INVESTIGATION OF DAM FAILURES

by

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RESUME

Dam disasters are an increasing problem. Its significance is heightened especially when viewed in its broad and complex perspectives. Study of dam failures in the past resulted in the advancement of our knowledge of actual behaviour of dams and also offers substantial potential for the future. Yet, for certain reasons, such mission-oriented investigations are seldom carried out or supported. The data on a large number of accidents, failures and disasters of dams and embankments have been studied to identify the dimensions of the problem as well as to assess the implications of the existing methods of ensuring safety of dams and reservoirs. It is suggested that a comprehensive approach to the overall problem of dam-safety is a viable method for instituting satisfactory schemes of action. Undoubtedly some degree of cognizance of this formalism is indicative in the scant literature on this topic. However, this approach has yet to be fully tested so that its actual benefits and costs are determined.

The thesis comprehensively surveys all available major literature on various aspects of the safety of dams and reservoirs. The focus in on a broad re-evaluation of existing concepts which has important implications in observed dam failure mechanisms. Thus, the study provides a deeper insight into the complex safety problems and suggests a useful modus operandi for improvement of dams and reservoirs.

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References

PUBLICATIONS ON THE THESIS

(1) "Dam Failures: An Assessment", paper presented at the Annual Meeting, Engineering Institute of Canada, Ottawa, September 16-18, 1970 (to be published in the Engineering Journal).

(2) "Human Dimensions of Dam Disasters", Resources Paper, Department of Environment and Renewable Resources, Ottawa, 1971.

(3) "Social Science Aspects of Dam Failures", to be presented at the Specialty Conference, American Society of Civil Engineers, Lincoln, Nebraska, 1971.

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CHAPTER I

INTRODUCTION

Nature has provided an unlimited storage of water in the seas. About ninety-seven per cent of all the water in the world, that is one quadrillion(10^{15}) acre-feet, is in the oceans. Unfortunately, most of this water is saline, whereas freshwater is an essential resource for the growth of our standard of living. Thus far it has not been possible in general, to exploit economically the vast storage of saline water for consumptive uses. Also man has failed to adapt himself to the direct use of saline water. On the whole, our fresh water requirements are supplied to a large extent by the operation of the natural hydrologic system, consisting primarily of evaporation, rainfall and streamflow. However, the various elements of the hydrologic cycle create complex problems that require a solution. In fact, the solution of several problems that arise from the stochastic nature of streamflow is to be found in the creation of a suitable storage reservoir. Other likely means of exercising control over hydrologic system devised so far do not offer any hope of eliminating the role of storage reservoirs.

NEED OF DAMS AND RESERVOIRS

There are frequent floods in the rivers and streams. At times the flows of water in the rivers, streams, lakes and the underground acquifers are so less that droughts occur. The natural hydrologic system thus creates many critical problems in water management. Human habitations are devastated by floods, and even the rich and prosperous civilizations are often severely strained by prolonged droughts. To mitigate these problems it is necessary to regulate the flow in rivers and lakes. In spite of the recent developments in cloud-seeding technology, it has not been possible to control the meteorological factors so that the entire hydrologic system may be operated in conjunction with the varying demands of freshwater and the limitations of the natural streams and channels. Thus, some measure of control is exercised on extreme fluctuations of the hydrologic system, by the creation of reservoirs which, in turn, requires construction of dams, embankments, dykes, etc. These provisions impose on some natural embankments and earth surfaces conditions of stress and strain to which they would otherwise not be subjected.

Also, storage of water by damming natural reservoirs or valleys is by far the most economic means known to our civilization to control and eliminate flood and drought. Impounding of water to meet higher damands during the period of low natural streamflow has required damming of natural valleys. So long as man remains dependent on the meteorologic factors, dams will continue to provide the much needed relief against the uncertainties of floods and droughts.

With the development of cheap nuclear energy and desalination technology, it may be possible to purify the sea water to the required standards and pump it up economically from the sea to the highest elevations. Obviously better water management practices could eliminate human sufferings casued by droughts. Even then it would not be possible to dispense with dams, embankments and reservoirs, as they are as yet the only effective means of flood control.

FAILURE OF DAMS AND RESERVOIRS

The destructive power of floods is well known to man since times immemorial. Experience shows that dams have been able to provide effective protection against floods to a large measure. Yet some failure of dams, that have resulted in heavy losses of lives and properties, have offered an opportunity to doubt their suitability, effectiveness, and reliability in providing necessary protection against flood disasters. In fact, failure of dams has much deeper implications compared to the nature induced flood disasters.

Construction of dams permits the regulation of flow in the downstream channel of rivers. Such a regulation gives incentive to development in the areas which would otherwise be flooded in the absence of the dam. However, failure of the dam virtually destroys almost all lives and properties lying on the path of the resulting flood and causes a serious socio-economic loss. Several items can be listed as the social costs of dam failures. It almost wipes out the total economic and social activities generated in its neighbourhood, it creates a strong public opposition to building of dams in future, and also it deprives society of the enormous benefits the dam would have projected had it survived and continued to operate. All these costs are seldom assessed, or at least realized, by those responsible for safety supervision of dams and reservoirs.

DIMENSIONS OF DAM SAFETY

Like the problem of pollution, reservoir hazards and dam disasters have caused increasing concern. There is also certain similarity between the two problems.

In the past, failures of dams and reservoirs received only scant public attention. The primary reasons were lack of public recognition of the problem as well as less frequent catastrophic consequences of previous dam disasters. Only in the recent past certain major reservoir disasters engaged public attention. The growth of population, affluence and standard of living of the people have had serious implications in expanding the public realization of the menace of dam failures, accidents and disasters. Obviously with increasing social awareness, the standards of safety demanded by people have enhanced considerably. In addition, with the rapidly increasing heights and numbers of large dams, the disaster potential of these dams have increased enormously. Thus, these two effects - enhanced safety demands and disaster potential - on our safety consciousness have resulted in increasing awareness for dam-safety. Such is true for pollution too.

With this concept in mind, it was considered necessary to make an in-depth study of the various aspects of dam safety. It was observed that the entire field of dam safety was characterized by a piece-meal approach to the whole problem of safety of dams. Hardly any comprehensive study on the safety of dams and reservoirs is now available to serve as a framework for such an investigation.

Dam failures have caused some degree of professional concern. However, the academics have shown very little, if any, interest in the overall problems and consequences associated with dam failures. This apathy stems from the fact that almost nothing is known in a comprehensive manner about the causes and implications of dam and reservoir failures. Difficulty in obtaining information on dam failures has rendered this field of enquiry apparently unproductive. Thus, the present study should be of considerable interest to all professionals.

At the 1970 Annual Conference of the Engineering Institute of Canada in Ottawa, several commentators pointed out the great significance of the current problems of dam safety. As such the study would serve as a basis for further research and would provide the required framework for a more comprehensive in-depth study of the problem.

CONTENTS OF THE THESIS

The thesis is the result of a comprehensive study of some 300 dams failures. In Chapter II, a brief overview of various contributions made toward documenting different types of dam and reservoir failures has been made. In fact, it briefly summarizes the numerous data sources of dam failures and their useful contents.

In Chapter III, some major dam failures have been studied in considerable depth. It thus focuses attention on some serious and current problems of dam safety and shows the effectiveness of available techniques to alleviate future problems of dam safety. The failures have been assessed in order to identify the serious shortcomings in the whole field of dam technology.

Chapter IV is a general overview of all the dam failures studied and discusses various mechanisms of failure in different types of dams, i.e. earthfill, masonry and concrete dams and cofferdams. It provides some useful data that are of great benefit to dam designers, supervision engineers and the academic researchers interested in the investigation of the behaviour of dams.

Chapter V discusses the various devices available for the safety of dams and reservoirs. It discusses in detail some of the technologic factors and methodologies which are important for the supervision of safety of dams and shows how failure of dams can be prevented by their successful application.

Chapter VI discusses the human dimensions of dam diaster and then summarizes some of the important dam-safety measures which could eliminate or minimize possible dam disasters along with their undesirable social and human consequences. It would remind the engineers and managers of their responsibilities to the society as well as their primary role in dam safety supervision, i.e. elimination or minimization of the social and economic costs of dam disasters.

CONCLUSION

In a nutshell, the study focuses attention on the various problems involving dam failures as well as the safety measures required to alleviate these problems to an acceptable limit. The study of dam and reservoir failures thus reveal that a comprehensive attack on dam failures could only be a satisfactory scheme of action. Firstly, there is a need to develop effective and efficient means to monitor and control the behaviour of dams and secondly, it is also relevant to devise ways and means to minimize the potential dam disaster costs. In fact, a major problem in the latter task is to implement the measures already available so that further knowledge can be obtained for meaningful assessment of benefits and costs. Obviously, a comprehensive safety system is probably the only answer to the whole question of dam disasters.

CHAPTER II

STATISTICS

Literature survey of dam and reservoir failures indicate that one of the earliest efforts to compile such accounts is due to Hill(1)* in 1902. On the other hand, probably the latest collection of statistics on various dam disasters, accidents and failures, far exceeding any available published listings on dam failures, has been catalogued along with a bibliography for each event by Babb and Mermel.(2) Last but not least is a draft compilation of dam disasters by Gruner,(3) which though yet unpublished, has the largest number of recorded events (dam failures) throughout the world. The material presented in this manuscript can hardly be considered complete and thus further work would be required to develop a comprehensive catalogue of dam disasters. At any rate, Table I shows in chronological order the names of various investigators and their contributions to the compendium on dam failures.

Schuyler,(4) in 1905, documented details of some dam failures in his famous treatise on reservoirs. Later, in 1920, Jorgensen(5) briefly described some one hundred dam failures that occurred between 1873 and 1918. In fact, he listed failures of all conceivable types of dams except arch dams; he did, however, notice a number of freak dams of this type. Jorgensen, thus predicted the failure of some arch dams with faulty alignment or in highly stressed condition in advance.

In 1930, a list of total, partial and incipient failures of 305 dams was compiled by R. J. Tipton(6) under the directions of Hinderlider. This listing was submitted to the American Society of Civil Engineers after subsequent revision in January, 1932. It really marked the beginning of a fully systematic

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*See references at the end of the report

TABLE I

INVESTIGATORS ON DAM FAILURES

S.N.	Investigators	Year	Reference Number	Remarks
1	Hill	1902	(1)	Some dam failures classified.
2	Schuyler	1905	(4)	Accounts of some dam failures.
3	Jorgensen	1920	(5)	About 100 dam failures described.
4	Wegman and Noetzli	1927	(6)	
5	Merriman	1930	(7)	
6	Hinderlider	1932	(8) and (9)	About 305 dam failures listed.
7	Schatz and Boesten	1936	(12)	Same as Hinderlider's list.
8	Middlebrooks	1953	(14)	Failures of Earth Dams.
9	Justin, Hinds and Creager	1945	(15)	Partial list of Earth Dam failures.
10	Mallet	1951	(16)	
11	Navarro	1953	(17)	Lists large number of dam failures.
12	Ambraseys	1960	(18)	Only earthquake failures of earth dams.
13.	Rao	1960	(20)	Failures of earth dams .
14	Casal	1960	(23)	
15	Morgenstern	1963	(25)	Lists 16 drawdown failures.
16	Sherard et al	1963	(19)	Discusses large number of earth dam failures.
17	Gruner	1966	(26)	About 300 dam failures have been classified.
18	Schnitter	1967	(28)	
19	Babb and Mermel	1968	(27)	About 700 dam failures have been cataloged.
20	Gruner	Undated	(31)	More than 1000 dam failures arranged in alphabetical order. Not yet complete .

effort to develop a comprehensive catalogue of dam failures, and thus was a significant contribution to the knowledge of reservoir safety, also inducing better awareness of reservoir failure problems throughout the world.

Hinderlider(7) recognized eighteen major causes of dam failures and catagorized all known failures to that date (1930) under subheads shown in Table II.

