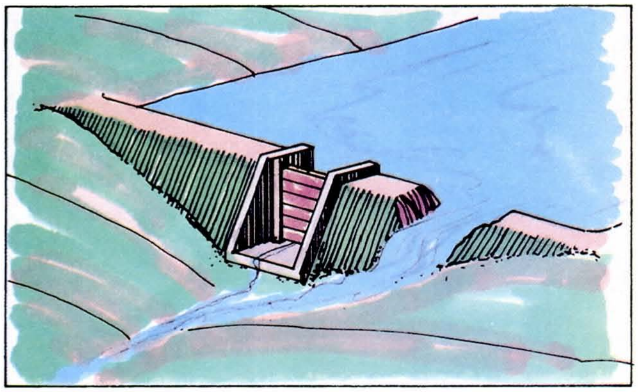
stoplogs



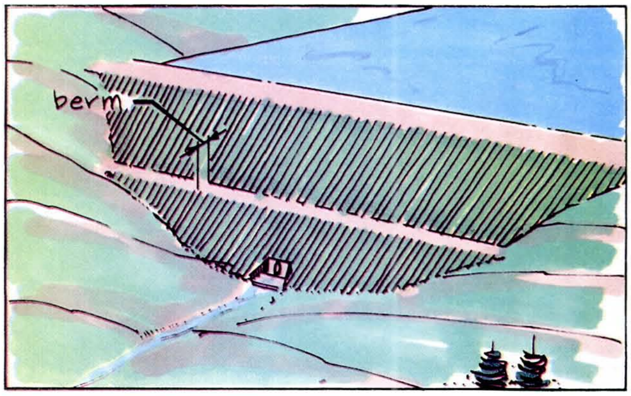
slump area



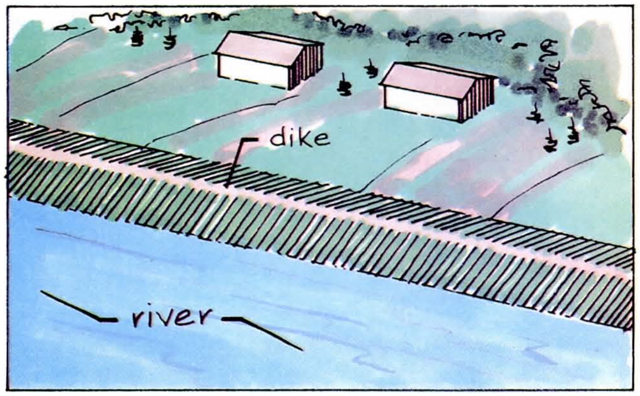
controlled drawdown



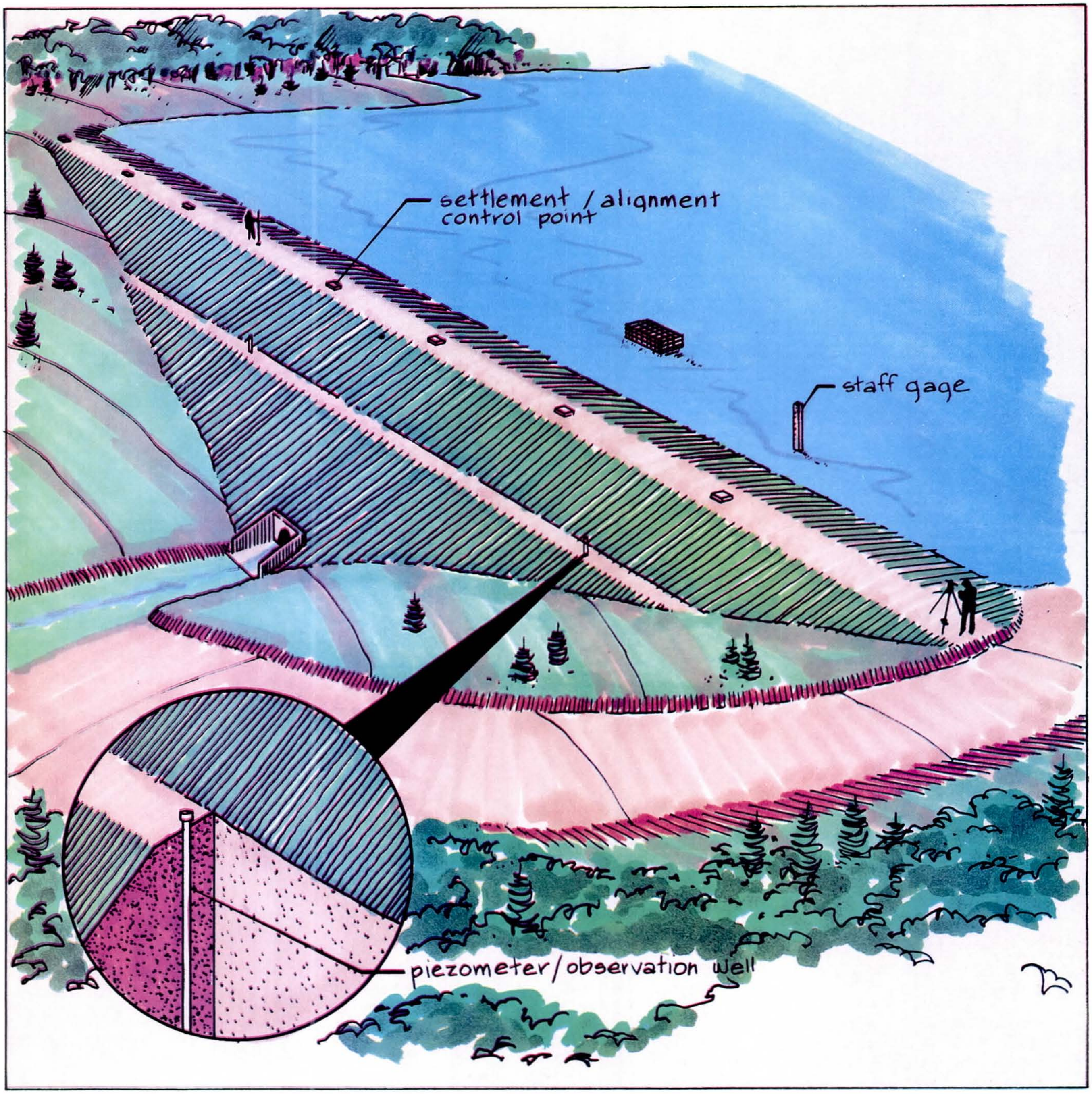
uncontrolled breach



berm



dike (levee)



instrumentation

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CHAPTER 3

YOUR STATE GOVERNMENT'S ROLE IN DAM SAFETY

I. Historical Background

Recognizing the potential hazard created by construction of a dam, the State Legislature acted in 1929 to provide the general public with protection from loss of life or property through malfunction or failure of a dam. With this legislation, the supervision of the dams and reservoirs of Arizona became a State responsibility and was delegated to the State Highway Engineer. This jurisdiction extended to all dams within the State that were 15 feet or more in height from ground level to the spillway crest, or that impounded more than 10 acre-feet.

House Bill 3, enacted by the first session of the 30th Legislature and signed into law by the Governor on April 13, 1971, transferred the responsiblity for the supervision of dams program from the Highway Department to the Water Commission. The House Bill provided for the transfer of the activity in its entirety and specified that all nonfederal dams should be under jurisdiction of the State Water Engineer.

On July 1, 1971, the program for Supervision of Safety of Dams under the direction of the Arizona Water Commission, was fully operational, and with its own staff assumed complete responsibility for the program.

The Arizona Revised Statutes, defining the program, were changed by Senate Bill 1271, which was signed by Governor Williams on April 26, 1973 and went into effect July 26, 1973. Significant among the changes was in increase in the jurisdictional size of dams to 25 feet or more in height or with a storage capacity of 50 or more acre-feet. The statutes were again revised in 1977, specifically excluding mine tailings dams from state jurisdiction.

II. Current Regulations

The Arizona Revised Statutes (ARS 45-701. to 45-717; a copy is included in the Appendix) currently give regulatory responsibility to the Arizona Department of Water Resources (formerly the Arizona Water Commission). Arizona law includes all of the major areas suggested by both the U.S. Committee on Large Dams and the national Association of State Dam Safety Officials. The State has the authority to and does (1) Review and approve plans to construct, enlarge, repair, alter or remove dams, (2) Perform periodic inspections of construction, (3) Issue licenses to operate, (4) Investigate every dam at least once each five years, (5) Issue orders as needed to correct deficiencies, (6) Adopt rules, (7) Require a fee as part of the application process, (8) Require emergency action plans.

Additionally, the statutes establish the duties and responsibilities of both the State and the individual dam owners regarding the proper maintenance and repair of dams, as outlined in Table 1. The law requires owners of any dam to properly maintain the structure, giving due regard for life and property while applying sound and accepted engineering principles.

Currently, there are about 190 dams within the state that fall under these regulations (an inventory showing the number and locations of dams in each county is included in the Appendix). Nineteen are concrete or masonry. With the exception of one steel dam, the remainder are earth and/or rock. The average dam is 37 feet high and stores 3330 acre-feet of water. Ownership is as follows: City, County and State Governments 81, Private Corporations and Individuals 72, Water Development and Irrigation Districts 36, and Boy Scouts 1.

TABLE 1

DUTIES AND RESPONSIBILITIES REQUIRED BY LAW

General Item	State Responsibility	Owner Responsibility
1. Maintenance & repair, in general	Supervise and monitor the maintenance & up-keep of dams within the state. (Ref. ARS 45-712.A.)	Maintain and keep in good repair and/or operating condition, by exercising prudence and due regard for life or property, and by applying sound engineering principles. (Ref. ARS 45-715.B.)
2. Standards for Proper Maintenance	Prepare guidelines for the maintenance and/or operation of dams within the state. (Ref. ARS 45-712.A.)	Maintain and/or operate in accordance with all of the rules, regulations & guidelines of the Department.
3. Inspections	Make engineering inspections of all dams as frequently as warranted. (Ref. ARS 45-712.A.)	Cooperate with state employees conducting engineering inspections; provide access to site and furnish upon request any pertinent information.
4. Correcting Deficiencies	Issue orders directing owners within a certain time-frame to correct any maintenance and/or operating condition deficiencies found during inspection of the dam. (Ref. ARS 45-712.A.)	a. Comply with provisions of any state order requesting correction of any deficiency noted. b. Failure to comply with an order of the Department is a Class 2 Misdemeanor and every day is a separate offense. (Ref. ARS 45-716. & 45-717.)
5. Emergency Conditions	In emergency conditions, take whatever measures are appropriate (eg. lowering the lake), and may recover the cost from the dam owner. (Ref. ARS 45-712.)	If emergency conditions exist, contact local and state emergency authorities and the State DWR (See Section V of this Chapter.

III. Inspections

In order for you to carry out your first duty and responsibility of providing general maintenance for your dam, it is recommended that you perform regular inspections of the dam. These inspections, along with the State's periodic inspections, should make you aware of any problems that develop so that appropriate corrective measures can be taken in a timely manner. To assist you in your inspection, a checksheet has been provided in the Appendix, with explanations given in Chapter 5 - Inspections.

The State's periodic inspection program involves an on-site inspection of your dam at least once every 1 to 5 years (depending on size, downstream hazard potential and condition) by an experienced staff member of the Department of Water Resource's Safety of Dams Section. This inspector will be evaluating how well your dam is being maintained and will identify any noticeable deficiencies or problems. An inspection report is then prepared and sent to you, outlining the results of the inspection, noting any problems or deficiencies, and recommending what corrective measures should be taken. A nominal inspection fee of $50.00 plus $1.00 per foot of height is charged for each state inspection.

If deficiencies are noted by these inspections and corrective measures are recommended, it may be necessary for you to obtain the services of a registered professional engineer who is experienced in the design, construction and maintenance of dams. For your information a list of some qualified engineering firms that have experience in this area is provided in the Appendix.

IV. Application Requirements

Any routine maintenance or **minor** repair work needed to be done on your dam does **not** require approval from the Arizona Department of Water Resources. However, if major repair or rehabilitation work is envisioned for your dam, approval from the Department is required **prior** to beginning the work. Submit an application for approval, including appropriate plans and specifications. (A copy of an application form and instructions are included in the Appendix). A $50 filing fee and a further fee based on the estimated cost of the project must accompany the application form. Staff from the Safety of Dams Section of the Department will review the proposed work and make its recommendation to the Director of the Department. Once the Director approves the proposed work, an approved copy of your application will be issued to you, authorizing you to proceed with the repairs.

Be sure to submit your application for a permit at least 2 to 3 months before you plan to begin major repairs or rehabilitation work. Once you have submitted a **complete** application to the State, you should expect it to take from 60 to 90 days before the Department will act on your application.

V. State Contacts

General Information

If you would like some information about the Dam Safety Program, or any other information concerning your dam, you should contact:

Arizona Department of Water Resources
Safety of Dams Section
99 E. Virginia
Phoenix, Arizona 85004
(602) 255-1541

Insurance

Insurance is a worthwhile opportunity for you to "Share the Risk" of being an owner of a dam. For information concerning insurance for your dam, you can contact:

Arizona Insurance Department

801 E. Jefferson, Room 200	402 W. Congress
Phoenix, Arizona 85034	Tucson, Arizona 85701
(602) 255-5400	(602) 628-5386

Emergencies

In extreme emergencies, such as, if:

1. Your dam should be overtopped by floodwaters;
2. You anticipate your dam will be overtopped by floodwaters in the very near future;
3. Your dam begins to fail; or
4. You anticipate your dam will fail in the very near future;

Immediately notify:

--- Arizona Department of Public Safety (602) 262-8011

--- Your local county Emergency Services/Civil Defense Director/Coordinator (roster included in Appendix)

--- State of Arizona, Emergency Services Division (602) 244-0504

--- Arizona Department of Water Resources (602) 255-1541 (Monday-Friday, 8:00 am - 5:00 pm)

VI. Financing

For private dam owners, the usual financing sources available for obtaining funds for maintenance/repair work are banks, savings and loans, credit unions, etc. For cities, towns and conservancy districts, taxes usually provide the necessary funding for operation and maintenance/repair expenses. The State currently has no specific funding program for assisting dam owners with any of their expenses.

CHAPTER 4

LIABILITY AND RESPONSIBILITY OF DAM OWNERS

Dam ownership carries with it significant legal responsibilities. The dam owner should be aware of the potential liabilities and how to conscientiously deal with these liabilities.

This chapter will deal with general legal and insurance matters to help you minimize exposure to liability due to dam ownership or operation. You will become familiar with the responsibilities imposed by dam ownership. Since this guidebook is intended to provide general guidance to dam owners, it cannot answer specific legal issues. Dam owners and operators should obtain competent legal counsel when dealing with specific issues.

I. Potential Liability Problems for Dam Owners

A dam owner should first be familiar with the legal obligation to maintain a dam in a safe and reasonable condition. The **general rule** is that a dam owner is responsible for its safety. Liability can be imposed upon a dam owner if he or she fails to maintain, repair or operate the dam in a safe and proper manner. This liability can apply not only to the dam owner, but also to any company that possesses that dam, or any person who operates or maintains the dam. If an unsafe condition existed prior to ownership of the dam, the new dam owner could not be absolved of liability should the dam fail during his term of ownership. Thus, the owner must carefully inspect the structural integrity of any dam prior to purchase and then provide inspection, maintenance and repair thereafter.

Since the dam owner is responsible for dam safety, it is important to note what you must do to comply with that legal duty. The dam owner must do what is necessary to avoid injuring persons or property. This usually applies to circumstances and situations which can be anticipated. A dam owner would generally not be responsible for those circumstances that a reasonable person could not anticipate. One key action is almost universally recognized: In order to meet your responsibility to maintain your dam in a reasonable and safe condition, virtually every jurisdiction will require a dam owner to conduct regular inspections of the dam and maintain and/or repair deficient items. Regular inspections by qualified professionals are virtually mandated if a dam owner is to identify all problems and correct them.

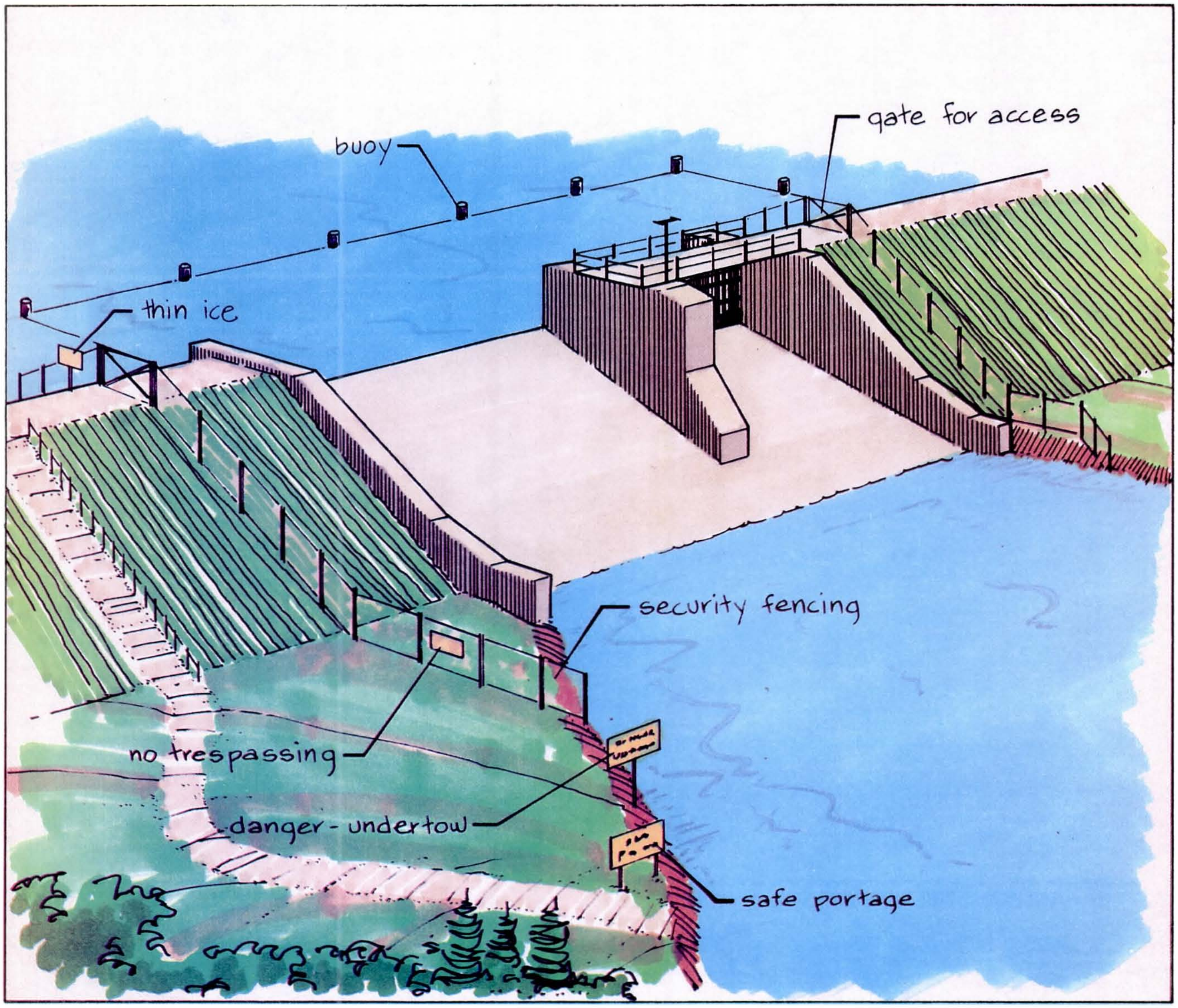
II. Potential Personal Injury Liability

Dams and impoundments are popular places, even if located in remote areas. A dam may be visited by employees, contractors, invited visitors or trespassers. The presence of these persons is a potential liability to the dam owner. Liability or worker's compensation insurance should cover employees, contractors or invited guests. However, the trespasser presents a unique problem.

The majority of trespassers at a dam site are probably members of the public who wish to use the site for fishing, boating or swimming. While they may mean no harm, their unauthorized use of the site is a serious liability problem for the dam owner.

The dam owner is responsible for making and keeping his premises safe. The general rule is that the dam owner must avoid conduct or conditions which could injure any person, even one who trespasses. If the dam owner knows that an unsafe condition exists he is responsible to correct it and/or post warnings (See Page 4-4). Typical dangers at a dam site include fast moving water, open spillway (pipes) and thin ice. A particularly dangerous area is the spillway which not only has fast moving water but undertow at the spillway bottom.

Owners of dams are charged with greater responsibility when the trespassers are children. By reason of children's inability to understand the danger which a condition may pose, a dam owner is expected to protect children from the dangers of a dam site. In effect, this rule requires you to anticipate what parts of the facility would be particularly attractive to children. Since signs may not adequately warn children, security fencing is necessary. Dam sites located near state or county roads, campgrounds or picnic areas, or near populated areas will attract many more people. These popular dam sites require frequent visits by the dam owner to inspect and assure safety.



typical warning signs

III. Potential Liability Due to Operation of the Dam

In addition to liability problems arising out of dam ownership, operation of the dam is also a significant legal issue. First and foremost is the simple **right to operate.** State law requires a permit to construct, repair and/or operate a dam. Your state dam safety agency should be consulted for particular matters regarding this issue. (Also, read Chapter 3 of this Guidebook.) In addition, a dam on a navigable stream may involve federal government regulations which may govern operation.

Beyond the basic permitting question, all dam owners must consider the effect of dam operation on the rights of other water users, whether they are upstream or downstream from the facility. For both upstream and downstream users, this responsibility includes a duty to avoid negligent flooding of their property.

A general rule in all states is that the dam owner must protect downstream land owners from additional flooding, if those downstream owners have come to rely on the existence and operation of the dam to reduce flooding. The extent of this duty will vary from state to state, so the dam owner is advised to consider dam operations in the light of the downstream land owners' expectations and dependency on the dam to prevent flooding.

In situations where there is no specific duty to protect downstream owners from flooding, the dam owner must still operate the dam conscientiously. As the dam owner, you must be in a position to clearly show that your dam did not increase flooding.

Upstream users may also have the right to be protected from damage caused by operation of the dam. Therefore, the dam owner is advised to assess the legal as well as the physical impact of **any** change in the level of the impoundment, including dam removal.

IV. Environmental Concerns

While there are an infinite number of potential environmental issues, a few basic areas of concern should be addressed before a dam is purchased or its method of operation altered. Since this guidebook cannot address all environmental issues, you should seek professional evaluation of potential environmental problems. However, we can give you some general guidance.

Dams with gates for regulating the impoundment and downstream flow can cause water levels to fluctuate. These fluctuations can cause gain or loss of wetland habitat affecting fish spawning, waterfowl, and shorebird nesting. Fluctuations of water levels can also increase shore erosion, cause unsafe ice conditions and the like. At this time, virtually all states have laws concerning wetlands.

Variations in the impoundment and downstream elevations can also impact fish in the impoundment or the river. Evaluation of existing fisheries and the impact of changes will require consultation with the state agency responsible for fisheries.

Within a dam impoundment, it is likely that sediments have accumulated over the years. Release of these sediments downstream by operation of the dam, changing the impoundment level, or removing the dam could result in significant damage and liability to the dam owner. In addition, release of sediments downstream could adversely impact plant and wildlife for significant time periods. It is also quite common that the sediments contain pollutants. Thus, the dam owner should carefully consider the possible impacts of dam operation and how it affects the environment.

V. A Final Word About Liability

The dam owner is liable. This section on liability is only a general introduction to the many issues regarding dam owner liability. The discussion is only intended to provide a basis for you to consider liability potentials and to encourage you, the dam owner, to seek competent legal counsel and/or technical experts to help resolve your problems. Where the ownership and operation of dams and impoundments are concerned, the old saying, "an ounce of prevention . . ." is appropriate. Following it will truly save you the "pound of cure."

VI. Insurance

The purpose of this section is to provide dam owners with general information about dam insurance. The primary goal of dam insurance is to share the risk and protect the assets and financial well being of the dam owner. Insurance cannot make a dam safe, or make an inherently faulty construction or renovation project into a good one. Inadequate coverages or insufficient limits on those coverages, coupled with a major loss, can mean the financial ruin of a dam owner. In order to obtain insurance and get a reasonable rate, the dam owner will have to show that the dam meets all state standards with regard to design, construction and operation.

When insuring a dam, the owner should select and involve a competent insurance agent or broker as early as possible. Whenever a dam project requires new construction, reconstruction or renovation, any lender involved will be very interested in the adequacy of the dam owner's insurance program.

The primary job of the agent is to serve as a contact point between the client and the insurance companies, and to place the insurance coverage with appropriate companies. The agent, depending on his skill, dedication and relationship with insurance markets, can greatly affect both the premiums quoted by the companies and the availability of certain coverages. Although most types of insurance have standard contract forms, many of the details of the coverage are open to negotiation and can be tailored to meet the needs of the dam owner, except in those areas mandated by law. It is important to work with your agent to define conditions of the policy that are of real importance and those that need modification.

Although the size of an agency is not an indication of its quality or experience, large national firms will frequently have more extensive consulting services available. Also, dams are an unusual risk to most insurance companies and large agencies often have personnel who have worked with other dam owners and industries with dam experience.

Contact your insurance agency or state insurance commissioner (Chapter 3) to get a list of insurance agencies who may assist you with dam insurance.

There are various types of insurance the dam owner should consider, in consultation with his agent. These include the standard "All Risk" property damage policy with a flood coverage amendment, business interruption insurance, boiler machinery coverage, general liability, automobile liability, workers compensation, and umbrella liability policy coverage.

Because of the many types of insurance protection required and available, the development of an effective insurance program requires care and planning. If you involve a qualified insurance agent in the early planning and work diligently to define your insurance needs, then an effective and economic program can be developed.

CHAPTER 5

INSPECTION

The purpose of a dam inspection program is to identify problems and/or unsafe conditions. Periodic inspections, conducted either by the dam owner or by an engineer, may be required under state statute. Inspection is an integral part of a proper maintenance program for a dam. Failure to correct identified maintenance and repair items could result in the failure of a dam. Failure of a dam through lack of inspection and maintenance can severely impact upstream and downstream properties. Widespread personal injury and property damage could result. Severe financial losses would be suffered, not only from the dam failure, but through liability claims against the owner.

This chapter will introduce the dam owner to two levels of dam inspection. The major parts of a dam will be defined. Common dam problems and how to recognize them will be explained. Finally, the dam owner will learn where to get additional technical assistance.

I. When to Inspect

An inspection program for the dam starts during construction and continues on a regular basis for the life of the dam. **A dam is an active structure**, constructed of materials that are subject to erosion, corrosion and deterioration by wind, water, ice and temperature. Manmade influences such as vandalism may speed up deterioration or damage to the dam. Water passing over, under and through a dam exposes it to continuing deterioration and possible weakening of its structural elements. A well documented inspection program will observe this deterioration, and identify needed repairs and maintenance items that, if carried out, can prevent structural failure of the dam.

All dams, new or old, require constant monitoring and inspection. It has been well documented by the engineering profession that a high percentage of dam failures occur during or just after the initial filling of the impoundment. The force of water on the dam as the impoundment is filled causes significant stresses in the dam. Similarly as a dam ages, inspection remains important because deterioration and accumulated stresses take their toll.

II. Types of Dam Inspection

An inspection program usually has at least two types of inspection. The first type is regularly conducted by the dam owner or operator and is called an operation and maintenance type inspection. This owner inspection includes, but is not limited to, frequent visual inspections of the dam surface, and recording of data from instrumentation at the site.

The second type of inspection, the engineering inspection, will provide a thorough, systematic evaluation of the dam condition. This engineering inspection often requires special equipment and should be performed by a qualified engineer or by a consulting firm with experience in the construction and inspection of dams. The frequency of engineering inspections may be dictated by state statute.

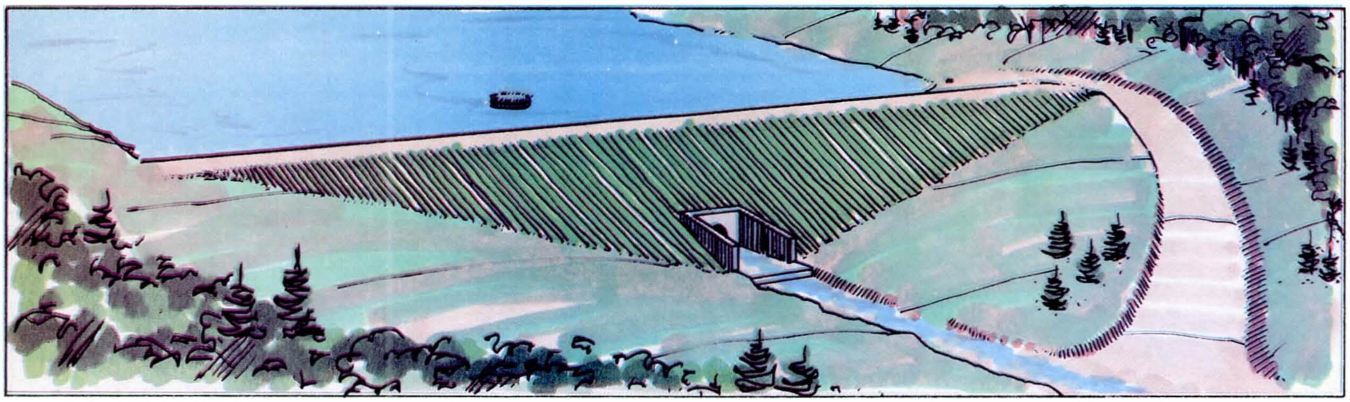
III. What Dam Parts Need Owners Inspection

To conduct an owners inspection, the dam owner should be familiar with the type and parts of the dam, and with the words commonly used to describe them. The brief glossary in Chapter 2 of this guidebook will give you most of the terms you will need to know.

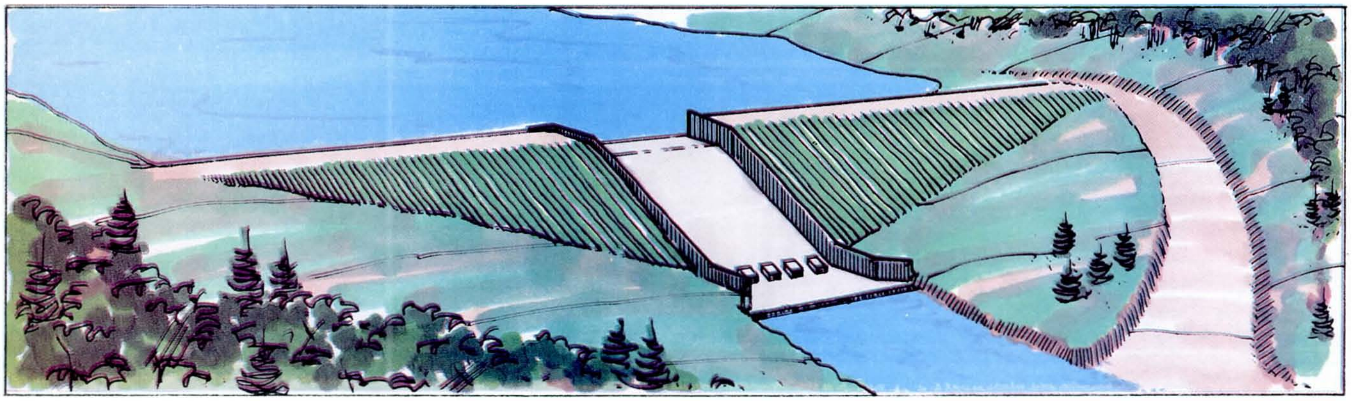
There are many possible dam types and designs. Some are shown on Pages 5-4 and 5-5. Despite the differences in dam types and styles, most dams will have principal parts that are typical. Some of these parts are the spillway and/or outlet works and associated mechanical equipment; earth embankments; abutment areas; operating equipment; and appurtenant works. Pages 5-6 and 5-7 show principal parts of dams.

Spillway Outlet Works

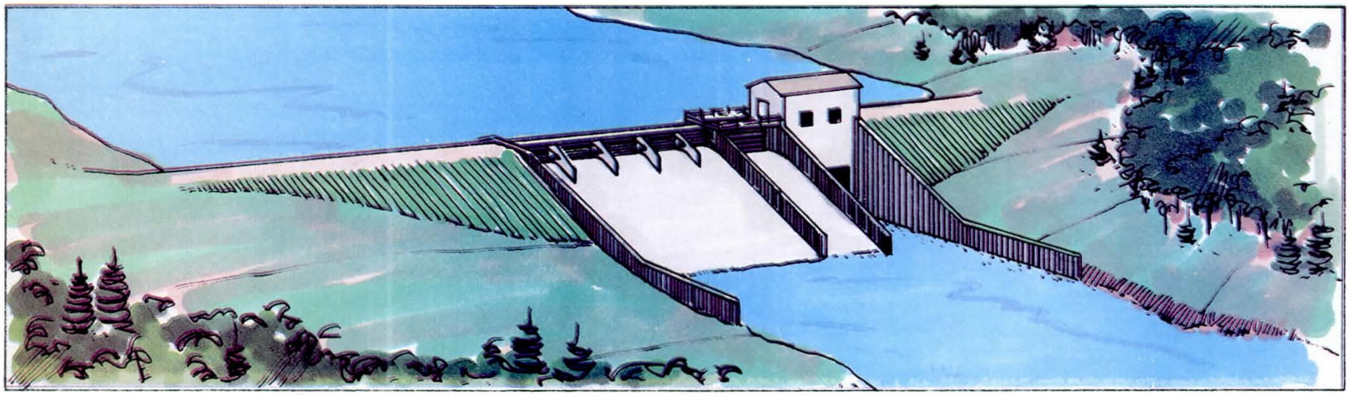
Spillway and/or outlet works are a means of passing water around or through a dam structure. The spillway may or may not have gate control. The spillway should be of sufficient capacity to permit passage of flood flows without overtopping the dam. Common terms used to describe spillway or outlet works are as follows:



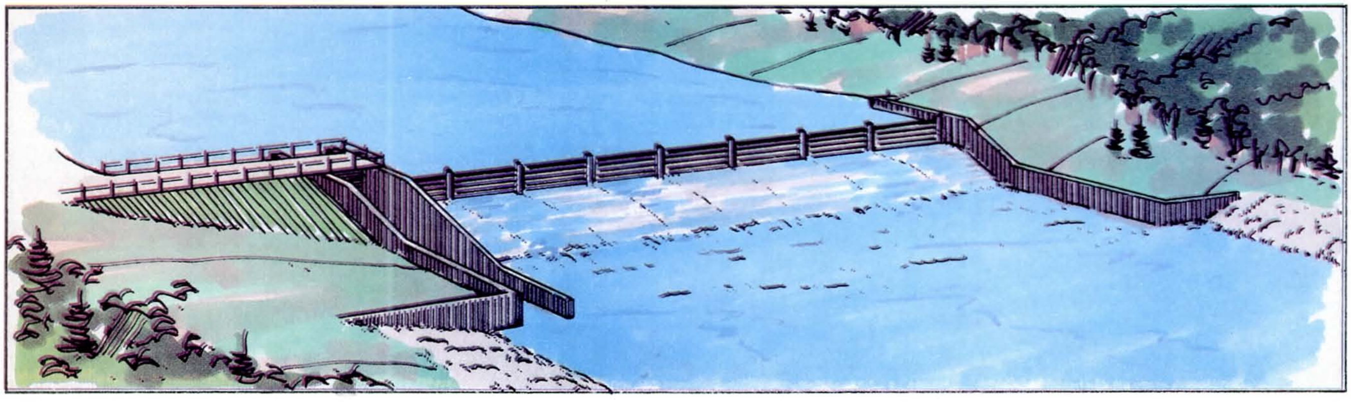
earth embankment with drop inlet and emergency spillway



earth embankment with overflow spillway

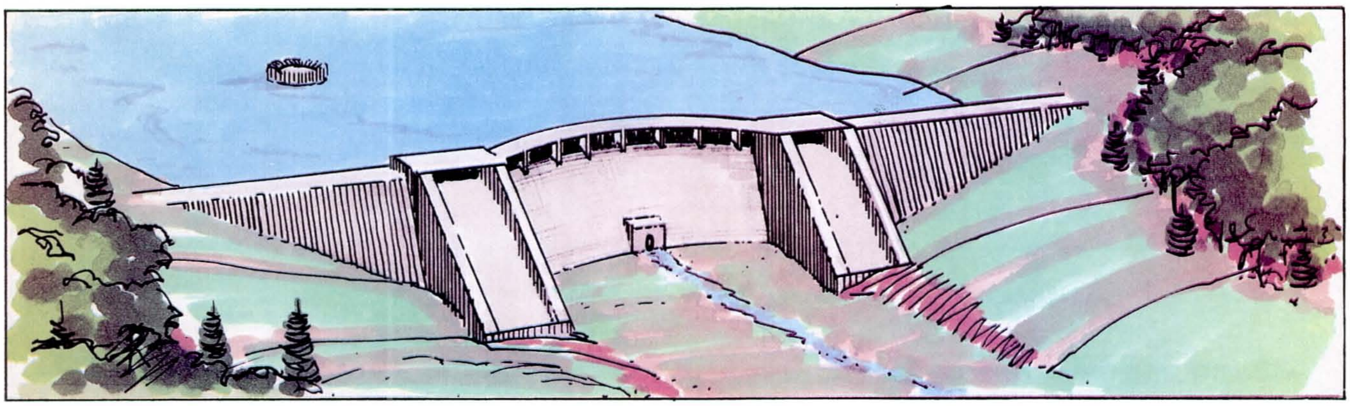


gated dam with powerhouse and tainter gate spillway

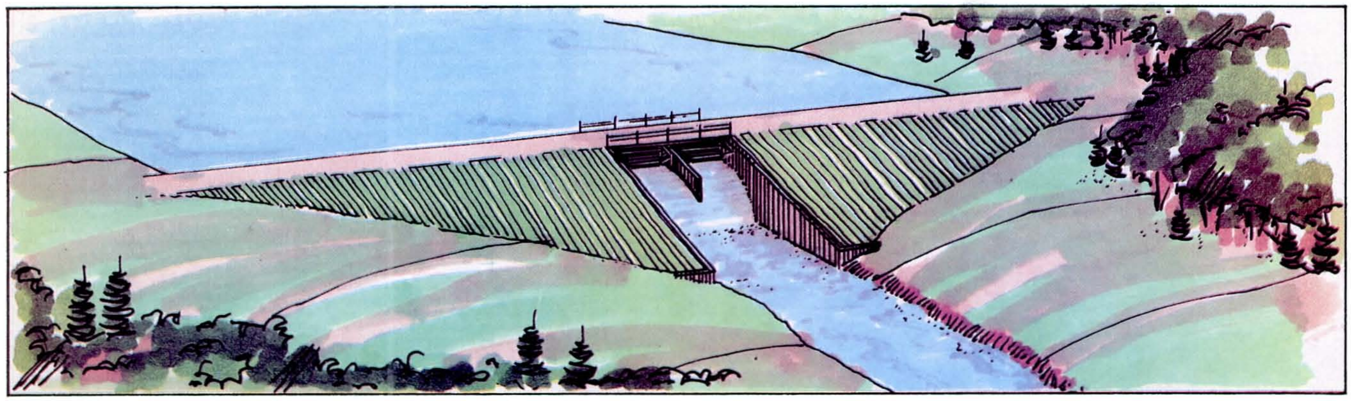


gravity dam with stoplog spillway

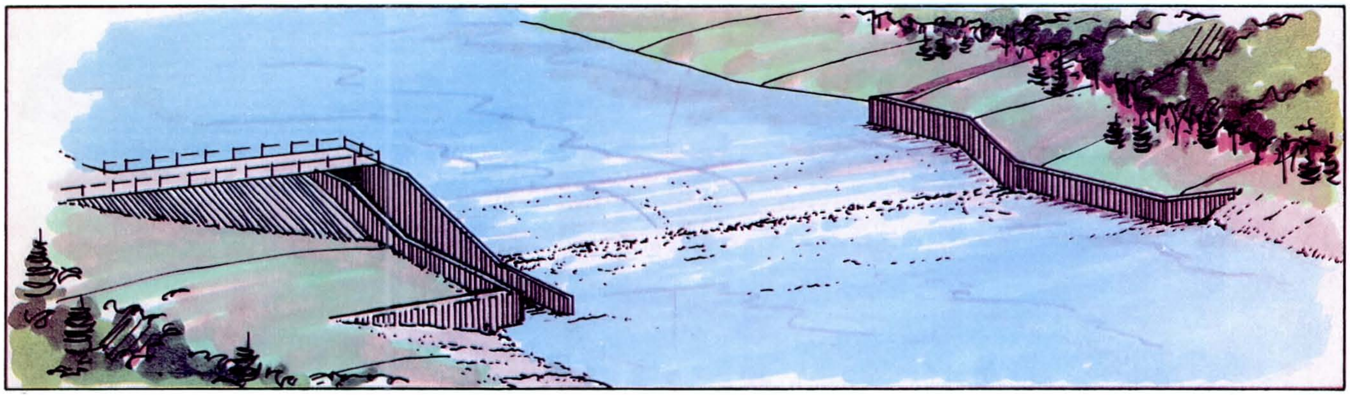
Typical Spillways



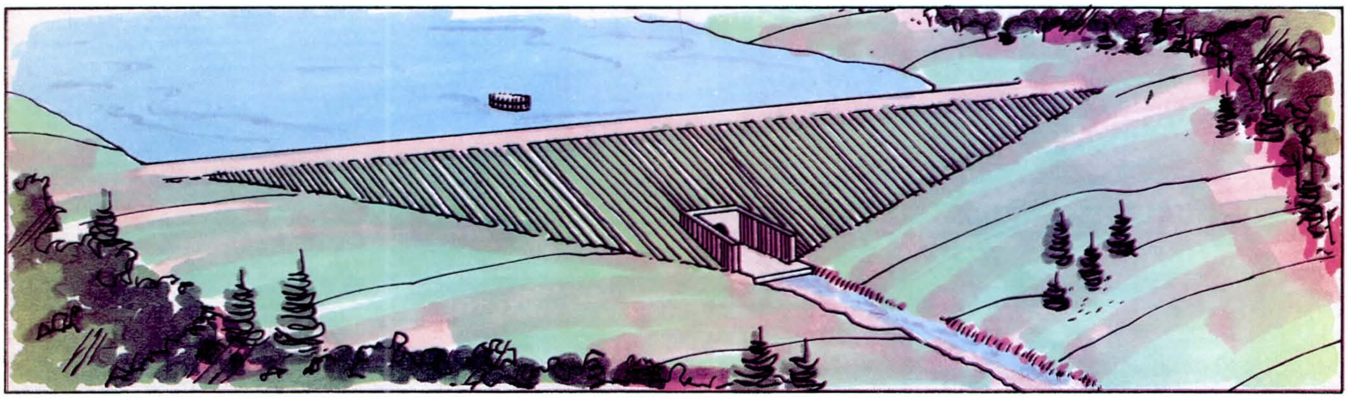
arch / buttress with pipe outlet



rock fill / timber crib with tainter gates

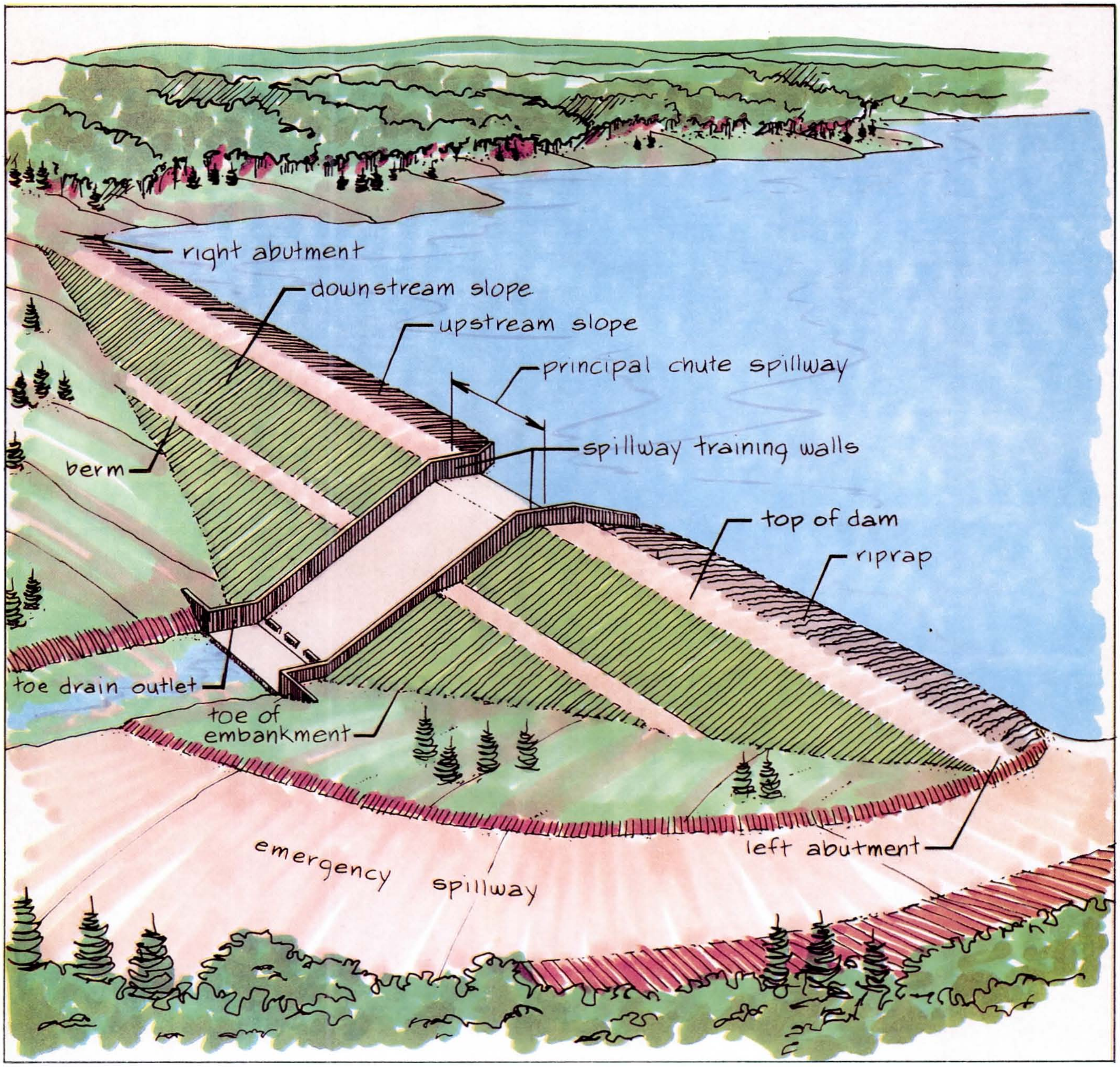


concrete gravity with overflow spillway

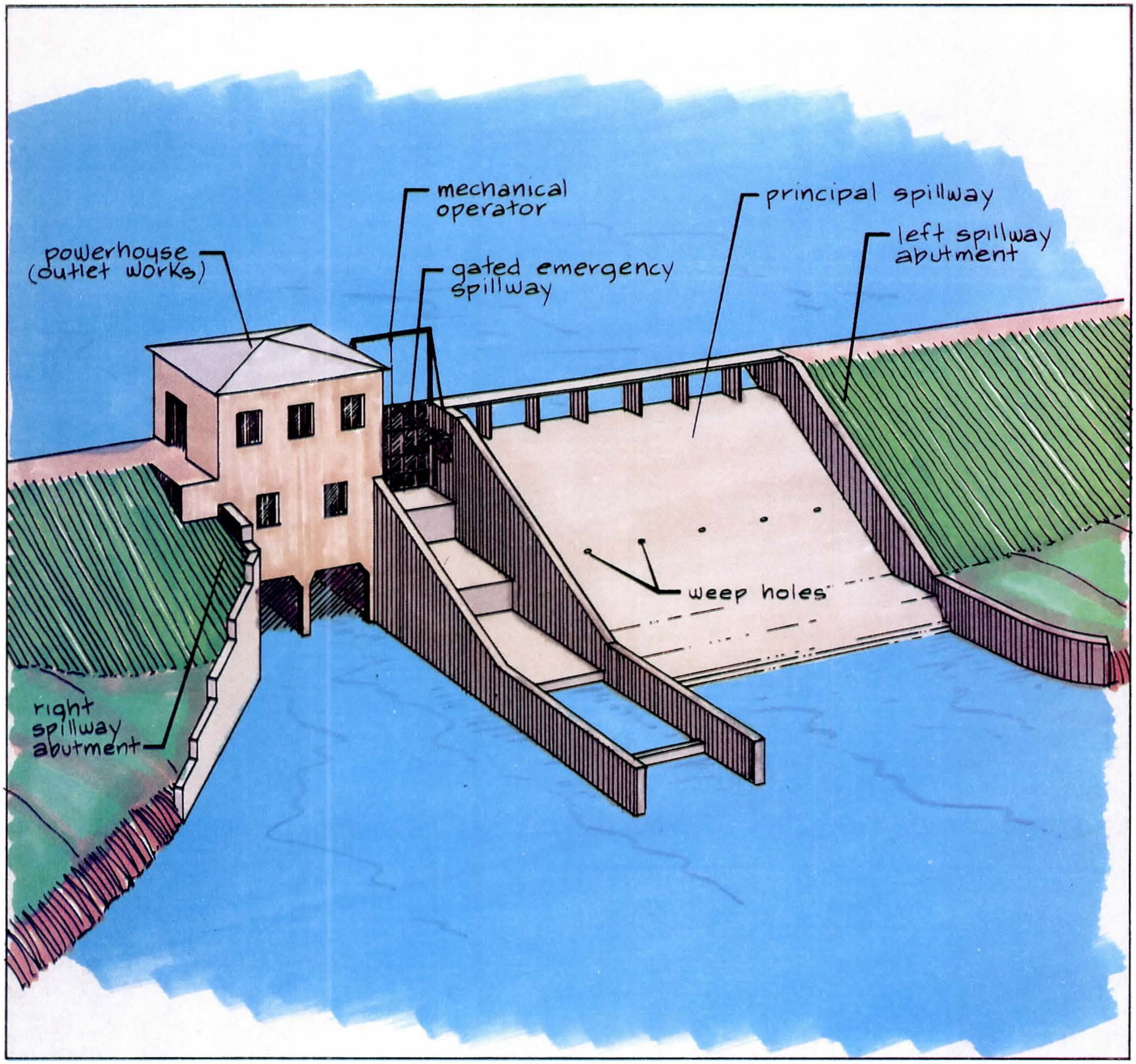


earth embankment with drop inlet

Typical Dams



principal parts of a dam



principal parts of a dam

Principal Spillway - The principal spillway may be a concrete chute, concrete pipe, or metal pipe, with or without gates. In either case, the principal spillway passes water and regulates the water level of the impoundment. Inspection of this structure for cracks, settlement, rusty pipes, pipe joint separation, pipe alignment and deterioration is very important.

Emergency Spillway - An emergency spillway is normally used when there is too much water for the principal spillway to handle. Besides checking for settlement or cracks, make sure it isn't blocked with logs, fences, tree/brush growth or other debris. If your dam doesn't seem to have an emergency spillway, advice is available from your state dam safety agency.

Intake and Outlet Works - These may be gates, valves or stoplogs which are used to regulate water for a specific purpose. For example, an intake and outlet works structure would be the gates in front of a hydroelectric powerhouse or an irrigation channel. The operation of the gates should be checked. Gates, chains, cables and guide channels should be checked for rust and cracks, to be sure they are in good condition. Any logs or debris should be removed.

Trash Racks - These are part of the principal spillway gates or intake and outlet works of the dam. Trash racks and spillway gates must be inspected on a periodic basis to make sure they are unobstructed, operating well and allowing free flow of water.

Appurtenant Works

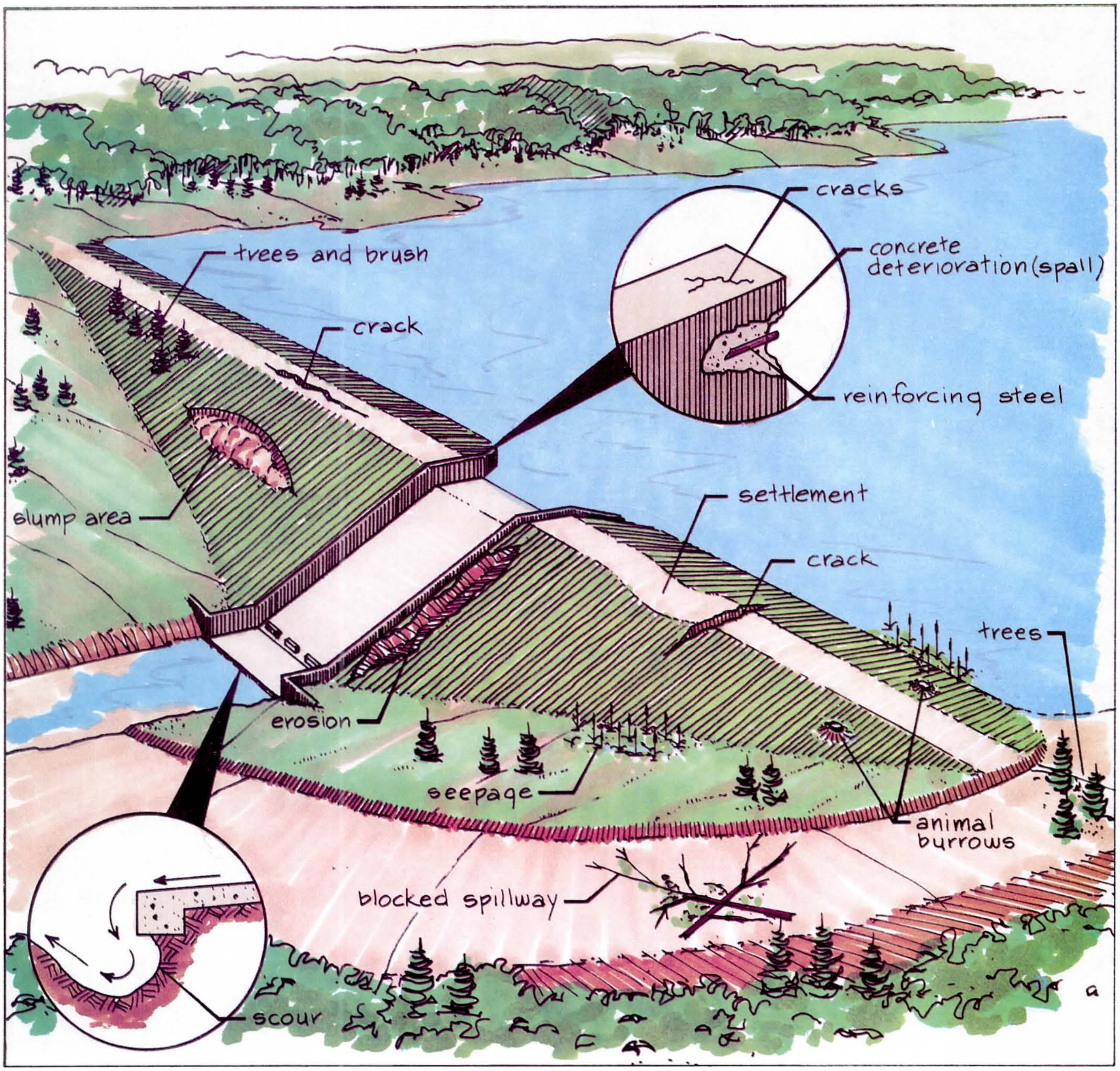
All mechanical and dam operation related equipment should be frequently inspected to ensure it will operate as designed. If equipment hasn't been used recently, the inspection should include a test operation. One important item is the drawdown facility and whether it is operational.

Besides the basic parts described above, dams often have many other components. Operational parts should be checked frequently and maintained.

Earth Embankments

Earth embankments are a main part of most dams. Concrete or masonry spillways may have earth embankment portions that connect the spillway to the natural ground area. This combination of earth embankments and some type of spillway is common.

Earth embankments are very susceptible to deterioration by weathering and the erosive forces of wind, water, ice and temperature changes. The older the structure, the more the owner must be concerned about the embankment. Observations and inspections should be concentrated on seepage (piping), cracks, settlements, slumps, erosion, scour, vegetative cover and animal burrows (Page 5-10). Each one of these is an indicator of potential problems.



potential problem indicators

Seepage - The passage of water through and/or underneath the earth embankment or at the contact between the embankment and other structures is called seepage. It appears on the downstream side of the embankment and can be indicated by cattails or other wet environment vegetation. Sometimes a drain system is provided to collect and control the seepage water. If a collection system is not provided, water normally finds its way to the natural stream bed, creating an erosion channel. Water can also saturate the slope, leading to slumping of the embankment face. Seepage must be monitored for the presence of soil particles. If soil is moving, a piping condition (internal erosion) may exist. This is a dangerous condition and requires immediate contact with the state dam safety agency and/or engineer.

Cracks - Cracks in the earthen embankment indicate movement of the dam and/or foundation. Cracks can weaken the structure, leading to potential failure. Cracks should be repaired and monitored to detect any subsequent changes. Before cracks are repaired, seek advice from an engineer.

Settlement - Settlement indicates a loss of material or compression of material either within the dam or the foundation. It causes a decrease in elevation of all or a part of the dam. If the settlement is not uniform, cracks may form or depressions (low spots) may result. The settlement of the top of the dam reduces the amount of freeboard. Observable settlement or settlement that changes with time should be documented and evaluated by an engineer.

Scour - Scour is underwater erosion caused by the rapid flow and turbulence of water from a spillway or high velocity seepage. Because scour generally occurs underwater, it cannot always be directly observed. Scour can sometimes be indicated by an island forming downstream. Probing can be used to detect it. Scour beneath a concrete slab may cause the slab to crack or collapse. A special effort, even hiring a specialist in underwater inspection, is often warranted to identify this condition.

Vegetative Cover - The earth embankment dam should have a suitable cover of grasses. Woody vegetation such as brush, trees and shrubs is undesirable since it inhibits the inspection of the dam and can cause other problems. For example, the root systems of large trees help water get into the dam or large trees can be uprooted during a storm and cause gaps in the embankment. Also, brush is good habitat for burrowing animals. It is very important that grass be cut and brush and trees not be allowed to grow on embankments. If you have trees on your embankment, see your state dam safety agency for advice on removal.

Animal Burrows in the earth embankment are a source of potential problems. Animal burrows result in a loss of earth material, and can provide seepage paths. Animals should be removed and holes backfilled with soil. Their habitat should be destroyed.

Surface Erosion - When vegetative cover is removed due to water action or traffic patterns (people, vehicles or animals), erosion can occur. Deep ruts or gullies could form and threaten the safety of the dam. Frequently used paths should be stabilized with stone aggregate or pavement. Eroded areas should be filled and seeded.

Slumps (slide or slough) - A slow or sudden movement of an earth embankment slope is an indication of instability and requires immediate response. Often cracks will also form which could lead to further slumping. The slumps can occur on the upstream or downstream side. Contact the state dam safety agency or engineer for advice.

Abutment Areas

Abutment areas are the contact points between the ends of the earth embankment or spillway and the natural ground, or the contact points between embankments and spillways. These areas should be closely and frequently inspected for any signs of seepage, cracks or erosion. Though seepage is common in this area, it can cause serious problems.

Owner's Checklist

To help you start your dam inspection, a simple checklist is included as Page 5-15. Your state agency has more detailed inspection/checklist forms.

IV. An Engineering Inspection

The owner may be required by state statutes to have a formal inspection conducted by an engineer. An engineering inspection should include the following procedures: a review of past inspections, dam history, construction plans and specifications, analysis of all data and information gathered including that from an on-site inspection.

Review Process

The review will only be as good as the data that is available to the engineering consultant. If the owner has conducted his inspections, that information should be provided to the consultant. The first inspection by the engineer should include a history of the dam with photographs if possible. Quite often design plans and specifications are not available for older dams. It is also important for the engineer to know about any dam modifications or repairs. A review of the state dam safety agency's files on the dam is a good source for dam history and records of modifications.

When conducting the inspection, the engineer should obtain all available monitoring data. This includes impoundment levels, discharge rates, gate openings, occurrences of floods

OWNER'S INSPECTION CHECKLIST

Dam Name: ______________________________

Date of Inspection: ______________________________

Your Name: ______________________________

	NO	YES	IF YES
Surface cracks?	☐	☐	Contact state agency or engineer.
Slumping or cracking on the upstream or downstream side?	☐	☐	Contact state agency or engineer.
Erosion from runoff, wave action or traffic?	☐	☐	Repair and stabilize.
Embankment/spillway seepage?	☐	☐	Contact state agency or engineer.
Water muddy?	☐	☐	
Top of the dam settled?	☐	☐	Contact state agency or engineer.
Loss of riprap?	☐	☐	Contact state agency or engineer.
Trees, brush or burrows on embankment?	☐	☐	Clear trees, brush and seed.
Spillways blocked?	☐	☐	Clear spillway immediately.
Exposed metal rusty?	☐	☐	Clean and paint.
Concrete deterioration or cracks?	☐	☐	Contact state agency or engineer.
Cracks or uneven movement?	☐	☐	Contact state agency or engineer.
Scour?	☐	☐	Contact state agency or engineer.
Pipe joint separation?	☐	☐	Contact state agency or engineer.
Gates operational?	☐	☐	Repair and make operational.
Trash racks blocked?	☐	☐	Clean out debris.

and excessive precipitation, observation well (piezometer) readings and settlement/alignment data. This information will indicate how the dam has functioned.

A book or diary of activity occurring at the dam is very useful. Many dam owners make entries on a daily basis. If not established, one should be started. This log book should contain visual observations, dates of inspections, visitors to the dam site, gate openings and settings, water levels, maintenance performed and any unusual occurrences. Keeping a log book could also prove to be invaluable in the event of litigation proceedings.

Past inspection reports are also important. These reports assist the engineer in evaluating the structure.

Provided with the information above, the engineer will become familiar with the dam and develop a good base for a sound analysis of the dam structure.

Inspection Procedure

The owner should review or have the state dam safety engineers review the proposed inspection procedure before authorizing the engineer to proceed. The initial inspection should include an underwater inspection. The engineer should be recording each inspection detail on a checklist. The checklist usually is included in the report as an appendix. It should be emphasized that **the checklists are not inspection reports.** Inspection reports must always include professional judgements and opinions and be thoroughly documented (names, dates, photographs, measurements, etc.).

Analysis

The owner will not be directly involved in the detailed analysis conducted by his engineer. The owner should insist that the engineer has identified problem areas and deficiencies; recommended remedial actions for problem areas; established a priority list for dealing with the problems; included a cost estimate for the repair items or other inspections; and contacted the state dam safety agency to arrange for a third party review of the dam safety inspection report.

V. Repair Priorities

Timely repairs are a must after problem areas have been identified. Conducting the two level inspection program will provide evidence that the dam owner has properly cared for the dam only as far as the recommendations for repair are followed. Scheduling the repair of problem areas will guide a good rehabilitation and maintenance program.

The owner should be aware of the various items that will be provided in an engineering inspection report. They are emergency indicators, warning indicators and clues to potential problems.

Emergency Indicators - During a visual inspection of the dam, emergency indicators would alert the engineer to a severe problem at the dam that could result in a dam failure. The engineering consultant should notify the owner and the appropriate state agency of the observed deficiency in the dam. Accompanying such notification, an immediate course of action should be provided to the owner. This information will be verbal and may be given during the on-site visit. Written confirmation should follow. At this point, an emergency indicator would activate the emergency action plan which is discussed in Chapter 9 of this guidebook.

Warning Indicators - To a lesser degree, warning indicators represent items of concern, requiring close watching and monitoring. At this point, the engineer should notify the owner and the state dam safety agencies about the level and type of response that would be needed.

A warning indicator may result in on-site spoken notification to the owner, followed by a written letter prior to final drafting of the inspection report.

Clues to Potential Problems - Clues to potential problems are items that need further detailed technical analysis. In the final report, the engineer will notify the owner of these items. Corrective actions may require further detailed study and analysis, or remedial repairs and programmed maintenance of the items.

VI. Where To Get Help

Manuals or literature available from the state dam safety agency will help you develop an owner's inspection program. State personnel are also available to review and discuss an inspection program with you. See Chapter 3 for more details.

The state dam safety agency can provide you with a list of qualified engineering consultants to assist you in conducting an engineer's inspection. Further, professional engineering societies can provide you with guidelines for selecting an engineering consultant. In general, the owner should check the qualifications of an engineer based on:

Experience - Request references on previous dam inspections and construction projects.

Expertise - Engineering consultants should have expertise in hydrology, hydraulics, structural design and analysis, and geotechnical engineering.

VII. Summary

A good inspection program is an integral part of the operation and maintenance of the dam. Dam inspection identifies deficiencies and the maintenance required to ensure the integrity of the structure and to protect life, health and property. The dam owner, working with a qualified engineering consultant and in cooperation with the state dam safety agency, can develop a sound inspection program.

VIII. References

For more information about dam inspection, see these publications.

1. "Safety of Existing Dams, Evaluation and Improvement," by Committee on Safety of Existing Dams, National Academy Press, Washington, D.C., 1983.

2. "Guidelines and Forms for Inspection of Illinois Dams," State of Illinois Department of Transportation, Division of Water Resources, February 1984.

3. "Operation and Maintenance Inspection for Dams," Field Guide, United States Department of Agricultural Forest Service, January 1981.

4. Operation, Maintenance and Inspection Manual for Dams, Dikes, and Levies, by George E. Mills, Ohio Department of Natural Resources, Division of Water and Dam Inspection Section.

5. "Dam Safety Manual," State Engineer's Office, State of Colorado, June 1983.

CHAPTER 6

MAINTENANCE

I. Why Maintain?

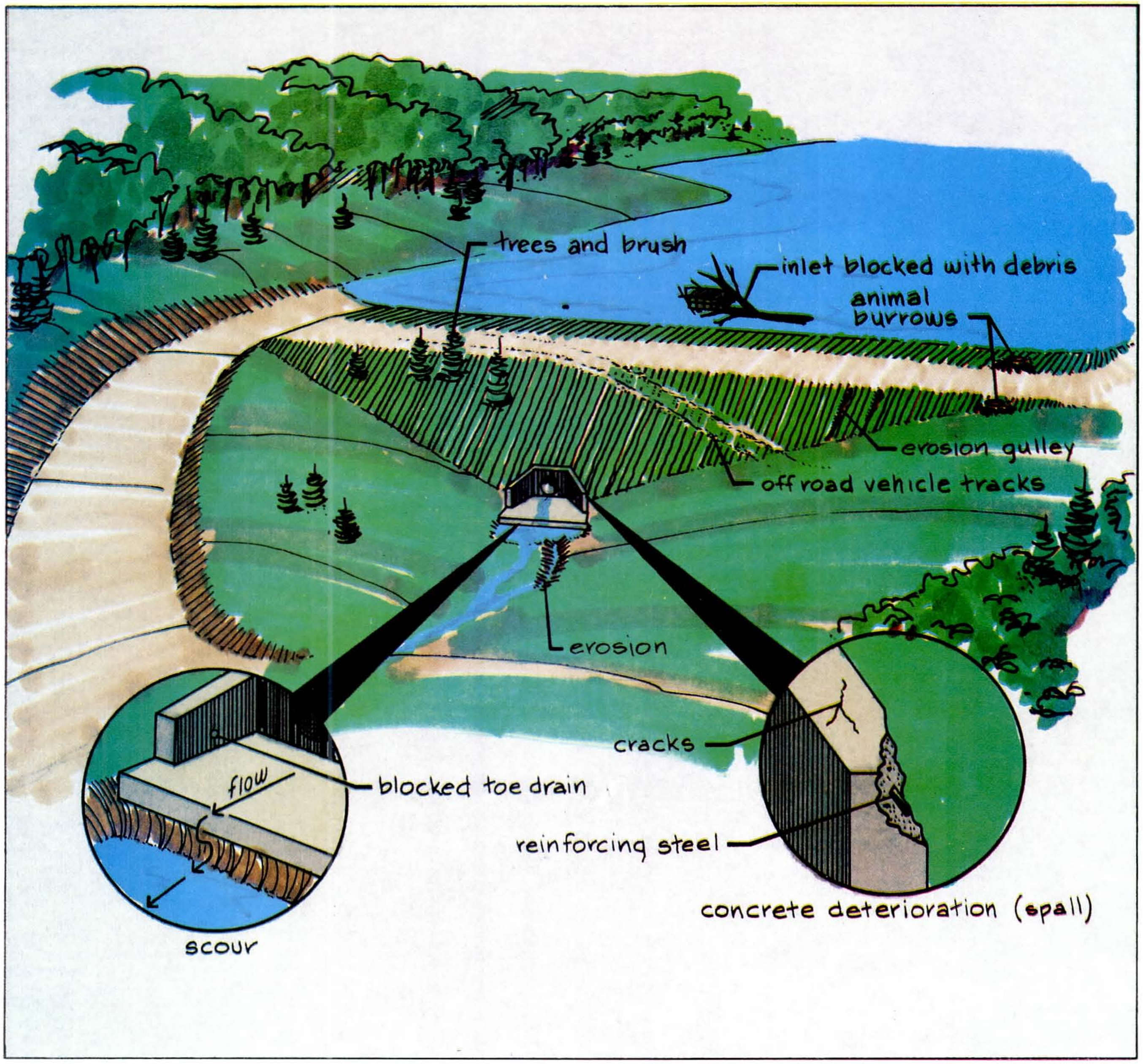
A good maintenance program will prolong the life of the dam and protect against deterioration.

A poorly maintained dam will deteriorate. Nearly all dam components and materials are susceptible. A good maintenance program provides not only protection for the owner, but for the public in general. The cost of a proper maintenance program is small compared to the cost of major repairs (caused by lack of maintenance) or litigation. Lack of proper maintenance can be used against the dam owner in civil and/or criminal negligence suits resulting from damages caused by a failure of the dam.

II. Common Maintenance Items

The dam owner can develop a basic maintenance program from his systematic and frequent inspections. An inspection should be performed monthly and after major flood events. During each of the inspections, a checklist of items to be maintained and items to be observed should be recorded. See Chapter 5, Page 5-15 (Owner's Inspection Checklist).

Minor and routine maintenance items, such as cutting grass, painting and equipment repair, should be regular and may be handled by the owner. The maintenance items illustrated on Page 6-2 should have special attention.



common maintenance items

Spillways

Spillway entrances and/or intakes must be clear of all debris to allow uninterupted flow. Debris often accumulates after storm events. A plugged spillway can lead to a rising pool elevation which may overtop and breach the dam. Regular and timely removal of trash and debris is recommended. Trash rack maintenance or replacement is a critical item. Undersized or improperly spaced trash racks can significantly reduce spillway capacity and endanger the dam. Your dam safety agency should be contacted. Emergency spillways (earth overflow type) should be mowed and checked for erosion after each use. Metal outlet pipes can also deteriorate by rusting and corrosion. Leaking joints are also a problem that will need attention.

Discharge Areas

High flow can cause underwater erosion (scour) which may affect the stability of the dam, spillway slabs or walls. Some spillways have a plunge pool or a stilling basin to dissipate energy. Frequent surveillance of the discharge area is required to identify excessive erosion. It may be advisable to inspect or to probe underwater. If erosion is minor, the owner can probably correct these items himself. However, if severe, professional assistance will probably be required.

Drainage

Most concrete spillway structures and earthen embankments require the control of seepage and uplift pressures with drains. With the concrete spillway, this is accomplished with small drain holes (weep holes) in the concrete wall to permit water to drain from behind the wall. These holes should be periodically inspected and cleaned to prevent plugging. Seepage through earth embankments can be controlled with a toe drain. Periodic inspections and cleaning of these drains will ensure they are free flowing.

Slope Protection

Trees and brush should not be allowed to grow on embankments or in emergency spillways. Periodic cutting/controlling of the embankment vegetation and adjacent areas is recommended. It will also prevent the growth of trees and brush.

Upstream slope protection is needed to protect against erosion. This can be provided by coarse riprap with a properly designed soil or fabric filter underneath. Failed riprap should be repaired. If the failures are frequent, professional help should be obtained.

Concrete

Concrete spalling and cracking is a common problem. Spalling concrete (chipped, flaked or "pock-marked") or cracks can lead to further deterioration of the structure over a period of time. Spalled concrete can expose steel reinforcement and further deteriorate the structural integrity of the dam. When it is observed, it should be repaired and patched to protect the structural integrity of concrete parts of the dam. Concrete joints and concrete pipe joints should be periodically checked and kept water tight.

Appurtenant Works

Dams may have some type of mechanical, electrical and/or hydraulic equipment used to activate gates, valves, hoists, air systems and other moving parts. These mechanical items are essential to the safety of the dam because they provide the means to pass flood flows and to draw down the impoundment in case of emergencies. Instrumentation which is used to monitor dam performance also requires maintenance.

Gates and valves are part of the overall water retaining structure. They must be structurally sound and functional. These parts will need lubrication and painting. The dam owner should be familiar with gates and valves and their operation.

Hoists which move gates, slide gates, stoplogs, etc. are either mechanically or hydraulically operated. Mechanical hoists can be hand operated or powered by electrical motors or gas engines. Hydraulic systems require a compressor which can either be operated by an electric motor or gas engine or an accumulator tank. Some hydraulic devices can also be manually operated. It is recommended that all components be test operated at least annually. Backup electric generators may be advisable in some situations.

Sometimes air systems (bubblers) are provided near gates in climates where ice can form on the impoundment. In the northern states, it is common to find thick ice in the impoundment that can exert extreme pressure against the dam and its operating gates. The air system may not keep the gates from freezing, but will help reduce the ice buildup around the gate structure. The air system and compressor should be tested to make sure they are functional.

III. Summary

This chapter lists common maintenance items that should be addressed by the dam owner. Failure to maintain a dam in a proper operating condition can lead to potential liability and could cause failure of the dam. Systematic and proper maintenance will help to extend the life of the dam and may prevent a failure.

IV. References

1. "A Recommended Maintenance Program for Owners of Dams" by Charles L. Hahn, MS, P.E., Division of Water, Ohio Department of Natural Resources.

2. "Maintenance of Equipment" by Bruce O. Frudden, Mead & Hunt, Inc., Madison, Wisconsin.

3. "Dam Safety Manual," State Engineer's Office, State of Colorado, June 1983.

4. "Operation, Maintenance and Inspection Manual for Dams, Dikes and Levees, Ohio Department of Natural Resources, Division of Water, Dam Inspection Section, November 1983.

CHAPTER 7

REPAIR, ALTERATION AND REMOVAL

An engineering inspection report, or even the owner's inspection, may identify the need for repair or modifications of the dam. Repair is performed to essentially restore a dam to its approved design condition. The dam owner may need to seek engineering assistance to develop design plans and specifications for the repair, alteration or removal of the dam. The dam safety agency should be consulted. Such repairs or removal of the dam may require permits. Permit requirements are described in Chapter 3. Please read that chapter carefully. This chapter will identify some of the issues the dam owner must address.

I. Seek Help from Engineers and Contractors

If an engineering inspection reveals the need for repairs and/or further investigations, then an engineer should be retained to prepare plans and specifications and oversee the repair work. It is very important that the dam owner select a qualified engineer in the area of dam design, construction, operation and inspection.

Just as important as selecting a qualified engineer, the owner must select a qualified contractor. The construction or repair work activity around a water retaining structure such as a dam requires specialized skills, knowledge and experience. It is important that the dam owner and/or engineer seek references from the contractor to prequalify them for the work. The competitive bidding process is not always the best procedure. Although it may appear to save the dam owner some cost, long-term effects from an improper repair job or cost overruns due to contractor inexperience can also occur.

II. Common Deficiencies and Remedies

Identifying dam deficiencies and remedies will probably require the assistance of an engineer who is able to assess the relative severity of a problem, which most laymen cannot do. Some maintenance items, if not promptly attended to, can deteriorate to the point where major repair is required.

Common items that could require repair include slumping and settlement, loss of riprap, seepage, piping, scour, embankment erosion, and severe concrete and metal deterioration.

A common hydraulic deficiency is an undersized spillway (that is, inadequate capacity), through which a major flood cannot be passed without overtopping the dam. This deficiency is serious and can be expensive to correct. The state dam safety agency can help the owner evaluate the spillway capacity.