Observations of Hinderlider are interesting to note. He showed that Jorgensen's conclusion that "earth dams fail quite freely," drawn a decade ago, remained effectively valid and was probably further strengthened by the statistics. However, it should be remembered that the number of earth dams built is greater than those of other types; the capital and recurring costs of earth dams are also lesser and the factor of safety is rather low, being comparatively weaker against external forces. At any rate, little evidence is available in these statistics to support Takase's(8) contention that the factor of safety of earth dams tend to increase with age unless disturbed by some external forces in the early stages of its life.

Middlebrooks,(9) in 1953, presented a compilation of 206 earth dam failures, the majority of them occurring in North America. The height of these dams varied between 6 feet to 250 feet and the last failure on record occurred in 1940. About 34 per cent of the dam failures were due to overtopping of the structure by flood water. Most of the earth dams that were overtopped failed with few exceptions. Some were overtopped during construction by clogging of outlets, while others failed due to inadequate spillway.

Piping was responsible for the failure of approximately 30 per cent of the dams. The following are possible ways in which piping has been reported

TABLE II

Causes of Dam Failures

1. Overtopping due to inadequate spillway
2. Inadequate spillway, Overtopping by flood wave due to failure of dam above.
3. Inadequate cut-offs. Porous foundation allowing leakage and erosion under earth dam, and/or sliding in rigid types.
4. Faulty construction; material not properly compacted in earth dams; poor construction in masonry dams.
5. Inadequate cut-offs around conduits in earth and rock-fill dams.
6. Faulty design of section; slopes too steep in earth dams; section too light in masonry dams.
7. Inadequate means for stream control during construction.
8. Excessive quantities of clay or other classes of fine material.
9. Ice-pressure or disintegrating effect of.
10. Improper operation or inadequate maintenance.
11. Burrowing rodents.
12. Poor materials, including soluble salts.
13. Unstable or structurally weak foundation.
14. Conduits through earth or rock-fill dams not properly supported to prevent settlement or failure. Improper location of valves.
15. Insufficient provision against erosion from back wash below dam or spillway.
16. Earthquakes.
17. Miscellaneous and undetermined.
18. Failure of bottom in small waterworks reservoirs.

Note: The above causes of dam failure were identified by Tipton (Ref. 8) and Hinderlider (Ref. 9).

to occur in earth dams:

- (a) Piping through dam body and settlement cracks,
- (b) Piping along outlet,
- (c) Piping under spillway,
- (d) Foundation piping,
- (e) Piping between fill and foundation,
- (f) Piping through abutment,
- (g) Transverse cracking and piping, and
- (h) Sloughing.

Other reasons of unsatisfactory performance and failure of earth dams are seepage, settlement, slides and poor design and construction. It may be noted that improper maintenance, inadequate overall supervision of design, construction and operation are quite important factors that cause dam failures. Full understanding of the causes of failures, therefore, is a prerequisite to the design of a satisfactory preventive and corrective safety system for dams and reservoirs. The following mechanisms could further be identified from Middlebrook's statistics:

- (1) Seepage
    - (a) embankment seepage, and
    - (b) seepage through foundation.
- (2) Settlement
    - (a) embankment settlement,
    - (b) foundation settlement,
    - (c) settlement beneath spillway, and
    - (d) settlement due to saturation.

(3) Slides
  - (a) core pressure slide (during construction),
  - (b) seepage slide,
  - (c) drawdown slide,
  - (d) Foundation slide,
  - (e) upstream slope slide,
  - (f) embankment slide,
  - (g) earthquake slide,
  - (h) slide due to core being too large.

(4) Poor design and construction
  - (a) erosion due to spillway,
  - (b) spillway collapsed,
  - (c) break in dam,
  - (d) conduit break,
  - (e) abutment failure at contact or crack,
  - (f) core too flat,
  - (g) foundation failure during construction,
  - (h) riprap displaced by wave action and loss of filter through riprap or concrete blocks,
  - (i) wave erosion of concrete on upstream slope,
  - (j) floating logs displaced hand placed riprap,
  - (k) slopes too steep,
  - (l) poor compaction,
  - (m) disintegration of porous concrete.

Then, in 1945, Justin, Hinds and Creager (10) reported a partial list of earth dam failures in their monumental treatise "Engineering of Dams."

TABLE III

STATISTICAL DATA OF DEFECTS AND FAILURES OF EARTH DAMS DUE TO SEISMIC ACTIVITY

S.N.	Name of Dam	Country	Year of Contr.	Type	Height (ft)	Age	Accel.	Damage
1	SAN ANDREAS	USA	1870	E	95	36	25%	C
2	UPPER CRYST. SPRINGS	USA	1878	E	75	28	25%	C; Sh
3	OLD SAN ANDREAS	USA		E	28		25%	Sh
4	LAKE RANCH	USA	1877	E	38	29	25%	ND
5	UPPER HOWELL	USA	1878	E	36	28	20%	C;S
6	LOWER HOWELL	USA	1877	E	38	29	20%	B
7	Name not known (I)	USA		E&C			15%	F
8	BEAR GULCH	USA	1896	E	45	10	12%	ND
9	LAGUNITA	USA	1900	E	15	6	20%	ND
10	PILARDITOS	USA	1866	E	95	40	10%	ND
11	SAN LEANDRO	USA	1876	E&H	125	30	7%	ND
12	PIEDMONT	USA	1905	E	51	1	5%	C;S
13	TENESCAL	USA	1868	E&H	105	38	7%	ND
14	VOLCANO LAKE	Mexico		E	12	15	5%	F
15	FAIRMONT	USA	1912	H	121	4	4%	ND
16	ONO	Japan	1912	E	124	11	15%	CC;S
17	UPPER MURAYAMA	Japan	1923	E	80	1	12%	CC;S
18	LOWER MURAYAMA	Japan	1927	E	101	4	12%	C
19	Tokyo Embankments	Japan		E	-		25%	F
20	SHEFFIELD	USA	1917	E	30	8	10%	F
21	CHATS ORTH	USA	1918	E	44	12	6%	C;IS
22	Hawkes Bay Embankment	N.Z.		E			20%	F
23	MALPASSO	Peru	1936	R	255	2	4%	ND
24	LAGUNA	USA	1938	E	50	2	4%	
25	Imperial Val. Canals	USA		E				F
26	MORENO	USA	1896	R	165	44	2%	ND
27	COGOTI	Chile	1938	R	275	5	14%	S
28	BRIDGEPORT	USA	1924	E	70	19	3%	ND
29	BOZ'SUISKAYA	USSR	1943	E	90	2	7%	ND
30	SUMMIT	USA		E			8%	CC

S.N.	Name of Dam	Country	Year of Constr.	Type	Height (ft)	Age	Accel.	Damage
31	RUBY	USA					12%	
32	Hosorogi Embankment	Japan		E	28		45%	F
33	NORTH END	USA		E			8%	C;S
34	POGGIO CONCELLI	Italy	1943	E	56	7	7%	ND
35	YUBA	USA	1910	E	28	41	4%	S;SL
36	BONQUET	USA	1934	E	190	18	7%	ND
37	BUENA VISTA	USA	1890	E	17	62	25%	S;CC;IS
38	DRY CANYON	USA	1912	H	67	40	5%	S;IS;C
39	ISABELLA	USA	1953	E	185		4%	
40	SOUTH HAIWEE	USA	1913	H	91	39	5%	S;IS;C
41	TEJON RANCH	USA	1946	E	32	6	25%	ND
42	Name not known (II)	USA		E			25%	F
43	TINEMAHA	USA	1928	E	32	24	3%	ND
44	LONG VALLEY	USA	1940	E	126	12	4%	ND
45	DEER CREEK	USA		E			7%	C
46	DVVIS	USA	1950	R	138		2%	ND
47	LAHONTAN	USA	1915	E	125	39	3%	ND
48	COLEMAN	USA		E&C			8%	F
49	SAGUSPE	USA		E			8%	F
50	ROGERS	USA		E&C			7%	F
51	ARCATA	USA	1937	E	55	17	8%	ND
52	EUREKA	USA		E			8%	ND
53	SAINT MARY	USA	1928	E	55	27	7%	C;S
54	KAIRAKUMSKAYA	USSR	1957	H	117	1	4%	ND
55	MINGUECHAURSKAYA	USSR	1957	H	271	1	4%	ND
56	CACHUMA	USA	1953	E&R	275	4	1%	ND
57	PINZANES	Mexico	1956	R	180	1	4%	ND
58	HEBGEN	USA	1913	E	87	46	10%	CC;S

E = Earth dam
R = Rock-fill
H = Hydraulic (aH = Semi-hydro.)
C = Concrete (gravity-type)

Ambrasays also tabulated data on length, crest width, upstream slope, downstream slope, impervious element, foundation conduit, epicentre distance, and seismic intensity for the above dams. For the full data the original paper may be seen.

Similarly, Mallet(11) in France, and Navarro and Segva(12) in Spain, published accounts of a large number of dam and reservoir failures throughout the world.

In 1960, Ambraseys(13) presented a paper "on the seismic behaviour of earth dams," documenting in a comprehensive manner the results of various investigations during 26 different earthquakes which affected 58 earth dams throughout the world. The statistical data compiled in the paper has been summarized in Table III. The first of the recorded earthquakes described in this paper occurred in San Francisco on April 18, 1906. There were about 13 dams within the area of its influence. The last but not least of the seismic events described occurred at Hebgen Lake on August 17, 1959. This dam suffered some damage, including cracking and settling of the concrete spillway with attendant rather alarming leakage after the shocks.

Rao,(14) in 1960, documented in brief particulars, the type of failure, causes of failure and remedial measures undertaken in case of six earthdams in the United States, two in the United Kingdom and six of them in India. The dam failures studied are mentioned in Table IV. They occurred between 1918 and 1958. The height of the dams varied between 29 feet and 255 feet. The causes of most of these failures can be discerned in a more comprehensive listing due to Middlebrooks, mentioned earlier.

Morgenstern,(15) in 1963, reported the results of his investigation on stability of earth dam slopes during rapid drawdown. He listed 16 drawdown failures of earth embankments. Necessity to design upstream slopes safe against rapid drawdown was emphasized on account of frequent failures of this type observed in the past. There can thus be little doubt regarding the value of the statistics on dam failures to identify the directions of productive research.

TABLE IV

FAILURES DOCUMENTED BY RAO(20)

S.N.	Name of Dam	Country	Height in feet	Year of Failure
(1)	Calveras Dam	U.S.A.	240 ft.	1918
(2)	Apishapa Dam	U.S.A.	102 ft.	1923
(3)	Saluda Dam	U.S.A.	208 ft.	1932
(4)	Marshall Creek Dam	U.S.A.	90 ft.	1937
(5)	Fort Peck Dam	U.S.A.	255 ft.	1939
(6)	Kingsley Dam	U.S.A.	160 ft.	1941
(7)	Harrogate Dam	U.K.	29 ft.	1953
(8)	Chingford Embankment	U.K.	34 ft.	1937
(9)	Ashti Dam	India	58 ft.	1883, 1933
(10)	Palakmati Dam	India	48 ft.	1953
(11)	Ahraura Dam	India	75 ft.	1953
(12)	Arwar Dam	India	41 ft.	1956, 1957
(13)	Guddah Dam	India	93 ft.	1956, 1957
(14)	Kaddam Dam	India	74 ft.	1958

TABLE V

FAILURES AND DAMAGES DOCUMENTED BY SHERARD ET AL(19)

S.N.	Name of the dam	Country	Type of Failure
1.	Schofield	Utah, U.S.A.	Piping
2.	Sinkar Creek	Idaho, U.S.A.	Sloughing
3.	Fork Peck	Montana, U.S.A.	Hydraulic fill slide, downstream boils
4.	Apishapa	Colorado, U.S.A.	Foundation settlement and Piping
5.	Mud Moutain	Washington, U.S.A.	Cracking
6.	Cherry Valley	California, U.S.A.	"
7.	Neversink	New York, U.S.A.	"
8.	Waco	Texas, U.S.A.	Slides
9.	Tappan	Ohio, U.S.A.	"
10.	Clendening	Ohio, U.S.A.	"
11.	North Ridge	Canada	"
12.	Chingford	U.K.	"
13.	Lafayette	California, U.S.A.	"
14.	Muirhead	Scotland	"
15.	Marshall Creek	Kansas, U.S.A.	"
16.	Fruit Growers	Colorado, U.S.A.	
17.	Great Western	Colorado, U.S.A.	"
18.	Costilla	New Mexico, U.S.A.	"
19.	Belle Fourche	Montana, U.S.A.	"
20.	San Andreas	California, U.S.A.	Earthquake damage
21.	Upper Crystal Springs	California, U.S.A.	"
22.	Pilarcitos	California, U.S.A.	"
23.	Ono	Japan	"
24.	Lower Maurayama	Japan	"
25.	St. Mary	California, U.S.A.	"
26.	South-Haiwee	California, U.S.A.	"
27.	Hebgen	Montana, U.S.A.	"
28.	Sheffield	California, U.S.A.	Earthquake damage, flow slide due to liquefaction
29.	Dry Canyon	California, U.S.A.	Earthquake damage aggravated by leaching
30.	Buena Vista	California, U.S.A.	"
31.	Point of Rocks	Colorado, U.S.A.	Damage due to wave action.
32.	Belle Fourche	South Dakota, U.S.A.	"
33.	Minatare	Nebraska, U.S.A.	"
34.	Kingsley	Nebraska, U.S.A.	"
35.	Harold	California, U.S.A.	"
36.	Throttle	New Mexico, U.S.A.	Damaged by burrowing rodents
37.	Heart Lake	Colorado, U.S.A.	"
38.	Vermillion	California, U.S.A.	Damage due to leaching
39.	Red Mountain	California, U.S.A.	Flow Slide due to liquefaction
40.	Crane Valley	California, U.S.A.	Downstream Creep in rockfills
41.	Oued Kébir	North Africa	"
42.	Lovewell	Nebraska, U.S.A.	Drying crack formation.

In an excellent treatise on earth and rock-fill dams by Sherard and others(16) some 42 earth dam failures have been documented in sufficient detail. The type of failure suffered by these dams is given in Table V. Whereas most of these failures would be referred to briefly later in this report, full details may be had from the original text(16) and the references cited in it. Sherard et al also considers the critical knowledge of the past failures and damages an essential part of the training of an earth dam designer. Research and diffusion of information regarding dam failures is, therefore, considered an inevitable task.

It should also be emphasized that theory is no substitute for practical experience. Development of a theory replaces empirical thumb rules, which are at times deceptive and lead to erroneous conclusions. However, theory development largely depends on systematic empirical data and real practical experience. And since theories must be based on field observation, their application also needs empirical backing. Thus theories of dam design should be tested and reviewed on the basis of available failure date, which are actually optimum performance indicators.

Gruner,(17) in 1966, compiled a list of 300 dam failures, tabulating some relevant data and a bibliography for each of the dams. He identified seventeen different causes of failure; and the dams were classified under three distinct catagories - under 50 feet in height, over 50 feet in height and unknown height. An extract of the data is presented in Table VI with the percentage occurrence of each type of dam failure. Some interesting observations made from the tabulations is noted below:

TABLE VI

NUMBER/PERCENTAGE OF DAM FAILURES UNDER DIFFERENT CATEGORY

S.N.	Causes of dam failure	Under 15m	Over 15m	Unknown height	Total
(1)	Piping	11	10	9	30
		12.0%	7.0%	12.0%	10.0%
(2)	Piping along outlet	13	8	3	24
		14.0%	6.0%	4.0%	8.0%
(3)	Seepage under weir	--	--	6	6
				8.0%	2.0%
(4)	Seepage on dam	1	8	2	11
		1.0%	6.0%	2.5%	4.0%
(5)	Seepage on embankment	14	12	7	33
		15.0%	9.0%	9.0%	11.0%
(6)	Overtopping of dam	3	6	5	14
		3.0%	5.0%	6.5%	4.5%
(7)	Overtopping of embankment	37	20	27	84
		42.0%	15.0%	36.0%	28.0%
(8)	Disintegration of fill or foundation	--	2	--	2
			1.5%		1.0%
(9)	Subsidence of foundation	3	2	2	7
		3.0%	1.5%	2.5%	2.0%
(10)	Settlement of fill	1	5	3	9
		1.0%	4.0%	4.0%	3.0%
(11)	Slide Hydraulic fill	--	9	--	9.
			7.0%		3.0%
(12)	Drawdown slide	--	10	2	12
			7.0%		3.0%
(13)	Faulty design of dam	--	2	--	2
			1.5%		1.0%
(14)	Faulty embankment design	4	12	4	20
		4.0%	9.0%	5.0%	6.5%
(15)	Poor workmanship and poor material	2	10	--	12
		2.0%	7.5%		4.0%
(16)	Hostile Action	--	3	1	4
			2.5%	1.5%	1.0%
(17)	Various causes	3	13	5	21
		3.0%	10.5%	6.5%	7.0%
	Total	92	132	76	300

(i) Overtopping of embankment is by far the most frequent type of failure encountered among dams of all possible heights. Overtopping of embankments under 15 meters in height is about 42 per cent. But only 15 per cent of dams over 15 meters in height were embankments that failed by overtopping. Overtopping of other types of dams have been comparatively less.

(ii) Piping has also been a dominant cause of failure among dams and embankments.

(iii) Seepage through the foundation and embankment is another phenomena by which a large number of dams and embankments have failed.

(iv) All other mechanisms of failure appear to have a much smaller frequency.

(v) Failures have also been much greater with dams and embankments of smaller height.

(vi) Quite a large number of dams have been reported under the category of "unknown height". In fact, their performance shows greater semblance to the dams under 15 meters in height. This probably explains the poor accountability and record-keeping of such dams. Improper documentation has been an overwhelming shortcoming of the past activities in this field.

Babb and Mermel(2) recently prepared an up-to-date catalogue of dam disasters, failures and accidents that have occurred throughout the world. The total number of dams listed in the catalogue is 688, considering the Perregaux Dam to be the same as the El Habra Dam, and Saint Denis and Cheurfas as alternative

TABLE VII

SUMMARY OF STATISTICS ON ACCIDENT AND FAILURE OF DAMS

S.N.	Name of Country	Number of all types	Masonry$^{(1)}$ dams
(1)	Algeria	12	6
(2)	Argentina	1	0
(3)	Australia	9	0
(4)	Austria	2	0
(5)	Brazil	4	0
(6)	Bulgaria	2	1
(7)	Canada	18	6
(8)	Ceylon	5	0
(9)	Chile	3	0
(10)	China	4	1
(11)	Columbia	2	0
(12)	Czechoslovakia	1	0
(13)	Egypt	3	2
(14)	Finland	2	1
(15)	France	12	4
(16)	Germany	5	2
(17)	Great Britain	41	1
(18)	Ghana	1	0
(19)	India	40	14
(20)	Indonesia	1	0
(21)	Iraq	6	6
(22)	Ireland	2	0
(23)	Israel	1	0
(24)	Italy	5	4
(25)	Japan	5	3
(26)	Jordan	1	0
(27)	Kenya	1	0
(28)	Korea	2	1
(29)	Mexico	3	1
(30)	Norway	3	1
(31)	Pakistan	3	1
(32)	Panama	1	0
(33)	Peru	1	1
(34)	Scotland	2	0
(35)	Scuth Africa	2	1
(36)	Spain	17	14
(37)	Sudan	1	1
(38)	Sweden	3	3
(39)	Switzerland	10	1
(40)	Syria	3	1
(41)	Tunisia	1	0
(42)	Turkey	3	3
(43)	Uruguay	1	1
(44)	U.S.A.	440	70
(45)	U.S.S.R.	2	1
(46)	Vietnam	1	0
(47)	Yemen	1	0
	Total	688	153

Only a very small number of Timber Crib and Steel dams are listed in the compilation. Therefore the remaining can be considered as Earth and/or Rock-fill structures.

names to Sig River and Oran Cheurfas respectively. To this long list, Little(18) adds another twelve odd dam failures, which may be of some interest. These dams are: Abberton, East Lee, Edinburgh, Ghrib, Guadalupe, Laurel Hill, Santa Ana Acaxochitlan, Siburua, Sir Adam Beck, Stillhouse Hollow, Sutherland, and Zamfirovo.

Table VII shows how these 688 failures are distributed among 47 countries of the world. It appears from the table that the United States is a "major offender" with as many as 440 failures and accidents of dams at its credit. Examination of the comparative figures of other countries reveals that such a conclusion may not be true for the following reasons:

(i) Almost complete reporting of dam failures in the United States.

(ii) No comparable coverage of foreign dams is available from records.

(iii) Failure of small dams and embankments is often considered insignificant and seldom reported in many countries.

Two strategic observations of great salience that are made in the catalogue are also worth noting:

(i) Proper documentation and diffusion knowledge regarding dam safety is as yet far from satisfactory.

(ii) The collection is intended to be used for undertaking documentation of various dam failures.

Another catalogue of dam disasters, compiled by Edward Gruner(3) of Switzerland, is still in a manuscript form. Though considerable data has been collected in this compendium, far exceeding those collected by Babb and Mermel, the compilation is yet far from complete. Research and analyses based on this data could be of much benefit in understanding problems of dam safety. Thus, statistical survey of dam failures is an important avenue of research, and especially so when the results can be useful in developing profitable safety models for dams and reservoirs.

CHAPTER III

SOME CASE STUDIES

Dam failures can be best understood by a comprehensive study of available data on past failures. Such a study was recently suggested by Biswas(1)*. Schnitter(2) briefly reviews the history of dam engineering pointing out the great influence of past failures in a secular growth of dam building science. It was Aristotle who said: He who sees things grow from the beginning will have the best view of them. The beginnings of hydrologic concepts as a result of construction and failure of the dams and impoundments of prehistory is clearly demonstrated by Biswas(3,4). Gruner,(5) however, provides ample data on the basis of which some useful case studies could be documented.

Several statistical compilations of dam-failure have been discussed earlier in this report. One of the recent published compendium is due to Babb and Mermel(6). Babb and Mermel point out therein that "the collection will be of value to anyone who may undertake a comprehensive search and compilation of all existing reports on failures of dams." An attempt has therefore been made with this idea in mind in the present report which overviews as many as possible, if not all, of the existing published and unpublished reports, papers, documents, and personal communications.

In this chapter we propose to study in some detail a few of well-known dam failures and disasters which have had a significant impact on the science of dam building. These typical cases of dam failures have been documented in a chronological sequence emphasizing the various aspects of their problems, diagnosis, the limitations of diagnosis and possible

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* See references at the end of the report.

solutions. Thus, the study has a unique characteristic of lumping together the various complex safety problems, limitations of diagnosis, possible solutions, and also the present safety research needs for the dams and reservoirs.

In the sections to follow failures of some major dams and reservoirs will be discussed in detail. Some important analytical resources for investigating dam failures are borrowed from Biswas(1,3,4) and Schnitter,(2) Hinderlider,(7) while much of the raw data is drawn from Gruner,(5) and Babb and Mermel.(6)

a. *Sheffield Dam*

The Sheffield dam, near Santa Barbara, California, was an earth-roll dam. Figure I shows a representative section of the dam at its maximum height. Its upstream slope was protected by a facing of concrete slabs, on a 4-ft. thick clay blanket that was carried 10 ft. into the foundation to form a cut-off. The dam failed in June, 1925, due to a moderate earthquake having an intensity of IX RF. At the time of the failure, the dam was subjected only to a head of 20 ft. of water. The earthquake was responsible for moving a 300-ft. long embankment from the central portion of the dam, thus, releasing 45 million gallons of water.