III. Dam Removal

Dam removal or breaching so as not to damage downstream areas is the choice an owner must sometimes make after considering safety conditions and costs of repair and removal. It is recommended that an engineer be engaged not only to prepare plans and specifications, but also to supervise construction (actually "destruction") and removal phases. The engineer should provide documentation that all environmental protection measures are taken, and assure that upstream and downstream users are protected during removal of the dam.

Removal does not necessarily mean total removal of the earth embankment. Removal may simply involve taking away the stoplogs and lowering the impoundment. The state dam safety agency should be contacted to determine the extent of removal.

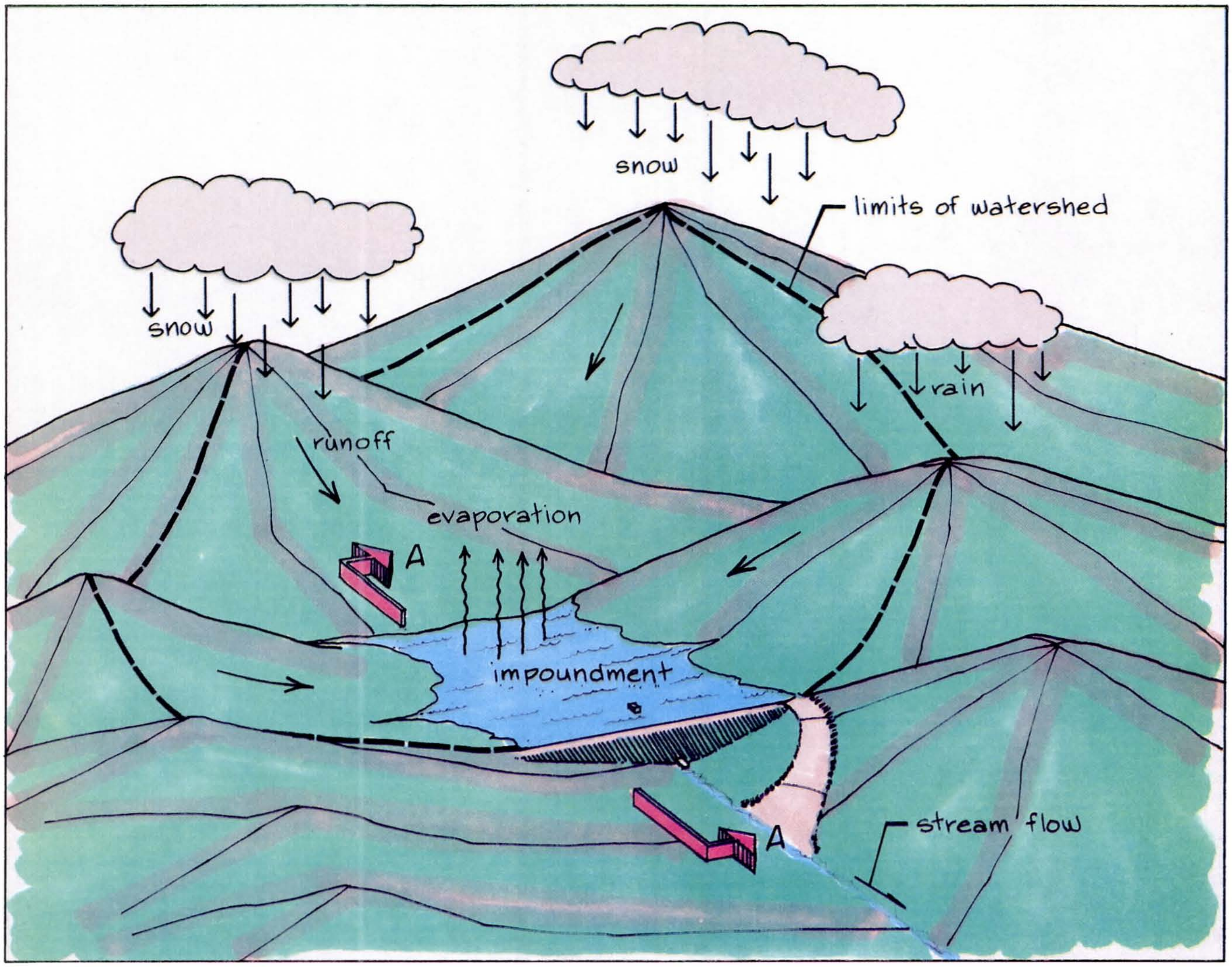
CHAPTER 8

OPERATION

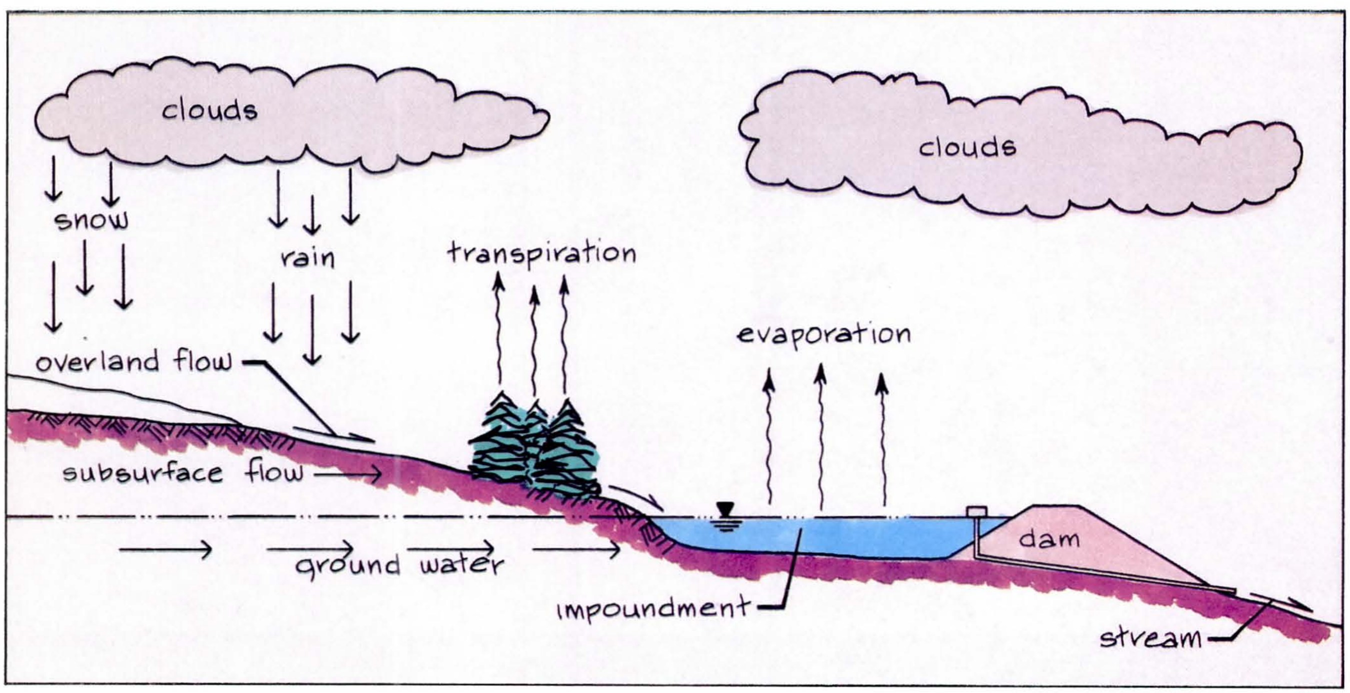
The purpose of a dam is to create an impoundment which may be used for flood control, irrigation, water supply, recreation, hydroelectric power production, land development, or fish and wildlife habitat. Proper operation will ensure that the purposes are achieved. This chapter will discuss the daily operation of the dam and what the dam owner should understand to safely operate his dam.

I. Know Your Watershed

A watershed is the area of land drained by a river or river system. Within a watershed, precipitation will infiltrate into the ground, move as surface runoff, evaporate and transpire. Surface water runoff is part of the Hydrologic Cycle shown in Page 8-2. Although a detailed study of hydrology and the hydrologic cycle is not the intent of this section, the dam owner should recognize that river flow is dependent on precipitation (the quantity of rainfall or snow in the watershed area) as well as soil types, land use, slope and other factors. Large, rapid river flows into an impoundment can result in rapid changes in impoundment levels. High river flows are usually the result of heavy precipitation within a watershed. However, other combinations of climatic and ground conditions can cause large runoff. An example would be a



hydrologic cycle



section A-A

moderate rain on frozen ground, combined with snow melt. In this case, there would be little or no infiltration and nearly all the rainfall would move as surface runoff. Many other situations could occur.

In any event, the dam owner must be prepared to respond correctly to high flows by adjusting the dam gates.

There are two approaches to understanding the watershed. One would be the scientific approach and the other would be the "rule of thumb" approach. The scientific approach is beyond the scope of this guidebook, but guidance is available from your state dam safety agency.

The "rule of thumb" approach is developed from experience and observation. By compiling a history of precipitation, ground conditions and corresponding river flow, the dam owner can become aware of the length of time between rainfall events and flow increases at the dam.

Specific information is available to develop the history of a particular river and watershed. For many river basins, the United States Geological Survey (USGS), a branch of the U.S. Department of Interior, maintains a stream gaging station that provides information on river flow. Dam owners should contact the state dam safety agency for locations of USGS gaging stations. Stream flow data from gaging stations is published yearly. Precipitation data can be obtained from the National Weather Service. Based on a review of this information and consultation with the dam safety agency, the dam owner can learn more about the response of the river to a particular rainfall event.

For smaller river basins, the USGS can also provide information or develop a correlation study to the particular dam site. Dam safety agencies within the state government can also provide you with guidance or sources of information concerning flood events in and around your particular watershed. An inquiry to the state agency involved with flood insurance studies developed for your particular area can also provide useful information.

Other sources of information are flood forecast stations operated by the Weather Service, as well as television and radio weather stations.

II. Plan Your Operation

Your response to rainfall events will be determined by the operation plan you develop with the watershed information. To pass flood flows, critical equipment must be operable and well maintained.

It is also advisable to have backup equipment to operate gates and pass flood flows in case equipment breaks or there is power failure.

In developing an operation plan, the owner should also be aware that log jams, debris and ice buildup can block a spillway and prevent passing the flood flow.

The operation plan should be in writing. This will provide you, the dam owner, with a logical set of instructions for yourself, your operating personnel, or future owners of the dam.

The operation plan should also provide for limiting access. Fencing, gates and locks should be provided on the structure so unauthorized people do not damage or misoperate the dam. Particular areas to fence are the spillways, mechanical equipment and the outlet tubes. Signs that detail the specific dangers must be posted. For example, if electrical transmission lines from a power generating plant are essential in the dam's operation, signs should state "High Voltage." Another example is a spillway sign stating "Danger of Swift Currents, Undertow and Thin Ice." Page 4-4 shows some of the recommended features of a limited access dam. The operation plan should include maintenance of these signs.

III. Operation Plan Details

Detailed operation plans are a good idea and may be required by the state. This plan should be passed on to new owners. If there is an operating plan already, it can be kept with this guidebook.

A sample operation plan which you can use to get started is on Pages 8-6 and 8-7. The operation plan addresses the questions: Who? What? When? How? and Where? For example, to detail the cleaning of trash racks, the owner should know: **who**

SAMPLE DAM OPERATION PLAN

Dam Name: ______________________________

Date: ______________________________

Owner Name: ______________________________

WHO

1. Who operates the dam? (Owner or other agent/employee.)

   Address: ______________________________

   Telephone number: (   ) ______________________________

2. Who is the backup operator?

   Address: ______________________________

   Telephone number: (   ) ______________________________

3. Who maintains the dam:

   Address: ______________________________

   Telephone number: (   ) ______________________________

4. Who must be called in an emergency?

   Address: ______________________________

   Telephone number: (   ) ______________________________

WHAT

1. What downstream structures would be affected by a flood?

2. What minimum flow, if any, is required for downstream users?

3. What impoundment levels are required to protect upstream users?

   Maximum Elevation ______________
   Normal Elevation ______________
   Minimum Elevation ______________

SAMPLE DAM OPERATION PLAN (Continued)

WHEN

1. When are gates operated during storm events?

2. When are gates operated during normal conditions?

WHERE

1. Where is emergency power?

2. Where is engineering assistance?

HOW

1. How are gates operated?

2. How often is mechanical equipment operated?

is responsible; **what** problems can develop; **what** equipment is used; **when** or **how** the trash racks should be cleaned; and **where** they are located.

Your operation plan should specify how and when to release water in normal and flood times, what equipment is needed and who is responsible. The operation plan must take into account that a minimum flow release may be required for downstream users, as well as fishery and wildlife habitat protection. These minimum flows should be determined in cooperation with downstream water users and the state dam safety agency.

In addition to the items on Pages 8-6 and 8-7, the operation plan should detail the impoundment levels required to protect upstream users. This may require that minimum, normal and maximum impoundment levels be established. If the impoundment is raised, flowage rights or easements must be checked. The operation plan should also contain the Emergency Action Plan which will be discussed in further detail in Chapter 9.

Any drawdown of the impoundment will require detailed calculations of the amount of flow that can be released from the structure without causing downstream flooding. Investigating silt load in the impoundment and analyzing its potential for downstream damage should be addressed. Provisions for a drawdown should be in the operation plan, but it must be carefully planned by consulting with your state dam safety agency or other qualified professionals. Also, the rate of the drawdown (or speed at which the pond level is dropped) should be controlled so as not to cause structural damage to the dam or any damage to properties upstream or downstream from the dam.

CHAPTER 9

EMERGENCIES

I. Can an Emergency be Anticipated

Dams should be designed with sufficient safety factors so as not to fail. However, conditions beyond the control of the dam owner and engineer can occur due to natural forces, mistakes in operation, negligence or vandalism. The purpose of this chapter is to identify typical dam failure scenarios. We will explain the effect of dam failures on upstream and downstream users and help the reader formulate an emergency action plan. The emergency action plan will include procedures for warning local units of governments. Local evacuation plans should be coordinated and developed with the local government agency.

Since the existance of a dam can pose a threat to public health, safety and welfare, the dam owner is responsible for keeping these threats to a minimum. A carefully conceived and implemented emergency action plan is one positive step the dam owner can take.

II. Types of Failures

In preparing emergency plans, two types of failures are usually considered. They are termed **rainy day** and **sunny day** failures. A rainy day failure could occur when heavy precipitation, in excess of that normally observed in the watershed above the dam, leads to a high runoff period. If the high water was to overtop the dam or add too much pressure, a rapid failure could result. The dam owner should be an astute weather watcher and be responsive to precipitation events.

A normal storm event could lead to overtopping the dam if the outlet works are plugged with debris, if the gates jammed or were broken, or if a power failure prevented operation of key mechanisms. All the items can be controlled by proper operation and maintenance of the dam.

Dams have also failed without any heavy precipitation. These failures are called **sunny day** failures. They are usually the result of neglected inspection programs and poor maintenance and operation of the dam. As an example, failure to consider embankment seepage could lead to piping (internal erosion). A sunny day failure could be caused by vandalism of the outlet works, such as damage to gate mechanisms, or if the outlet works are plugged with debris. Sunny day failures are more likely at unattended dams than frequently visited dams.

Both rainy and sunny day failures can occur at new dams. New dams are very susceptible during initial filling and for a few years after filling. In fact, many dams have failed their first filling. Emergency action plans are a good idea for all dams. Some states even require one prior to construction.

The downstream effect of a dam failure can be devastating. When a break in a dam (breach) develops, water discharge increases due to the uncontrolled release of water stored by the dam. Destruction of homes and property has been well publicized. The force of water through existing bridges and culverts and over roads can cause their collapse. It has been documented that the flood wave from a small dam can overtop roads and wash cars from the roadway. Overtopping of the roads also makes them impassable for emergency vehicles. Dam failures can kill people.

Damage to the environment and to upstream users from a dam failure can also be catastrophic. A breach in the dam and rapid loss of the impounded water can cause heavy silt loads to be passed downstream. These sediments, after a period of time, will settle out, clogging and covering the flooded land and stream bed. Fish and wildlife habitat can also be damaged. Upstream slopes can fail and boaters could be washed downstream.

The Federal Emergency Management Agency (FEMA) has prepared guidelines for the preparation of emergency action plans for dam owners. The following sections of this guidebook discuss, in general, the contents of an emergency action plan. Each should be modified for a specific dam site. Dam owners may need to retain an engineer to help prepare the emergency action plan.

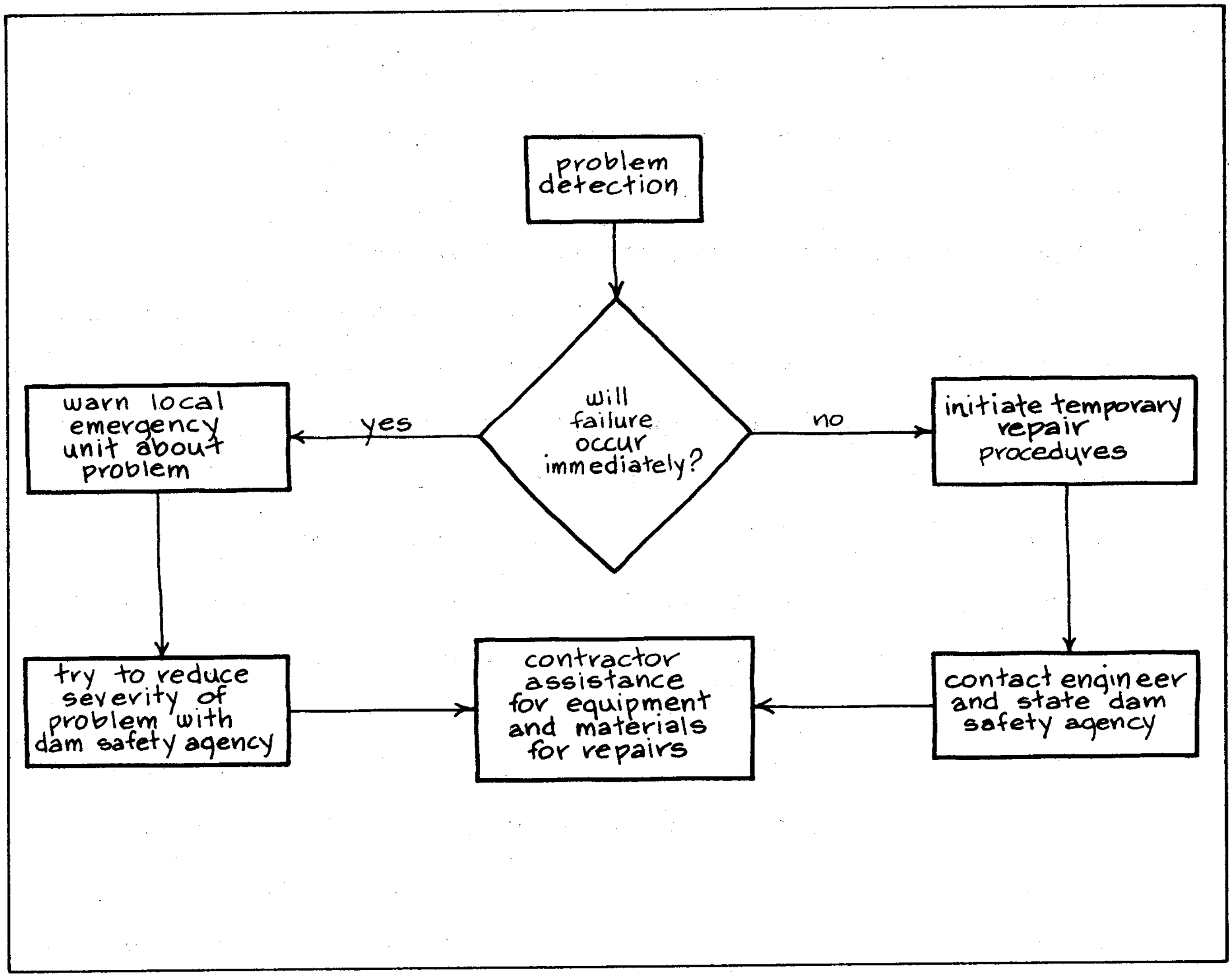
III. Emergency Action Plans

Emergency action plans for dam failures usually consist of three sections: preplanning, on-site assessment and initiating warning and evacuation plans. Each part of the plan calls for action by the dam owner.

Preplanning for an emergency may require detailed analysis by an engineer. (Normally, the state dam safety group would not supply this service). The engineer will analyze and determine flooding that would happen after a dam failure. He or she will prepare flooding maps to be used for the evacuation portion of the emergency action plan. The analysis will also predict and map flood wave height and speed. All affected downstream land owners and buildings will be identified, so you can develop a list of who to contact with flood warnings and evacuation notices.

It is also important to provide early warning of possible dam failure. This may include installation of sound alarms at unmanned dams. The consulting engineer or your state dam safety group can advise you as to whether or not alarms are needed.

During the time preceding a possible dam failure, the dam owner is responsible for implementation of the emergency action plan. Page 9-5 is a simplified "flow chart" to help you make decisions and respond correctly to a failure of your dam. Two



dam owners decision and notification procedures types of failures are normally considered, the instantaneous or rapidly developing failure (the failure **will occur immediately**), and the slowly developing failure.

For a slowly developing failure, there may be time to take remedial or corrective actions to reduce the impact of the failure. For example, a controlled drawdown of the impoundment could be done. For the rapidly developing failure, immediate contact with local emergency authorities is essential.

The dam owner is responsible to provide early warning of the problems at the dam to the local emergency unit. The owner is responsible to convey the message; however, it is the ultimate responsibility of the local and state agencies to make the decision to initiate evacuation plans and re-entry plans.

It is a good idea to have a telephone list like that shown on Page 9-7. Keep this list up to date and readily accessible. Many dam owners post it by their phones. In an emergency, it will save time searching for a number. Also, if you or your normal operator are absent, the substitute can rapidly respond to the emergency.

IV. Summary

The development of an adequate emergency action plan requires coordination between the dam owner and the local and state agencies. It is important that each individual participating in the warning procedures and evacuation plan be provided with a copy of the plan. The plan should be updated annually and reviewed by all participants involved.

DAM OWNER/OPERATOR TELEPHONE LIST

1. Local Police/Sheriff Department

( ) ______________

2. State Police/Patrol

( ) ______________

3. Downstream and Upstream Dams and Operators

- Dam ______________________________

  Telephone ( ) ______________

- Dam ______________________________

  Telephone ( ) ______________

- Dam ______________________________

  Telephone ( ) ______________

4. Downstream Residence/Business

( ) ______________

5. Hospital/Ambulance

( ) ______________

6. Contractor

   Name ______________________________

   Telephone ( ) ______________

7. State Dam Safety Agency

   Name ______________________________

   Telephone ( ) ______________

8. Engineer

   Name ______________________________

   Telephone ( ) ______________

Post this list in a prominent place at the dam and give a copy to all of your operators.

Developing and preparing an emergency action plan gives the dam owner the ability to make correct responses in times of emergency. As we have stated throughout this guidebook, you are responsible for the operation, maintenance and activities at your dam. This remains true - **even in an emergency.**

V. References

1. "Emergency Action Planning Guidelines for Dams," by Subcommittee on Emergency Action Planning of Interagency Committee on Dam Safety, Federal Emergency Management Agency, February 1985.

CHAPTER 10

FINANCIAL ASSISTANCE

This guidebook points out that the dam owner is responsible for the repair, security, maintenance, and operation of the dam. It is common that the dam owner will need professional assistance in fulfilling these responsibilities. Repairs and maintenance of a dam can also be costly. So the dam owner is often faced with the dilemma of the lack of finances along with the responsibility of maintaining a safe dam. This dilemma is not easily resolved. Conscientious dam owners always prepare for the time when costly repairs will be needed.

There are many possible sources of financial assistance including federal, state, local government and private sources. However, chances of obtaining federal, state and local governmental financial assistance are remote. Financial assistance is further complicated by the fact that many programs depend on the dam location, type of ownership, jobs created and the like. The dam owner should not be discouraged by the many conditions to be met to obtain such financial assistance. You should instead **become innovative** in applying for assistance.

This chapter will present some basic information on sources of financing for dam and dam safety projects from federal and state sources. Funding through private sources and hydroelectric projects will also be described.

I. Public Sector Funding

The federal government has played an important role in providing funds for dam owners. Obtaining federal funding often requires meeting complex and varied requirements, and doing a myriad of paperwork. The successful use of these programs requires the dam owner to present the project with innovation and sophistication. In addition, most programs require local matching funds, while private projects must utilize funds from private sources.

The U.S. Department of Agriculture, Farmers Home Administration (FMHA) has several different programs to assist dam owners with renovation projects. The agency functions as a lender of last resort, only when private sector funding is not available at reasonable terms.

The Environmental Protection Agency, through its office of Research and Development, offers a series of grants related to energy and water quality. Other programs are available through the Soil Conservation Service and the U.S. Forest Service.

The U.S. Department of Commerce, Economic Development Administration provides business development assistance loans under three different programs to businesses and industries. Hydroelectric power projects may qualify if they are generating power for new or expanded business and industrial operations.

The U.S. Department of Interior, Bureau of Reclamation serves the 17 westernmost contiguous states and offers several financing programs to public and privately owned dams in that region. The Bureau's Regional and Washington offices may be contacted for additional information on the following programs:

Irrigation Distribution System Loans

Irrigation Systems Rehabilitation and Betterment Loans

Small Reclamation Projects

State government funds may be requested for dam repair projects. Since these programs are constantly changing, any definitive statement regarding the availability of funding from the sources is not possible. However, as a general guideline, Page 10-4 is included to indicate applicability of various funding programs for a given state. Specific information regarding such programs should be obtained from the administering agency, or from the state dam safety agency.

II. Private Funding Sources

Dam owners are most often faced with funding dam repair projects without government assistance. Owner generated funding could come from conventional sources such as commercial banks, savings and loan organizations, insurance companies, or private foundations.

In order to tap such conventional funding sources, the borrower must "sell the project." This will undoubtedly require the owner to retain specialists in the field of dam repair or modification to assure that proper engineering and construction practices are followed.

FUNDING SOURCES BY STATE

	Illinois	Indiana	Michigan	Minnesota	Nebraska	Wisconsin
Community Development Block Grants	XX	XX	XX	XX	XX	XX
Bureau of Reclamation					XX	
Great Plains Conservation Program					XX	
State Certified Development Co.			XX	XX	XX	XX
Rural Development Loan Fund Intermed			XX	XX		XX
Small Business Revitalization	XX			XX	XX	
State Equity/Venture Capital Corporation		XX	XX			XX
Industrial Revenue Bonds-Local Level	XX	XX	XX	XX	XX	XX
Industrial Revenue Bonds-State Level	XX		XX		XX	
Umbrella Bonds			XX	XX	XX	
Loan Guarantees		XX		XX		
Direct Loans	XX			XX		
Development Credit Corp.			XX		XX	
Job Training	XX	XX	XX		XX	
Property Tax Abatements	XX	XX	XX	XX		
Investment Tax Credit	XX	XX	XX			
Job Creation Tax Credit		XX				
Energy Tax Credit	XX	XX	XX	XX	XX	XX
Industrial Fuels	XX	XX	XX	XX	XX	XX
Machinery and Equipment	XX	XX	XX	XX	XX	XX
Pollution Abatement	XX	XX	XX	XX	XX	XX
Water Supply Loans and Grants		XX			XX	
Water Recreation Loans and Grants			XX		XX	
Flood Control Loans and Grants		XX			XX	

Source: "Financing Dam Safety Projects"

A dam owner whose dam benefits others may attempt to spread the costs of dam maintenance and repair by forming a lake management district, a non-profit organization or unit of government. In this way, dues from the membership could be used to finance dam repair projects. Refer to Chapter 3 for specific state statutes that provide for this. It is advisable for the dam owner to establish a "sinking fund" in preparation for the day when repairs or modifications to his dam will be necessary.

III. Hydroelectric Possibilities

The rising cost of fossil fuels, together with the high cost of nuclear generated electricity, has renewed interest in the use of water to generate electricity. Currently, there are many dams which are retired hydroelectric sites or dams which could be retrofitted to hydroelectric generation. A dam owner would be well advised to research the possibilities of retrofitting the site for the purpose of generating electricity. The biggest benefit from retrofitting is that the dam may become income producing and hence self-supporting. The revenue can vary greatly depending on the flow characteristics of the river, environmental constraints, size of dam and other variables.

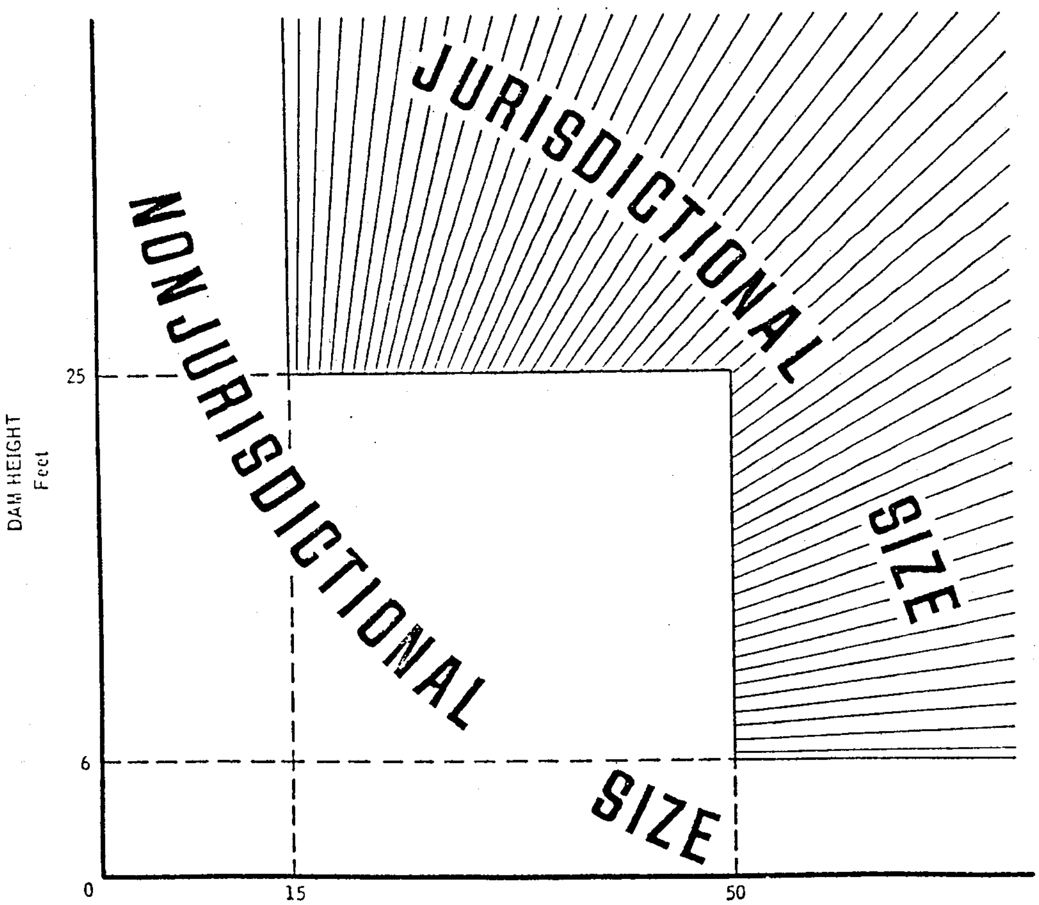
Hydroelectric sites are often expensive to develop. Besides the conventional sources previously listed, there are hydroelectric developers who specialize in financing and developing these projects.

There are certain Internal Revenue code benefits that apply to small hydroelectric projects. Also, investment tax credits may be available. However, as is the case with Federal Legislation, changes in the law can occur, and thus specialists knowledgeable in these areas should be consulted regarding specific projects.

IV. References

"Financing Dam Safety Projects," Federal Emergency Management Agency, Washington, D.C. 20472.

STATE OF ARIZONA
DEPARTMENT OF WATER RESOURCES
ENGINEERING DIVISION/SAFETY OF DAMS SECTION



Acre-Feet
RESERVOIR CAPACITY

A JURISDICTIONAL DAM is either twenty-five feet or more in height or stores more than fifty acre-feet. If it is less than six feet in height regardless of storage capacity or does not store more than fifteen acre-feet regardless of height, it is not in jurisdiction.

THE HEIGHT is the vertical distance from the lowest elevation of the outside limit of the dam at its intersection with the natural ground surface to the spillway crest elevation.

THE CAPACITY is the maximum storage, in acre-feet which can be impounded by the dam when there is no discharge of water.

STATE OF ARIZONA

DEPARTMENT OF WATER RESOURCES

ENGINEERING DIVISION
SAFETY OF DAMS SECTION

ARIZONA REVISED STATUTES
TITLE 45-WATERS, CHAPTER 3, ARTICLE I

SUPERVISION OF DAMS, RESERVOIRS AND PROJECTS

SEPTEMBER 1983

ARIZONA REVISED STATUTES - TITLE 45, WATERS, CHAPTER 3 - DAMS AND RESERVOIRS, ARTICLE I SUPERVISION OF DAMS, RESERVOIRS AND PROJECTS

ARTICLE I - SUPERVISION OF DAMS, RESERVOIRS AND PROJECTS

§ 45-701. Definitions

In this article, unless the context otherwise requires:

1. "Dam" means any artificial barrier, including appurtenant works for the impounding or diversion of water except those barriers for the purpose of controlling liquid borne material, twenty-five feet or more in height or the storage capacity of which will be more than fifty acre feet, but does not include any such barrier which is or will be less than six feet in height, regardless of storage capacity, or which has or will have a storage capacity not in excess of fifteen acre feet, regardless of height.

2. "Height" means the vertical distance from the lowest elevation of the outside limit of the barrier at its intersection with the natural ground surface to the spillway crest elevation.

3. "Owner" includes any person or entity who owns, controls, operates, maintains, manages or proposes to construct or modify a dam, except the United States government and its agents or instrumentalities, if a safety program at least as stringent as the state program is applicable to and enforced against such agent or instrumentality.

4. "Person" means any person, firm, association, organization, partnership, business trust, corporation, company or district.

5. "Storage capacity" means the maximum volume of water that can be impounded by the reservoir when there is no discharge of water.

§ 45-702. Jurisdiction of director of water resources; records; rules and regulations

A. All dams shall be under the jurisdiction of the director of Water Resources. Dams of the state, or any political subdivisions thereof, or dams of public utilities, and all dams within the state except those of the United States or its instrumentalities are included within the jurisdiction conferred by this §. It is unlawful to construct, reconstruct, repair, operate, maintain, enlarge, remove or alter any dam except upon approval of the director.

B. The records pertaining to dam supervision shall be public documents. The director shall adopt and revise rules of procedure and regulations and issue general orders to effectuate this article.

§ 45-703. <u>Approval by director of proposed dams or enlargements of existing dams; application for construction or enlargement</u>

A. Construction of a dam or enlargement of an existing dam shall not be commenced until a written approval of plans and specifications has been obtained from the director.

B. A separate application for each dam shall be filed with the director upon forms provided by him, reciting the name and address of the owner or his agent, the location, type, size and height of the proposed dam and appurtenant works, the storage capacity of the reservoir, and such other information as the director requests. The application shall also set forth the area of the drainage basin, rainfall and stream flow records, flood flow records and estimates and other similar information required by the director. The director may require information concerning subsoil and foundation conditions and may require that the site be drilled or otherwise prospected.

C. When the physical conditions and the size of the dam do not require the information provided in subsection B, such information may be waived by the director.

D. The means, plans and specifications by which the stream or body of water is to be dammed, by-passed or controlled during construction shall be stated in the application, or such means, plans and specifications shall be submitted to the director for approval prior to beginning construction. The director shall have the same authority over the construction and maintenance of such means of damming, by-passing or controlling the stream or body of water during construction of the dam as he has over similar work on the dam itself.

E. The application shall further state the proposed time of beginning and completing construction, the estimated cost of construction, the use to which the impounded or diverted water is to be put, and shall be accompanied by maps, plans and specifications and state such details and dimensions as the director may require. The maps, plans and specifications shall be a part of the application.

F. Prior to the approval of plans and specifications, the director may require a surety company bond in an amount sufficient to secure the costs to the state in assuring the safety of any dam left partially constructed. The bond may be required only when the director questions the financial ability of the owner or contractor, or otherwise deems the bond advisable.

§ 45-704. Estimated cost of dam; application fees

A. The estimated cost of the dam or alterations thereof shall include the cost of all labor and materials entering into the construction of the dam and appurtenant works. The cost of the preliminary investigation and surveys, the construction plant and all other items properly included in the cost of the dam shall be chargeable to the cost of the dam.

B. Application fees shall be based on the cost of the dam. Applications shall be accompanied by a filing fee of fifty dollars. There shall be a further fee paid at the time of filing the application, or upon receipt of the proceeds of bonds and prior to beginning actual construction if the dam is to be constructed from the proceeds of bonds, on the following basis:

1. For the first two hundred fifty thousand dollars, one per cent of the estimated cost.

2. For the next seven hundred fifty thousand dollars, one-half of one per cent.

3. For the next four million dollars, one-fourth of one per cent.

4. For all cost in excess of five million dollars, one-tenth of one per cent.

C. An application shall not be considered until the filing fee has been paid, and no construction shall be permitted until the additional fees required by subsection B are received.

§ 45-705. Charges against irrigation projects; disposition of proceeds

A. Upon all projects for which approval is required by the state certification board, or which involves examination, supervision and inspection by the director, whether in connection with the construction of a dam or otherwise, the following shall be paid:

1. For irrigation projects of any kind involving twenty-five thousand acres or less, an annual tax levy of ten cents per acre shall be levied and collected.

2. For such irrigation projects in excess of twenty-five thousand acres, an annual tax levy of five cents per acre shall be levied and collected.

B. The levy shall be made only in the years required for construction of the project, and shall be made and collected in the same manner as provided for the levy and collection of taxes made for other expenses of the particular district. Such collections shall be transmitted to the state treasurer and credited to the state general fund.

C. All fees collected by the director under this article shall be paid to the State treasurer who shall credit them to the state general fund.

D. The fees provided by this article shall be required of all applicants including the state and its departments, institutions or agencies.

§ 45-706. <u>Approval of repair, alteration or removal of dam</u>

A. Before commencing the repair, alteration or removal of a dam, application shall be made for written approval by the director, except as otherwise provided by this article. The application shall state the name and address of the applicant, shall adequately detail the changes it proposes to effect and shall be accompanied by maps, plans and specifications setting forth such details and dimensions as the director requires. The director may waive any such requirements. The application shall give such other information concerning the dam and reservoir required by the director, such information concerning the safety of any change he may require, and shall state the proposed time of commencement and completion of the work. The application shall otherwise conform to the requirements of § 45-703.