Different opinions have been expressed regarding the mechanism of failure. Andrews (8) attributed the failure to the opening of joints between concrete slabs of the upstream lining and the resultant cracking of the clay blanket. This created uplift in the central fractured section which culminated in the sliding of the central part of the embankment. According to Nunn, (9) the earthquake caused vertical fissures which caused a section of the dam to be washed away. The Corps of Engineers, however, suggested that the failure occurred along a shear surface because of the instability caused by the horizontal earthquake acceleration. (10)

A recent comprehensive study of the Sheffield dam failure, by Seed et al, (11) based on the data available from the Corps of Engineers, concluded that:

a. The sliding occurred due to the liquefaction failure of the loose saturated silty sand near the base of the embankment. Similar opinion has also been

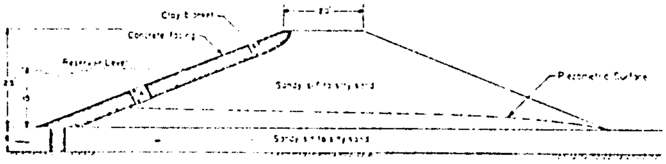


FIG. I EMBANKMENT TRANSECTION AT THE HIGHEST SECTION OF THE SHEFFIELD DAM.
SOURCE: SEED ET AL (REF. 11)

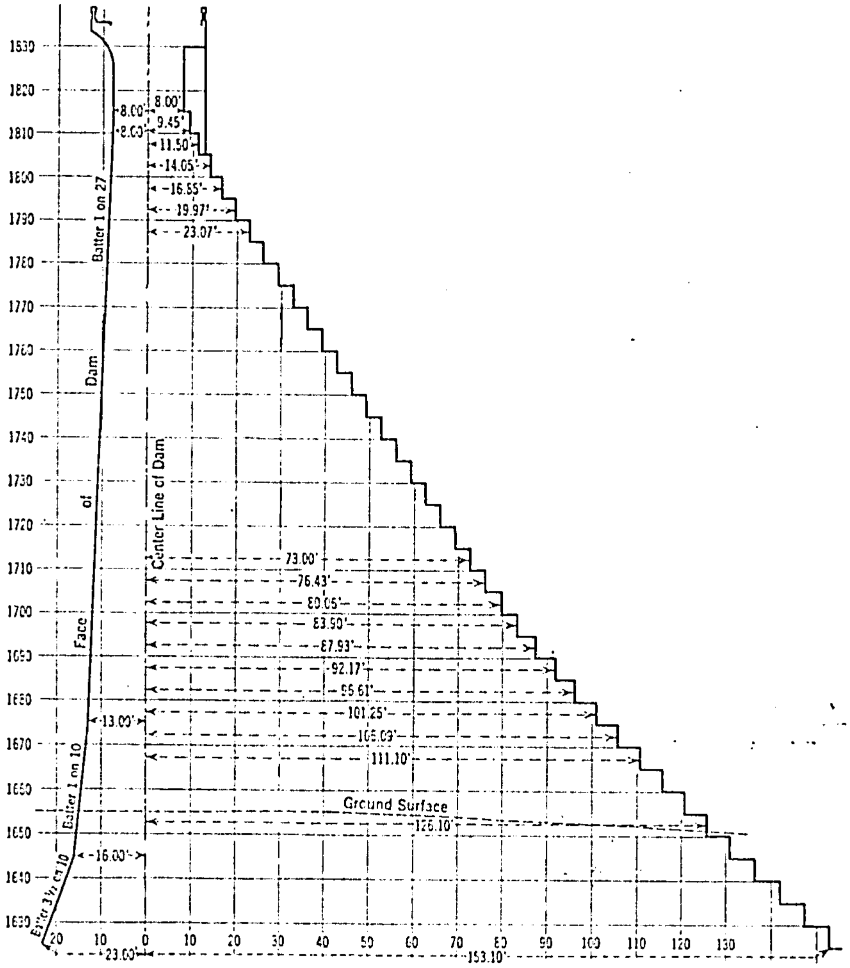


FIG. II (a) DEEPEST SECTION OF THE SAINT FRANCIS DAM. SOURCE: REPORT ASCE COMMITTEE (REF. 64) expressed by Sherard et al.(12)

b. The dam would not have failed if the maximum ground acceleration due to the earthquake was 0.1g. The most probable value of ground acceleration at the time of failure was about 0.15g.

c. A reasonable compaction of the foundation or the embankment soil to about 90% of the standard AASHO compaction test, would have prevented the occurrence of the failure.

d. Ground accelerations due to earthquakes of the order of 0.5g can be expected depending on the active characteristics of an area. Analysis of the performance of embankments during past earthquakes and evaluation of the ability of the analytical procedures to predict this performance would provide the necessary tools to examine the seismic behaviour of dams and will be useful in designing dams that are likely to be subjected to major earthquakes.

It is evident that significant knowledge and understanding of the behaviour of dams can be obtained from the study of dam failures.

Ambraseys(8) reported the spectacular performance of the San Andreas dam, of California located on a fault. The dam although sheared during a severe earthquake did not give way. This was interpretted by some authorities as clear indication of the adequacy of conservative design provisions to withstand severe earthquake stresses on dams. Quite on the contrary, the Sheffield dam-failure is a direct contradiction of the above hypothesis. It is generally believed that Sheffield failure was one of those rare seismic behaviour of dams in which the structure was damaged and destroyed by a moderate earthquake. Rightly therefore Seed and others (11) point out that many lessons can be learnt by engineers from this failure. Research on earthquake resistant design of dams has therefore made a substantial progress in recent times.

b. *Saint Francis Dam*

Saint Francis Dam, built in 1926, across San Francisquito Creek in California, about 40 miles north of Los Angeles, was a concrete arched gravity dam. The dam failed on the midnight of March 12, 1928 in reservoir-full condition, and the resulting flood washed away 426 people and 700 homes.(6) The disaster has been well documented in literature(57,58,59,60,61,62,63) and extensively investigated by six officially appointed commissions(64) and numerous specialists(65,66,67,68,69,70,71,72,72,74). The tragedy of the Saint Francis dam failure initiated a serious re-examination of the stability of existing dams and re-emphasized the necessity of introducing "greater conservation and sounder judgement" in the design and construction of such structures.(75) The increasing public pressure coupled with the failure of the dam was responsible for state legislation on public supervision of dams and reservoirs in California in 1929.(76)

Some important details of the dam, before and after the failure, are presented in table I. Also, figure II shows the plan, elevation and the deepest section of the dam.

The dam was designed purely as a gravity section despite its arch shape in plan. No provision for free-board was made. Also the structure was not checked for overturning against forces other than horizontal water pressure. The rocks beneath the left abutment and part of the right foundation consisted of layers of laminated mica schist, with dip parallel to the slope of the left abutment and strike normal to the long chord of the arch. Half way up the right abutment a fault plane formed the contact between the schist and a dark reddish conglomerate, that lay beneath the flat foundation slope. The abutment on the left side was notched into the steep canyon rock, whereas the excavation in the remaining portion

...2

SAINT FRANCIS DAM

TABLE I

Owner of the dam : City of Los Angeles, California

Construction period: April, 1924 to May, 1926

Name of River : San Francisquito Creek

Failure : Midnight of March 12, 1928

Dam type : Concrete Arched-Gravity (radius 500 ft. at the upstream crest face)

Length of dam : 700 ft

Length of wing wall: 500 ft (westerly end)

Dam height : 205 ft

Crest thickness : 16 ft

Base width : 176 ft

Cost of the dam : $1.5 million

Losses : 426 lives;
700 houses.

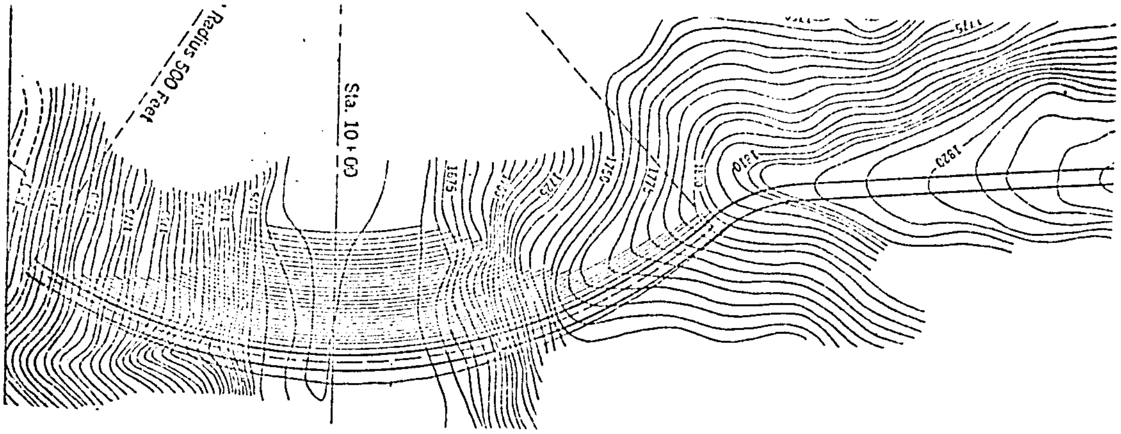
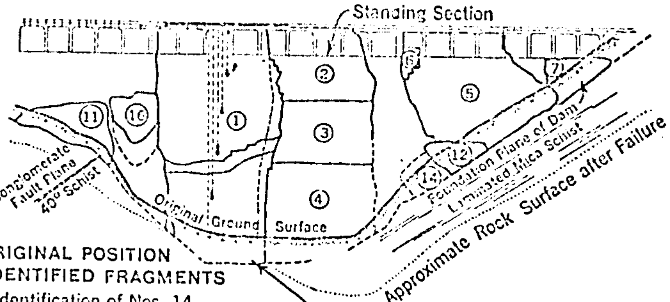
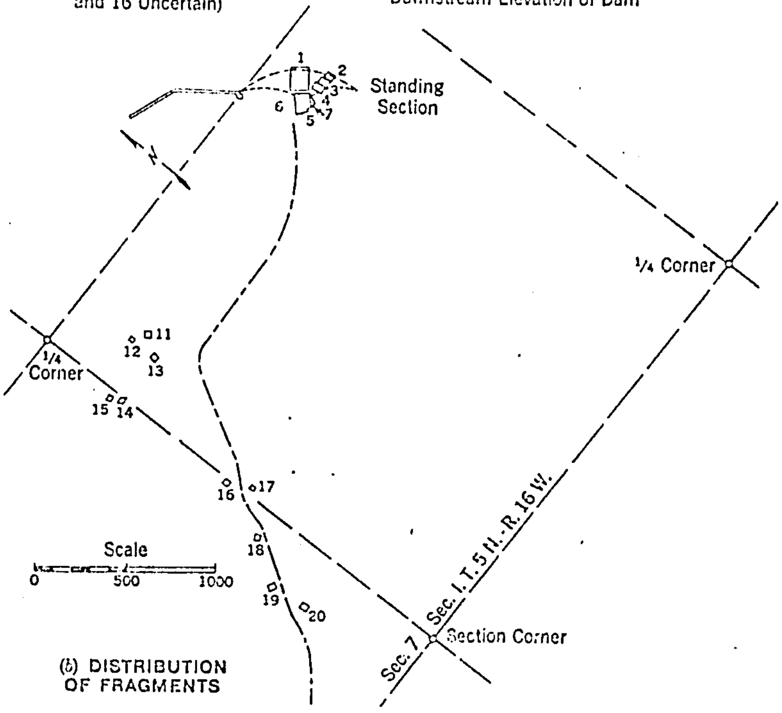


FIG. II (b) PLAN OF THE SAINT FRANCIS DAM. SOURCE: REPORT OF ASCE COMMITTEE (REF. 64)



(a) ORIGINAL POSITION OF IDENTIFIED FRAGMENTS (Identification of Nos. 14 and 16 Uncertain)



(b) DISTRIBUTION OF FRAGMENTS

FIG. II (c) DOWNSTREAM ELEVATION AND DISTRIBUTION OF FRAGMENTS BENEATH THE SAINT FRANCIS DAM. SOURCE: REF. 64.

was made by sluicing or by steam shovel. The spillway openings, located near the center of the dam in two groups, was controlled by sliding gates.