B. When repairs are necessary to safeguard life and property, they may be started immediately, but the director shall be notified forthwith of the proposed repairs and of work under way, and they shall be made to conform to his orders.

§ 45-707. <u>Approval or disapproval of applications; commencing construction</u>

A. Upon receipt of an application, the director shall approve, disapprove or approve subject to conditions necessary to insure safety.

B. A defective application shall not be rejected, but notice of the defects shall be sent to the applicant by registered mail. If the applicant fails to file a perfected application within thirty days, the original shall be canceled unless further time is allowed.

C. No application shall be approved in less than ten days from its receipt, nor shall an application be retained more than sixty days after it is filed unless the director finds that additional information is necessary.

D. If the director disapproves an application, one copy shall be returned with a statement of his objections. If an application is approved, the approval shall be attached, to the application and a copy returned by registered mail. Approval shall be granted under terms, conditions and limitations which the director deems necessary to safeguard life and property.

E. Construction shall be commenced within one year after the date of approval of the application or such approval is void. The director upon written application and good cause shown may extend the time for commencing construction. Notice by registered mail shall be given to the director at least ten days before construction is commenced.

§ 45-708. Inspections and investigations during construction; modifications; notice

A. During the construction, enlargement, repair, alteration or removal of a dam the director shall make such inspections, investigations or examinations as he deems necessary to enforce the provisions of his approval and the plans and specifications as approved. If thereafter as the work progresses the director believes amendments, modifications or changes are necessary to insure safety, he shall revise the approval.

B. If, during construction, reconstruction, repair, alteration or enlargement of any dam, the director finds the work is not being done in accordance with the provisions of the approval and the approved plans and specifications, he shall give written notice by registered mail or personal service to the person who received the approval and to the person in charge of construction at the dam. The notice shall state the particulars in which compliance has not been made, and shall order immediate compliance with the terms of the approval, and the approved plans and specifications. The director may order that no further construction work be undertaken until such compliance has been effected and approved by the director. A failure to comply with the approval and the approved plans and specifications shall render the approval revocable unless compliance is made after notice as provided by this section.

§ 45-709. Notice of completion; license of final approval; removal of dam

A. Immediately upon completion or enlargement of a dam, notice of completion shall be given to the director. As soon as possible thereafter supplementary drawings or descriptive matter showing or describing the dam as actually constructed shall be filed with the director which shall include:

1. A record of all grout holes and grouting.

2. A record of permanent location points and bench marks.

3. A record of tests of concrete or other material used in the construction of the dam.

4. A record of other items of permanent value bearing on safety and permanence of construction.

B. When an existing dam is enlarged, the supplementary drawings and descriptive matter need apply only to the new work.

C. The owner of a completed dam shall file an affidavit of the total cost of the dam comprised of items set forth in § 45-704, and furnish such evidence in support thereof as the director requires. No license of final approval shall issue until the affidavit is filed. The completed dam shall be inspected by the director, and upon finding that the work has been done as required and that the dam is safe, he shall issue a license of final approval forthwith, subject to such terms as he deems necessary for the protection of life and property. In the event the total cost exceeds the estimated cost, the fee shall be recomputed in accordance with the schedule in § 45-704, subsection B. The owner shall pay the difference between the fee already paid and the recomputed fee.

D. Pending issuance of the license, the dam shall not be used except on written consent of the director, subject to conditions he may impose.

E. When a dam is removed the owner shall file with the director evidence showing that a sufficient portion has been removed to permit the free passage of flood waters. Before final approval of the removal of the dam the director shall inspect the work to ascertain its safety.

§ 45-710. Petition for review

Except as otherwise provided in this article, a petition for review by a board of review of any approval, disapproval or order of the director concerning plans, specifications, construction or maintenance pertaining to any dam may be filed by the owner or applicant, or by three land owners whose property would be endangered by the failure of the dam.

§ 45-711. Time for filing petition; board of review

A. The petition for review shall be in writing and shall be filed with the director within ten days after issuance of the approval, disapproval or order of which complaint is made. Upon receipt of the petition, the director shall prepare a list of ten qualified experts. Within ten days the petitioner shall select three individuals from the list who shall then serve as the board of review. The board shall serve at the expense of the petitioners. Within thirty days from its designation, or within such further time as the director allows, the board shall report to the director and he shall forthwith affirm, change or modify the report, and his action shall be final and not subject to further review. No board of review shall be appointed to consider any action taken by the director relative to emergency regulation and control of a dam under § 45-712.

B. Pending examination, change or modification by the director, his approval, disapproval or order issued shall remain operative. Operations shall be suspended if an applicant or owner files a petition for a board of review unless the director orders work to proceed because of emergency conditions.

§ 45-712. Supervision over maintenance and operation; remedial measures; lien

A. Supervision over the maintenance and operation of dams to safeguard life and property is vested exclusively in the director. He shall make complete inspections, require reports from owners or operators and shall issue rules, regulations, and orders necessary to secure maintenance and operation of dams which will safeguard life and property.

B. If the director determines that the dam under consideration is dangerous to the safety of life and property, and that there is not sufficient time to issue and enforce an order relative to its maintenance or operation, or if the director believes that imminent floods threaten the safety of the dam under consideration, the director shall immediately employ remedial measures necessary to protect life and property.

C. In applying remedial measures the director may lower the water level of a reservoir by releasing water impounded, may completely empty the reservoir, may destroy the dam or reservoir or such portions as appear necessary, or may construct, reconstruct, repair or enlarge the dam, and may exercise any other control of the dam, reservoir and appurtenances essential to safeguard life and property. The director shall remain in full charge and control of the dam, reservoir and appurtenances until they have been rendered safe or the emergency has terminated.

D. The costs and expenses of the control, regulation and abatement provided by this §, including costs of construction work done to render the dam, reservoir, or appurtenances safe, shall constitute a lien against all property of the owner, and the lien shall be prior and superior to all other mortgages, liens or encumbrances or record. The lien shall have the force and effect of a mechanic's and materialman's lien, and may be foreclosed at any time within two years.

E. The lien referred to in subsection D may be perfected and foreclosed in advance of construction or repair or after completion of the repairs. If in advance, the lien shall be perfected by the filing of an affidavit of the director setting forth the estimate of the costs of construction or repair with the county recorder in the county in which the dam is located in the same manner as prescribed for mechanics' liens in Title 33, Chapter 7, Article 6 and may be foreclosed in the same manner as a mechanic's and materialman's lien. When the affidavit is filed, the amount set forth in the affidavit shall be a lien in such amount against all property of the owner. If the actual cost of construction or repair exceeds the estimated cost, the director may amend the affidavit setting forth the additional estimated cost. If the estimated cost exceeds the actual costs of construction or repair at completion, the director shall file an amended affidavit at completion. If a lien is perfected in advance and the construction or repair is not commenced with two years from the date of perfection, the lien shall be void. The director shall file a satisfaction of lien upon payment of the costs of construction or repair by the owner.

§ 45-713. Inspection upon complaint

Upon receipt of a written complaint that the person or property of the complainant is endangered by any dam, the director shall inspect such dam unless his records disclose that the complaint is without merit. If the complainant insists upon an inspection and deposits with the director an amount sufficient to cover costs of inspection, the inspection shall be made. If an unsafe condition is found, the director shall cause it to be corrected, and the deposit shall be returned. If the complaint was without merit the deposit shall be paid into the general fund.

§ 45-714. Investigations for review of design and construction

The director shall make investigations and assemble data for a proper review and study of the design and construction of dams, reservoirs and appurtenances, and shall make watershed investigations to facilitate decisions on public safety. The director or his representatives may enter upon private property for such purposes.

§ 45-715. Liabilities of state and owners of dam in action for damages

A. No action shall be brought or maintained against the state, or any of its departments, agencies or officials thereof, or any of their employees or agents, for damages sustained through the partial or total failure or any dam or its maintenance by reason of control and regulation thereof by any of them pursuant to duties imposed upon them under the provisions of this chapter.

B. Nothing in this article shall relieve any owner or operator of a dam from the legal duties, obligations and liabilities arising from such ownership or operation.

§ 45-716. Violations; classification

A. It is unlawful for an owner, director, officer, agent, employee, contractor or his agents to construct, reconstruct, repair, enlarge, alter or remove a dam without an approval as provided in this chapter, or contrary to an approval issued. It is unlawful for the agents or employees of the director to permit such work to be done without immediately notifying the director.

B. A person who violates this article, except as otherwise provided, is guilty of a class 2 misdemeanor, and each day such violation continues constitutes a separate offense.

§ 45-717. Action and procedures to restrain violations

A. The director may take any legal action proper and necessary for the enforcement of this chapter.

B. An action or proceeding under this § may be commenced whenever any owner or any person acting as a director, officer, agent or employee of any owner, or any contractor or agent or employee of such contractor is:

1. Failing or omitting or about to fail or omit to do anything required of him by this chapter or by any approval, order, rule, regulation or requirement of the director under the authority of this chapter; or

2. Doing or permitting anything or about to do or permit anything to be done in violation of or contrary to this chapter or any approval, order, rule, regulation or requirement of the director under this chapter; or

3. In the opinion of the director, in any manner in violation of this chapter.

C. Any action or proceeding under this section shall be commenced in a court of appropriate jurisdiction in which:

1. The cause or some part thereof arose; or

2. The owner or person complained of has his principal place of business; or

3. The person complained of resides.

RULES AND REGULATIONS
PERTAINING TO THE
SUPERVISION OF DAMS

CHAPTER 15

DEPARTMENT OF WATER RESOURCES

(Authority: A.R.S. § 45-101)

All former Sections renumbered. Refer to Historical Notes following each Section (Supp. 82-5).

ARTICLE 1. DEFINITIONS, FEES, PROCEDURAL RULES FOR HEARINGS

ARTICLE 12. DAM SAFETY PROCEDURES

NOTE: THESE RULES AND REGULATIONS ARE THE MOST CURRENT ON FILE WITH THE SECRETARY OF STATE.

THE FOLLOWING CLERICAL CHANGES SHOULD BE NOTED:

1. THE STATE WATER ENGINEER IS NOW THE DIRECTOR OF THE DEPARTMENT OF WATER RESOURCES.
2. THE DIRECTOR'S OFFICE IS LOCATED AT 99 E. VIRGINIA, PHOENIX, ARIZONA.

ARTICLE 1. DEFINITIONS, FEES, PROCEDURAL, RULES FOR HEARINGS

R12-15-151. Fee schedule

The Department shall not accept or take action on an application or filing without payment of the appropriate fee as listed below. Payment may be made by cash, check or by entry in an existing Department fee credit account established pursuant to R12-15-152.

7. SAFETY OF DAMS

Application for review..................................50.00

Review of plans, studies

As a portion of Dam Cost (in dollars):

first 250,000..1.00%

next 750,000.. .50%

next 4,000,000...................................... .25%

over 5,000,000...................................... .10%

Safety Inspections

Per Inspection.......................................50.00

Plus, per foot of height............................ 1.00

9. OTHER CHARGES

Photocopies.. .20each

Computer reports: First page of report.................15.00

Additional pages.................... .25each

Certified "True Copies"............................. 2.50each

R12-15-152. Fee credit account

Any person who may pay more than two hundred dollars in fees annually may apply to the Department to have a fee credit account established for periodic billing. The Department retains discretion to refuse to establish a credit account and shall set reasonable terms for payment and interest for any credit account established pursuant to this Rule. Any person who has established a credit account for fees with the Department who does not comply with the terms of the account shall lose the privilege of maintaining such an account.

ARTICLE 11.

ARTICLE 12. DAM SAFETY PROCEDURES

R12-15-1201. General provisions

A. The State Engineer's office is located at 222 North Central Avenue, Suite 800, Phoenix, AZ. All notices and contracts with the State Engineer shall be sent to and made at this address.

B. Forms with respect to these Rules and Regulations may be picked up or requested by mail at the address of the State Engineer. Copies of these Rules will also be available at the office of the State Engineer. A copy of the application form is shown as Exhibit A following this Article.

Historical Note

Adopted eff. Nov. 2, 1978 (Supp. 78-6). Former Section R12-15-01 renumbered without change as Section R12-15-1201 eff. Oct. 8, 1982 (Supp. 82-5).

R12-15-1202. Professional engineering requirement

A. The plans and specifications accompanying an application for construction of a new dam or alteration, repair, enlargement, or removal of an existing dam shall be prepared by or under the direction of a professional engineer registered under the laws of Arizona, having proficiency in civil engineering as related to dam technology.

B. Engineers of the United States Soil Conservation Service who design and construct dams for owners other than the United States are not required to be registered in Arizona for purposes of these Rules.

Historical Note

Adopted eff. Nov. 2, 1978 (Supp. 78-6). Former Section R12-15-02 renumbered without change as Section R12-15-1202 eff. Oct. 8, 1982 (Supp. 82-5).

R12-15-1203. Application procedure

A. An application for the construction of a new dam or enlargement, repair, alteration, or removal of an existing dam shall be prepared in duplicate and sent to the State Engineer on forms furnished by the State Engineer.

B. The application shall include at least the following information:

1. One complete set of plans and specifications prepared by a registered professional engineer (or U.S. Soil Conservation Service engineer).

2. Geotechnical engineering data including the results of foundation and materials exploration.

3. Engineering design data including basis assumptions as to loads and limiting stresses and as to methods of analyses for all structures, including the dam.

4. Hydraulic engineering data used in determining capacity of spillways and outlet works and hydrologic data used in deriving required spillway capacity.

C. Upon completion of the project, the total cost shall be tabulated and the fee recomputed in accordance with the law. If the recomputed fee exceeds the fee paid with the application then the owner shall pay the difference between the fee already paid and the recomputed fee.

D. Plans for the proposed work shall be filed in the form of paper prints. Notification of any changes required by the State Engineer will be given to the applicant. Thereafter, the drawings designated for approval by the State Engineer shall be submitted in triplicate together with two sets of specifications. After approval by the State Engineer one set of signed prints and approved application shall be returned to the applicant, one set of the drawings shall be retained for the permanent State records of the State Engineer and the third set shall be used for construction.

Historical Note

Adopted eff. Nov. 2, 1978 (Supp. 78-6). Former Section R12-15-03 renumbered without change as Section R12-15-1203 eff. Oct. 8, 1982 (Supp. 82-5).

R12-15-1204. Final inspection and license of approval

A. Upon completion of construction, the State Water Engineer shall be notified in writing and shall finally inspect the work as soon as practicable.

B. After final inspection by a field engineer, the applicant is required to file the following:

1. Affidavit of cost of construction. Attach breakdown of costs, including engineering.

2. Additional fee if final cost exceeds estimated cost, pursuant to Arizona Revised Statutes.

3. As-constructed plans, in the form of paper prints.

C. A license of approval shall be issued by the State Engineer after payment of all fees and upon a finding that the dam and reservoir are safe to impound water. No water shall be stored nor shall the reservoir be used without written permission of the State Engineer, pending issuance of a license of approval.

Historical Note

Adopted eff. Nov. 2, 1978 (Supp. 78-6). Former Section R12-15-04 renumbered without change as Section R12-15-1204 eff. Oct. 8, 1982 (Supp. 82-5).

R12-15-1205. Plans and specifications

A. Engineering drawings shall be in sizes ranging from 22 inches by 36 inches to 28 inches by 42 inches. Letter size drawings are not permitted. Drawings shall be prepared on conventional drawing material such that clear legible prints can be obtained. Submittal of blue line or black line prints for final approval and signature will be satisfactory. In preparing the drawings, each sheet shall contain, in addition to the normal title block in the lower right hand corner, space approximately 4" x 5" somewhere in proximity to the lower right hand corner for application of the State Engineer signature block.

B. The minimum requirements of maps and drawings for small dams consist of the following:

1. Location map of the dam and the drainage basin above the structure.

2. A topographic map of the dam, spillway, outlet works and the reservoir. The scale should be large enough to accurately locate the dam and appurtenances and to indicate cut and fill lines. Elevations should be to a real datum base, rather than an assumed elevation. Contour intervals should be compatible with height and size of the dam and its appurtenances. An area capacity curve of the reservoir can be shown on this sheet.

3. Profile and section of the dam. The profile of the dam may be drawn to different horizontal and vertical scales. However, the maximum section of the dam should be drawn to a true scale. For a small dam the outlet conduit can be shown on the maximum section if this is typical of the proposed construction.

4. Details of the outlet works. This should include the intake structure, the gate system, conduit details, the trashrack, and the downstream outlet structure. Details should be sufficiently complete to accurately lay out the structure and build it. Schematic drawings are not acceptable.

5. Plan, profile, and control section of the spillway. This will also include details of any concrete work that may be contemplated. A complex control structure, a concrete chute, or an energy dissipating device for a terminal structure, will require additional design details.

C. Drawings required for technical review of a major structure cannot be listed in detail because each dam is different. The following maps and drawings are required for the typical large structure:

1. Drainage basin above the proposed dam.

2. Location map for all foundation drill holes, auger holes, test pits, trenches, and borrow areas. Also, bench marks with elevations, reference points, permanent ties, should be shown. These may be shown on a reservoir map.

3. Log of foundation drill holes and auger holes.

4. Topographic map of the damsite.

5. A topographic map of the reservoir, with area and capacity curves.

6. Plan, profile and sections of the dam, all at natural scales. Camber, crest details, interior drains, and zone details should be included.

7. Foundation plan showing excavation with proposed grout and drain holes.

8. Outlet works showing plan, profile, sections, and details.

9. Spillway showing plan, profile, sections and details.

10. Details of diversion scheme if applicable.

D. If there is question as to whether a dam is considered a small dam or a large dam the owner may present any information he deems pertinent to the State Engineer and the State Engineer will then decide which category applied.

E. Specifications concerning the proposed method of construction shall be filed in duplicate with the application. The specifications shall include a detailed description of the work to be performed and a statement of the requirements for the various types of materials that will enter into the permanent construction, including but not limited to, foundation preparation, placement of materials and concrete quality control. Also any special techniques should be carefully described.

F. If not included in the specifications, the construction schedule and a statement of the sequence of construction operations shall be filed in duplicate with and form a part of the application.

Historical Note

Adopted eff. Nov. 2, 1978 (Supp. 78-6). Former Section R12-15-05 renumbered without change as Section R12-15-1205 eff. Oct. 8, 1982 (Supp. 82-5).

R12-15-1206. Construction control

A. The owner is responsible for safety during and after completion of any construction of a new dam, enlargement, repair, alteration, or removal of an existing dam, and is responsible for inspections and completion of the work in accordance with the plans and specifications approved by the State Engineer.

B. The State Engineer shall determine that proper construction control is being exercised by the owner or the owner's engineer and any unsatisfactory condition shall be remedied by the owner or his engineer with the contractor.

C. The foundation of the dam shall be inspected by the State Engineer or his designee. The foundation shall not be covered over prior to such inspection and written approval of the foundation by the State Engineer or his designee. Inspection for the foundation for the outlet and spillway structures is also required and written approval of the State Engineer or his designee is required before covering over these foundations.

D. The State Engineer and his designee shall have access to the job for purposes of inspecting all phases of the construction, including, but not limited to, mechanical installations, concrete work, placement methods and strength test records.

E. Any deviations from the approved plans and specifications must be approved in writing by the State Engineer or his designee before proceeding therewith. Any problems encountered during construction which required any deviation from the plans and specifications approved by the State Engineer shall be immediately reported to the State Engineer.

Historical Note

Adopted eff. Nov. 2, 1978 (Supp. 78-6). Former Section R12-15-06 renumbered without change as Section R12-15-1206 eff. Oct. 8, 1982 (Supp. 82-5).

State of Arizona
Department of Water Resources
Divison of Safety of Dams

Application No. ______________________ Filed ______________
(Applicant shall not fill in above blanks)

APPLICATION FOR THE APPROVAL OF THE PLANS AND SPECIFICATIONS FOR THE CONSTRUCTION, ENLARGEMENT, REPAIR, ALTERATION OR REMOVAL OF A DAM AND RESERVOIR

(This application involves in no way the right to appropriate water. To secure the right to appropriate water, application has to be made to the Department of Water Resources, Division of Water Rights on forms which will be furnished upon request.)

This application is for the ______________________ of the ______________________ Dam.
(Construction, Repair, Alteration, Etc.)

LOCATION OF DAM

This dam is in ______________________ County, in the __________ ¼, Sec. __________, Tp. __________

R. __________, G&SR, B&M, and is located on ______________________
(Creek, River or Watershed)

tributary to ______________________.
(Creek or River)

OWNER

Name ______________________

Address ______________________
(Street and Number, or P.O. Box)

____
(City) (State) (Zip) (Telephone)

If this application is for construction of a new dam complete all items (1 thru 21) except Item 15. For alteration, repair, enlargement or removal of a dam complete Items 15 thru 21 and those other items where a change is being made.

DESCRIPTION OF DAM AND RESERVOIR

1. Type of dam ______________________
(Earth, Rock, Concrete Gravity, Concrete Arch)
2. Crest length ______________________ ft. Crest width ______________________ ft.
3. Slope, upstream ______________________ Slope, downstream ______________________
4. Dam crest elevation ______________________ ft. Spillway crest elevation ______________________ ft.
5. Dam height is __________ feet (Measured from original ground level at the downstream toe to the spillway crest).
6. Volume of material in dam ______________________ cubic yards.
7. Water surface elevation is __________ feet at the time of maximum spillway discharge.
8. Spillway (type, size and capacity) ______________________.
9. Outlet (type, size and capacity) ______________________.
10. Reservoir capacity at spillway crest elevation is ______________________ acre feet.
11. Reservoir surface area at s[illegible]ay crest elevation is ______________________ acres.

(See Reverse Side)

HYDROLOGIC DATA

12. Maximum Recorded Rainfall ______________ inches in ______________ hours.

Date ______________ Location ______________

13. Maximum Recorded Streamflow ______________ cubic feet per second.

Date ______________ Location ______________

14. Drainage Area ______________ square miles.

GENERAL INFORMATION

15. Description of Work (repair, alteration, etc.) ______________

____

____

____

____

16. Use of Stored Water ______________

17. What provisions to divert flood flows during construction? ______________

____

18. Construction will begin ______________ (Date) Estimated Completion ______________ (Date)

19. Estimated cost of dam, reservoir, and appurtenances: $______________

20. Fees accompanying this application: $______________

21. Investigations, plans and specifications prepared by ______________

____

Signed: ______________

Address: ______________

Legal capacity if other than owner: ______________

____

Date: ______________

APPROVAL OF APPLICATION No. ______, INCLUDING THE PLANS AND SPECIFICATIONS

This is to certify that Application No. ______, including the plans and specifications for ______________ Dam and Reservoir has been examined and the same is hereby approved, subject to the following terms and limitations:

1. Construction work shall be started within one (1) year from date.

2. No foundations or abutments shall be covered by the material of the dam until the Department has been given an opportunity to inspect and approve the same.

____

____

____

Dated this ______ day of ______________, 19 ______.

DIRECTOR, Department of Water Resources

By C ef, Divisior ' Safety of Dams

STATE OF ARIZONA
DEPARTMENT OF WATER RESOURCES
ENGINEERING DIVISION/SAFETY OF DAMS SECTION

INSTRUCTIONS FOR FILING AN APPLICATION

OCTOBER 1986

An application is required for construction of a new dam or enlargement, repair, alteration, or removal of an existing dam. Applications shall be prepared in duplicate and sent to the Department of Water Resources upon forms which will be furnished free on request. The Department's address is: 99 East Virginia, Phoenix, Arizona 85004; Attention: Engineering Division, Safety of Dams Section.

In addition to the application form, two complete sets of engineering drawings, specifications and an engineering design report shall be considered a part of the application and shall be submitted to the Department with the proper application fees. The drawings, specifications and engineering report shall be prepared by a professional engineer registered in Arizona and experienced in the design and construction of dams. The engineer's seal and signature shall appear on all drawings, the specifications and the design report. The Director may waive or enlarge any requirements for information to accompany an application.

As prescribed in the Statutes, no application shall be given consideration unless accompanied by a filing fee of $50 and a further fee based upon the estimated cost of the project (see paragraph on Fee Requirements) as well as appropriate support data.

Plans for the proposed work shall be filed in the form of paper prints. After review of the plans and specifications, the Department will notify the applicant of any required changes. A conference may also be arranged to work out revisions which will meet the Department's requirements. The revised drawings shall be submitted to the Department in triplicate along with two sets of revised specifications for approval. Upon approval, one set of signed prints and the approved application will be returned to the applicant, one signed set of drawings retained for permanent State record and the third set retained for use by the Department during construction. A half-size set, if available, will be acceptable for the construction set.

FEE REQUIREMENTS

Payment of the application fees, including the filing fee and the further fee, is required for all new construction, alteration, repair, enlargement or removal applications for dams. The filing fee for all applications is $50.00. The further fee is based upon the estimated project cost. The project cost shall include all costs associated with construction of the dam and appurtenant works. Preliminary investigations and surveys, engineering design, supervision of construction and any other engineering costs shall be included. All costs of construction of the project shall also be included.

Based upon these total costs the further fee will be computed according to the following schedule:

1. For the first $250,000, one (1%) percent of the estimated cost.
2. For the next $750,000, one-half of one (0.5%) percent.
3. For the next $4,000,000, one-fourth of one (0.25%) percent.
4. For all costs in excess of $5,000,000, one-tenth of one (0.1%) percent.

Upon completion of the project, the actual total cost shall be tabulated and the fee recomputed for this amount in accordance with the schedule above. If the recomputed fee exceeds the fee paid with the application be $50.00 or more, then the owner shall pay the difference between the fee already paid and the recomputed fee. If the recomputed fee is less than the original fee by an amount of $50.00 or more, then the owner shall be entitled to a refund by the amount of the difference between the fee already paid and the recomputed fee. When the amount indicated in less than $50.00 there will be no refund, and likewise there will be no additional fee if the indicated amount is less than $50.00.

Example of amount of fee to accompany application.

ESTIMATED COST	$6,420,000.00
Filing Fee	50.00
1% x $250,000	2,500.00
0.5% x $750,000	3,750.00
0.25% x $4,000,000	10,000.00
0.10% x $1,420,000	1,420.00
TOTAL FEE	$ 17,720.00

If the actual cost for this project were $6,482,500.00 the recomputed fee would be:

ACTUAL COST	$6,482,500.00
Filing Fee	50.00
1% of $250,000	2,500.00
0.5% x $750,000	3,750.00
0.25% x $4,000,000	10,000.00
0.10% x $1,482,500	1,482.50
Recomputed Fee	$ 17,782.50
Original Fee	$ -17,720.00
DIFFERENCE	$ 62.50

In this case the owner would be required to pay an additional fee of $62.50. If the actual cost were $6,320,000.00 then the recomputed fee would be $17,620.00, the difference would be $100.00, in the owner's favor, and the owner would be entitled to a refund of $100.00.

If, after review of the final cost statement by the Department, a refund is indicated, a refund may be obtained by requesting it in writing. Upon receipt of a refund request, the Department will initiate the refund process.

PLANS AND SPECIFICATIONS

ENGINEERING DRAWINGS - All drawings submitted shall be from 22" x 36" to 28" x 42" in size.

Drawings should be prepared on conventional drafting material such that clear, legible prints can be obtained. Submittal of blue line or black line prints for final approval and signature will be satisfactory. In preparing the drawings, each sheet shall contain, in addition to the normal title block in the lower right hand corner, a space at least 2-1/2" x 4" in proximity to the lower right hand corner for application of the Department's approval signature block.

Drawings accompanying the application shall include:

1. A topographic map of the dam, spillway, outlet works and the reservoir on a scale large enough to accurately locate the dam and appurtenances and to indicate cut and fill lines. Elevations shall be to a real datum base, rather than an assumed elevation. Contour intervals shall be compatible with the height and size of the dam and its appurtenances.
2. Area and storage capacity curves and tables for the reservoir.
3. Spillway and outlet rating curves and tables.
4. A location map showing all exploration drill holes, test pits, trenches, adits, borrow areas and bench marks with elevations, reference points and permanent ties.
5. Geologic information including geologic maps of the damsite and reservoir area at scales compatible with the site and geologic complexity; soils and geologic profile along the dam centerline, showing logs of exploration drill holes, test pits, trenches, and adits.

6. A foundation profile showing the existing ground and proposed cut and fill elevations.
7. A Profile and sufficient cross sections at the dam to adequately describe it. Embankment camber, crest details, interior drains and zone details must be shown. The profile of the dam may be drawn to different horizontal and vertical scales. As a minimum, a maximum section of the dam shall be included. It shall be drawn to a true scale (vertical = horizontal). The outlet conduit may be shown on the maximum section if this is typical of the proposed construction.
8. A foundation plan showing excavation with proposed grout and drain holes.
9. Details of the outlet works, including the intake structure, the gate system, conduit details, the trashrack and the downstream outlet structure.
10. The plan, profile and control section of the spillway, including details of any concrete work that is contemplated. A complex control structure, a concrete chute or an energy dissipating device for a terminal structure will require additional design details.
11. Hydrologic data, drainage area and flood routing criteria, as appropriate.

The Director may waive or enlarge any requirements for information to accompany an application.

SPECIFICATIONS - The specifications shall include a detailed description of the work to be performed and a statement of the requirements for the various types of materials that will enter into the permanent construction. Of particular importance are those sections describing foundation preparation, placement of materials and concrete quality control. Any special techniques should also be carefully described.

If not included in the specifications, the construction schedule and a statement of the anticipated sequence of construction operations shall be filed in duplicate with the application.

DESIGN REPORT

In addition to plans and specifications, a design report is required for all structures. As a minimum, this report should contain the following:

1. Hydrology calculations and a summary table of data used in determining the required spillway capacity and freeboard.
2. Hydraulic characteristics and engineering data of the structure used in determining the capacity of outlet works and spillway.
3. Results and analysis of subsurface investigation including logs of test borings and geologic cross-sections.
4. Material testing results and the location of test pits and the logs of these pits.
5. Design of the grout curtain and cap.

6. Sample calculations and basic assumptions on loads and limiting stress as for the reinforced concrete design.

7. A stability analysis of the dam including appropriate seismic loading, safety factors and embankment zone characteristics. The seismicity of the project area and activity of faults in the vicinity must be discussed.

8. Geologic investigation of the damsite and reservoir basin.

9. Plans to adequately compensate for geological weakness in the dam foundation or in the abutment areas.

10. Flow net considerations including the cutoff trench design or other cutoff facilities.

11. Internal drainage design including instrumentation necessary to monitor the drainage system.

12. Foundation treatment and abutment contact design.

13. Post-construction vertical and horizontal movement monitoring system. Systems for monitoring piezometric levels and seepage flows. Strong motion instrumentation may be required at some sites.

14. A statement of the designer's intent with regard to testing frequencies, foundation guidelines, etc.

The Director may waive or enlarge any requirements for information to accompany an application.

CONSTRUCTION CONTROL

Application approval is valid for a one year period in which construction must begin. If construction does not begin within one year, the Department must review the application again in light of changes which may have occurred since the approval was originally given. Upon written application and good cause shown by the owner, the time for commencing construction may be extended.

The owner and his engineer shall assure that construction of a new dam, or enlargement, repair, alteration or removal of an existing dam is carried out in accordance with the plans and specifications approved by the Director. Construction supervision shall be under the direction of a registered professional engineer having proficiency in the design and construction of dams.

The Safety of Dams Section will inspect construction and determine that proper construction control is being exercised by the owner's engineer. Any unsatisfactory condition shall be remedied by the owner or his engineer with the contractor.

The Department shall have access to the damsite for purposes of inspecting all phases of the construction including the foundation, embankment or concrete placement, inspection and test records and mechanical installations.

The owner or his engineer shall immediately report to the Department any conditions encountered during construction which require any deviation from the plans and specifications approved by the Director. The owner or his engineer shall promptly submit a written request for approval of any necessary change and sufficient data to justify the proposed change. Construction pursuant to the proposed change may not commence without the written approval of the Director.

AFTER COMPLETION

Upon completion of construction, the Department shall be notified to that effect in writing and a final inspection will be made as soon as practicable.

As soon as possible after completion of the work and final inspection by an engineer from the Section, the following shall be filed by the owner or his engineer:

A. An Affidavit of the actual cost of construction. Attach a detailed breakdown of the costs, including all engineering costs (see paragraph on fee requirements).

B. An additional fee or refund request, as appropriate (see paragraph on fee requirements).

C. One set of full sized as-constructed plans, in the form of paper prints.

D. Construction records such as grouting, materials testing and permanent bench marks.

E. A brief completion report summarizing the salient features and causes for changes or deviations from the approved drawings and specifications which were made during the construction phase.

F. An Emergency Alert Plan including inundation map.

G. An operating manual for the dam and its appurtenant structures including schedules for surveillance activities.

Upon completion of these items and finding that the dam has been constructed in accordance with the approved plans and specifications, a license of final approval will be issued. Pending issuance of a license, use of the reservoir shall require written permission from the Department.

LIST OF REFERENCES

Included below is a brief list of references which have proven useful in coping with basic dam design problems. The list is not intended to be all-inclusive. However, most of these references do include comprehensive bibliographies which may provide additional assistance in locating more detailed reference materials. When complex dam design problems are encountered, it is advisable to retain a qualified specialist.

AMERICAN SOCIETY OF CIVIL ENGINEERS, U.S. COMMITTEE ON LARGE DAMS Design and Construction of Dams 1967.

ARIZONA DEPARTMENT OF WATER RESOURCES, ENGINEERING DIVISION, SAFETY OF DAMS SECTION, Guidelines for the Determination of Spillway Capacity Requirements, (Revised 1986).

CEDERGREN, H.R. - Seepage, Drainage, and Flow Nets, Second Edition New York: John Wiley and Sons, Inc. 1977.

COMMITTEE ON SAFETY OF EXISTING DAMS. Safety of Existing Dams--Evaluation and Improvement. Prepared under auspices of Water Science and Technology Board, Commission on Engineering and Technical Systems, National Research Council. Washington, D.C.: National Academy Press. 1983.

DAVIS, C.V. and K.E. SORENSEN, New York: McGraw-Hill Book Company, Inc., Handbook of Applied Hydraulics, 3rd Edition. 1969.

HANSEN, E.M., J.T. RIEDEL, and F.K. SCHWARZ. Probable Maximum Precipitation Estimates--Colorado River and Great Basin Drainages. Hydrometeorological Report 49. Silver Spring, Maryland: National Weather Service, National Oceanic and Atmospheric Administration, U.S. Department of Commerce. 1977.

KING, H.W. and E.F. BRATER, Handbook of Hydraulics, 5th Edition, New York: McGraw-Hill Book Co., Inc. 1963.

SHERARD, J.R., R.J. WOODWARD, S.F. GIZIENSKI & W.A. CLEVENGER, Earth and Earth-Rock Dams, New York: John Wiley and Sons, Inc., 1963.

U.S. DEPARTMENT OF THE ARMY, CORPS OF ENGINEERS, Recommended Guidelines for Safety Inspection of Dams.

U.S. DEPARTMENT OF THE INTERIOR, BUREAU OF RECLAMATION, Design of Small Dams, A water resources technical publication, 2nd Edition, Washington, D.C.: U.S. Government Printing Office, 1973.

OWNERS LETTERHEAD

AFFIDAVIT OF TOTAL COST

ARIZONA DEPARTMENT OF WATER RESOURCES
DIVISION OF SAFETY OF DAMS
99 EAST VIRGINIA, SUITE 150
PHOENIX, ARIZONA 85004

GENTLEMEN:

I, ____
(OWNER)

AM THE ____
(OWNER, OR AUTHORIZED REPRESENTATIVE OF OWNER)

OF THE ____
(NAME OF DAM AND RESERVOIR)

THE FINAL TOTAL COST OF CONSTRUCTION (REPAIR, ALTERATION) OF THE DAM AND APPURTENANT WORKS TO COMPLETE THEREOF IS AS FOLLOWS:

* 1. ENGINEERING

1.1 PRELIMINARY INVESTIGATIONS AND SURVEYS $__________

1.2 PRECONSTRUCTION SURVEYS, GEOLOGIC, HYDROLOGIC, TESTING, DESIGN .... $__________

1.3 CONTRACT ADMINISTRATION AND CONSTRUCTION SUPERVISION .... $__________

1.4 CONSTRUCTION QUALITY CONTROL TESTING $__________

TOTAL ENGINEERING .... $__________

* 2. CONSTRUCTION CONTRACT PAYMENTS

** 2.1 FINAL PAYMENT FOR CONTRACT BID QUANTITY LIST .... $__________

** 2.2 FINAL PAYMENT FOR CHANGE ORDERS TO BID QUANTITY LIST .... $__________

TOTAL CONSTRUCTION CONTRACTS .... $__________

TOTAL CONSTRUCTION COST .... $__________

I HEREBY DECLARE UNDER PENALTY OF PERJURY THAT THE ABOVE STATEMENT IS TRUE AND CORRECT.

EXECUTED ON ______________ AT ______________ , ARIZONA.

____
(OWNER'S SIGNATURE)

____
(NOTARY) (DATE)

* The cost breakdown must include all applicable costs as indicated. For projects with two or more features an allocation of total project cost items to each appropriate feature may be made. Allocations of project cost items may be combined when properly identified to fit the individual circumstances.

** Attach forms showing contract bid quantities with prices and final pay quantities including change order items.

State of Arizona
DEPARTMENT OF WATER RESOURCES
Division of Safety of Dams

LICENSE OF APPROVAL

Pursuant to Chapter 3, Title 45-Waters, of the Arizona Revised Statutes, the DIRECTOR, Department of Water Resources authorizes the use of: .... Dam and Reservoir, Application Number .... Located in Sec. ...., Tp. ...., R. ...., G. & S.R.B. & M. .... County, State of Arizona to impound water in accordance with and subject to the following terms and conditions:

....

....

....

....

This license of approval supersedes every previous consent for use issued by the State of Arizona relative to said dam and reservoir.

Witness my hand and seal of the Arizona Department of Water Resources

.... day of ...., 19 ....