The investigation of the failure disclosed that a deep slide had occurred at the left abutment eroding the schist to a depth of 40 feet, whereas the conglomerate in the right abutment eroded only about 30 feet. The slip surface at the left side was active up to two weeks after the catastrophe.

Measurements were carried out before and after the disaster. Considerable horizontal and vertical deformations and/or displacements were noted in the remnant structure (the wing-walls, the standing central section, and the neighbouring rocks of the canyon) soon after the collapse. Water was found seeping from concrete joints and cracks for some days after the failure. Also, the two abutments were breached and washed away by the flood wave while the central deepest section remained standing.

A water surface recording gage, recovered from the structure, provided a useful data for analysis of events that occurred prior to the failure. For a full one year before the catastrophe, the head of water in the reservoir increased from 149 feet to 205 feet, which is the full capacity of the reservoir. This first prolonged application of high water pressure on the dam foundation could be considered to have some connection with the ultimate failure. Also, the observation of numerous large cracks, that formed on the dam and the wing walls a few months before the actual breaching of the dam, could have served as a warning against further filling of the reservoir. This warning, however, was not heeded.

During the investigations, it was also pointed out that the structure suffered from several deficiencies. No foundation cut-off was provided below the upstream face. Nevertheless a small key wall had been inserted along the right abutment. There was neither any grouting done in the foundation nor had there been any provision for foundation drainage for the relief of uplift pressure. In fact, leakage of water at the dam base was noted during the first filling of the reservoir. Unfortunately this warning was also ignored. Later, the leakage substantially increased and a few weeks before the failure drains had to be installed beneath the right abutment. The above discussion shows that many observations, made prior to the final breaching of the Saint Francis Dam, indicated the possibility of a disastrous failure; yet the symptoms observed were not taken very seriously. Obviously, inadequate understanding of the implications of the observed data could be blamed for the catastrophic failure. Since then considerable research effort has been directed to the development of a methodology for systematic analysis and observation of dams and reservoirs. Lately, Serafim and Guerreiro (45) have thus shown that a successful observation of dams is possible with a team of skilled experts.

The six enquiry commissions, which separately looked into the causes of failure of the Saint Francis Dam, came to the following general conclusions: (9)

(1) The weaknesses inherent in the foundation was the cause of the dam failure; (47,58,59)

(2) concrete in the dam was quite satisfactory;

(3) Pasadena seismograph records revealed that an earthquake was not responsible for the failure;

(4) gravity dams built on proper foundation cannot be considered unsafe;

(5) state supervision over design, construction and operation of dams was considered a necessity for future safety.

Differences in opinion, however, exist regarding the starting point of the dam break. Some believe that the rupture began at the right abutment; others blamed the left abutment while the rest avoided a definitive conclusion. As both the abutments ultimately failed, Godfrey (65) pointed out that the entire dam was in reality three dams joined in one. While two of these were essentially identical, the third (middle) one was in fact different. The base width of the "middle" dam was 80 percent of height, whereas the end dams had a base size 65 percent of height. Calculations of Godfrey showed that the "wing" dams had no factor of safety, and hence the underpressure was able to lift the dam base, allowing water to percolate beneath the dam. Thus the two end dams were floated and washed away. It was, therefore, obvious that the real mistake lay in the neglect of full uplift in designing the dam. The "middle" dam, however, could not be carried away on account of the larger base width. Jorgensen (67) therefore, recommends a base width of more than 75 percent of height for safe design of dams.

It was also stated that the failure of the dam was due to "incompetent" geological formations and it was not feasible to erect a safe concrete gravity dam at this location. (64) The percolation of water under pressure moistened the schist layers lying parallel to the steep slope of the canyon. The wetting was thus responsible for the increased slide tendency of the abutment slope, and this should have contributed to the ultimate collapse of the two abutments by slide. Observations on the actual failure slopes and their active nature, as seen after the disaster, also testify to the above failure hypothesis. The Saint Francis dam failure has, therefore, been generally regarded as a man-made disaster. (63)

Unfortunately, occurrence of hard rock formations is somewhat rare in most chosen dam-sites. The final slide of the abutment slopes indicate that water penetration had undermined the rock beneath the dam. According to Turner (66) injection of neat cement grout into the underlying foundation material at a pressure almost double of the full reservoir head could have saved the dam. Unfortunately grouting was never resorted to. The presence of pores and crevices in the foundation material is potentially dangerous on account of the possibility of water percolation under a high head, into these empty cells, that develop an appreciable pore pressure. This would seriously weaken and undermine the strength and stability of the underlying material. It was, therefore, necessary to prevent the occurrence of such a situation by grouting.

Jakobsen (68) also shows that provision of foundation drains is necessary for safety if percolation of water in the foundation is apprehended. Excessive leakage through the drains provide sufficient advance warning to prevent a sudden failure with resultant loss of life. He also added that if the dam had been designed for full uplift, it would have cost no more than 25 percent above its actual cost. Obviously, the saving was not sufficient to justify the risk.

Gerry (69) draws attention to an interesting but most ignored aspect of the weakness of the dam. He shows that the curvature was certainly a defect. A straight dam, utilizing the same amount of material, could have been much stronger. Thus, under similar conditions it is absolutely necessary to assess the safety of arch action in combined dams as this one.

Whereas Marsh (70) shows, on the basis of available data, that the left abutment of the dam failed first, Finkle (71) finds evidence of greater settlement under the right abutment causing leaking horizontal cracks that weakened the structure to the point of ultimate collapse. It is also suggested that a careful measurement of displacement and rotation at various points in dams be made in order to be able to detect the dangerous movements, which culminate in the failure of dams. Appreciable movement of the dam was noticed prior to its failure and, according to Grunsky,(73) this movement accelerated just before the final failure. Floris(72) shows that the sliding friction factor of the Saint Francis Dam is far in excess of what is permitted by conservative practice. A chemical analysis of water percolating from the rocks of the dam abutment have been reported by Morris(74) which shows a comparatively high content of dissolved solids. This also, probably, explains the partial weakening of the rock structures beneath the foundation.

In short, it may be stated, that Saint Francis Dam failure was a consequence of the combined effect of several important errors in design and construction. Nevertheless, most of these errors are yet to be fully understood in the light of many complex forces brought into play in a dam-reservoir system. Unfortunately, many similar factors have been responsible for several other dam-disasters that followed the Saint Francis dam-failure . Extensive research to establish some of the basic relationships in dam-reservoir systems may have to be carried out in order to enable a safe design of these systems . Obviously, conflicting hypotheses continue to be offered for most major dam-failures hitherto; very little progress has thus been made in obtaining a logical comprehensive framework for testing these hypotheses.

c. *Veg de Tera Dam*

The 112-ft. high Veg de Tera Dam, located on the River Tera, a tributary of the Douro River in the Province of Zamora in Spain, was a buttress dam. It had 28 masonry buttresses. On January 10, 1959, when the reservoir was full, about 17 of the buttresses collapsed, and approximately 330-ft. long section was washed away by the resulting flood. The flood played havoc with the village of Rivadelgo, and nearly 140 persons lost their lives. Adverse public reaction to the consequences of the disaster was primarily responsible for the initiation of legal proceedings against ten Spanish engineers who were associated with the dam. The court found the director and three staff engineers of Monocabril Hydroelectric Company that owned the structure "guilty of neglect and insufficient construction supervision". The convinction was unexpected (14).

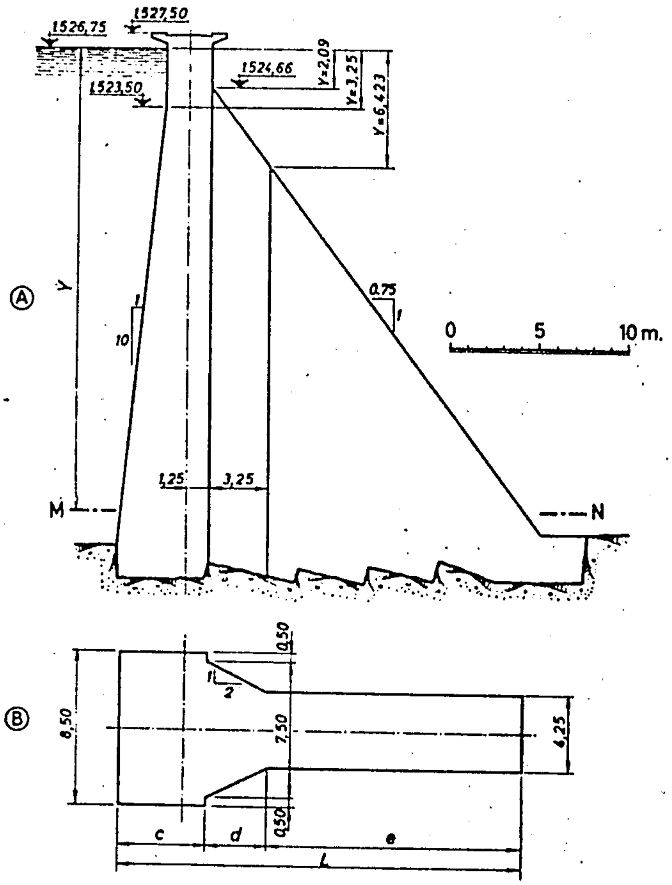
The construction of the dam began in 1954, and was completed in 1957. According to a U.S. State Department Report: "construction was suspended during winter and poor bond resulted between old and new masonry .... (and) high rate of leakage through masonry caused shifting of stresses in dam." (6) Apparently, the defective joints between concrete and masonry failed due to full reservoir stresses which it could not resist. While this defect might have contributed to the ultimate failure of the dam, it is not quite clear whether the collapse was solely due to this factor.

The report of the inquiry on the Veg de Tera disaster was published after a long delay, on April 4, 1963, in La Actualidad Espanola, a Spanish illustrated weekly from Madrid. It, however, concluded that the assumption of equality of the modulus of elasticity of the masonry and its constituent rock aggregate for design calculations was in error (15). The laboratory facilities available probably did not provide a suitable method for assessing the modulus of elasticity of masonry.

Extensive theoretical and experimental studies were conducted in Spain on buttress dams after the failure of the Veg de Tera dam. The studies (16) indicated that:

1) Calculation of stresses in the buttress by traditional methods, ignoring the influence of foundation, is likely to be in error within a certain distance from the foundation joint.
2) Analytical methods which takes account of the foundation influence is unreliable on account of numerous simplifying assumptions.
3) It is not enough to check the highest buttress for the greatest tensile stress. It may occur in smaller buttresses under some specific conditions.
4) The behaviour of the dam is also dependent on the effect of load transmission between continguous buttresses.
5) A reduced model study may be the only appropriate method for reproducing the actual behaviour of the structure.

Obviously, the distribution of stresses within the buttresses were affected by the different buttress heights as a result of varying foundation level. Hence, the designers of the Veg de Te ra dam were completely unaware of the actual stresses. Evidently, defective



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	Y<2,09	2,09<Y<3,25	3,25<Y<6,423	Y>6,423
c	2,50		0,10Y+2,175	
d	0,00	0,75Y-1,5675		3,25
e	0,00			0,75Y-4,8175
L	2,50	0,75Y+0,9325	0,85Y+0,6075	

FIG. III (A) LATERAL ELEVATION (B) PLAN AND (C) DIMENSIONS OF THE VEG DE TE RA DAM MODEL TESTED AFTER THE FAILURE.
Source: Guitart (Ref. 16).

construction joints and incorrect assumptions regarding the strength of the masonry should not be fully blamed for the failure. It, however, remains an open question as to which of these factors really initiated the failure mechanism. The failure indicates the lack of any knowledge about the exact dam behaviour. Nevertheless, it should be realized that the Zamora court did not consider the lack of knowledge at the present state-of-the-art in deciding legal liability for the failure.