Kathleen Ferris

STATE OF ARIZONA

DEPARTMENT OF WATER RESOURCES

ENGINEERING DIVISION/SAFETY OF DAMS SECTION

D R A F T

GUIDELINES FOR THE DETERMINATION OF SPILLWAY CAPACITY REQUIREMENTS

PREPARED BY:

DAN ROGER LAWRENCE, CHIEF

ENGINEERING DIVISION/SAFETY OF DAMS SECTION

OCTOBER 1986

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INTRODUCTION

The Arizona Revised Statutes assign the responsibility for the supervision of the safety of dams to the Director of the Department of Water Resources. The statutes require that, in order to maximize protection of the public against loss of life and property by virtue of the failure of the dam, the construction, repair, enlargement, maintenance and operation of such dam must be under the approval and supervision of the Director, acting under authority vested in the Department of Water Resources.

One of the most important among the many factors affecting the safety of a dam is that of the adequacy of the emergency spillway. The Department of Water Resources is frequently asked to provide guidelines and hydrologic criteria for spillway and freeboard requirements for dams within jurisdiction. These guidelines have been prepared to assist the owner, his engineer, and other interested individuals involved in the design or modification of spillways on jurisdictional dams. The Guidelines will be reviewed periodically and revised as needed.

The following procedures and requirements will apply to all existing dams which are being enlarged or improved, dams which are being reevaluated for safety and proposed new dams which will be under the statutory jurisdiction of the Director.

GENERAL REQUIREMENTS

The basis for assigning the hydrologic requirements for spillway capacity determination lies primarily in the potential hazard posed by the dam. The hazard classification assigned to a dam is dependent on many factors all of which must be carefully evaluated in terms of their effects on the safety of the dam and on the magnitude of economic, environmental and human losses in the event of failure of the dam. These factors include, but are not necessarily limited to: height of dam; storage capacity; existing and probable future downstream development; uses of reservoir; operational procedures; condition of dam; type of dam; type of spillway; site and foundation geology; size, slope, material composition and configuration of downstream channel; distance from dam to nearest significant downstream development; and the relative location of the spillway to the dam.

Spillways acceptable to the Department of Water Resources must be sized in accordance with the classification of hazard potential for the dam and may range in capacity from a size capable of safely passing the outflow from a storm with a recurrence interval of 100-years for dams of low hazard potential to that of the Probable Maximum Flood for high hazard potential dams.

The minimum acceptable size of the spillway must have a capacity large enough to safety handle the 100-year flood inflow.

DOWNSTREAM HAZARD POTENTIAL CLASSIFICATION

Classification of downstream hazard potential has no relationship to the condition of the dam but rather is dependent on an evaluation of probable loss of life and damage downstream in the event of a dam failure. The hazard potential classification for each dam is determined by the Department in accordance with the following table. This assessment will be reevaluated periodically and revised as needed.

TABLE 1
HAZARD POTENTIAL CLASSIFICATION

CATEGORY	URBAN DEVELOPMENT	ECONOMIC LOSS
Low	No permanent structure for human habitation	Minimal (undeveloped to occasional structures or agriculture
Significant	No urban development and no more than a small number of habitable structures*	Appreciable (Notable agriculture, industry, or other structures)
High	Urban development with more than a small number of habitable structures*	Excessive (extensive community, industry, agriculture)

* Because this definition does not cite a specific number of lives that could be lost, some difficulty has been experienced in determining whether dams should be categorized as having "significant or high hazard potential". The issue is clarified by emphasizing that the hazard potential classification should be based on the density of downstream development containing habitable structures. For example, dams located upstream of isolated farmhouses would be classified as having significant hazard potential, and those located upstream of several houses or residential development would be classified as having high hazard potential.

SIZE CLASSIFICATION

Dams are classified into small, medium and large sizes. A numerical rating procedure, based on the descriptive characteristics of height and reservoir capacity has been developed to determine the dam size classification.

Height is measured from the lowest elevation of the outside limit of the dam (usually the downstream toe) to the spillway crest, or top of spillway gates if so equipped. For dams with no spillway, the height is measured to the crest of the dam.

Capacity, in acre-feet, is measured to the spillway crest or top of the spillway gates, if so equipped. For dams with no spillway, capacity is measured to the dam crest.

The categories and corresponding rating factors are shown below:

TABLE 2
SIZE CLASSIFICATION

CATEGORY Height (feet)	RATING FACTOR
6-24	0
25-39	1
40-59	2
60-79	3
80-99	4
100+	5

CATEGORY Reservoir Capacity (acre-feet)	RATING FACTOR
15-499	0
500-999	1
1,000-2,999	2
3,000-9,999	3
10,000-24,999	4
25,000+	5

A numerical rating is computed for each dam by adding the corresponding rating factors for each of the two categories. For example, a dam that is 65 feet in height and has a reservoir capacity of 22,000 acre-feet would have a rating of (3+4=7).

Small dams have a rating in the range 0-2, medium dams in the range 3-7 and large dams, 8 or greater.

REQUIRED HYDROLOGIC CRITERIA

An evaluation of the performance and capacity of an existing spillway or a hydrologic design study for a spillway at a proposed dam is required to determine the ability of the structure to safely pass a flood whose magnitude is established on the basis of the size and hazard potential classifications assigned to the dam.

The Inflow Design Flood (IDF) for a specific spillway is determined by the runoff hydrograph selected primarily on the basis of the size and hazard classifications assigned to the dam. As there are many factors to consider in the selection of the magnitude of this flood, it is not the purpose of these guidelines to require a specific flood frequency, volume or rainfall depth for each classification. However, the following table does provide ranges of flood magnitudes from which the Inflow Design Flood may be selected on the basis of the designated hazard potential and size classifications. These ranges of flood magnitudes generally define the limits acceptable to the Department of Water Resources for use as the basis for sizing the spillway.

TABLE 3
SPILLWAY CAPACITY REQUIREMENTS
RECOMMENDED SPILLWAY DESIGN FLOODS

HAZARD CATEGORY	SIZE DESIGNATION	INFLOW DESIGN FLOOD MAGNITUDE
Low	Small	100-year
	Medium	100-yr. to 1/2 PMF
	Large	1/2 PMF
Significant	Small	100-yr. to 1/2 PMF
	Medium	1/2 PMF
	Large	1/2 PMF to PMF
High	Small	1/2 PMF
	Medium	1/2 PMF to PMF
	Large	PMF

The flood magnitudes shown in the above table are derived from rainfall depths for various durations and severities of storms. Both general frontal and thunderstorm type storms should be studied with due consideration given to tropical storm potential and orographic influences that may greatly increase rainfall amounts.

Recorded rainfall and flood flows in Arizona are rather sparse, and the period of record is usually short. Consequently, rainfall data are usually obtained from data published by the National Weather Service as listed in the References. Synthetic flood hydrographs are than developed by modeling the watershed's rainfall/runoff response and employing the unit hydrograph approach.

The peak inflow flow rate usually has a greater influence than the runoff volume on the spillway capacity requirement for a dam with a small reservoir storage that is subject to storm inflow from a large watershed. In this case, the Inflow Design Flood (IDF) peak flow is essentially equal to the peak outflow rate. Conversely, a reservoir that is relatively large compared to contributing watershed will usually attenuate the IDF peak; in this case, the spillway peak discharge may be considerable less than the IDF peak.

A spillway capacity less than outlined above will be acceptable, for (1) all new dams, (2) existing dams which are being enlarged or improved, and (3) dams being reevaluated for safety, where the owner (or his engineer) can demonstrate to the Department that the incremental damages due to failure of the dam are insignificant and will not cause loss of life. The analysis shall be based upon the dam failure caused by a flood which just exceeds the routing capacity of the reservoir. The result shall be compared to the pre-failure conditions such as the spillway discharge and any reasonable rainfall runoff occurring between the dam site and the point(s) of interest below the dam. The burden of proof rests with the owner.

RESERVOIR ROUTING REQUIREMENTS

The adequacy of the spillway for an existing dam is normally determined by routing the Inflow Design Flood through the reservoir and spillway. Flood routings for spillway capacity determinations will normally be required to commence with the reservoir storage level at the spillway crest elevation. Infrequent exceptions would be: (1) normal conservation storage level is below the spillway crest of a reservoir without a flood storage pool, (2) the normal upper surface of the conservation pool is limited to a level that is coincident with the bottom level of the flood control pool allocation or (3) the reservoir is used exclusively for flood control and would normally be empty. Deviations from the normal starting level of routing at the spillway crest elevation must be considered on the basis of risk and reservoir operating procedure.

FREEBOARD REQUIREMENTS

Total freeboard (the distance between the top of the dam and the spillway crest) is determined by the type of dam, the maximum water surface during discharge of the Inflow Design Flood, maximum anticipated wave height and runup, and by economic factors. The minimum permissible total freeboard shall be four feet.

Residual freeboard (the distance between the maximum water surface and the top of the dam) depends on dam type, wave height and runup, the slope and finish of the upper part of the upstream face, and the Inflow Design Flood. Generally, the minimum permissible residual freeboard for an earthfill or rockfill dam shall be the greater of either the sum of wave height and runup or three feet. This requirement may be reduced in those cases where the Inflow Design Flood is the 1/2 PMF or greater.

The minimum residual freeboard for a concrete dam of any type without either a parapet wall or protection against overpour shall be the same as that of an earthfill or rockfill dam.

Concrete dams provided with parapet walls exceeding the minimum residual freeboard height, or concrete dams provided with adequate splash impact protection at the toe, need no other residual freeboard requirements except those which the owner may wish to provide.

DEFINITIONS

The following definitions may be helpful to those concerned with the design of an emergency spillway. The terminology is largely based on data published by Federal agencies.

100-Year Flood - The flood runoff whose magnitude is expected to be equaled or exceeded, on the average, once in 100 years. Stated another way, it is a flood that has a one percent change of being equaled or exceeded in any one year.

Concrete Dam - Any dam constructed of concrete. Some examples are: arch, gravity, arch-gravity, slab and buttress, multiple arch. A dam having only a concrete facing should not be referred to as a concrete dam.

Drainage Area - The area that drains naturally to a particular point on a river or stream.

Earth Dam (Earthfill Dam) - An embankment dam in which more than 50% of the total volume is formed of compacted fine-grained material obtained from a borrow area.

Embankment Dam (Fill Dam) - Any dam constructed of excavated natural materials or of industrial waste materials.

Fetch - The straight line distance between a dam and the farthest reservoir shore. The fetch is one of the factors used in calculating wave heights in a reservoir.

Flood - The runoff from rainfall or snowmelt of significant magnitude and often related to a theoretical frequency of occurrence. Flood is inflow to the water control structure.

Flood Routing - The determination of the attenuating effects of storage on a flood passing through a valley, channel, or reservoir.

Hydrograph - A graphical representation of discharge, stage, or other hydraulic property with respect to time for a particular point on a stream. (At times the term is applied to the phenomenon the graphical representation describes: hence a flood hydrograph is the passage of flood discharge past the observation point).

Inflow Design Flood (IDF) - The reservoir flood inflow whose magnitude has been selected for design requirements based on the size and assigned hazard classification of the dam. The magnitude of the IDF may range from the 100-year flood to the PMF.

Masonry Dam - Any dam constructed mainly of stone, brick, or concrete blocks that may or may not be joined with mortar. A dam having only a masonry facing should not be referred to as a masonry dam.

Maximum Water Surface (MWS) - The maximum elevation of the reservoir water surface attained during routing of the Inflow Design Flood (IDF).

Normal Water Surface (NWS) - The storage level at which the reservoir is usually operated. This level is usually at or below the spillway crest, except in the few instances where the storage level is normally maintained above the spillway crest by means of gates or flashboards.

Outlet Works - A closed channel under the dam or through an abutment for the discharge of water. An outlet works may be controlled or uncontrolled. An outlet works is subject to plugging by debris and is generally not eligible for classification as a spillway.

Parapet Wall - A solid wall built along the top of a dam for ornament, for the safety of vehicles and pedestrians, or to prevent overtopping.

Peak Flow - The maximum instantaneous discharge that occurs during a flood. It is coincident with the peak of a flood hydrograph.

Probable Maximum Flood (PMF) - The flood runoff that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region.

1/2 PMF - That flood hydrograph with ordinates equal to one-half the corresponding ordinates of the Probable Maximum Flood Hydrograph.

Probable Maximum Precipitation (PMP) - The precipitation depth which generates the PMF.

Reservoir Capacity - The storage capacity of the reservoir when the storage level is at the crest of the spillway, or at the top of permanently mounted spillway gates in closed position. For dams with no spillway the capacity is measured to the dam crest.

Reservoir Routing - The computation by which the interrelated effects of the inflow hydrograph, reservoir storage, and discharge from the reservoir are evaluated.

Residual Freeboard - The vertical distance between the maximum water surface elevation and the minimum dam crest elevation.

Rockfill Dam - An embankment dam in which more than 50% of the total volume comprises compacted or dumped pervious natural or crushed rock.

Spillway - A structure over or through which flood flows are discharged. If the flow is controlled by gates, it is considered a controlled spillway; if the elevation of the spillway crest is the only control, it is considered an uncontrolled spillway.

Spillway Design Hydrograph (SDH) - The routed outflow flood derived from the Inflow Design Flood (IDF). In some cases the IDF and the SDF hydrographs are essentially identical; however, the SDF hydrograph will usually have a lower peak discharge value because of attenuation of the IDF peak due to reservoir routing. The SDH peak discharge is the maximum discharge capacity of the spillway.

Surcharge Storage - The storage volume above the spillway crest.

Total Freeboard - The vertical distance between the spillway crest and the crest of the dam.

REFERENCES

1. Arizona Compilation of Rules and Regulations, R12-15-151, R12-15-152, and R12-15-1201 to R12-15-1206.

2. Arizona Revised Statutes, Title 45, Chapter 3, Aricle I.

3. Atlas 2, Precipitation-Frequency Atlas of the Western United States, Volume VIII-Arizona, U.S. Department of Commerce, National Oceanic and Atmospheric Administration, 1973.

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5. Design of Small Dams, 2nd Edition, U.S. Department of the Interior, Bureau of Reclamation, Revised 1977.

6. EM 1110-2-1411, Standard Project Flood Determinations, U.S. Army Corps of Engineers, 1952.

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8. HEC-1, Flood Hydrograph Package, U.S. Army Corps of Engineers, Revised October 1981.

9. HEC-2, Water Surface Profiles, U.S. Army Corps of Engineers, Revised March 1982.

10. Hydrometeorological Report No. 5, Thunderstorm Rainfall, U.S. Weather Bureau, 1947.

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14. Instructions for Filing an Application, Arizona Department of Water Resources, May 1985.

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AZ. DEPT. OF WATER RESOURCES DAMS UNDER JURISDICTION NOV, 5, 1986

DAM NO.	DAM NAME	SECTION	TOWNSHIP N or S	RANGE E or W	HEIGHT-FT	CAPACITY-AF	TYPE
1.001	UDALL	21	14 N	27 E	24	5000	EARTH
1.002	CONCHO SPRINGS	7	12 N	26 E	20	918	EARTHROCK
1.003	LYMAN	9	11 N	28 E	65	30600	EARTHROCK
1.005	TYLER RESERVOIR	7	8 N	27 E	9	375	EARTH
1.007	NORTON	29	8 N	27 E	10	232	EARTH
1.008	ELLIS WILTBANK	14	8 N	27 E	12	205	EARTH
1.010	BUNCH RESERVOIR	36	8 N	27 E	22	512	EARTH
1.011	NELSON	28	8 N	30 E	34	770	EARTH
1.012	WHITE MOUNTAIN	17	7 N	27 E	16	2390	EARTH
1.014	RIVER RESERVOIR #3	6	7 N	28 E	58	2146	EARTHROCK
1.015	HOG WALLOW	19	7 N	28 E	7	1000	EARTH
1.016	POOL CORRAL	29	7 N	28 E	12	990	EARTH
1.018	ATCHISON	13	7 N	28 E	8	205	EARTH
1.019	CANYON	13	7 N	28 E	17	65	EARTH
1.020	MEXICAN HAY LAKE	2	7 N	28 E	9	821	EARTH
1.021	EAGAR-SLADE	18	7 N	29 E	18	522	EARTH
1.024	GLEN LIVET	6	7 N	30 E	18	110	EARTH
1.025	GLEN COE	1	7 N	29 E	14	81	EARTH
1.026	JARVIS	20	7 N	30 E	7	120	EARTH
1.027	NUTRIOSO	31	7 N	30 E	6	145	EARTH
1.028	LEE VALLEY	4	6 N	27 E	20	399	EARTH
1.029	RIVER #1	3	6 N	27 E	19	724	EARTH
1.030	CRESCENT LAKE	18	6 N	28 E	22	5800	EARTH
1.031	BIG LAKE	29	6 N	28 E	24	9300	EARTH
1.033	ROGERS	3	6 N	29 E	12	183	EARTH
1.036	LUNA	16	6 N	31 E	27	1390	EARTH
1.043	SHEEP SPRINGS	31	8 N	27 E	28	393	EARTH
1.044	BOYNTON LAKE	4	9 N	26 E	21	98	EARTH
1.046	CORONADO GENERATING STATION	5	13 N	29 E	53	5600	EARTH
1.048	SPRINGERVILLE GEN. STA. ABF #1	36	11 N	29 E	27	122	EARTH
2.002	PARKER CANYON	18	23 S	19 E	80	3710	EARTH
3.001	KAIBAB	15	22 N	2 E	36	890	EARTHROCK
3.002	WEST CATARACT CREEK	30	22 N	2 E	36	415	EARTH
3.004	RAILROAD	33	22 N	2 E	42	215	MASONRY
3.006	STEEL #1	5	21 N	1 W	30	97	STEEL
3.007	MASONRY #2	4	21 N	1 W	48	247	MASONRY
3.008	McLELLAN	4	21 N	1 E	24	60	GRAVITY
3.009	CITY	4	21 N	2 E	35	111	EARTH
3.010	DOGTOWN	12	21 N	2 E	32	1129	EARTH
3.013	WILLOW SPRINGS	29	11 N	14 E	80	3654	EARTHROCK
3.017	LOWER LAKE MARY	17	20 N	8 E	35	8600	EARTH
3.018	UPPER LAKE MARY	27	20 N	8 E	38	16576	EARTH
3.019	MORTON	36	18 N	10 E	12	285	EARTH
3.022	HAY LAKE	19	16 N	11 E	26	5500	EARTH
3.023	COCONINO	25	19 N	9 E	30	255	EARTH
3.024	KINNIKINICK	35	18 N	10 E	13	2820	EARTH
3.025	WOODS CANYON	13	11 N	13 E	43	1014	EARTH
3.027	KNOLL	16	12 N	12 E	58	1550	EARTH
3.030	BLUE RIDGE	33	14 N	11 E	150	15000	ARCH
3.034	BEAR CANYON	29	12 N	13 E	50	1430	EARTH
3.039	CHEVELON CANYON	14	13 N	14 E	84	7000	EARTHROCK
3.042	ASHURST LAKE	13	19 N	9 E	8	3924	EARTH
3.043	FREDONIA	13	41 N	2 W	23	1136	EARTH
3.044	CONTINENTAL #1	18	21 N	8 E	32	71	EARTH
3.045	CONTINENTAL #2	19	21 N	8 E	36	611	EARTH
3.046	CONTINENTAL #3	19	21 N	8 E	42	195	EARTH
3.047	ODELL	22	18 N	7 E	14	206	EARTH
3.048	NAVAJO EVAPORATION POND 60-2	35	41 N	9 E	50	606	EARTH
3.049	NAVAJO EVAPORATION POND NE-1	36	41 N	9 E	33	384	EARTH
4.010	GOLD GULCH	24	1 N	13 E	72	370	EARTH
4.012	RAFFINATE	24	1 N	13 E	95	23	EARTH
4.013	GOLD GULCH #2	24	1 N	13 E	99	420	EARTHROCK
4.014	ASARCO 82	14	5 S	15 E	43	113	EARTH

5.002	ROGERS RESERVOIR	25	6 S	24 E	24	412	EARTH
5.004	CLUFF RANCH #3	23	7 S	24 E	29	140	EARTH
5.005	RIGGS FLAT	26	8 S	23 E	45	123	EARTH
5.006	CENTRAL DETENTION	4	7 S	25 E	20	539	EARTH
5.007	FRYE CREEK	7	8 S	25 E	91	135	ARCH
5.008	RIGGS RESERVOIR	34	7 S	25 E	31	39	EARTH
5.009	LEBANON RESERVOIR #2 (LOWER)	13	8 S	25 E	22	145	EARTH
5.010	LEBANON RESERVOIR #1 (UPPER)	14	8 S	25 E	40	96	EARTH
5.014	JUDY WASH RETARDING	20	7 S	27 E	21	159	EARTH
5.016	GRAVEYARD WASH RETARDING	30	7 S	26 E	26	1270	EARTH
5.017	FREEMAN WASH RETARDING	24	7 S	25 E	21	400	EARTH
5.018	STOCKTON WASH RETARDING	28	7 S	26 E	31	6704	EARTH
5.019	FRYE CREEK RETARDING #3	14	7 S	25 E	29	1900	EARTH
5.021	ROPER	8	8 S	26 E	17	313	EARTH
5.022	SAVAGE	27	7 S	25 E	30	141	EARTH
5.023	HARALSON	34	7 S	25 E	29	73	EARTH
5.024	GRANT MORRIS	25	6 S	25 E	43	194	EARTH
5.025	HOWARD	23	6 S	25 E	44	171	EARTH
5.026	CHESLEY-WAMSLEE	16	6 S	25 E	41	1276	EARTH
5.027	FOOTE WASH	26	7 S	25 E	38	2782	EARTH
5.028	NO NAME WASH	27	7 S	26 E	22	251	EARTH
5.029	LEE	6	6 S	25 E	60	211	EARTH
5.030	INDIAN FARMS	27	5 S	25 E	49	161	EARTH
5.031	BILLINGSLEY	1	6 S	24 E	62	2234	EARTH
6.002	SILVER BASIN	33	4 S	29 E	140	5200	EARTHROCK
6.003	TAILINGS WATER RECLAIM	11	5 S	29 E	45	17	EARTH
6.005	COLUMBINE	26	4 S	29 E	42	537	EARTH
7.017	CAVE CREEK	4	4 N	3 E	53	11000	MULTIPLE
7.021	McMICKEN	13	4 N	2 W	23	16800	EARTH
7.022	WADDELL	21	6 N	1 E	170	157590	MULTIPLE
7.023	CAMP DYER DIVERSION	28	6 N	1 E	35	537	GRAVITY
7.024	GILLESPIE	28	2 S	5 W	21	3600	MULTIPLE
7.028	WHITE TANKS #3 FRS	8	2 N	2 W	21	2655	EARTH
7.029	WHITE TANKS #4 FRS	6	1 N	2 W	14	1036	EARTH
7.030	LITCHFIELD PARK FRS	15	2 N	1 W	6	150	EARTH
7.031	FOUNTAIN HILLS	14	3 N	6 E	31	322	EARTH
7.032	FOUNTAIN HILLS #7 FRS	17	3 N	6 E	40	110	EARTH
7.033	FOUNTAIN HILLS #4 FRS	10	3 N	6 E	20	150	EARTH
7.035	WEST PARK FRS	20	3 N	3 E	29	109	EARTH
7.036	EAST PARK FRS	29	3 N	3 E	26	21	EARTH
7.038	FOUNTAIN HILLS #36 FRS	4	3 N	6 E	36	284	EARTH
7.039	FOUNTAIN HILLS #6 FRS	4	3 N	6 E	25	164	EARTH
7.040	FOUNTAIN HILLS #11 FRS	9	3 N	6 E	34	165	EARTH
7.041	FOUNTAIN HILLS #19 FRS	22	3 N	6 E	24	72	EARTH
7.042	BUCKEYE FRS #1	3	1 N	5 W	26	8195	EARTH
7.043	GUADALUPE	8	1 S	4 E	27	298	EARTH
7.044	BUCKEYE FRS #2	7	1 N	3 W	16	780	EARTH
7.045	BUCKEYE FRS #3	10	1 N	3 W	21	920	EARTH
7.046	NORTH MTN. FLOOD DETENTION #2A	16	3 N	3 E	21	59	EARTH
7.047	NORTH MTN. FLOOD DETENTION #3	22	3 N	3 E	22	66	EARTH
7.048	SUNNYCOVE FRS	11	7 N	5 W	40	219	EARTH
7.049	SUNSET FRS	11	7 N	5 W	20	55	EARTH
7.050	SPOOKHILL FRS	31	2 N	7 E	15	992	EARTH
7.051	DETENTION BASIN #7	17	3 N	3 E	23	120	EARTH
7.052	SADDLEBACK FRS	21	2 N	8 W	21	7600	EARTH
7.053	HARQUAHALA FRS	31	3 N	8 W	38	8000	EARTH
7.054	PVNGS EVAPORATION POND	4	1 S	6 W	30	6200	EARTH
7.055	NEW RIVER FRS	35	5 N	1 E	74	43520	EARTH
7.056	DREAMY DRAW FRS	20	3 N	3 E	40	317	EARTH
7.057	ADOBE FRS	21	4 N	2 E	40	15650	EARTH
7.058	CAVE BUTTES FRS	4	4 N	3 E	99	46600	EARTH
7.059	THUNDERBIRD PARK RESERVOIR	18	4 N	2 E	44	37	EARTH
7.060	SIGNAL BUTTES FRS	12	1 N	7 E	17	1365	EARTH
7.061	APACHE JUNCTION FRS	8	1 N	8 E	22	500	EARTH

8.002	JOSHUA CROSBY	27	35 N	9 W	14	52	EARTH
8.008	W. F. CATTLE COMPANY	9	24 N	12 W	23	220	EARTH
8.009	SHORT CREEK SOUTHSIDE #2	32	42 N	6 W	33	40	EARTH
8.010	SHORT CREEK SOUTHSIDE #1	5	41 N	6 W	33	60	EARTH
9.003	CLEAR CREEK #1	10	18 N	16 E	7	350	MASONRY
9.004	FIVE MILE WASH	29	17 N	21 E	25	15	EARTH
9.005	WOODRUFF	32	16 N	22 E	25	15	EARTH
9.007	MILLETT SWALE	20	12 N	22 E	24	1550	EARTH
9.009	LONE PINE	14	11 N	21 E	98	10800	EARTH
9.011	DAGGS	10	11 N	22 E	57	5160	EARTHROCK
9.013	JAQUES	10	9 N	22 E	65	6200	EARTH
9.014	SCOTT	13	9 N	22 E	40	1200	EARTH
9.016	LAKESIDE	23	9 N	22 E	21	1200	EARTHROCK
9.018	WOODLAND	31	9 N	23 E	18	90	EARTH
9.019	FOOL HOLLOW	12	10 N	21 E	60	3217	EARTHROCK
9.020	BLACK CANYON	24	11 N	15 E	60	1581	EARTH
9.021	LAKE OF THE WOODS	23	9 N	22 E	10	83	EARTHROCK
9.027	CHOLLA BOTTOM ASH POND	13	18 N	19 E	70	2300	EARTH
9.028	CHOLLA FLY ASH POND	30	18 N	20 E	80	18000	EARTH
9.029	CHOLLA COOLING POND	26	18 N	19 E	13	2200	EARTH
9.030	TROPHY LAKE	7	11 N	22 E	33	105	EARTH
9.031	CLEAR CREEK #2	10	18 N	16 E	12	498	EARTH
9.032	TWIN LAKES	7	14 N	19 E	13	4380	EARTH
9.033	SCHOENS	23	12 N	21 E	108	23400	EARTHROCK
10.002	ROCKING K RANCH ESTATES	16	15 S	16 E	25	115	EARTH
10.006	GOLDER #2	9	11 S	14 E	8	60	EARTH
10.007	DUVAL LEACH FLOOD #1	15	18 S	12 E	36	135	EARTH
10.008	LOWER ROSE CANYON	16	12 S	16 E	41	117	GRAVITY
10.012	ARIVACA	7	22 S	11 E	30	1037	EARTH
10.013	KENNEDY PARK	27	14 S	13 E	18	90	EARTH
10.014	SABINO CANYON RESERVOIR	20	13 S	15 E	60	47	EARTH
10.015	GREEN VALLEY	36	17 S	13 E	12	219	EARTH
11.002	POWERLINE FRS	5	1 S	8 E	32	4194	EARTH
11.005	MAGMA RETARDING	35	3 S	9 E	21	4960	EARTH
11.006	FLORENCE RETARDING	21	4 S	10 E	21	5010	EARTH
11.011	VINEYARD ROAD FRS	9	1 S	8 E	16	4310	EARTH
11.012	RITTENHOUSE FRS	2	2 S	8 E	17	4060	EARTH
11.013	MINERAL CREEK ARCH FRS	35	2 S	13 E	155	9580	ARCH
12.001	ORO BLANCO	24	22 S	10 E	18	66	EARTH
12.005	PENA BLANCA	23	23 S	12 E	60	975	EARTHROCK
12.006	LAKE PATAGONIA	25	22 S	14 E	82	7540	EARTH
12.008	KINO SPRINGS	6	24 S	15 E	19	145	EARTH
13.001	CANYON MOUTH	24	22 N	7 W	22	400	EARTH
13.002	RAILROAD EMBANKMENT	35	23 N	6 W	30	840	EARTH
13.003	PAN	9	22 N	5 W	56	703	MASONRY
13.004	WILLISCRAFT	22	21 N	3 W	25	58	EARTH
13.005	FORT ROCK RANCH	9	20 N	10 W	16	110	EARTH
13.013	WILLOW CREEK	11	14 N	2 W	65	6900	ARCH
13.014	GRANITE CREEK	13	14 N	2 W	81	4598	ARCH
13.015	LYNX CREEK	22	14 N	1 W	59	113	ARCH
13.016	MESA RESERVOIR	22	14 N	1 W	25	131	EARTH
13.017	LOWER GOLDWATER	15	13 N	2 W	67	212	GRAVITY
13.018	UPPER GOLDWATER	15	13 N	2 W	52	552	BUTTRESS
13.019	HASSAYAMPA CHECK	36	12 N	2 W	40	43	GRAVITY
13.020	LYNX LAKE	8	13 N	1 W	70	1472	EARTHROCK
13.021	BILLINGSLEY #2	11	10 N	6 W	28	187	EARTH
13.022	BILLINGSLEY #3	35	11 N	6 W	23	350	EARTH
13.026	BILLINGSLEY #4	11	10 N	6 W	10	103	EARTH
13.027	STEHR LAKE	20	11 N	7 E	19	287	EARTH
13.034	SLURRY POND #1	8	22 N	7 W	32	33	EARTH
13.035	BOULDER FLOOD BASIN	29	5 N	9 W	45	128	EARTH
13.036	MAMMOTH WASH DETENTION	12	14 N	10 W	120	890	EARTH
13.038	JUMBO	18	21 N	2 W	40	114	EARTH

ARIZONA DEPARTMENT OF WATER RESOURCES

CHECKLIST FOR
EVALUATION OF OPERATIONAL DAM

Name of Dam, Dam No., Reservoir Storage Level, Freeboard and Contacts.

MAIN STRUCTURE:

CONCRETE STRUCTURE:

CONCRETE CONDITIONS: See A.C.I. Guide for condition survey of concrete. Cracks, disintegration, efflorescence, exudation, pitting, popout, erosion, scaling, spalls, drummy, corrosion, chemical attack, stains. Deterioration, continuing serviceability of concrete surface. Location and description of structural cracking, differential movements. Settlement, heaving, deflection, junctions. Drains foundation, joint, face functioning properly? Flowrate. Seepage: Flowrate, increasing? Detrimental? Monolith and construction joints: movement, distress, leakage, filler. Foundation Conditions.

EMBANKMENT:

CONDITION OF EMBANKMENT: Damage by erosion, rodents, livestock; sloughing, sinkholes or spalling of dam or abutments, slope protection. Is crest level? Slope stability: Are slopes regular? Areas of settlement. Undesirable vegetation. Leakage: Location, nature, and flowrate. Saturated embankment or abutment, location. Drainage system, nature and rate of flow. Instrumentation: Survey records, piezometers, etc. Cracking: Transverse, longitudinal, location, station, upstream or downstream slope, crest, dimensions, length, depth, width, cause, dessication, settlement, other. Compare with previous conditions.

OUTLET WORKS:

GENERAL GATE: Size, type, location, open or closed during inspection? Operable? Last operated. Condition of approach, air vents, trashrack, stilling basin, outlet channel. Conduit: (Size), corrosion, cavitation, cracking, joint separation leakage, erosion.

CONCRETE STRUCTURES: See A.C.I. Guide for condition survey of concrete: Cracks, scaling, spalls, popouts, corrosion, chemical attack, stains.

SPILLWAY:

Type of spillway - Condition of approach, control section, channel, stilling basin, outflow channel - Gates, open, closed, operable, flashboards, in, out, loss of freeboard, undermining, debris, obstruction, scour, condition of lining, drains (quantity), does capacity appear adequate?

REMARKS AND RECOMMENDATIONS:

CHANGES SINCE LAST INSPECTION: Reservoir, shoreline, landslide areas, upstream hazards (flooding), downstream hazards, owner, staff, operation, unusual developments and events, high water for period. Normal operation.

OVERALL CONDITION OF DAM: Excellent, good, fair, poor - Needed improvements or studies - by, (date) - Comments to owner. Is hazard classification correct? Inspection frequency? Suggest approximate date of next inspection. Date - Photos.

PARTIAL LIST OF ARIZONA CONSULTANTS
WITH EXPERIENCE IN DAM CONSTRUCTION

Black & Veatch Engineers & Architects
3020 E. Camelback Rd., Suite 155
Phoenix, Arizona 85016
Phone No.: (602)-957-2795

John Carollo Engineers
1314 N. 3rd St., Suite 300
Phoenix, Arizona 85004
Phone No.: (602)-257-9200

Coen Engineering Corp.
930 W. Birchwood
Mesa, Arizona 85202
Phone No.: (602)-835-1111

Coe & Van Loo
4550 N. 12th Street
Phoenix, Arizona 85014
Phone No.: (602)-264-6831

Dames & Moore
Pointe Corporate Center
7500 N. Dreamy Draw, Suite 145
Phoenix, Arizona 85020
Phone No.: (602)-371-1110

DMJM/Adam, Hamlyn, Anderson
300 W. Clarendon, Suite 400
Phoenix, Arizona 85013
Phone No.: (602)-264-1397

Franzoy & Corey
Engineers & Architects
5030 E. Sunrise Drive
Phoenix, Arizona 85044
Phone No.: (602)-838-8626

Geological Consultants
2822 W. Northern Ave., Suite B
Phoenix, Arizona 85021
Phone No.: (602)-864-1888

Harza Engineering Co.
5025 E. Washington St., Suite 120
Phoenix, Arizona 85034
Phone No.: (602)-244-0992

HDR Infrastructure
100 W. Clarendon, Suite 1222
Phoenix, Arizona 85013
Phone No.: (602)-264-0731

Robert R. Koons, P.E., Inc.
4645 S. Lakeshore Drive, Suite 7
Tempe, Arizona 85282
Phone No.: (602)-820-3104

Leedshill-Herkenhoff, Inc.
Airport Center
120 N. 44th St., Suite 300
Phoenix, Arizona 85034
Phone No.: (602)-267-8894

Wayne Linthacum, P.E.
3002 E. Montecito Ave.
Phoenix, Arizona 85016
Phone No.: (602)-955-9174

MK Engineering, Inc.
10240 N. 31st Ave., Suite 213
Phoenix, Arizona 85021
Phone No.: (602)-997-4050

PRC Engineering
4131 N. 24th St., Suite 110
Phoenix, Arizona 85016
Phone No.: (602)-954-9191

Sergent, Hauskins & Beckwith
3940 W. Clarendon
Phoenix, Arizona 85019
Phone No.: (602)-272-6848

The Earth Technology Corporation
3116 W. Thomas Rd., Suite 601
Phoenix, Arizona 85017
Phone No.: (602)-269-7501

Western Technologies
663 W. Second Ave.
Mesa, Arizona 85202
Phone No.: (602)-834-3964

NOTE: THE DEPARTMENT DOES NOT GUARANTEE THE COMPLETENESS OF THIS LIST. FURNISHING OF THIS LIST SHALL NOT BE CONSIDERED AS RECOMMENDATION OR ENDORSEMENT OF ANY OR ALL OF THE FIRMS BY THE DEPARTMENT OF WATER RESOURCES.

9/25/86 copy with changes made / VJ

STATE OF ARIZONA

EMERGENCY SERVICES/CIVIL DEFENSE DIRECTORS/COORDINATORS

State of Arizona Division of Emergency Services Director, Mr. Richard A. Colson	5636 E. McDowell Rd. Phoenix, AZ 85008	244-0504 (Off) 955-2049 (Res)	

APACHE COUNTY	CO-DIRECTORS:		
	C. Art Lee Sheriff	P.O. Box 518 St. Johns, AZ 85936	337-4364 X 290 (Off) 337-4996 (Res)
	Clarence Bigelow County Manager	P.O. Box 428 St. Johns, AZ 85936	337-4364 X 204 (Off) 333-2176 (Res)
	COORDINATORS:		
	Monty M. Stansbury Coordinator	P.O. Box 238 St. Johns, AZ 85936	337-4364 X 295 (Off) 333-2075 (Res)
	Archie WATKins ~~Bob Gilchrist~~ Undersheriff	P.O. Box 518 St. Johns, AZ 85936	337-4364 X 290 (Off) ~~337-4683 (Res)~~
Eagar	Gordon C. Henrie ES Director	P.O. Box 78 Eagar, AZ 85925	333-4128 (Off) 333-2990 (Res)
	Kathleen Stewart ES Assistant	P.O. Box 78 Eagar, AZ 85925	333-4128 (Off) 333-2030 (Res)
COCHISE COUNTY	Sam Rumore Director	P. O. Box 696 Bisbee, AZ 85603	432-5703 X 496 (Off) 378-6669 (Res)
	John N. (Dutch) Schultz Deputy Director	P.O. Box 696 Bisbee, AZ 85603	432-5703 X 497 (Off) 458-2212 (Res)
Benson	George McMinimy Director	P. O. Box 2228 Benson, AZ 85602	586-2211 (Off) 586-3956 (Res)
Bisbee	James Busk Director	306 Purdy Lane Bisbee, AZ 85603	432-7127 (Off) 432-4756 (Res)
Douglas	Edward J. Greenough Director	1400 - 10th St. Douglas, AZ 85607	364-2481 (F.D.) 364-7721 (Res)
Huachuca City	Terry McGriff Director	500 N. Gonzales Blvd. Huachuca City, AZ 85616	456-1354 (Off) 456-1388 (Res)

STATE OF ARIZONA

EMERGENCY SERVICES/CIVIL DEFENSE
DIRECTORS/COORDINATORS

Sierra Vista	Captain Bruce Thompson	1327 Fry Blvd. Sierra Vista, AZ 85635	458-3319 (Off) 456-1512 (Res)
Tombstone	George Kruse City Clerk	P. O. Box 339 Tombstone, AZ 85638	457-3562/2202 (Off) 457-3721 (After Hours)
Willcox	Bill Morales	151 West Maley Willcox, AZ 85643	384-4271 X 221 (Off) 384-2894 (Res)
COCONINO COUNTY	Joe Richards, Sheriff Acting Director	P. O. Box 39 Flagstaff, AZ 86002	774-4523 (Off) 526-2212 (Res)
	Dave McPherson Coordinator	Kachina Village Fire Dept. 568 Kona Trail Kachina Village, AZ 86001	525-1717 (Off) 525-1323 (Res)
FlagSTAFF	DENNIS MARTIN Fire MARSHAL		774-5281 (OFF) 774-1538 (RES)
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Page	VACANT		
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Williams	Cary Price, Fire Chief ~~Joe Richards, Sheriff~~ Acting Director	RT1, Box 142 ~~P. O. Box 39~~ ~~Flagstaff, AZ 86002~~ Williams, AZ 86046	635-2633 (off) ~~774-4523 (Off)~~ ~~526-2212 (Res)~~ 635-4809 (Res)
GILA COUNTY	Carmen Corso Director	County Courthouse 1400 E. Ash St. Globe, AZ 85501	425-3231 X 360 (Off) 425-4745 (Res)
Globe	Carmen Corso Director	County Courthouse 1400 E. Ash St. Globe, AZ 85501	425-3231 X 360 (Off) 425-4745 (Res)
Hayden	Daryl Reinertson Coordinator	520 Ray Ave. Hayden, AZ 85325	356-6205 (Off) 356-6098 (Res)

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	Tom Candelaria Administrator	County Courthouse/Box 908 Clifton, AZ 85533	865-2072 (Off) 687-1300 (Res)
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	William Lamb C. D. Coordinator	7022 N. 58th Dr. Glendale, AZ 85301	931-5614 (Off)
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Guadalupe	Vacant C. D. Director	8413 S. Avenida del Yaqui Guadalupe, AZ 85283	820-9193 (Off)
Mesa	Bruce L. Solomon C. D. Coordinator	55 N. Center St. Mesa, AZ 85201	834-2290 (Off) 969-8879 (Res)
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Capt. Terry Meissner	832 CSG/DW Luke AFB, AZ 85309-5000	856-7386 (Off) 843-5760 (Command Post)
Capt. Steven Spies	82 ABG/XPRD Williams AFB, AZ 85224	988-6689/988-6545 (Off) 969-7461 (Toll Free Switchboard) 981-6892/981-2269 (Res)

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Kingman	Lou Sorenson Director	310 N. 4th St. Kingman, AZ 86401	753-5561 (Off) 753-5244 (Res)
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	Ed Koury Director of Admin.	Navajo County Gov. Complex P. O. Box 668 Holbrook, AZ 86025	524-6161 X 405 (Off) (24 hr.)
Heber/ Overgaard	Charles Settle Fire Chief	P. O. Box 200 Overgaard, AZ 85933	535-4346 (F.D.) 535-4315 (Res)
Holbrook	George E. "Teen" DeSpain	P. O. Box 70 Holbrook, AZ 86025	524-6225 (Off) 534-3802 (Res)
Pinetop/ Lakeside	R. W. Arthur	Town of Pinetop - Lakeside Police Department P. O. Box 1429D Lakeside, AZ 85935	367-5835 (24 hr)
Show Low	VACANT		
Snowflake	John Stewart	P. O. Box AE Snowflake, AZ 85937	536-7688 (P.D.) 536-7557 (Res)
Taylor	Gerald Gullick	Taylor Town Hall P. O. Box 158 Taylor, AZ 85939	536-7366 (Off) 536-4966 (Res)
Winslow	Melvin Alkire	215 Taylor Winslow, AZ 86047	289-2091 (Off) 289-2722 (Res)

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Green Valley	VACANT Coordinator	Green Valley Dist. 601A N. LaCanada Green Valley, AZ 85614	625-0139 (Off) 625-0607 625-0856
Oro Valley	Werner Wolff Coordinator (Chief of Police)	680 W. Calle Concordia Oro Valley, AZ 85704	742-5445 (Off) (742-5229) **EM ONLY DO NOT GIVE OUT**
South Tucson	George Felix Director	South Tucson Fire Dept. P. O. Box 7307 South Tucson, AZ 85713	622-3309 (Off) 886-7011 (Res)
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Casa Grande	Leo Hall Coordinator	520 N. Marshall Casa Grande, AZ 85222	836-8156 (Off) 836-8445 (Res)
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Apache Junction	H. Reed Cox Coordinator	1001 N. Idaho Rd Apache Junction, AZ 85220	982-8260 (Off)

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	~~Tom Sauser~~ Assistant Director	26 East Camino de Aramonte Rio Rico, AZ 85621	629-6546 (Off) 1-800-231-2838 (24 hours) 281-8223 (Res)
	TIM BEITEL ~~Robert Linnemann~~ Coordinator	P.O. BOX 4017 ~~1505 Pendleton Dr.~~ ~~Rio Rico, AZ 85621~~ TUBAC, AZ 85640	011-526-31-2-7610 ~~281-8585~~ (Off) 281-8491 (Res)
~~Nogales~~	JOKEY SPENCER ~~Jose de la Ossa~~ Coordinator	P.O. BOX 322 ~~1018 Grand Ave.~~ ~~Nogales, AZ 85621~~ ELGIN, AZ 85611	455-5524 ~~287-6548~~ (Off) ~~281-9137~~ (Res) 455-5638
Patagonia	William Bergier Coordinator	P. O. Box 146 Patagonia, AZ 85624	458-9540 (Off) 394-2026 (Res)
Tubac	Bill Grubb Coordinator	P.O. Box 2881 Tubac, AZ 85646	398-2255 (Off) 398-2600 (24 hours)
YAVAPAI COUNTY	Harry Stevens Director	Courthouse Plaza Prescott, AZ 86301	445-7450 (Off) 445-9733 (Res)
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Chino Valley	Joe Amore Director	P.O. Box 406 Chino Valley, AZ 86323	636-4223 (Off) 636-4649 (Res)
Cottonwood	DON EBERLE ~~Mark Tracy~~ Fire Chief	827 N. Main Cottonwood, AZ 86326	634-~~5561~~ 2741 (Off) (24 hours) 634-7241 (Off) 634-~~4246~~ (24 hours)
Jerome	LYNN FORNIERE ~~Jim Allen~~ ~~Acting~~ Police Chief/Coordinator	Box 335 Jerome, AZ 86331	634-2245 (Off) (24 hours)
Prescott	Jim Culbreth Acting City Manager	P. O. Box 2059 Prescott, AZ 86302	445-3500 X 220 (Off) 778-2077 (Res)
	Ron Prince Fire Chief	1700 Iron Springs Rd. Prescott, AZ 86301	445-5555 (Off) 778-2895 (Res)
Prescott Valley	Steven Thompson Town Manager	P. O. Box 25456 Prescott Valley, AZ 86312	772-9207 (Off) (24 hours)
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San Luiz	Ed Jenkins Dir. Public Safety	P. O. Box 3740 San Luiz, AZ 85349	627-8881 (Off) 783-2456 (Res)
Somerton	Paul DeAnda Coordinator	P. O. Box 638 Somerton, AZ 85350	627-8866 X 28 627-2011 (Emerg 24 hr)
* WELTON	MAY BROOKS COORDINATOR	28871 OAKLAND AVENUE WELTON, AZ. 85356	785-3348 (OFF + RES)
Yuma	John Teague Director	298 W. 4th St. Yuma, AZ 85364	783-1271 (Off) X 261 782-2037 (Res)

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A Synopsis of NWS Models for Predicting the Flooding Due to Dam Failures

by D.L. Fread*

April 3,1985

1. INTRODUCTION

Catastrophic flooding occurs when a dam is breached and the impounded water escapes through the breach into the downstream valley. Usually the response time available for warning is much shorter than for precipitation-runoff floods. Dam failures are often caused by overtopping of the dam due to inadequate spillway capacity during large inflows to the reservoir from heavy precipitation runoff. Dam failures may also be caused by seepage or piping through the dam or along internal conduits, slope embankment slides, embankment cracks or liquefaction of earthen dams from earthquakes, and landslide-generated waves within the reservoir. Middlebrooks (1952) describes earthen dam failures occurring within the U.S. prior to 1951. Johnson and Illes (1976) summarize 300 dam failures throughout the world.

During the last decade some major improvements were made in models which predict the changing celerity and magnitude of a flood wave emanating from a breached (failed) dam and propagating through the downstream valley. Such improvements included consideration of the breach dynamics, use of the one-dimensional equations of unsteady flow to route the flood wave through the downstream valley, and consideration of the effects of downstream bridge-embankments, dams, and dead storage areas on the propagating wave.

The National Weather Service (NWS), having the responsibility to advise the public of downstream flooding when there is a failure of a dam, has developed three models to aid NWS hydrologists who are called upon to forecast the extent of flood inundation and available evacuation time. This paper briefly describes the three models (BREACH, DAMBRK, SMPDBK) used to forecast dam-break floods. These models are also used extensively for a multitude of purposes by planners, designers, and analysts who are concerned with possible future flood inundation due to dam-break floods and/or reservoir spillway floods, or any specified flood hydrograph.

Essentially, BREACH can be used to predict the size and timing of the development of the breach in earthen dams. DAMBRK can be used to develop the outflow hydrograph due to a breached dam (earthen or concrete) and determine the extent and timing of the flooding that occurs at various locations downstream of the dam. SMPDBK can do the same thing as DAMBRK except in a relatively simple manner which usually yields more approximate results.

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2. BREACH

An earthen dam is subject to possible failure from either overtopping or piping waters which erode a passage (breach) through the dam. The breach formation is gradual with respect to time and its width as measured along the crest of the dam usually encompasses only a portion of the dam's crest length. In many instances, the bottom of the breach progressively erodes downward until it reaches the bottom of the dam; however, in some cases, it may cease its downward progression at some intermediate elevation between the top and bottom of the dam. The size of the breach, as constituted by its depth and its width (which may be a function of the depth), and the rate of the breach formation determine the magnitude and shape of the resulting breach outflow hydrograph. This is of vital interest to hydrologists and engineers concerned with real-time forecasting or evacuation planning for floods produced by dam failures.

BREACH is an enhanced version of a mathematical model (Fread, 1984b) for predicting the breach characteristics (size, shape, time of formation) and the breach outflow hydrograph. The model is physically based on the principles of hydraulics, sediment transport, soil mechanics, the geometric and material properties of the dam, and the reservoir properties (storage volume, spillway characteristics, and time dependent reservoir inflow rate). The dam may be either man-made or naturally formed as a consequence of a landslide. In either, the mechanics of breach formation are very similar, the principal difference being one of scale. The landslide-formed dam is often much larger than even the largest of man-made earthen dams. The critical material properties of the dam are the internal friction angle, cohesion strength, and average grain size diameter (D_{50}).

The BREACH model differs from the parametric approach which the author has used in the NWS DAMBRK Model (Fread, 1977, 1984a). The parametric model uses empirical observations of previous dam failures such as the breach width-depth relation, time of breach formation, and depth of breach to develop the outflow hydrograph. The breach erosion model can provide some advantages over the parametric breach model for application to man-made dams since the critical properties used by the model are measurable or can be estimated within a reasonable range from a qualitative description of the dam materials. However, it should be emphasized that even if the properties can be measured there is a range for their probable value and within this range outflow hydrographs of varying magnitude and shape will be produced by the model. The hydrologist or engineer should investigate the most critical combination of values for the dam's material properties. It is considered essential when predicting breach outflows of landslide dams to utilize a physically based model since observations of such are essentially non-existent, rendering the parametric approach infeasible.

2.1 General Description

The breach erosion model (BREACH) simulates the failure of an earthen dam as shown in Fig. 1. The dam may be homogeneous or it may consist of two materials, an outer zone with distinct material properties (ϕ - friction angle, C - cohesion, D_{50} - average grain size (mm), and γ - unit weight) and an inner core with its ϕ, C, D_{50}, and γ values. Also, the downstream face

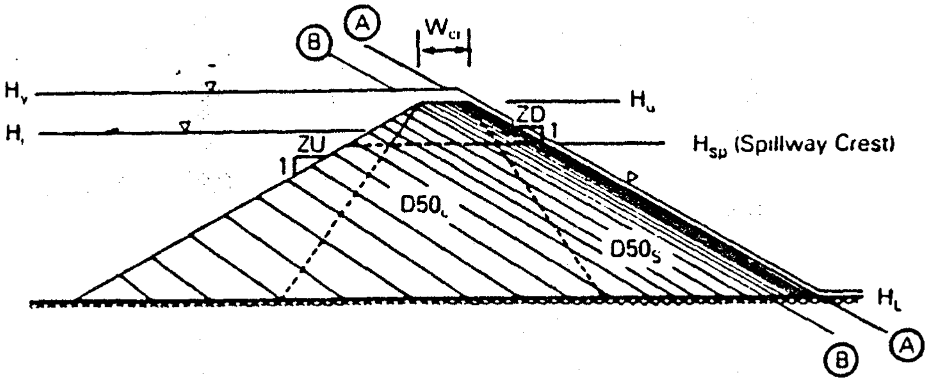

Figure 1. Side View of Dam Showing Conceptualized Overtopping Failure Sequence.

of the dam may be specified as having: 1) a grass cover with specified length of either good or fair stand, 2) material identical to the outer portion of the dam, or 3) material of larger grain size than the outer portion. The geometry of the downstream face of the dam is described by specifying the top of the dam (H_u), the bottom elevation of the dam (H_ℓ) which can also denote the original streambed elevation or the lowest level that the breach will form, and its slope as given by the ratio 1 (vertical) : ZD (horizontal). Then, the geometry of the upstream face of the dam is described by specifying its slope as the ratio 1 (vertical) : ZU (horizontal). If the dam is man-made it is further described by specifying a flat crest width (W_{cr}) and a spillway rating table of spillway flow vs. water elevation, in which the first elevation represents the spillway crest. Naturally formed landslide dams are assumed to not have a flat crest or, of course, a spillway.

The storage characteristics of the reservoir are described by specifying a table of surface area (S_a) in units of acre-ft vs. water elevation, the initial water surface elevation (H_i) at the beginning of the simulation, and a table of reservoir inflows (Q_i) in cfs vs. the hour of their occurrence (T_i).

If an overtopping failure is simulated, the water level (H) in the reservoir must exceed the top of the dam before any erosion occurs. The first stages of the erosion are only along the downstream face of the dam as denoted by the line A-A in Fig. 1 where, initially if no grass cover exists, a small rectangular-shaped rivulet is assumed to exist along the face. An erosion channel of depth-dependent width is gradually cut into the downstream face of the dam. The flow into the channel is determined by the broad-crested weir relationship:

$$Q_b = 3\ B_o(H-H_c)^{1.5} \tag{1}$$

in which Q_b is the flow into the breach channel, B_o is the instantaneous width of the initially rectangular-shaped channel, and H_c is the elevation of the breach bottom. As the breach erodes into the downstream face of the dam, the breach bottom elevation (H_c) remains at the top of the dam (H_u), and the most upstream point of the breach channel moves across the crest of the dam towards the dam's upstream face. When the bottom of the erosion channel has attained the position of line B-B in Fig. 1, the breach bottom (H_c) starts to erode vertically downward. The breach bottom is allowed to progress downward until it reaches the bottom elevation of the dam (H_ℓ) or in unusual circumstances to an elevation that may be specified as lower than the bottom of the dam.

If the downstream face of the dam (line A-A in Fig. 1) has a grass cover, the velocity of the overtopping flow along the grassed downstream face is computed at each time step by the Manning equation. This velocity is compared with a specified maximum permissible velocity for grass-lined channels (see Chow, 1959). Failure of the downstream face via erosion is initiated at the time when the permissible velocity is exceeded. At that time a single rivulet having dimensions of one (ft) depth x two width is instantly created along the downstream face. Erosion within the rivulet is allowed to proceed as in the case where a grass cover does not exist. The velocity (v) along the downstream face is computed as follows:

$$q = 3(H-Hc)^{1.5} \tag{2}$$

$$y = \left[\frac{qn'}{1.49(1/ZD)^{0.5}}\right]^{0.6} \tag{3}$$

$$n' = aq^b \tag{4}$$

$$v = q/y \tag{5}$$

in which q is the overtopping flow per foot of crest length, (H-Hc) is the hydrostatic head (ft) over the crest, n' is the Manning coefficient for grass-lined channels (Chow, 1959), a and b are fitting coefficients required to represent in mathematical form the graphical curves given in Chow.

If a piping breach is simulated, the initial water level (H) in the reservoir must be greater than the assumed center-line elevation (H_p) of the initially rectangular-shaped piping channel before the size of the pipe starts to increase via erosion. The bottom of the pipe is eroded vertically downward while its top erodes at the same rate vertically upwards. The flow into the pipe is controlled by orifice flow, i.e.,

$$Q_b = A\,[2g(H-Hp)/(1 + fL/D)]^{0.5} \tag{6}$$

in which Q_b is the flow (cfs) through the pipe, g is the gravity acceleration constant, A is the cross-sectional area (ft^2) of the pipe channel, (H-Hp) is the hydrostatic head (ft) on the pipe, L is the length (ft) of the pipe channel, D is the diameter or width (ft) of the pipe, and f is the Darcy friction factor computed from a mathematical representation of the Moody curves (Morris and Wiggert, 1972) and the breach material average grain size (D50). As the top elevation (H_{pu}) of the pipe erodes vertically upward, a point is reached when the flow changes from orifice-control to weir-control. The transition is assumed to occur when the following inequality is satisfied:

$$H < H_{pu} + 2(H_{pu} - H_p) \tag{7}$$

The weir flow is then governed by Eq. (1) in which H_c is equivalent to the bottom elevation of the pipe and B_o is the width of the pipe at the instant of transition. Upon reaching the instant of flow transition from orifice to weir, the remaining material above the top of the pipe and below the top of the dam is assumed to collapse and is transported along the breach channel at the current rate of sediment transport before further erosion occurs. The erosion then proceeds to cut a channel parallel to and along the remaining portion of the downstream face of the dam between the elevation of the bottom of the pipe and the bottom of the dam. The remaining erosion process is quite similar to that described for the overtopping type of failure with the breach channel now in a position similar to line A-A in Fig. 1.

The preceding general description of the erosion process was for a man-made dam. If a landslide dam is simulated the process is identical except, due to the assumption that the landslide dam has no crest width (W_{cr}), the erosion initially commences with the breach channel in the position of line B-B in Fig. 1. A failure mode of overtopping or piping may be initiated for a landslide-formed dam.

2.2 Breach Width

The method of determining the width of the breach channel is a critical component of the breach model. In this model the width of the breach is dynamically controlled by two mechanisms. The first, assumes the breach has an initial rectangular shape as shown in Fig. 2. The width of the breach (B_o) is governed by the following relation:

$$B_o = B_r\ y \tag{8}$$

in which B_r is a factor based on optimum channel hydraulic efficiency and y is the depth of flow in the breach channel. The parameter B_r has a value of 2 for overtopping failures while for piping failures, B_r is set to 1.0. The model assumes that y is the critical depth at the entrance to the breach channel, i.e.,

$$y = 2/3(H-H_c). \tag{9}$$

The second mechanism controlling the breach width is derived from the stability of soil slopes (Spangler, 1951). The initial rectangular-shaped

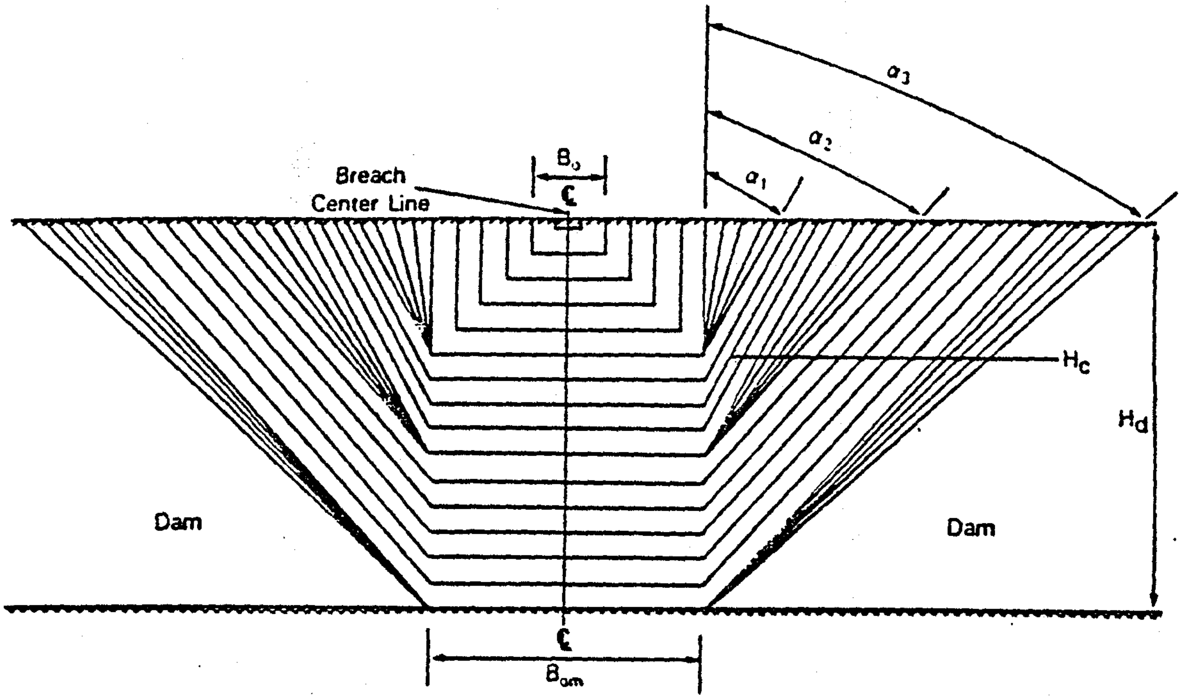

Figure 2. Front View of Dam with Breach Formation Sequence.

channel changes to a trapezoidal channel when the sides of the breach channel collapse, forming an angle (α) with the vertical. The collapse occurs when the depth of the breach cut (H_c') reaches the critical depth (H') which is a function of the dam's material properties of internal friction (ϕ), cohesion (C), and unit weight (γ), i.e.,

$$H_k' = \frac{4\ C \cos\phi\ \sin\theta'_{k-1}}{\gamma\ [1 - \cos(\theta'_{k-1} - \phi)]} \quad . \; . \; . \; k = 1,2,3 \tag{10}$$

in which the subscript k denotes one of three successive collapse conditions as shown in Fig. 2, and θ is the angle that the side of the breach channel makes with the horizontal. Angles (θ) and (α) are computed as follows:

$$\theta = (\theta'_{k-1} + \phi)/2 \tag{11}$$

$$\alpha = \pi/2 - \theta \tag{12}$$

The subscript (k) is incremented by 1 at the instant when

$$H_k > H_k'. \tag{13}$$

where: $H_k = H_c' - y/3$ (14)

The term (y/3) is subtracted from H_c' to give the actual free-standing depth of breach cut in which the supporting influence of the water on the stability of the sides of the breach is taken into account. Through this mechanism, it is possible for the breach to widen after the peak outflow through the breach has occurred since the flow depth (y) diminishes during the receding flow.

Erosion is assumed to occur equally along the bottom and sides of the breach channel except when the sides of the breach channel collapse. Thereupon the breach bottom is assumed not to continue to erode downward until the volume of collapsed material along the breach is removed at the rate of the sediment transport capacity of the breach channel at the instant of collapse. After this characteristically short pause, the breach bottom and sides continue to erode.

2.3 Reservoir Level Determination

Conservation of mass is used to compute the change in the reservoir water surface elevation (H) due to the influence of reservoir inflow (Q_i), spillway outflow (Q_{sp}), crest overflow (Q_o), breach outflow (Q_b), and the reservoir storage characteristics. The conservation of mass over a time step (Δt) in hours is represented by the following:

$$\bar{Q}_i - (\bar{Q}_b + \bar{Q}_{sp} + \bar{Q}_o) = S_a \frac{\Delta H}{\Delta t} \frac{43560}{3600} \tag{15}$$

in which ΔH is the change in water surface elevation during the time interval (Δt), and S_a is the surface area in acres at elevation H. All flows are expressed in units of cfs and the bar (-) indicates the flow is averaged over the time step. Rearranging Eq. (15) yields the following expression for the change in the reservoir water surface:

$$\Delta H = \frac{0.0826\Delta t}{S_a} (\bar{Q}_i - \bar{Q}_b - \bar{Q}_{sp} - \bar{Q}_o) \tag{16}$$

The reservoir elevation (H) at time (t) can easily be obtained from the relation,

$$H = H' + \Delta H \tag{17}$$

in which H' is the reservoir elevation at time $t-\Delta t$.

The reservoir inflow ($\bar{Q}_i$) is determined from the specified table of inflows (Q_i) vs. time (T_i). The spillway flow ($\bar{Q}_s$) is determined from the specified table of spillway flows (Q_s) vs. reservoir elevation (H). The breach flow (Q_b) is computed from Eq. (6) for piping flow. When the breach flow is weir-type, Eq. (1) is used when $H_c = H_u$; however, when $H_c < H_u$, the following broad-crested weir equation is used:

$$Q_b = 3\ B_o\ (H - H_c)^{1.5} + 2\ \tan(\alpha)\ (H - H_c)^{2.5} \quad (18)$$

in which B_o is the bottom width and α is given by Eq. (12). The crest overflow is computed as broad-crested weir flow from Eq. (1), where B_o is replaced by the crest length of the dam and H_c is replaced by H_u.

2.4 Breach Channel Hydraulics

The breach flow is assumed to be adequately described by quasi-steady uniform flow as determined by applying the Manning open channel flow equation at each Δt time step, i.e.,

$$Q_b = \frac{1.49\ S^{0.5}\ A^{1.67}}{n \qquad P^{0.67}} \quad (19)$$

in which $S = 1/ZD$, A is the channel cross-section area, P is the wetted perimeter of the channel, and n is the Manning coefficient. In this model, n is computed using the Strickler relation which is based on the average grain size of the material forming the breach channel, i.e.,

$$n = 0.013\ D_{50}^{\ 0.167} \quad (20)$$

in which D_{50} represents the average grain size diameter expressed in mm.

The use of quasi-steady uniform flow is considered appropriate because the extremely short reach of breach channel, very steep channel slopes (1/ZD) for man-made dams, and even in the case of landslide dams where the channel length is greater and the slope is smaller, contribute to produce extremely small variation in flow with distance along the breach channel. The use of quasi-steady uniform flow in contrast to the unsteady flow equations as used by Ponce and Tsivoglou (1981) in their breach erosion model greatly simplifies the hydraulics and computational algorithm. Such simplification is considered commensurate with the other simplifications inherent in the treatment of the breach development in dams for which precise measurements of material properties are lacking or impossible to obtain and the wide variance which exists in such properties in many dams. The simplified hydraulics eliminates troublesome numerical computation problems and enables the breach model to require only minimal computational resources.

2.5 Sediment Transport

The rate at which the breach is eroded depends on the capacity of the flowing water to transport the eroded material. The Meyer-Peter and Muller sediment transport relation as modified by Smart (1984) for steep channels is used, i.e.,

$$Q_s = 3.64\ (D_{90}/D_{30})^{0.2}\ \frac{D^{2/3}}{n}\ S^{1.1}\ (DS - 0.0054\ D_{50}\ \tau_c) \quad (21)$$

where:

$$\tau_c = a' \tau'_c \tag{22}$$

$$a' = \cos \Theta \, (1. - 1.54 \tan \Theta) \tag{23}$$

$$\Theta = \tan^{-1} S \tag{24}$$

$$\tau'_c = 10^{-1.21 - .19 \log R^*} \quad \ldots\ldots\ldots\ldots \; R^* < 30 \tag{25}$$

$$\tau'_c = 10^{-1.49 + .35(\log R^* - 1.48)} \quad \ldots\ldots\ldots \; 30 \leq R^* \leq 200 \tag{26}$$

$$\tau'_c = 0.062 \quad \ldots\ldots \; 200 < R^* < 25000 \tag{27}$$

$$S = \frac{1}{ZD} \tag{28}$$

$$R^* = 1524 \; D_{50} \; (DS)^{0.5} \tag{29}$$

in which Q_s is the sediment transport rate (cfs); D_{30}, D_{50}, D_{90} (mm) are grain sizes at which 30, 50, and 90 percent of the total weight is finer; D is the hydraulic depth of flow (ft), S is the slope of the downstream face of the dam; and τ'_c is the Shields' critical shear stress.

2.6 Breach Enlargement By Sudden Collapse

It is possible for the breach to be enlarged by a rather sudden collapse failure of the upper portions of dam in the vicinity of the breach development. Such a collapse would consist of a wedge-shaped portion of the dam having a vertical dimension (Y_c). The collapse would be due to the pressure of the water on the upstream face of the dam exceeding the resistive forces due to shear and cohesion which keep the wedge in place. When this occurs the wedge is pushed into the breach and then transported by the escaping water through the now enlarged breach. When collapse occurs, the erosion of the breach ceases until the volume of the collapsed wedge is transported through the breach channel at the transport rate of the water escaping through the suddenly enlarged breach. A check for collapse is made at each Δt time step during the simulation. The collapse check consists of summing the forces acting on the wedge of height, Y_c. The forces are those due to the water pressure and the resisting forces which are the shear force acting along the bottom of the wedge, the shear force acting along both sides of the wedge, the force due to cohesion along the sides and bottom of the wedge.

2.7 Computational Algorithm

The sequence of computations in the model are iterative since the flow into the breach is dependent on the bottom elevation of the breach and its width while the breach properties are dependent on the sediment transport capacity of the breach flow; the transport capacity is dependent on the breach size and flow. A simple iterative algorithm is used to account for the mutual dependence of the flow, erosion, and breach properties. An estimated incremental erosion depth ($\Delta H_c'$) is used at each time step to start the iterative computation. This estimated value can be extrapolated from previously computed incremental erosion depths after the first few time steps. The computational algorithm is described elsewhere (Fread, 1984b).

2.8 Computational Requirements

The basic time step (Δt) is specified; however when rapid erosion takes place the basic time step is automatically reduced to $\Delta t/20$. The specified value for the basic time step is usually about 0.02 hrs with slightly larger values acceptable for landslide dams. For typical applications, the BREACH model requires less than 10 seconds of CPU time on a Prime 750 computer and less than 2 seconds on an IBM 360/195 computer, both of which are mainframe computers. Although it has not been used on microcomputers, it would be quite amenable to such applications.

The model has displayed a lack of numerical instability or convergence problems. The computations show very little sensitivity to a reasonable variation in basic time step size. Numerical experimentation indicates that as the time step is increased by a factor of 4, the computed peak flow (Q_p), time of peak (T_p), and final breach dimensions vary by less than 10, 4, and 0.5 percent, respectively.

2.9 Model Applications

The BREACH model was applied to two earthen dams to determine the outflow hydrograph produced by a gradual breach of each. The first was the piping failure of the man-made Teton dam in Idaho, and the second was an overtopping failure of the landslide-formed dam which blocked the Mantaro River in Peru.

2.9.1 Teton Dam

The Teton Dam, a 300 ft high earthen dam with a 3000 ft long crest and 262 ft depth of stored water amounting to about 250,000 acre-ft, failed on June 5, 1976. According to a report by Ray, et. al (1976) the failure started as a piping failure about 10:00 AM and slowly increased the rate of outflow until about 12:00 noon when the portion of the dam above the piping hole collapsed and in the next few minutes (about 12 minutes according to Blanton (1977)) the breach became fully developed allowing an estimated 1.6 to 2.8 million cfs (best estimate of 2.3) peak flow (Brown and Rogers, 1977) to be discharged into the valley below. At the time of peak flow the breach was estimated from photographs to be trapezoidal shape having a top width at the original water surface elevation of about 500 ft and side slopes of about 1 vertical to 0.5 horizontal. After the peak outflow the outflow gradually decreased to a comparatively low flow in about five hours as the reservoir volume was depleted and the surface elevation receded. The downstream face of the dam had a slope of 1:2 and the upstream face 1:2.5. The crest width was 35 ft and the bulk of the breach material was a D_{50} size of 0.03 mm. The inflow to the reservoir during failure was insignificant and the reservoir surface area at time of failure was about 1950 acre-ft.

The BREACH model was applied to the piping generated failure of the Teton Dam. The center-line elevation for the piping breach was 160 ft above the bottom of the dam, and an initial width of 0.1 ft was used for the assumed square-shaped pipe. The material properties of the breach were assumed as follows: ϕ = 40 deg, C = 250 lb/ft^2, and γ = 100 lb/ft^3. The Strickler equation was judged not to be applicable for the extremely fine breach material, and the n value was computed as 0.013 from a Darcy friction factor based on the D_{50} grain size and the Moody curves. The computed outflow hydrograph is shown in Fig. 3. The timing, shape, and magnitude of the hydrograph compares quite well with the estimated actual values. The computed peak outflow of 2.3 million cfs agrees with the best estimate made by the U.S. Geological Survey and the time of occurrence is also the same. The computed breach width of 460 ft agrees closely with the estimated value

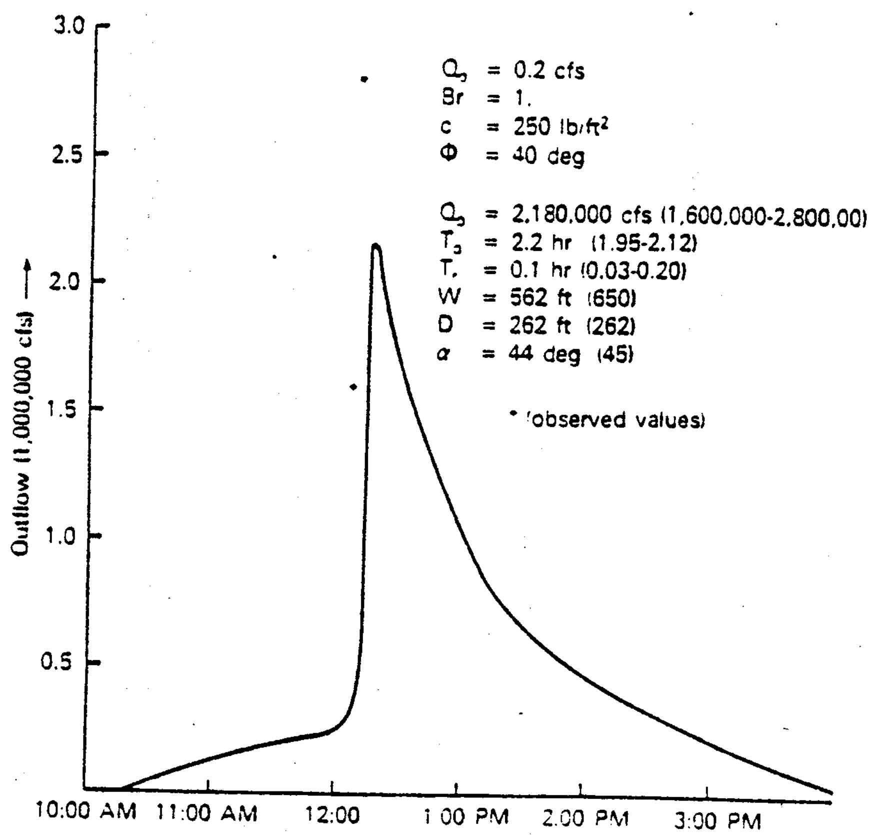

Figure 3. Teton Dam: Predicted and Observed Breach Outflow Hydrograph and Breach Properties.

of 500 ft at the elevation of the initial reservoir water surface. A larger estimated actual breach width of 650 ft breach width was reported by Brown and Rogers (1977); however this was the final breach width after additional enlargement of the breach occurred. The (BREACH) model produced a final width of 560 ft when the reservoir water elevation had receded to near the reservoir bottom; the additional widening of the breach during the recession of the outflow is due to the influence of the depth (y) in Eq. (14).

Sensitivities of the peak breach outflow (Q_p), time of peak flow (T_p) and the top width (W) of the trapezoidal-shaped breach to variations in the specified breach material properties, cohesive strength (C) and internal friction angle (ϕ), are shown in Fig. 4. The dashed lines apply to the Teton simulation. Peak outflow is moderately affected by the cohesion; however it is sensitive to the ϕ value which mostly controls the enlargement of the breach width. Q_p is sensitive to a full range of ϕ values, however the ϕ value may vary by ± 10 degrees with less than 20% variation in Q_p. The breach width (W) was moderately sensitive to variations in the cohesion (C), and somewhat more sensitive to the ϕ value. The time to peak outflow (T_p) was almost insensitive to variations in C and ϕ.