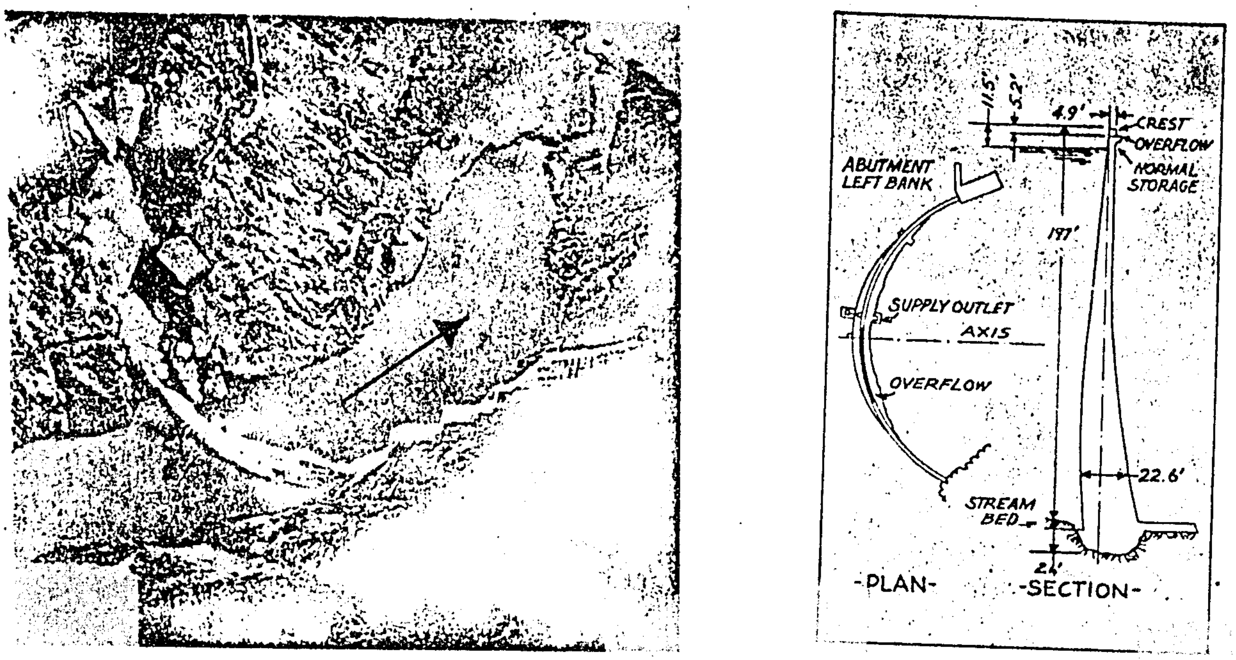
In fact, very little information and data have been made available on the Veg de Tera dam-failure. The investigation report was also diffused in a manner as to be of little benefit to the profession. This probably demonstrates the conventional reluctance of authorities to publicise the failure of dams administered by them. Figure III shows in detail the main characteristics of the type of buttresses in the Veg de Tera dam. The sketch has been reproduced from a study reported by Guitart(16) which attempts a thorough investigation of the distribution of stresses within the buttress dam. Eventually, the studies that followed the failure of this dam led to the evolution of a rational buttress dam technology.

d. *Malpasset Dam*

The Malpasset dam, built in 1954 on the Reyran River on the French Riviera was one of the thinnest arch dams (46) ever built for its height of 200 ft. The dam was one of the most advanced examples of the dam-builder's art and a product of the mind and heart of Andre Coyne, a prime mover in the evolution of arch dams. (47) Some salient features of the dam are shown in Figure VI and Table II presents relevant data on the structure. A foundation defect described as a clay seam $1\frac{1}{2}$ inch thick which became visible on the left bank after the failure of the dam, could not be detected in investigations prior to the construction of the dam. (48) Opinions differ regarding the possibility of its detection with the aid of existing techniques.

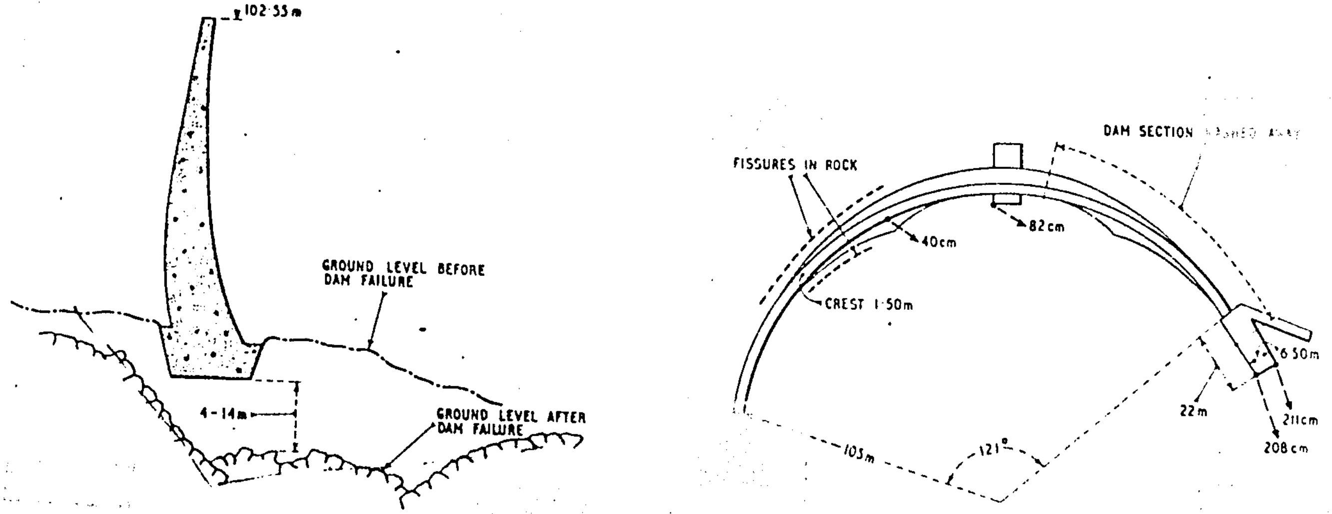
TABLE II

Name of dam :	Malpasset dam
Type :	dome or cupola type concrete arch dam
Height :	218 feet
Length of crest :	700 feet
Radius at top :	344 feet (u/s face)
Angle subtended :	135 degrees
Crest width :	5 feet
Base width :	23 feet
Volume of concrete :	62,500 cu. yds.
Theoretical design stress:	680 psi



(a) Malpasset canyon and the remnant dam after the failure. Source: Ref. 47.

(b) Plan and Section of the dam. Source: Ref. 47.



(c) Ground level at the dam site before and after failure. Source: Jaeger (Ref. 17)

(d) Plan representing displacements of the dam section. Source: Jaeger (Ref. 17).

FIG. VI MALPASSET DAM

On December 2, 1959 at 9:10 p.m. the dam failed, releasing some 40,000 acre-ft. of water. When the actual collapse occurred, the dam was subjected to a record head of water (about a foot below the highest water level) due to five days of unprecedented rainfall. According to the Inquiry Commission instituted by the French Ministry of Agriculture, the causes for the failure (17) were:

a. the arch ruptured as the left abutment gave way;

b. the left abutment moved about 208 cms in the tangential direction with a small radial component, by sliding, along well-defined lines of fissuration;

c. the mechanical strength of certain rocks undernearth the left abutment was less than expected;

d. the rock foundation was more deformable near the higher upstream rock fissure;

The Commission, after careful considerations, ruled out earth tremors, microseismic earth vibrations, sabotage, etc., as possible reasons for failure.

Various authorities like Terzaghi, Cambefort, Serafim and Bellier, have put forward different hypotheses regarding the actual mechanism of failure. According to Terzaghi, (17) the failure occurred by sliding along a continuous seam of weak material covering a large area. He felt that a conventional site exploration would have indicated that the site of the dam was a potentially dangerous one, but it would have been impossible to predict accurately the surface of least resistance in the rock along which the actual failure occurred.

Cambefort,(17) however, suggested that the buckling of the slender arch dam under axial loading was the primary cause of failure, and it resulted in the destruction of the abutment rocks and the eventual rotation of the dam. It has not been possible to justify the hypothesis by model studies made so far. The failure according to Serafim,(17) was caused by the high tensile stresses developed in the concrete, in a direction parallel to the foundation, near localized weak rocks, resulting in multiple fissuration of the dam. Bellier(18) pointed out that the dam rested on a rock dihedron, formed by the downstream fault plan dipping 45° and the upstream potential shear surfaces which were undetected by drilling. The weight of the dam and the arch thrust tended to compress and stabilize the dihedron. The high compression in the gneiss created an impervious cut-off in the foundation, against which seepage pressure built up gradually. Finally, the dihedron was subjected to a hydrostatic head of the reservoir level, which exceeded the weight of the foundation, and, thus caused the bank to fail.

Bellier's hypothesis, when viewed along with that of Terzaghi, seems to indicate the most plausible explanation of the failure mechanism. Unfortunately, both the left rock abutment and the foundation were washed away, and this made it extremely difficult to arrive at definite conclusions.

In a recent article, Jaegar(19) states,

1) The French experts and the U.S. researchers have come to the conclusion that the classifical tests were inadequate to detect the peculiarities of the fissured gneiss rock at Malpasset. A new set of tests devised at Paris also prove this point.

2) A grout curtain is known to seriously affect the overall stability of the dam and its foundation. The microfissured gneiss rock whose permeability varied with the state of stress probably had a similar effect on the Malpasset dam as revealed by a detailed stability analysis of the left abutment.

3) A large rock fracture along the dam heel, observed at the right abutment after the failure, and the movement and rotation of the base of the dam about the comparitively fixed crest, indicate that the progressive loosening of the arch from its rock foundation might have favoured the rupture. It is, however, difficult to conclude that better anchorage at the foundation could have saved the dam.

The Malpasset disaster clearly indicates that there is an urgent need for more intensive research in the field of safety of dams. The legal charges of homicides and injuries, caused due to the negligence, brought against the Chief Engineer for Rural Engineering in the French Court of law, indicate an increasing public concern about safety of dams. With the population growth and consequent encroachment of the flood plains dam failures are likely to cause increasing loss of lives and property damages. Thus, in all probability, the public concern on dam-failures is likely to increase.

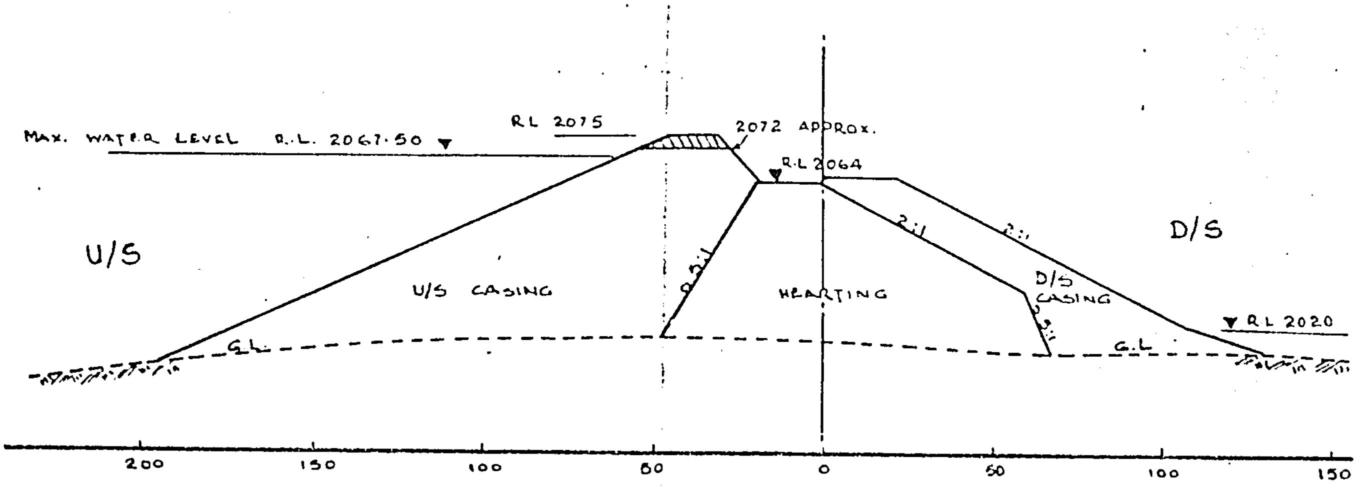
A close study of the Malpasset trial in the French Court also reveal an almost equalitarian dominance of two opposing schools of thought(49). One of them believed that the disaster was a consequence of grave negligence of engineers and bureaucrats whereas the other gave evidence to the effect that the failure and the defect were unforeseeable. Both, de facto, had some good reasons and justifications, and were also supported by some leading specialists and researchers. The arguments, therefore, clearly show a lack of sufficient understanding and available methodology for resolving the complex problems brought to the fore-front by the Malpasset failure. It should, nevertheless, be remembered that the investigation of dam site was not made in summary fashion, the dangerous displacement at the foot of the dam, though measured, were ignored, and no attention was paid to the capacity of the weak foundation to support the applied pressure. Hopefully, the lessons of Malpasset disaster will be able to teach engineers to avoid such an event in future.

e. *Panshet Dam*

The Panshet dam on the River Ambi at Panshet, India, is a 168 ft. high earth dam having a side-channel spillway with a design capacity of 17,200 cusecs. Due to a heavy rainfall of 70 in. in 23 days, the newly created reservoir was subjected to a high runoff, and it resulted in the failure of the Panshet dam, and the Khadakwasla dam downstream, causing unprecedented havoc and misery to the city of Poona. Nearly 95,000 people with directly affected by the flood, and about 5000 houses were either damaged or destroyed.(22)

The construction of the dam was started in 1957 and was scheduled for completion in 1962. Subsequently, the completion date was put forward to early 1961. Thus, when the filling of the reservoir was started, the construction of the dam was not complete. The unfortunate situation was a direct result of improper planning and management.