2.9.2 Mantaro Landslide Dam

A massive landslide occurred in the valley of the Mantaro River in the mountainous area of central Peru on April 25, 1974. The slide, with a volume of approximately 5.6×10^{10} ft^3, dammed the Mantaro River and formed

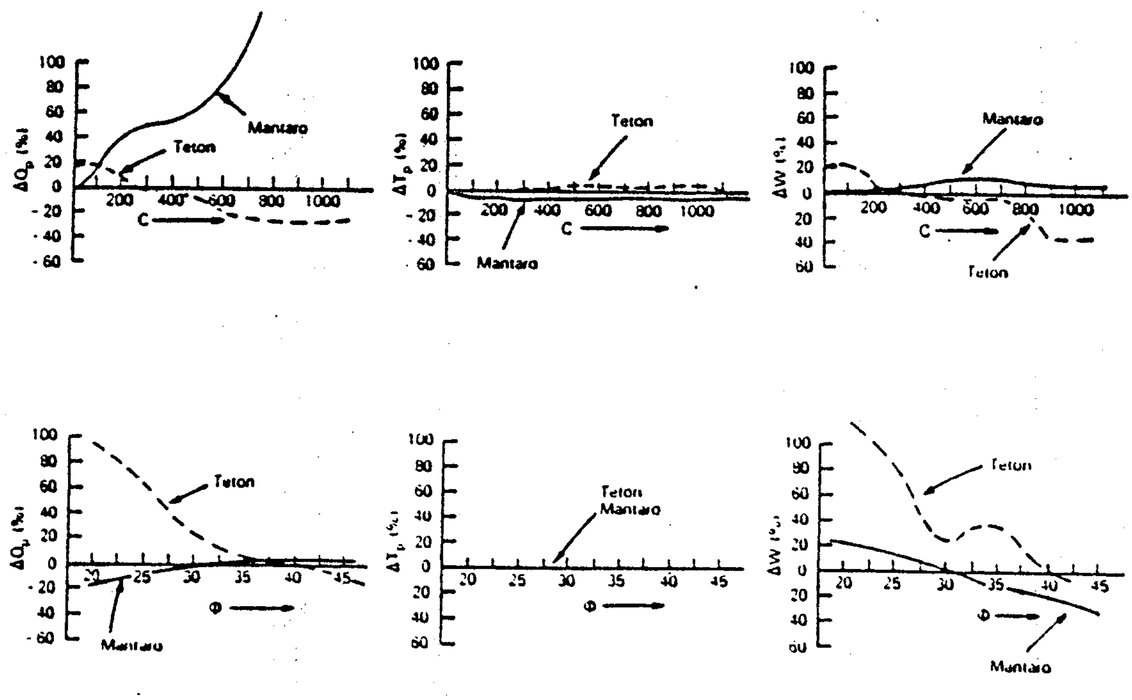

Figure 4. Sensitivity of Mantaro and Teton Predictions of Peak Outflow (Q_p), Time to Peak (T_p), and Breach Width (W) to Changes in the Material Properties of the Dam: Cohesion (C) and Internal Friction Angle (ϕ).

a lake which reached a depth of about 560 ft before overtopping during the period June 6-8, 1974 (Lee and Duncan, 1975). The overtopping flow very gradually eroded a small channel along the approximately 1-mile long downstream face of the slide during the first two days of overtopping. Then a dramatic increase in the breach channel occurred during the next 6-10 hrs resulting in a final trapezoidal-shaped breach channel approximately 350 ft in depth, a top width of some 800 ft, and side slopes of about 1:1. The peak flow was estimated at 353,000 cfs as reported by Lee and Duncan (1975), although Ponce and Tsivoglou (1981) later reported an estimated value of 484,000 cfs. The breach did not erode down to the original river bed; this caused a rather large lake to remain after the breaching had subsided some 24 hrs after the peak had occurred. The slide material was mostly a mixture of silty sand with some clay resulting in a D_{50} size of about 11 mm with some material ranging in size up to 3-ft boulders.

The BREACH model was applied to the Mantaro landslide-formed dam using the following parameters: $ZU = 17$, $ZD = 8.0$, $H_u = 560$ ft, $D_{50} = 11$ mm, $P_{or} = 0.5$, $S_a = 1200$ acres, $C = 30$ lb/ft^2, $\phi = 30$ deg, $\gamma = 100$ lb/ft^3 $B_r = 2$, and $\Delta t = 0.1$ hr. The Manning n was estimated by Eq. (20) as 0.020 and the initial breach depth was assumed to be 0.3 ft. The computed breach outflow is shown in Fig. 5 along with the estimated actual values. The timing of the peak outflow and its magnitude are very similar except for a somewhat more gradual rising limb of 10 hr compared to the estimated actual of 6 hr. The dimensions of the gorge eroded through the dam are similar as shown by the values of D, W, and α in Fig. 5.

The sensitivities of Q_p, T_p and W for variations in C and ϕ are shown in Fig. 4. The solid line denotes the Mantaro application. Most notably, Q_p is very sensitive to the cohesion (C) while much less sensitive to the internal friction angle (ϕ). T_p is almost insensitive to the value of C and quite insensitive to ϕ. W is not very sensitive to C and moderately sensitive to ϕ; a variation of $\pm$ 10 degrees in ϕ results in a change in W of less than 20%.

3. DAMBRK

The DAMBRK model represents the current state-of-the-art in understanding of dam failures and the utilization of hydrodynamic theory to predict the dam-break wave formation and downstream progression. The model has wide applicability; it can function with various levels of input data ranging from rough estimates to complete data specification; the required data is readily accessible; and it is economically feasible to use, i.e., it requires a minimal computation effort on mainframe computing facilities and can be used with microcomputers.

The model consists of three functional parts, namely: (1) description of the dam failure mode, i.e., the temporal and geometrical description of the breach; (2) computation of the time history (hydrograph) of the outflow through the breach as affected by the breach description, reservoir inflow, reservoir storage characteristics, spillway outflows, and downstream

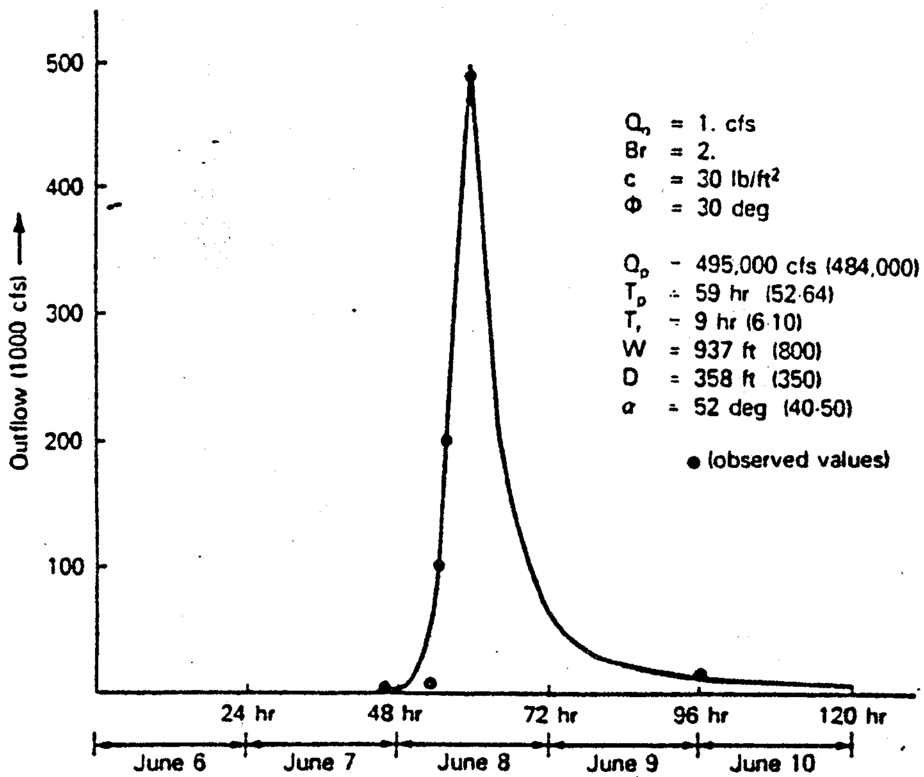

Figure 5. Mantaro Landslide Dam: Predicted and Observed Breach Outflow Hydrograph and Breach Properties.

tailwater elevations; and (3) routing of the outflow hydrograph through the downstream valley in order to determine the changes in the hydrograph due to valley storage, frictional resistance, downstream bridges or dams, and to determine the resulting water surface elevations (stages) and flood-wave travel times.

DAMBRK is an expanded version of a practical operational model first presented in 1977 by the author (Fread, 1977). That model was based on previous work by the author on modeling breached dams (Fread and Harbaugh, 1973) and routing of flood waves (Fread, 1974a, 1976).

3.1 Breach Description

The breach is the opening formed in the dam as it fails. Earthen dams which exceedingly outnumber all other types of dams do not tend to completely fail, nor do they fail instantaneously. The fully formed breach in earthen dams tends to have an average width ($\bar{b}$) in the range ($h_d < \bar{b} < 4h_d$) where h_d is the height of the dam. The middle portion of this range for $\bar{b}$ is supported by the summary report of Johnson and Illes (1976). Breach widths for earthen dams are therefore usually much less than the total length of the dam as measured across the valley. Also, the breach requires a finite interval of time for its formation through erosion of the dam materials by the escaping water. Total time of failure may be in the range of a few minutes to a few hours, depending on the height of the dam, the type materials used in construction, the extent of compaction of the materials, and the extent (magnitude and duration) of the overtopping flow of the escaping water. Piping failures occur when initial breach formation takes place at some point below the top of the dam due to erosion of an internal channel through the dam by escaping water. As the erosion proceeds, a larger and larger opening is formed; this is eventually hastened by caving-in of the top portion of the dam.

Concrete gravity dams also tend to have a partial breach as one or more monolith sections formed during the construction of the dam are forced apart by the escaping water. The time for breach formation is in the range of a few minutes.

Poorly constructed earthen dams and coal-waste slag piles which impound water tend to fail within a few minutes, and have average breach widths in the upper range or even greater than those for the earthen dams mentioned above.

In DAMBRK, the failure time (τ) and the size and shape of the breach are selected as input parameters similar to the approach used by Fread and Harbaugh (1973). The shape (see Fig. 6) is specified by a parameter (z) identifying the side slope of the breach, i.e., 1 vertical: z horizontal slope. The range of z values is: $0 < z < 2$. Rectangular triangular, or trapezoidal shapes may be specified in this way. For example, $z=0$ and $b>0$ produces a trapezoidal shape. The final breach size is controlled by the z parameter and another parameter (b) which is the terminal width of the bottom of the breach. As shown in Fig. 6, the model assumes the breach bottom width starts at a point and enlarges at a linear rate over the failure time interval (τ) until the terminal width is attained and the breach bottom has eroded to the elevation h_{bm} which is usually, but not necessarily, the bottom of the reservoir or outlet channel bottom. If τ is less than 10 minutes, the width of the breach bottom starts at a value of b rather than at a point. This represents more of a collapse failure than an erosion failure.

During the simulation of a dam failure, the actual breach formation commences when the reservoir water surface elevation (h) exceeds a specified value, h_f. This feature permits the simulation of an overtopping of a dam in which the breach does not form until a sufficient amount of water is flowing over the crest of the dam. A piping failure may be simulated when h_f is specified less than the height of the dam, h_d.

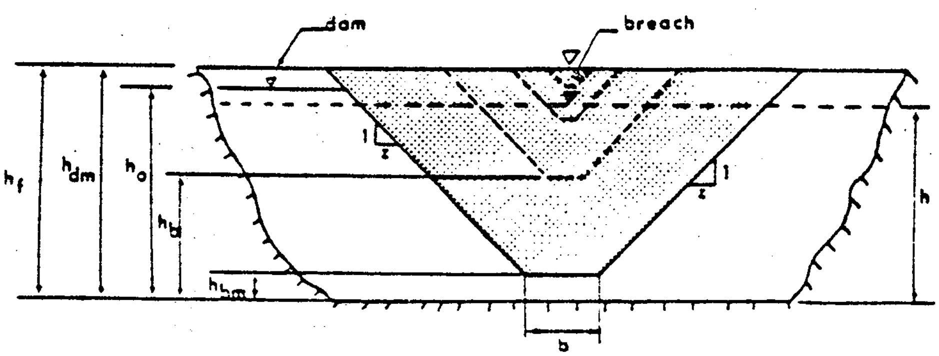

Figure 6. Front View of Dam Showing Formation of Breach

Selection of breach parameters before a breach forms, or in the absence of observations, introduces a varying degree of uncertainty in the model results; however, errors in the breach description and thence in the resulting time rate of volume outflow are rapidly damped-out as the flood wave advances downstream. For conservative forecasts which err on the side of larger flood waves, values for b and z should produce an average breach width ($\bar{b}$) in the uppermost range for a certain type of dam. Failure time (τ) should be selected in the lower range to produce a maximum outflow.

3.2 Reservoir Outflow Hydrograph

The total reservoir outflow consists of broad-crested weir flow through the breach and flow through any spillway outlets, i.e.,

$$Q = Q_b + Q_s \tag{30}$$

The breach outflow (Q_b) is computed as:

$$Q_b = c_1(h-h_b)^{1.5} + c_2(h-h_b)^{2.5} \tag{31}$$

where:

$$c_1 = 3.1\ b_i\ c_v\ k_s \tag{32}$$

$$c_2 = 2.45\ z\ c_v\ k_s \tag{33}$$

$$h_b = h_d - (h_d - h_{bm})\frac{t_b}{\tau} \qquad \text{if} \qquad t_b < \tau \tag{34}$$

$$h_b = h_{bm} \qquad \text{if} \qquad t_b > \tau \tag{35}$$

$$b_i = b\ t_b/\tau \qquad \text{if} \qquad t_b < \tau \tag{36}$$

$$c_v = 1.0 + 0.023\ Q^2/[B_d^2\ (h-h_{bm})^2\ (h-h_b)] \tag{37}$$

$$k_s = 1.0 \qquad \text{if} \qquad \frac{h_t - h_b}{h-h_b} \leq 0.67 \tag{38}$$

otherwise:

$$k_s = 1.0 - 27.8\ \left[\frac{h_t - h_b}{h-h_b} - 0.67\right]^3 \tag{39}$$

in which h_b is the elevation of the breach bottom, h is the reservoir water surface elevation, b_i is the instantaneous breach bottom width, t_b is time interval since breach started forming, c_v is correction for velocity of approach, Q is the total outflow from the reservoir, B_d is width of the reservoir at the dam, k_s is the submergence correction for tailwater effects on weir outflow (Venard, 1954), and h_t is the tailwater elevation (water surface elevation immediately downstream of dam).

The tailwater elevation (h_t) is computed from Manning's equation, i.e.,

$$Q = \frac{1.49}{n} S^{1/2} \frac{A^{5/3}}{B^{2/3}} \tag{40}$$

in which n is the Manning roughness coefficient, A is the cross-sectional area of flow, B is the top width of the wetted cross-sectional area, and S is the energy slope. Each term in Eq. (40) applies to a representative channel reach immediately downstream of the dam. The S parameter can be specified by the user; it does not change with time; if it is not specified, the model uses the channel bottom slope of the first third of the downstream valley reach. Since A and B are functions of h_t and Q is the total discharge given by Eq. (30), Eq. (40) provides a sufficiently accurate value for h_t if there are no backwater effects immediately below the dam due to downstream constrictions, dams, bridges, or significant tributary inflows. When these affect the tailwater, Eq. (40) is not used and the dam is treated as an internal boundary which is described in a following section on multiple dams and bridges.

If the breach is formed by piping, Eq. (31)-(39) are replaced by the following orifice flow equation:

$$Q_b = 4.8\ A_p(h-\bar{h})^{1/2} \tag{41}$$

where:

$$A_p = [2b_i+4z(h_f-h_b)]\ (h_f-h_b) \tag{42}$$

$$\bar{h} = h_f \qquad \text{if} \qquad h_t < 2h_f - h_b \tag{43}$$

$$\bar{h} = h_t \qquad \text{if} \qquad h_t > 2h_f - h_b \tag{44}$$

and h_d is replaced by h_f in Eq. (34) to compute h_b. However, if $\bar{h} = h_f$ and

$$h - h_b < 2.2(h_f-h_b) \tag{45}$$

the flow ceases to be orifice flow and the broad-crested weir flow, Eq. (31), is used.

The spillway outflow (Q_s) is computed as:

$$Q_s = c_s L_s (h-h_s)^{1.5} + c_g A_g (h-h_g)^{0.5} + c_d L_d (h-h_d)^{1.5} + Q_t \qquad (46)$$

in which c_s is the uncontrolled spillway discharge coefficient, h_s is the uncontrolled spillway crest elevation, c_g is the gated spillway discharge coefficient, h_g is the center-line elevation of the gated spillway, c_d is the discharge coefficient for flow over the crest of the dam L_s is the spillway length, A_g is the gate flow area, L_d is the length of the dam crest less L_s, and Q_t is a constant outflow term which is head independent. The uncontrolled spillway flow or the gated spillway flow can also be represented as a table of head-discharge values. The gate flow may also be specified as a function of time.

The total outflow is a function of the water surface elevation (h). Depletion of the reservoir storage volume by the outflow causes a decrease in h which then causes a decrease in Q. However, any inflow to the reservoir tends to increase h and Q. In order to determine the total outflow (Q) as function of time, the simultaneous effects of reservoir storage characteristics and reservoir inflow require the use of a reservoir routing technique. DAMBRK utilizes a hydrologic storage routing technique based on the law of conservation of mass, i.e.,

$$I - Q = dS/dt \qquad (47)$$

in which I is the reservoir inflow, Q is the total reservoir outflow, and dS/dt is the time rate of change of reservoir storage volume. Eq. (47) may be expressed in finite difference form as:

$$(I+I')/2 - (Q+Q')/2 = \Delta S/\Delta t \qquad (48)$$

in which the prime (') superscript denotes values at the time $t-\Delta t$ and the Δ approximates the differential. The term ΔS may be expressed as:

$$\Delta S = (A_s+A_s') (h-h')/2 \qquad (49)$$

in which A_s is the reservoir surface area coincident with the elevation (h).

Combining Eqs. (30), (31), (46), (48) and (49) result in the following expression:

$$(A_s+A_s') (h-h')/\Delta t + c_1(h-h_b)^{1.5} + c_2(h-h_b)^{2.5} + c_s(h-h_b)^{1.5}$$

$$+ c_g(h-h_g)^{0.5} + c_d(h-h_d)^{1.5} + Q_t + Q' - I - I' = 0 \qquad (50)$$

Since A_s is a function of h and all other terms except h are known, Eq. (50) can be solved for the unknown h using Newton-Raphson iteration. Once h is obtained, Eqs. (31) and (46) can be used to obtain the total outflow (Q) at time (t). In this way the outflow hydrograph Q(t) can be developed for each time (t) as to goes from zero to some terminating value (t_e) sufficiently large for the reservoir to be drained. In Eq. (50) the time step (Δt) is chosen sufficiently small to incur minimal numerical integration error. This value is preset in the model to $\tau/50$.

3.3 Downstream Routing

After computing the hydrograph of the reservoir outflow, the extent of and time of occurrence of flooding in the downstream valley is determined by routing the outflow hydrograph though the valley. The hydrograph is modified (attenuated, lagged, and distorted) as it is routed through the valley due to the effects of valley storage, frictional resistance to flow, and downstream obstructions and/or flow control structures. Modifications to the dam-break flood wave are manifested as attenuation of the flood peak elevation, spreading-out or dispersion of the flood wave volume, and changes in the celerity (translation speed) or travel time of the flood wave. If the downstream valley contains significant storage volume such as a wide flood plain, the flood wave can be extensively attenuated and its time of travel greatly increased. Even when the downstream valley approaches that of a uniform rectangular-shaped section, there is appreciable attenuation of flood peak and reduction in wave celerity as the wave progresses through the valley.

A distinguishing feature of dam-break waves is the great magnitude of peak discharge when compared to runoff-generated flood waves having occurred in the past in the same valley. The dam-break flood is usually many times greater than the runoff flood of record. The above-record discharges make it necessary to extrapolate certain coefficients used in various flood routing techniques and make it impossible to fully calibrate the routing technique.

Another distinguishing characteristic of dam-break floods is the very short duration time, and particularly the extremely short time from beginning of rise until the occurrence of the peak. The time to peak is in almost all instances synonymous with the breach formation time (τ) and, therefore, is in the range of a few minutes to a few hours. This feature, coupled with the great magnitude of the peak discharge, causes the dam-break flood wave to have acceleration components of a far greater significance than those associated with a runoff-generated flood wave.

A hydraulic routing technique (dynamic routing) based on the complete equations of unsteady flow is used to route the dam-break flood hydrograph through the downstream valley. This method is derived from the original equations developed by Barre De Saint-Venant (1871). In this method the important acceleration effects are properly considered. Also, the only coefficient that must be extrapolated beyond the range of past experience is the coefficient of flow resistance. It so happens that this is usually not a sensitive parameter in effecting the modifications of the flood wave due to its progression through the downstream valley. The dynamic routing technique properly considers the effect of downstream constrictions and flow control structures such as bridge-road embankments or dams.

The Saint-Venant unsteady flow equations consist of a conservation of mass equation, i.e.,

$$\frac{\partial Q}{\partial x} + \frac{\partial(A+A_o)}{\partial t} - q = 0 \tag{51}$$

and a conservation of momentum equation, i.e.,

$$\frac{\partial Q}{\partial t} + \frac{\partial(Q^2/A)}{\partial x} + gA(\frac{\partial h}{\partial x} + S_f + S_e) + L = 0 \tag{52}$$

where A is the active cross-sectional area of flow, A_o is the inactive (off-channel storage) cross-sectional area, x is the longitudinal distance along the channel (valley), t is the time, q is the lateral inflow or outflow per linear distance along the channel (inflow is positive and outflow is negative in sign), g is the acceleration due to gravity, S_f is the friction slope, and S_e is the expansion-contraction slope. The friction slope is evaluated from Manning's equation for uniform, steady flow, i.e.,

$$S_f = \frac{n^2|Q|Q}{2.21\ A^2\ R^{4/3}} \tag{53}$$

in which n is the Manning coefficient of frictional resistance and R is the hydraulic radius defined as A/B where B is the top width of the active cross-sectional area. The term (S_e) is defined as follows:

$$S_e = \frac{k\ \Delta(Q/A)^2}{2g\ \Delta x} \tag{54}$$

in which k (Morris and Wiggert, 1972) is the expansion-contraction coefficient varying from 0.0 to ±1.0 (+ if contraction, - if expansion), and $\Delta(Q/A)^2$ is the difference in the term $(Q/A)^2$ at two adjacent cross-sections separated by a distance Δx. L is the momentum effect of lateral flow assumed herein to enter or exit perpendicular to the direction of the main flow. This term has the following form: 1) lateral inflow, L = 0; 2) seepage lateral outflow, L = -0.5qQ/A; and 3) bulk lateral outflow, L = -qQ/A.

Eqs. (51)-(52) which are nonlinear partial differential equations, must be solved by numerical techniques. An implicit 4-pt. finite difference technique is used to obtain a solution to either set of equations. This particular technique (Fread, 1974) is used for its computational efficiency, flexibility, and convenience in the application of the equations to flow in complex channels existing in nature. In essence, the technique determines the unknown quantities (Q and h at all specified cross-sections along the downstream channel-valley at various times into the future; the solution is advanced from one time to a future time by a finite time interval (time step) of magnitude Δt. The flow equations are expressed in finite difference form for all cross-sections along the valley and then solved simultaneously for the unknowns (Q and h) at each cross-section. Due to the nonlinearity of the partial differential equations and their finite difference representations, the solution is iterative and a highly efficient quadratic iterative technique known as the Newton-Raphson method is used. Convergence of the iterative technique is attained when the difference between successive iterative solutions for each unknown is less than a relatively small prescribed tolerance. Usually, one to three iterations at each time step are sufficient for convergence to be attained for each unknown at all cross-sections. A more complete description of the solution technique may be found elsewhere (Amein and Fang, 1970; Fread, 1974a, Fread, 1977).

3.4 Tributary Inflows/Outflows

Unsteady flows associated with tributaries downstream of the dam can be added to the unsteady flow resulting from the dam failure. This is accomplished via the term q in Eq. (51). The tributary flow is distributed along a single Δx reach. Backwater effects of the dam-break flow on the tributary flow are ignored, and the tributary flow is assumed to enter perpendicular to the dam-break flow. Outflows are assigned negative values. Outflows which occur as broad-crested weir flow over a levee or natural crest may be simulated. The crest elevation, discharge coefficient, and location along the river-valley must be specified. The head is computed as the average water surface elevation, along the length of the crest, less the crest elevation.

3.5 Multiple Dams and Bridges

The dam-break flood forecasting model can simulate the progression of a dam-break wave through a downstream valley containing a reservoir created by another downstream dam, which itself may fail due to being sufficiently overtopped by the wave produced by the failure of the upstream dam. In fact, an unlimited number of reservoirs located sequentially along the valley can be simulated. When the tailwater below a dam is affected by flow conditions downstream of the tailwater section (e.g., backwater produced by a downstream dam, flow constriction, bridge, and/or tributary inflow), the flow occurring at the dam is computed by using an internal boundary condition at the dam. In this method the dam is treated as a short Δx reach in which the flow through the reach is governed by the following two equations rather than either Eqs. (51)-(52):

$$Q_i = Q_{i+1} \tag{55}$$

$$Q_i = Q_b + Q_s \tag{56}$$

in which Q_b and Q_s are breach flow and spillway flow. In this way, the flows Q_i and Q_{i+1} and the elevations h_i and h_{i+1} are in balance with the other flows and elevations occurring simultaneously throughout the entire flow system which may consist of additional dams which are treated as additional internal boundary conditions via Eqs. (55)-(56).

Highway/railway bridges and their associated earthen embankments which are located at points downstream of a dam may also be treated as internal boundary conditions. Eqs. (55)-(56) are used at each bridge; the term Q_s in Eq. (56) is computed by the following expression:

$$Q_s = C\sqrt{2g}\, A_{i+1}(h_i - h_{i+1})^{1/2} + C_d k_s (h-h_c)^{3/2} \tag{57}$$

in which C is a coefficient of bridge flow, C_d is the coefficient of flow over the crest of the road embankment, h_c is the crest elevation of the embankment, and k_s is similar to Eqs. (38)-(39).

3.6 Supercritical Flow

The DAMBRK model can simulate the flow through the downstream valley when the flow is supercritical. This type of flow occurs when the slope of the downstream valley exceeds about 50 ft/mi. Slopes less than this usually result in the flow being subcritical to which all preceding comments pertaining to the downstream routing apply. When the flow is supercritical, any flow disturbances cannot travel back upstream; therefore, the downstream boundary becomes superfluous. Thus, for supercritical flow, a downstream boundary condition is not required; however, an additional equation other than the reservoir outflow hydrograph is needed. To satisfy this requirement, an equation similar to Eq. (40) but with a time-dependent energy slope, is used at the upstream boundary. Multiple reservoirs on supercritical valley slopes must be treated using a storage routing technique such as Eq. (50) rather than the dynamic routing technique.

3.7 Floodplain Compartments

The DAMBRK model can simulate the exchange of flow between the river and floodplain compartments. The floodplain compartments are formed by one or two levees which run parallel to the river on either or both sides of the river, and other levees or road embankments which run perpendicular to the river. Flow transfer between a floodplain compartment and the river is assumed to occur along one Δx reach and is controlled by broad-crested weir flow with submergence correction. Flow can be either away from the river or into the river, depending on the relative water surface elevations of the river and the floodplain compartment. The river elevations are computed via Eqs. (51)-(52), and the floodplain water surface elevations are computed by a simple storage routing relation, i.e.,

$$V_\ell^{\,t} = V_\ell^{\,t-\Delta t} + \left(I^t - O^t\right) \Delta t/43560 \tag{58}$$

in which V_ℓ is the volume (acre-ft) in the floodplain compartment at time t or t-Δt referenced to the water elevation, I is the inflow from the river or adjacent floodplain compartments, and O is the outflow from the floodplain compartment to the river and/or to adjacent floodplain compartments. Flow transfer between adjacent floodplain compartments is also controlled by broad-crested weir flow with submergence correction. The broad-crested weir flow is according to the following:

$$I = c\, s_b \left(h_r - h_{fp}\right)^{3/2} \tag{59}$$

$$Q = c\, s_b\, (h_{fp} - h_r)^{3/2} \tag{60}$$

in which c is a specified discharge coefficient, h_r is the river elevation, h_{fp} is the water surface elevation of the floodplain, and s_b is the submergence correction factor, i.e.

$$s_b = 1.0 \qquad h_r < 0.67 \tag{61}$$

$$s_b = 1.0 - 27.8\, (H_r - 0.67)^3 \qquad h_r > 0.67 \tag{62}$$

$$H_r = (h_r - h_w)/(h_{fp} - h_w) \tag{63}$$

and h_w is the specified elevation of the crest of the levee. The floodplain elevation (h_{fp}) is obtained iteratively via a table look-up algorithm from the specified table of volume-elevation values. The outflow from a floodplain compartment may also include that from one or more pumps associated with each floodplain compartment. Each pump has a specified discharge-head relation given in tabular form along with start-up and shut-off operation instructions depending on specified water surface elevations. The pumps discharge to the river.

3.8 Routing Losses

Often in the case of dam-break floods, where the extremely high flows inundate considerable portions of channel overbank or valley flood plain, a measurable loss of flow volume occurs. This is due to infiltration into the relatively dry overbank material, detention storage losses, and sometimes short-circuiting of flows from the main valley into other drainage basins via canals. Such losses of flow may be taken into account via the term q in Eq. (51). An expression describing the loss is given by the following:

$$q_m = -0.00458\, V_L\, P/(L\, \bar{T}) \tag{64}$$

in which V_L is the outflow volume (acre-ft) from the reservoir; P is the volume loss ratio which may range from 0 to as high as 0.3; L is the length (mi) of downstream channel through which the loss occurs; and T is the average duration (hr) of the flood wave throughout the reach length L; and q_m is the maximum lateral outflow (cfs/ft) occurring along the reach L throughout the duration of flow. The mean lateral outflow is proportioned in time and distance along the reach L.

3.9 Landslide Generated Waves

Reservoirs are sometimes subject to landslides which rush into the reservoir displacing a portion of the reservoir contents, and thereby creating a very steep water wave which travels up and down the length of the reservoir. This wave may have sufficient amplitude to overtop the dam and precipitate a failure of the dam, or the wave by itself may be large enough to cause catastrophic flooding downstream of the dam without resulting in the failure of the dam as perhaps in the case of a concrete dam.

The capability to generate waves produced by landslides is provided within DAMBRK. The volume of the landslide mass, its porosity, and time interval over which the landslide occurs, are input to the model. Within the model, the landslide mass is deposited within the reservoir in layers during small computational time steps, and simultaneously the original dimensions of the reservoir are reduced accordingly. The time rate of reduction in the reservoir cross-sectional area creates the wave during the solution of the unsteady flow Eqs. (51)-(52), which are applied to he cross-sections describing the reservoir characteristics.

3.10 Model Testing

The DAMBRK model has been tested on five historical dam-break floods to determine its ability to reconstitute observed downstream peak stages, discharges, and travel times. Those floods that have been used in the testing are: 1976 Teton Dam, 1972 Buffalo Creek Coal-Waste Dam, 1889 Johnstown Dam, 1977 Toccoa (Kelly Barnes) Dam, and the 1977 Laurel Run Dam floods. However, only the Teton flood will be presented herein.

The Teton Dam, a 300 ft. high earthen dam with a 3,000 ft. long crest, failed on June 5, 1976, killing 11 people making 25,000 homeless, and inflicting about $400 million in damages to the downstream Teton-Snake River Valley. Data from a Geological Survey Report by Ray, et al. (1977) provided observations on the approximate development of the breach, description of the reservoir storage, downstream cross-sections and estimates of Manning's n approximately every 5 miles, indirect peak discharge measurements at three sites, flood peak travel times, and flood peak elevations. The inundated area is shown in Fig. 7.

The following breach parameters were used in DAMBRK to reconstitute the downstream flooding the downstream flooding due to the failure of Teton Dam: $\tau = 1.25$ hrs, $BB = 150$ ft, $z = 0$, $h_{bm} = 0.0$, $h_f = h_d = h_o = 261.5$ ft. Cross-sectional properties at 12 locations shown in Fig. 7 along the 60 mile reach of the Teton-Snake River Valley below the dam were used. Five top widths were used to describe each cross-section. The downstream valley consisted of a narrow canyon (approx. 1,000 ft. wide) for the first 5 miles and thereafter a wide valley which was inundated to a width of about 9 miles. Manning's n values ranging from 0.028 to 0.047 were provided from field estimates by the Geological Survey. Values of Δx between cross-sections gradually increased from 0.5 miles near the dam, to 1.5 miles near the downstream boundary at the Shelly gaging station (valley mile 59.5 downstream from the dam). The reservoir surface area-elevation values were obtained from Geological Survey topo maps. The downstream boundary was assumed to be channel flow control as represented by a loop rating curve given by Eq. (40).

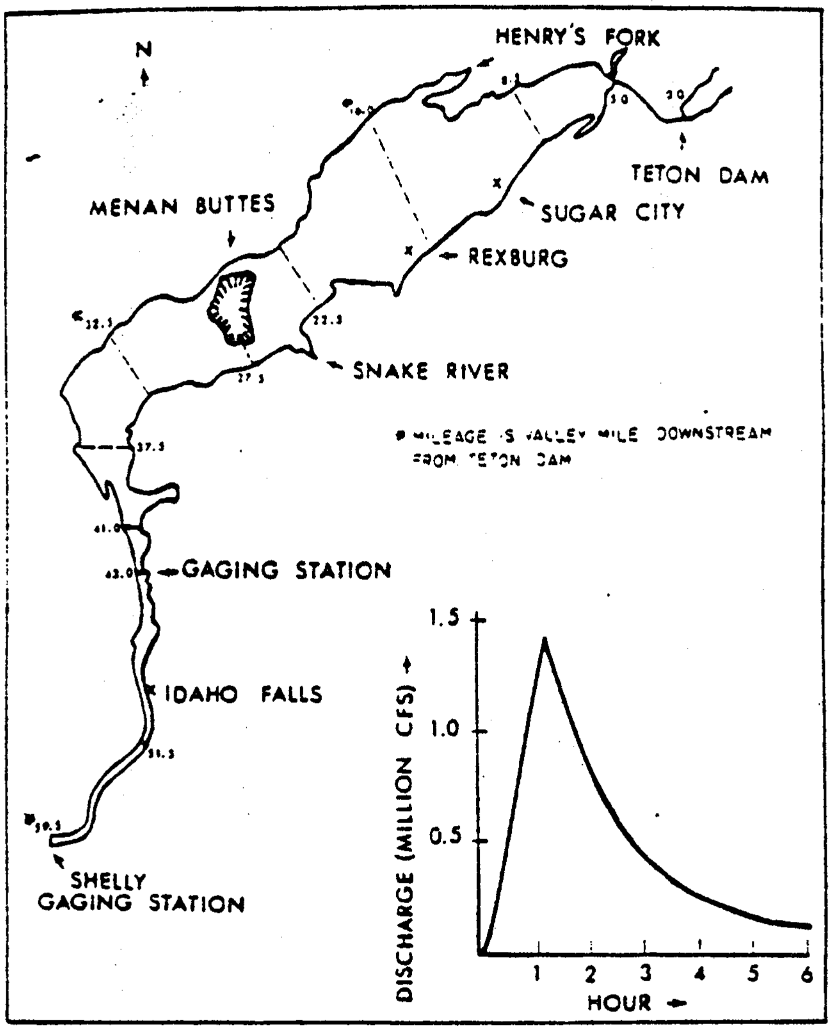

Figure 7. Outflow Hydrograph and Flooded Area Downstream of Teton Dam

The computed outflow hydrograph is shown in Fig. 7. It has a peak value of 1,652,300 cfs (cubic feet per second), a time to peak of 1.25 hrs, and a total duration of about 6 hrs. The peak is about 20 times greater than the flood of record. The temporal variation of the computed outflow volume compared within 5 percent of observed values. The computed peak discharge values along the 60-mile downstream valley are shown in Fig. 8 along with three observed (indirect measurement) values at miles 8.5, 43.0, and 59.5. The average difference between the computed and observed values is 4.8 percent. Most apparent is the extreme attenuation of the peak discharge as the flood wave progresses though the valley. Two computed curves were assumed, i.e., $q_m = 0$; and a second in which the losses were assumed to be uniform along the valley. The losses were assumed to vary from 0 to a maximum of $q_m = -0.30$ and were accounted for in the model through the q term in Eq. (51). Losses were due to infiltration and detention storage behind irrigation levees.

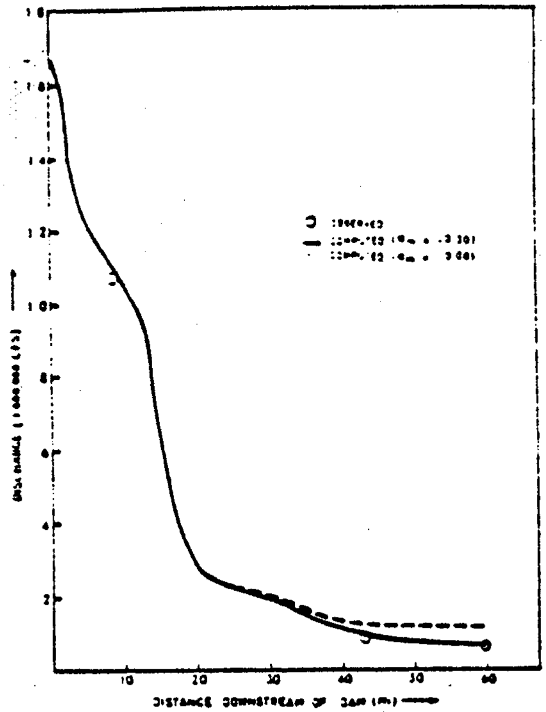

Figure 8. Profile of Peak Discharge from Teton Dam Failure

The a priori selection of the breach parameters (τ and BB) causes the greatest uncertainty in forecasting dam-break flood waves. The sensitivity of downstream peak discharges to reasonable variations in τ and $\bar{b}$ is shown in Fig. 9.

Although there are large differences in the discharges (+45 to -25 percent) near the dam, these rapidly diminish in the downstream direction. After 10 miles the variation is +20 to -14 percent, and after 15 miles the variation has further diminished (+15 to -8 percent). The tendency for extreme peak attenuation and rapid damping of differences in the peak discharge is accentuated in the case of Teton Dam due to the presence of the very wide valley. Had the narrow canyon extended all along the 60-mile reach to Shelly, the peak discharge would not have attenuated as much as the differences in peak discharges due to variations in τ and $\bar{b}$ would be more persistent. In this instance, the peak discharge would have attenuated to about 350,000 rather than 67,000 as shown in Fig. 9, and the differences in peak discharges at mile 59.5 would have been about 27 percent as opposed to less than 5 percent as shown in Fig. 9.

Computed peak elevations compared favorably with observed values. The average absolute error was 1.5 ft, while the average arithmetic error was only -0.2 ft.