As the first outlet gates were delivered late, and the headstock gears did not arrive, the gates were installed in their guides and hung on chains with an opening of 2 ft. The gate tower access bridge was also not delivered in time. Extensive model studies made by Rao(22) indicated that dynamic flow of water through the partially opened gates caused severe vibrations due to air entrainment and cavitation in the outlet culvert. The rough, unfinished, and uneven culvert invert was subjected to high pulsating flows causing periodic breaking of the water column which resulted in water-hammer effects. These adverse vibratory forces led to progressive disintegration of the arch voussoirs of the



Note: (i) The measurements are recorded in feet.

(ii) Subsidence first occurred under the latched portion of the upstream casing.

(iii) Details of portion below ground level is not shown. An outlet culvert with upstream gate passes beneath this section.

FIG. IV TRANSECTION OF THE PANSHET DAM OVER THE OUTLET CULVERT.

Source: Rao (Ref. 22).

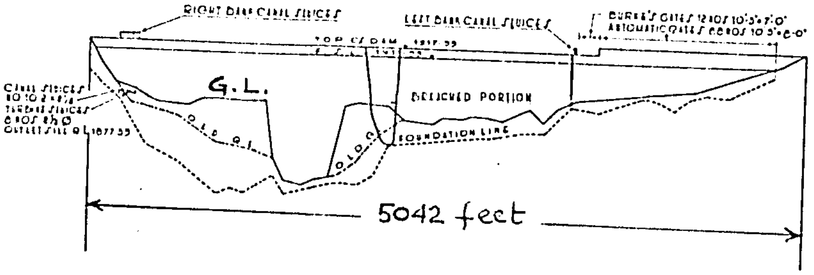
outlet culvert, causing subsidence (see figure IV). The earth embankment sank 4½ feet in only 2½ hours.(21)

It is highly surprising that no model test was performed to evaluate the performance of the designed gates under high heads. Obviously, the designers did not take seriously a similar accident in the Bhakra Dam in India,(21) in 1959, when the gate chamber floor in the right diversion tunnel failed. Heavy vibrations were caused by the use of the tunnel gates for regulating flows under high heads.

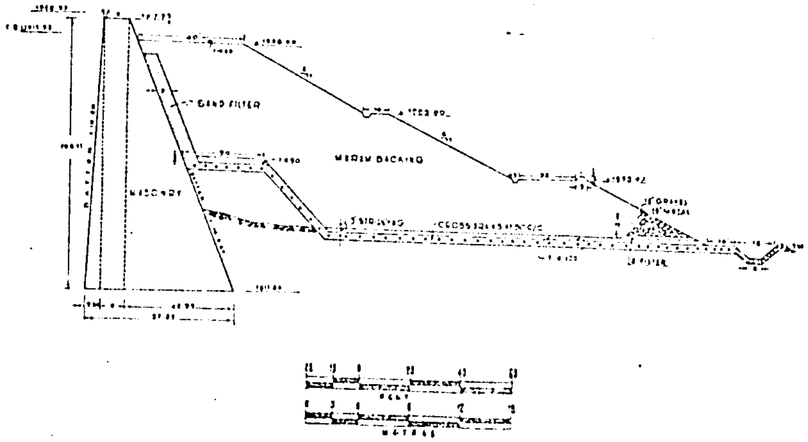
f. *Khadakwasla Dam*

The Khadakwasla, built in 1864 on the Mutha River near Poona, India is a rubble masonry dam having a maximum height of 100 ft. It was not designed for uplift forces or foundation drainage. As a result of some 70 in. of rainfall in 23 days and the consequent failure of the Panshet dam upstream, the Khadakwasla was overtopped by 9 ft. of water which created severe vibration problems. The 95-yr. old dam (see figure V) withstood such severe conditions for about 4 hours, and then failed in two stages. The waste weir section sheared off within three hours and then a triangular section of the dam broke open "like a door" near a step in the foundation. The failure occurred with the receding flood was 6 ft. above the top of the dam.(20)

A two-dimensional stability analysis(21) was carried out to determine the causes of failure after the mishap. The results are quite revealing. The maximum tension at the upstream heel of the deepest section of the dam, with full reservoir level and uplift, was found to be 56.0 p.s.i., and such conditions existed for about three months every year. With 9 ft. of water over the top of the dam, the calculated tension was found as 105 p.s.i. But, still the failure did not occur at the deepest section, even though the dam was subjected to a stress of 75% of the ultimate tensile strength of the masonry for a few hours. Under the same conditions of overtopping, the horizontal forces were found to be 1.1 times the total vertical forces, and the resultant force fell outside the base by 5 ft. (beyond the toe) for 3 to 4 hours. Still, it is surprising to note that the dam neither failed by sliding nor by overturning. The downstream backfill was scoured away by the overflowing water.



(a) Longitudinal Elevation looking upstream of the dam.



(b) Lateral section and downstream earth backing at the highest section.

FIG. V KHADAKWASLA DAM BREACHING. SOURCE: MURTI (REF. 20)

A three-dimensional photo-elastic model was made of the portion of the dam where failure occurred due to a step in the foundation (21). The cantelever elements were subjected to twisting action due to variable heights, and the actual tensile stresses were found to be 140 p.s.i. Obviously, the results indicated that the conventional analysis does not apply to such situations. Hence, there is a necessity to develop a suitable methodology for analyses of such stresses.

There are several lessons to be learned from the failure of the Khadakwasla dam.

a. There is a pressing need for conducting more intensive research on the existing design techniques and analyses of stability and strength of dams. It is also necessary to develop proper methodology for the evaluation of aging effects on the strength and stability of dams.

b. Designs which cannot be treated by conventional analyses should be tested by model studies.

c. The old dams should be strengthened, if need be, for additional safety.

d. Suitable means of outflow like fuse plugs should be provided in the downstream dams to protect them from the failure of dams upstream.

e. There is still much to learn about the mechanics of sliding and overturning of dams.

f. It seems that it is possible to design masonry dams from failures due to overtopping. Obviously, this would provide greater safety of the dams from unpredictable hydro-meteorologic conditions.

g. *Vaiont Dam*

The Vaiont Dam, built in 1961 across a narrow gorge of River Vaiont, a tributary of Piave River in Italy, was a concrete thin-arch dam. The 875-feet tall world's second highest dam supported on its back a huge 4-mile long reservoir that stored water for generation of hydroelectric power. It was constructed at the teeth of a serious local opposition, allowing bureaucratic forces to triumph over people's desires and wishes.

Some relevant details of the dam are summarised in Table III. Too, the Figure VII shows the location of the dam, the extent of the reservoir and the surrounding features. Three important reservations about the dam may be worth-noting:(23) (1) anchor tie-rods were used to strengthen the abutments; (2) a grout-curtain, 500-feet outward at the base, minimized the instability of the abutment wall surfaces; (3) the site of the dam was potentially dangerous from the point of view of landslides. Of primary importance was the commonly observed instability of the Vaiont Valley. A sequence of slide events, that occurred in the area, necessitated certain precautionary measures compatible with the expected slide problems.

After the occurrance of a 1.5-million cubic yard rock-slide on the left bank of the Vaiont Valley in 1960, a by-pass tunnel, 17-feet in diameter, was built through the right abutment in order to allow passage of water downstream when the reservoir gets blocked by a future slide. The reservoir elevation was restricted to a maximum level and geodetic stations on concrete pillars were installed in the potential slide area to control and monitor the dangerous creeping movement. Yet the

TABLE III

Name of dam	:	Vaiont Dam
Type	:	double-curved thin-arch concrete dam
Height	:	875 feet
Width at top	:	11 feet
Width at bottom	:	75 feet (near bottom plug)
Spillway	:	overflow type, with a two-lane highway deck spanning the crest
Powerhouse	:	underground in left abutment
Reservoir capacity	:	316,000 acre-feet
Date of construction	:	1961 (completed)
Extent of Slide	:	350 million cu. yds. (slide volume), reservoir filled up for 1.25 miles to a depth of 575 feet
Loss of lives	:	3000 (dead)

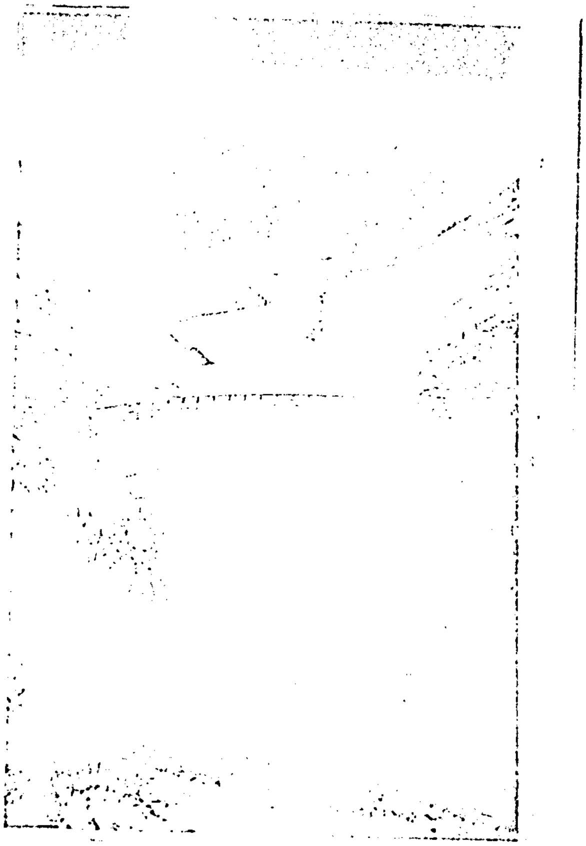
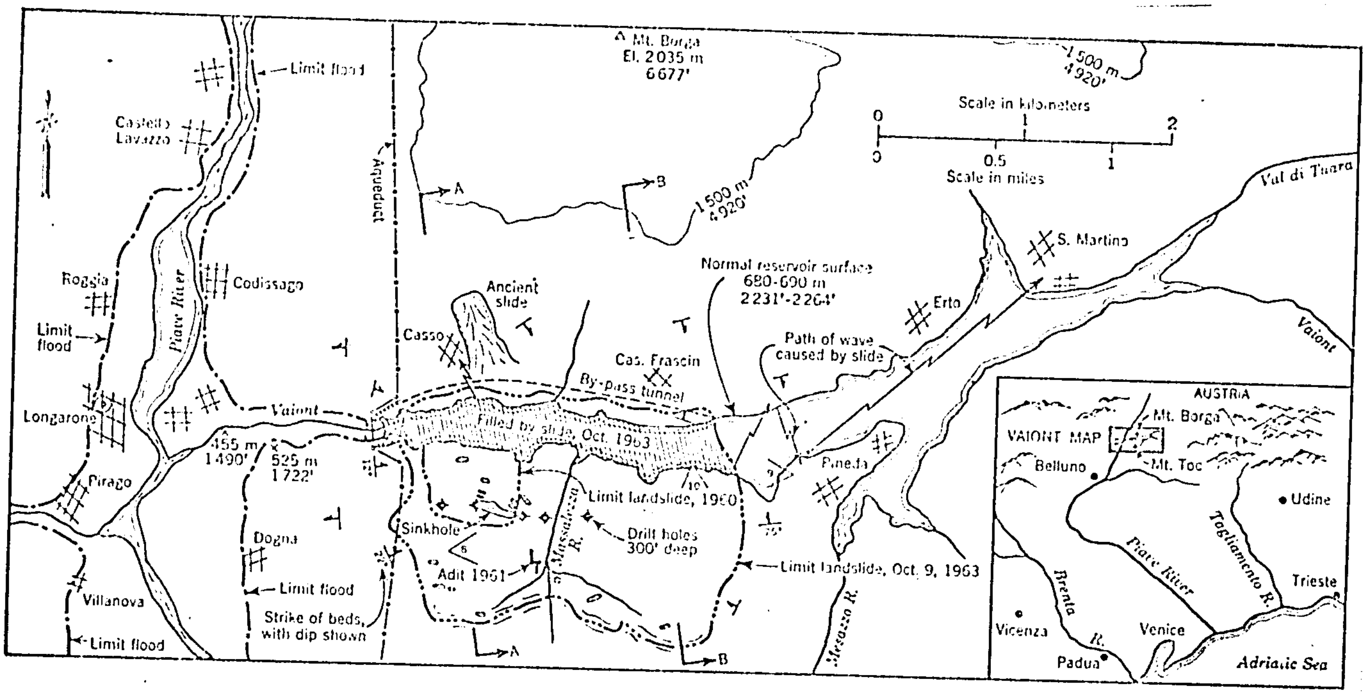