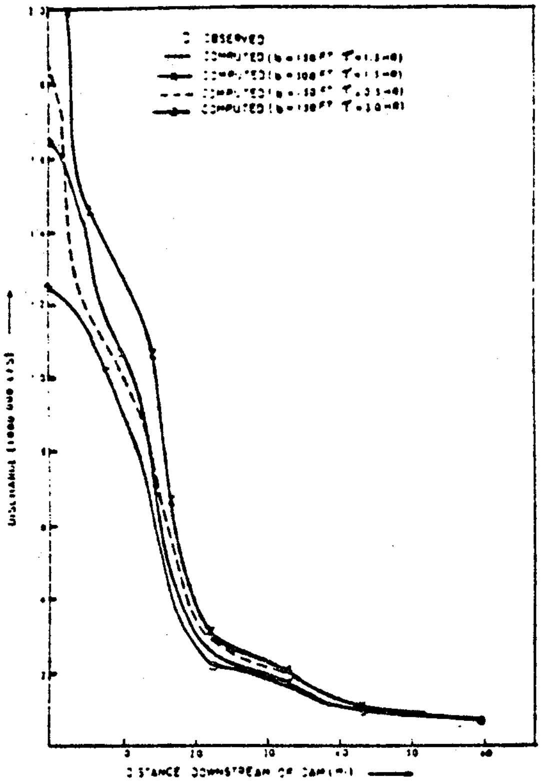

Figure 9. Profile of Peak Discharge from Teton Dam Failure Showing Sensitivity of Various Input Parameters.

The computed flood peak travel times and three observed values are shown in Fig. 10. The differences between the computed and observed are about 10 percent for the case of using the estimated Manning's n values and about 1 percent if the n values are slightly increased by 7 percent.

As mentioned previously, the Manning's n must be estimated, especially for the flows above the flood of record. The sensitivity of the computed stages and discharges of the Teton flood due to a substantial change (20 percent) in the Manning's n was found to be as follows: 1) 0.5 ft. in computed peak water surface elevations or about 2 percent of the maximum flow depths, 2) 16 percent deviation in the computed peak discharges, 3) 0.8 percent change in the total attenuation of peak discharge incurred in the 60-mile reach from Teton Dam to Shelly, and 4) 15 percent change in the flood peak travel time to Shelly. These results indicate that Manning's n has little effect on peak elevations or depths; however, the travel time is affected by nearly the same percent that the n values are changed.

A typical simulation of the Teton flood as described above involved 73 Δx reaches, 55 hrs of prototype time, and an initial time step (Δt) of 0.06 hrs. Such a simulation run required only 19 seconds of CPU time on an IBM 360/196 computer system; the associated cost was less than $5 per run. Microcomputer runs require about 10 min for the Teton simulation.

Information on similar testing of DAMBRK on the Buffalo Creek flood can be found in Fread (1977, 1984a). The results showed a similar degree of comparison between computed and observed values.

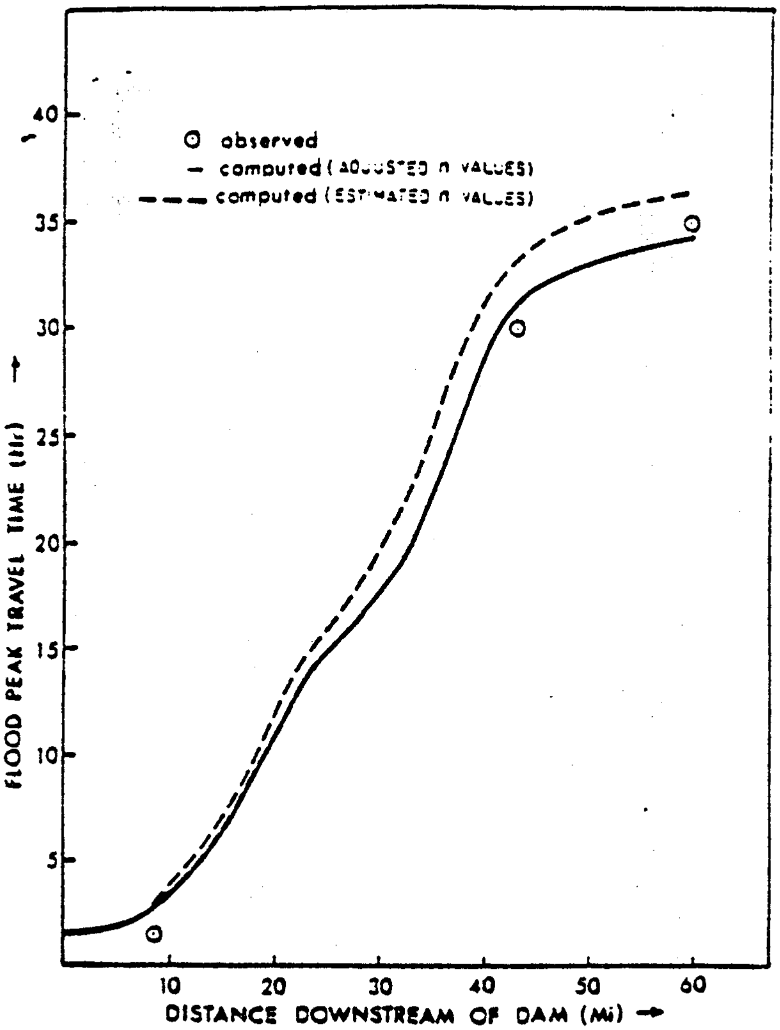

Figure 10. Travel Time of Flood Peak from Teton Dam Failure.

4. SMPDBK

SMPDBK is a simple model for predicting the characteristics of the floodwave peak produced by a breached dam (Wetmore and Fread, 1984). It will, with minimal computational resources (hand-held computers, micro-computers), determine the peak flow, depth, and time of occurrence at selected locations downstream of a breached dam. SMPDBK first computes the peak outflow at the dam, based on the reservoir size and the temporal and geometrical description of the breach. The computed floodwave and channel properties are used in conjunction with routing curves to determine how the peak flow will be diminished as it moves downstream. Based on this predicted floodwave reduction, the model computes the peak flows at specified downstream points. The model then computes the depth reached by the peak flow based on the channel geometry, slope, and roughness at these downstream points. The model also computes the time required for the peak to reach each forecast point and, if a flood depth is entered for the point, the time at which that depth is reached as well as when the floodwave recedes below that depth, thus providing a time frame for evacuation and fortification on which a preparedness plan may be based. The SMPDBK Model neglects backwater effects created by downstream dams or bridge embankments, the presence of which can substantially reduce the model's accuracy. However, its speed and ease of use together with its small computational requirement make it a attractive tool for use in cases where limited time and resources preclude the use of the DAMBRK Model. In such instances planners, designers, emergency managers, and consulting engineers responsible for predicting the potential effects of a dam failure may employ the model in situations where backwater effects are not significant for pre-event delineation of areas facing danger should a particular dam fail.

4.1 General Description

The SMPDBK model retains the critical deterministic components of the numerical DAMBRK model while eliminating the need for large computer facilities. SMPDBK accomplishes this by approximating the downstream channel as a prism, neglecting the effects of off-channel storage, concerning itself with only the peak flows, stage, and travel times, neglecting the effects of backwater from downstream bridges and dams, and utilizing dimensionless peak-flow routing graphs developed using the NWS DAMBRK model. The applicability of the SMPDBK model is further enhanced by its minimal data requirements; the peak flow at the dam may be calculated with only four readily accessible data values and the downstream channel may be defined by a single "average" cross-section, although prediction accuracy increases with the number of cross-sections specified.

Three steps make up the procedure used in the SMPDBK model. These are: (1) calculation of the peak outflow at the dam using the temporal and geometrical description of the breach and the reservoir volume; (2) approximation of the channel downstream of the dam as a prismatic channel; and (3) calculation of dimensionless routing parameters used with dimensionless routing curves to determine the peak flow at specified cross sections downstream of the dam.

4.2 Breach Description and Peak Outflow Computation

Since earthen dams generally do not fail completely nor instantaneously, the SMPDBK model allows for the investigation of partial failures occurring over a finite interval of time. And, although the model assumes a rectangular-shaped breach, a trapezoidal breach may be analyzed by specifying a rectangular breach width that is equal to the average width of the trapezoidal breach. Failures due to overtopping of the dam and/or failures in which the breach bottom does not erode to the bottom of the reservoir may also be analyzed by specifying an appropriate "H" parameter which is the elevation of the reservoir water surface elevation when breach formation commences minus the final breach bottom elevation (i.e., "H" is the depth to which the breach cuts).

The model uses a single equation to determine the maximum breach outflow and the user is required to supply the values of four variables for this equation. These variables are: 1) the surface area (A_s, acres) of the reservoir; 2) the depth (H, ft) to which the breach cuts; 3) the time (t_f, minutes) required for breach formation; and 4) the final width (B_r, ft) of the breach. These parameters are substituted into a broad-crested weir flow equation to yield the maximum breach outflow (Q_{bmax}) in cfs, i.e.

$$Q_{bmax} = Q_o + 3.1\, B_r \left(\frac{C}{\frac{t_f}{60} + \frac{C}{\sqrt{H}}}\right)^3 \tag{65}$$

where: $$C = \frac{23.4\, A_S}{B_r} \tag{66}$$

and Q_o is the spillway flow and overtopping crest flow which is estimated to occur simultaneously with the peak breach outflow.

Once the maximum outflow at the dam has been computed, the depth of flow produced by this discharge may be determined based on the geometry of the channel immediately downstream of the dam, the Manning "n" (roughness coefficient) of the channel and the slope of the downstream channel. This depth is then compared to the depth of water in the reservoir to find whether it is necessary to include a submergence correction factor for tail-water effects on the breach outflow (i.e., to find whether the water down-stream is restricting the free flow through the breach). This comparison and (if necessary) correction allows the model to provide the most accurate prediction of maximum breach outflow which properly accounts for the effects of tailwater depth downstream of the dam.

The maximum breach outflow must be corrected iteratively for submergence resulting from tailwater effects if the computed maximum outflow stage (h_{max}) is greater than (0.67 h_{weir}) where h_{weir} is the head over the weir (breach) at time t_f as expressed by the following relation:

$$h_{weir} = \left(\frac{C}{\frac{t_f}{60} + \frac{C}{\sqrt{H}}}\right)^2 \tag{67}$$

where C is defined by Eq. (66).

If the ratio of (h_{max}/h_{weir}) is greater than 0.67, a submergence correction factor must be computed as follows:

$$K_s^* = 1 - 27.8 \left[\frac{h_{max}}{h_{weir}} - 0.67\right]^3 \tag{68}$$

This value for K_s* is substituted into Eq. (69) to obtain an <u>averaged</u> submergence correction factor given by the following:

$$K_s^k = \frac{K_s^* + K_s^{k-1}}{2} \tag{69}$$

where the k superscript is the iteration counter and the first iteration value for K_s^o is 1. This correction factor is applied to the breach outflow as follows

$$Q_b^k = K_s^k \; Q_b^{k-1} \tag{70}$$

where: $Q_b^{k-1} = Q_{bmax}$ is the first iteration. The corrected breach outflow (Q_b^k) is then used to compute an outflow depth (h_{max}^k). The computation of h_{max} is described later via Eq. (75). Also, because there is decreased flow through the breach, there is less drawdown. Thus, the head over the weir (h_{weir}) must be recalculated using the relation:

$$h_{weir}^k = h_{weir}^{k-1} + \left(Q_b^{k-1} - Q_b^k\right) \frac{t_f \text{ (sec.)}}{2A_s \text{ (sq.ft.)}} \tag{71}$$

Now the ratio of the two new values, h_{max}^k / h_{weir}^k is used in Eq. (68) to compute a new submergence correction factor. If the new maximum breach outflow computed via Eq. (70) is significantly different (± 5%) from that computed in the previous iteration, the procedure is repeated. Generally, within two or three iterations the K_s value will converge and a suitable value for the maximum breach outflow (Q_b) is achieved which properly accounts for the effects of submergence.

4.3 Channel Description

The river channel downstream of the dam to the specified routing point is approximated as a prismatic channel by defining a single cross-section (an average section that incorporates the geometric properties of all intervening sections via a distance weighting technique) and fitting a mathematical function that relates the section's width to depth. This prismatic representation of the channel allows easy calculation of flow area and volume in the downstream channel which is required to accurately predict the amount of peak flow attenuation.

Approximating the channel as a prism requires three steps. First, topwidth vs. depth data must be obtained from topographic maps or survey notes. For each depth (h_i), a distance weighted topwidth $\bar{B}_i$ is defined producing a table of values that may be used for fitting (using least-squares or a log-log plot) a single equation of the form $\bar{B} = \bar{K}h^{\bar{m}}$ to define the prismatic channel geometry. The fitting coefficients ($\bar{K}$ and $\bar{m}$) are computed using the following least squares algorithm:

$$\bar{m} = \frac{\sum \left[(\log h_i)\ (\log \bar{B}_i)\right] - \dfrac{(\sum \log h_i)\ (\sum \log \bar{B}_i)}{I}}{\sum (\log h_i)^2 - \dfrac{(\sum \log h_i)^2}{I}} \tag{72}$$

$$\log \bar{K} = \frac{\sum \log \bar{B}_i}{I} - \bar{m}\ \frac{\sum \log h_i}{I} \tag{73}$$

$$\bar{K} = 10^{(\log \bar{K})} \tag{74}$$

After computing $\bar{K}$ and $\bar{m}$, the depth (h) may be computed for a given discharge (Q) by using the Manning equation, i.e.,

$$h = (Q/a)^b \tag{75}$$

where: $$a = \frac{1.49}{n} S^{1/2} \frac{\bar{K}}{(\bar{m}+1)^{5/3}} \tag{76}$$

$$b = 3/(3\bar{m}+5) \tag{77}$$

Also, S is the channel bottom slope (ft/ft), and n is the Manning n appropriate for the section of river-valley associated with the computed depth (h). In this manner h_{max} of Eq. (68) can be computed if Q_{bmax} is substituted for Q and the fitting coefficients $\bar{K}$ and $\bar{m}$ apply only to the tailwater section.

4.4 Downstream Routing

The peak outflow discharge determined in the preceding step may be routed downstream using the dimensionless routing curves. (See Fig. 11-13.) These curves were developed from numerous executions of the NWS DAMBRK Model and they are grouped into families based on the Froude number associated with the floodwave peak, and have as their X-coordinate the ratio of the downstream distance (from the dam to a selected cross-section) to a distance parameter (X_c). The Y-coordinate of the curves used in predicting peak downstream flows is the ratio of the peak flow at the selected cross section to the computed peak flow at the dam. To determine the correct family and member curve that most accurately predicts the attenuation of the flood, certain routing parameters must be defined.

The distinguishing characteristic of each curve family is the Froude number developed as the floodwave moves downstream. The distinguishing characteristic of each member of a family is the ratio of the volume in the reservoir to the average flow volume in the downstream channel. Thus it may be seen that to predict the peak flow of the floodwave at a downstream

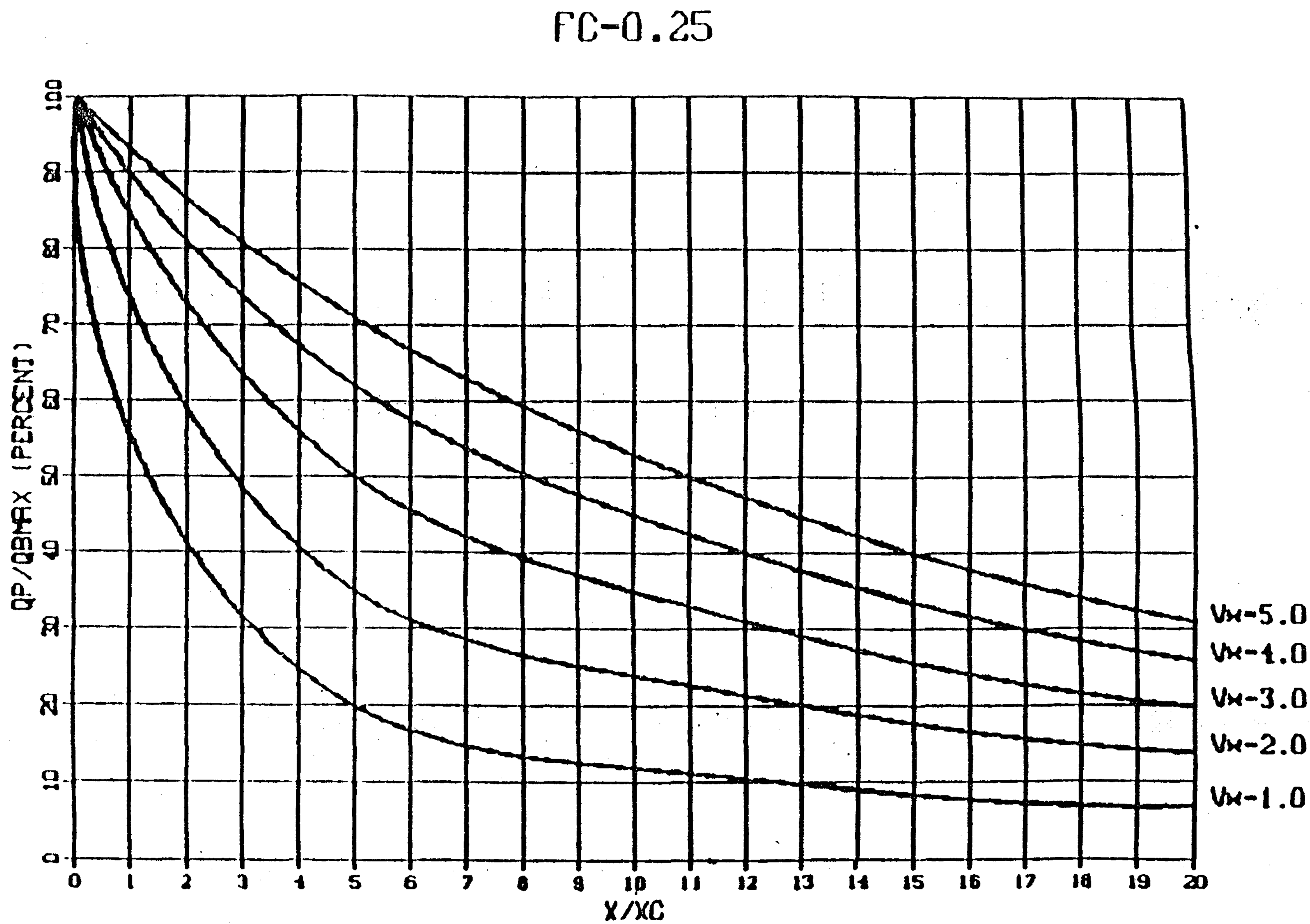

Figure 11. Simplified Routing Curves for F_c = 0.25.

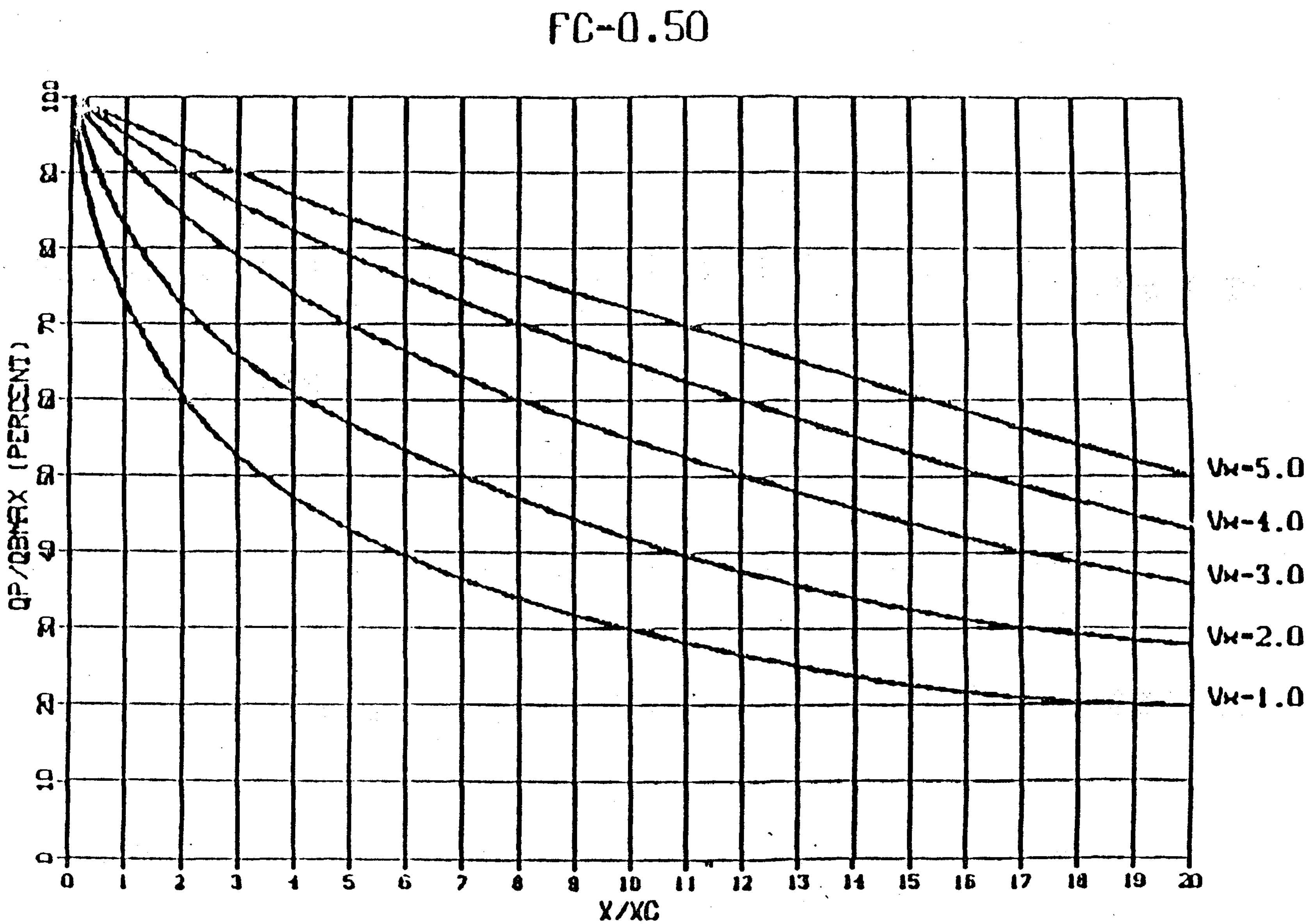

Figure 12. Simplified Routing Curves for F_c = 0.50.

FC-0.75

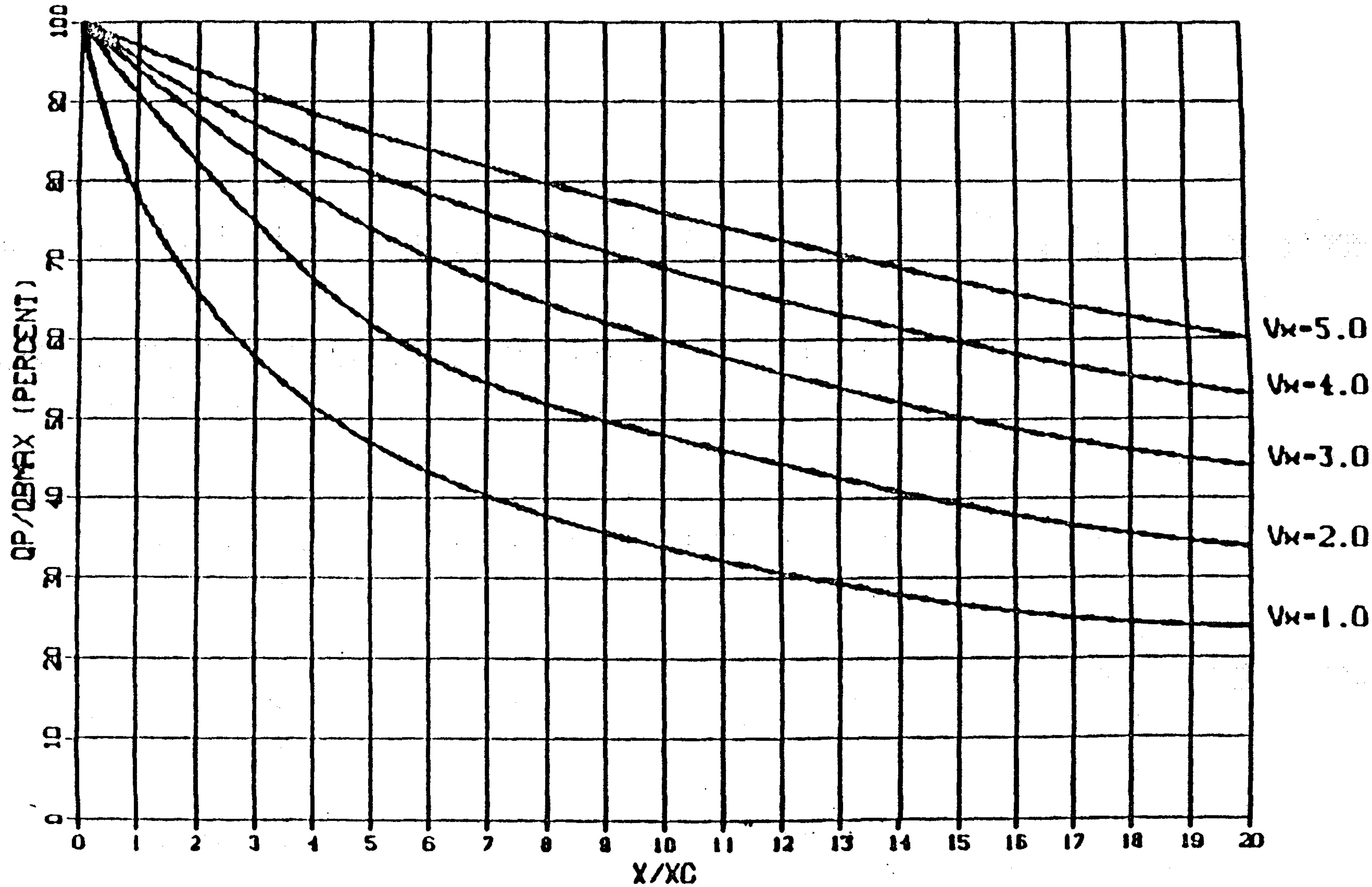

Figure 13. Simplified Routing Curves for F_c = 0.75.

point, the desired distinguishing characteristic of the curve family and member must be determined. This determination is based on the calculation of the Froude number and the volume ratio parameter. To specify the distance in dimensionless form, the distance parameter must also be computed.

The distance parameter (X_c) is calculated using Eq. (78) as follows:

$$X_c \text{ (ft)} = \frac{(\bar{m}+1)}{\bar{K}} \; \frac{VOL_r}{H_d^{\bar{m}+1}} \; \frac{6}{1 + 4\,(0.5)^{\bar{m}+1}} \tag{78}$$

where: VOL_r = volume in reservoir (cubic ft)

$\bar{K}$ & $\bar{m}$ = average channel geometry fitting coefficients

H_d = height of dam (ft)

Within the distance (X_c) in the downstream reach, the floodwave attenuates such that the depth at point X_c is h_x (see Fig. 14), which is a function of the maximum depth (h_{max}). The average depth ($\bar{h}$) in this reach is:

$$\bar{h} = \frac{h_{max} + h_x}{2} = \theta\, h_{max} \tag{79}$$

where θ is an empirical weighting factor that must be determined iteratively. The starting estimate for θ is 0.95.

The average hydraulic depth (D_c) in the reach is given by Eq. (80) as follows:

$$D_c = \frac{\theta\, h_{max}}{m+1} \tag{80}$$

The average velocity in the reach is given by the Manning equation, i.e.,

$$V_c = \frac{1.49}{n}\, S^{1/2}\, \left(D_c\right)^{2/3} \tag{81}$$

where S is the slope of the channel from the dam to the routing point.

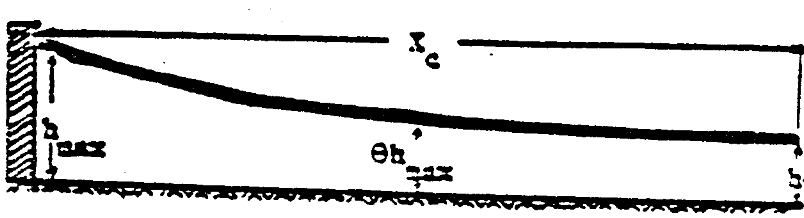

Figure 14. Attenuation of depth downstream of dam.

The average velocity (V_c) and hydraulic depth (D_c) are substituted into Eq. (82) to determine the average Froude number (F_c) in the reach as follows:

$$F_c = \frac{\bar{V}_c}{\sqrt{g D_c}} \tag{82}$$

where: g = 32.2 ft/sec^2 (acceleration of gravity).

The dimensionless volume parameter (V^*) that identifies the specific member of the curve family for the computed Froude number is the ratio of the reservoir storage volume to the average flow volume within the X_c reach. The average cross-sectional area of flow (A_c) is given by Eq. (83) as follows:

$$A_c = \bar{K}(\theta h_{max})^{\bar{m}} D_c \tag{83}$$

The volume parameter (V^*) is determined by dividing the average flow volume ($A_c X_c$) into the reservoir storage volume (VOL_r), i.e.,

$$V^* = \frac{VOL_r}{A_c X_c} \tag{84}$$

With the values of F_c and V^*, the specific curve (Fig. 11-13) can be used (interpolation may be necessary) to determine the routed discharge. The ordinate of the routing curve at $X^* = 1$ is the ratio of the peak flow (Q_p) at X_c to $Q_{b_{max}}$. Knowing Q_p, the stage (h_x) at X_c may be determined using Eq. (75) with the average channel fitting coefficients. The value of θ is checked by rearranging Eq. (79), i.e.,

$$\Theta = \frac{h_{max} + h_x}{2\, h_{max}} \tag{85}$$

If there is a significant difference in the new value of θ from the initial estimate of θ (e.g., ±5%), Eqs. (80)-(84) are recalculated and the new value of θ rechecked. Generally, within two iterations the value for θ will converge.

The distance(s) downstream to the forecast point(s) are non-dimensionalized using the following:

$$X_i^* = \frac{X_i}{X_c} \tag{86}$$

where X_i is the downstream distance to the i^{th} forecast point, i = 1,2,3,...The peak flow at X_i is determined from the proper family of routing curves and the ordinate of the specific V^* curve at X_i^*. Multiplying the value of this ordinate by $Q_{b_{max}}$ produces the peak flow (Q_p) at X_i miles downstream of the dam.

The time of occurrence of the peak flow at a selected cross section is determined by adding the time of failure to the peak travel time from the dam to that cross-section. The travel time is computed using the kinematic wave velocity which is a known function of the average flow velocity throughout the routing reach. The times of first flooding and "de-flooding" of a particular elevation at the cross section may also be determined.

The time of travel for the floodwave to X_i is computed by first calculating the reference flow velocity at the midpoint between the dam and X_i. The user must determine, from the routing curve, the peak flow ($Q_{x/2}$) at ($X_i/2$) miles downstream of the dam. This flow is multiplied by the factor ($0.3 + \bar{m}/10$) and substituted into Eq. (87) to find the reference depth (h_{ref}). Thus,

$$h_{ref} = \left(\frac{Q}{a}\right)^{b} \tag{87}$$

The reference hydraulic depth is given by Eq. (88), i.e.,

$$D_{x_i} = \frac{h_{ref}}{\bar{m}+1} \tag{88}$$

The reference flow velocity (V_{x_i}) in ft/sec is given by the Manning equation, i.e.,

$$V_{x_i} = \frac{1.49}{n} S^{1/2} \quad D_{x_i}^{\ 2/3} \tag{89}$$

This value for V_{x_i} is substituted into the wave celerity equation (Eq. (90)) to find the wave speed (c) in mi/hr, i.e.,

$$C = 0.682 \ V_{x_i} \quad [5/3 - 2/3 \ (\frac{\bar{m}}{\bar{m}+1})] \tag{90}$$

The time to peak is then given by Eq. (91) as follows:

$$t_{p_i} = t_f + \frac{X_i}{C} \tag{91}$$

where: t_p = time (hr) of peak occurrences

t_f = time (hr) of failure for dam

To compute the peak depth at mile X_i, K and m coefficients are fitted for that cross-section by substituting the specific depths and topwidths at mile X_i into Eqs. (72)-(74). Eq. (75) is used to find the peak depth (h_{x_i}) at mile X_i.

The SMPDBK allows the option to determine the time at which flooding commences and/or the time at which it ceases. To do this, a flow rate (Q_f) that corresponds with flood depth at the cross-section is computed as follows:

$$Q_f = a\, h_f^{\,b} \tag{92}$$

where: h_f = flood depth
and a and b are defined by Eqs. (76)-(77) using the K and m coefficients fitted for the cross-section at mile X_i.

This value for Q_f is substituted into Eq. (93) to determine the time to flooding (t_{fld}) as follows:

$$t_{fld} = t_{p_i} - \left(\frac{Q_{p_i} - Q_f}{Q_{p_i} - Q_o}\right) t_f \tag{93}$$

where: t_{p_i} = the time (hr) to peak calculated in Eq. (91)
t_f = the time (hr) of failure for the dam, and
Q_o = the flow (spillway/turbine/overtopping) other than flow.

To determine the time flooding ceases, t_d, the value of Q_f is substituted into the following relation:

$$t_d = t_{p_i} + \left(\frac{24.2\ VOL_r}{Q_{p_i} - Q_o} - t_f\right) \left(\frac{Q_{p_i} - Q_f}{Q_{p_i} - Q_o}\right) \tag{94}$$

where: VOL_r = the reservoir storage volume (ac-ft).

To route the peak flow downstream to cross-sections 3,4,..., the distance-weighted average cross-section must be determined between the dam and the routing point and new $\bar{K}$ and $\bar{m}$ parameters must be fitted to this average cross section. The distance-weighted average cross section may be determined as follows:

For each depth (h_i), the distance weighted topwidth ($\bar{B}_i$) is given by the relation:

$$\bar{B}_i = \frac{\frac{(B_{i,1} + B_{i,2})}{2}(X_2 - X_1) + \dots + \frac{(B_{i,J-1} + B_{i,J})}{2}(X_J - X_{J-1})}{(X_J - X_1)} \tag{95}$$

where: h_i = the i^{th} depth, i = 1,2,3 ... I (number of topwidths per cross-section

$B_{i,j}$ = the i^{th} topwidth (corresponding to the i^{th} depth h_i) at the j^{th} cross-section where j = 1,2,3, ... J (number of cross-sections)

$\bar{B}_i$ = the weighted i^{th} topwidth

X_j = the downstream distance to the j^{th} cross-section.

The table of values produced by defining a distance-weighted topwidth ($\bar{B}_i$) for each depth (h_i) may then be used for fitting a single equation of the form $B = \bar{K}h^{\bar{m}}$ to define the prismatic channel geometry. The fitting coefficients $\bar{K}$ and $\bar{m}$ may be computed using the least squares algorithm given in Eqs. (72)-(74).

With these weighted average $\bar{K}$ and $\bar{m}$ coefficients, the peak depth is recomputed at the dam using new routing parameters from Eqs. (79)-(84). The flow may then be routed to cross-section 3,4,.... by following the procedure given above.

4.5 Model Testing and Limitations

In both real-time forecasting and disaster preparedness planning, there is a clear need for a fast and economical method of predicting dam-break floodwave peak stages and travel times. The SMPDBK model fills this need, producing such predictions quickly, inexpensively and with reasonable accuracy. For example, in test analyses of the Teton and Buffalo Creek dam failures where the progression of the floodwave was not affected by backwater, approximating the channel as a prism, calculating the maximum breach outflow and stage at the dam, defining the routing parameters, and evaluating the peak stage and travel time to the forecast points required less than 20 minutes of time with the aid of a non-programable hand-held calculator while the average error in forecasted peak flow and travel time was 10-20% with stage errors of approximately 1 ft. Furthermore, comparisons of SMPDBK model results with DAMBRK model results from test runs of theoretical dam breaks show the simplified model produces average errors of 10% or less. The authors had the advantages, however, of prior experience with the model and possession of all required input data, the collection of which consumes precious warning response time in a dam-break emergency.

The SMPDBK Model can be a very useful tool in preparing for and during a dam failure event, however, the user must keep in mind the model's limitations (Fread 1981). First of all, as with all dam breach flood routing models, the validity of the SMPDBK model's prediction depends upon the accuracy of the required input data. To produce the most reliable results, the user should endeavor to obtain the best estimates of the various input parameters that time and resources allow. Secondly, because the model assumes normal, steady flow at the peak, the backwater effects created by downstream channel constrictions such as bridge embankments or dams cannot be accounted for and the model will predict peak depths upstream of the constriction that may be substantially lower than those actually encountered, while peak depths downstream of the constriction may be over predicted. Finally, because the "slowing down" of the floodwave caused by temporary off-channel dead storage is not accounted for by the model, the predicted time to peak at a certain point may be somewhat shorter than the actual time to peak. Recognizing these limitations and exercising good engineering judgment, the SMPDBK model may provide useful dam break flood inundation information with relatively small expense of time and computing resources.

5. SUMMARY AND CONCLUSIONS

Three NWS models for predicting the flooding due to dam failures were presented. The Breach Model can aid the hydrologist/engineer in determining the properties of the piping or overtopping initiated breach of an earthen dam. This information can be used in conjunction with historical breach data to create the dam breach hydrograph and route it through the downstream channel-valley using the complex DAMBRK Model or the simplified SMPDBK Model. The choice of either the DAMBRK or SMPDBK model is influenced by the available time, data, computer facilities, modeling experience, and required accuracy for each dam break analysis. Complexities in the downstream channel valley such as highway/railway embankment-bridges, significant channel constrictions, levee overtopping, flow volume losses, downstream dams, weirs, lakes require the DAMBRK Model to be used rather than the SMPDBK Model since latter model ignores such factors.

Notwithstanding the capabilities of state-of-the-art models (BREACH, DAMBRK, SMPDBK) the accuracy of the predicted magnitude and timing of downstream flood inundation can be subject to significant error (two feet or more in the crest profile) due to inaccuracies in the following: 1) the reservoir inflow computed from hydrologic precipitation-runoff models; 2) the breach characteristics; 3) the downstream cross-section properties; 4) the estimated flow resistance coefficients; 5) the neglected effects of transported debris of flow resistance and blockage of constricted cross sections; 6) the neglected infiltration and detention storage losses of flood volume; 7) the neglected sediment transport effects on bottom elevation and flow resistance of the downstream channel-flood plain and 8) the highly turbulent flows and complex flow patterns not adequately described by one-dimensional flow equations.

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