FIG. VII THE COMPLETED VAIONT DAM SHOWN ON THE RIGHT AND THE LIMITS OF SLIDE AND DESTRUCTIVE FLOOD WAVES SHOWN IN THE BOTTOM MAP.
SOURCE: Kiersch (Ref. 23).



prediction of slide was completely in error; and lack any effective control whatever created a situation of unexpected collapse.

On the night of October 9, 1963, the Vaiont dam, was overtopped by a 330-ft. flood wave which inflicted a loss of 3000 lives, the heaviest loss due to a dam failure ever known to history. The flood was caused by an unprecedented landslide, having an appropriate volume of 350 million cu. yds., which filled up 1.25 miles of the reservoir with slide materials up to a height of 575 ft. above the reservoir level. The dam can no longer be used for generation of hydro power as the cost of clearing the reservoir seems to be rather prohibitive. A by-pass tunnel under the right bank of the reservoir allows the water behind the "slide-dam" to be drained out into the river.

A technical board appointed by the Italian Government found "bureaucratic inefficiency, muddling, withholding of alarming information, lack of judgement and evaluation and lack of serious individual and collective consultation" as the real causes of the disaster. The report was accepted by the Italian Government, and resulted in the suspension and prosecution of those found responsible for the disaster. Fourteen engineers were prosecuted for manslaughter.

The causes that led to a gigantic Mont Toc slide into the reservoir have also been investigated by Kiersch (23) and Muller (24). Muller's conclusion "that the sliding could not possibly be foreseen by anybody in the form in which it actually took place and, in fact, nobody had foreseen nor predicted it," is in contradiction with the findings of the Italian Government. However, it was based on a thorough scientific investigation of pre- and post-disaster conditions and analyses. Jaegar(25,26) has considered Muller's work which focussed attention on the unexpected high velocity of the Mont Toc slide mass to be a major contribution to the mechanics of rock-slides.

The Vaiont disaster, however, indicated the extent of the static reserves of shells of the arch dams. In the Malpasset disaster, the failure of the rock abutments resulted in the complete destruction of the arch. It was, therefore, felt essential to strengthen the rock abutments against possible failure. Tie-rods were provided in the abutments of the Vaiont dam which resisted the tremendous forces of the rock-slide and the consequent overtopping, and did not fail in spite of the cracks in the abutments. It clearly suggests that it is necessary to strengthen the rock abutments of arch dams.

Mary(27) suggests, that the risks involved in arch dams are essentially those of the foundation and the abutment. This mechanism was vaguely understood until recently. Even though strengthening of the Vaiont dam was resorted to a-posteriori, it should be realized that the primary concern is to design a dam so that no problems develop at its abutments. The abutments of the Vaiont dam, being much too tangent, probably represented a significant design error. A similar error was responsible for the event of April 13, 1961, when the El Frayle dam almost failed in Peru.(39) Some have suggested that the wave generated by the slide at Vaiont jumped mostly over the dam without straining too heavily. This point of view, however, has not been experimentally confirmed. Also, the tie-rod strengthening may not be very effective.

One should realize the necessity of proper warning systems against such risks. Muller (28) points out that the suggestion of Stini, made some 40 years ago, to use the sensitivity of certain animals to detect slides in advance have not been implemented so far. Kiersch (23) also mentions the uneasiness of certain animals in the vicinity of the dam prior to a few days before the disaster.

h. *Baldwin Hills Dam*

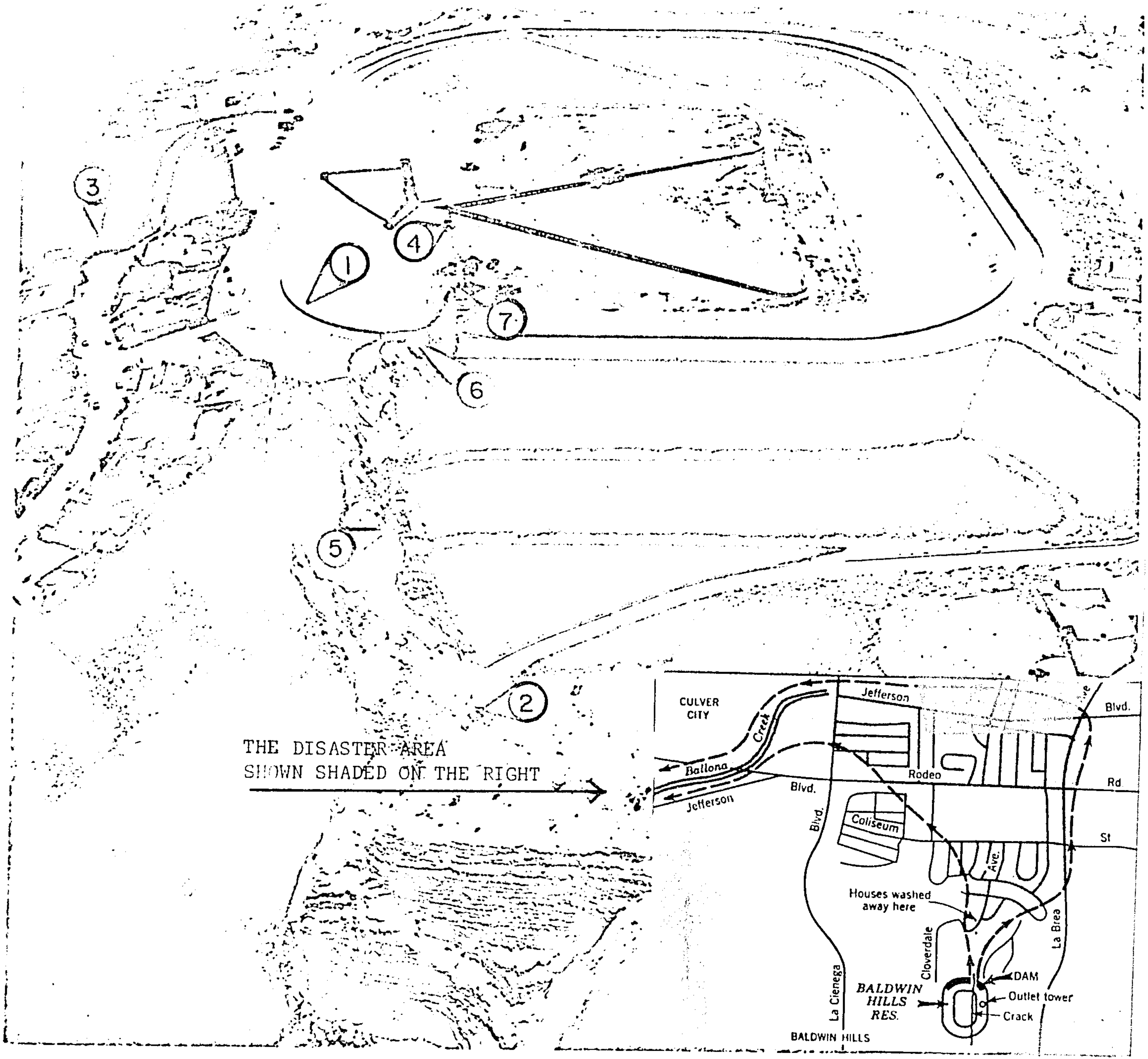
The dam, built in 1951, consisted of a main earthen embankment at the north end of the reservoir, and five minor ones to block the low-lying areas along the perimeter. The reservoir and the embankments had sandwich-type lining with pea-gravel drains to prevent seepage and to drain away all leakages. Such an expensive lining was necessary since the dam was built over a foundation having two fault zones running perpendicular to the northern embankment. During the site investigations, it was found that the faults were neither active nor potentially dangerous for the construction of a reservoir unless seriously undermined by seepage. Table IV presents the relevant data about the dam and Figure VIII shows details of the dam and the disaster area.

On December 14, 1963, the northern embankment of the dam, adjacent to the spillway, failed at a section over one of the fault lines, forming a V-shaped breach 90 ft. deep and 75 ft. wide. It caused substantial property damage, but, fortunately the loss of life was less because of the timely detection of the symptoms of the failure, and efficient warning, evacuation and rescue operations.(29)

An Engineering Board of Inquiry set up to investigate the failure reported that it was due to the gradual and progressive deterioration of the foundation which occurred because of subsidence along the fault zone, erosion under the undamaged blanket, and partially blocked drains (due to subsidence)(30). Eventually it led to the final rupture of the impervious blanket. Thus, the full reservoir water pressure, acting on the pervious

TABLE IV

Name of dam	Baldwin Hills Dam
Type	Earth fill off-stream dam
Height	200 feet
Devices in the foundation	Drainage wells, holes and settlement measuring devices.
Reservoir and embankment lining	sandwitch type with pea gravel drains
Fault zones	Two fault lines perpendicular to the northern embankment



EVENTS ON DAY OF FAILURE

1. Caretaker noticed an unusual sound of water at spillway intake at about 11:15 a.m.
2. At the catch basin, caretaker observed an unusually high flow of drainage water running through spillway pipe.
3. Caretaker entered inlet tunnel portal to reach drainage inspection chamber.
4. Inside the chamber, beneath the reservoir, drains were found to be discharging at an unusually high rate.
5. Leakage from reservoir was first observed on downstream side of dam at a point about 10 feet above 390 berm.
6. First surface evidence of possible breach occurred at Station 8+93.5, where a crack across the main dam opened and rapidly widened.
7. As flow through dam continued, an opening became evident on inside face of dam. The breach was complete by 3:38 p.m.

FIG. VIII THE BALDWIN HILLS DAM FAILURE SOURCE: JANSEN (REF. 30) AND JESSUP (REF. 29).

and erodible fault zone, created an opening through the abutment which made the overlying embankment collapse. The leaking water attracted the attention of the caretaker, and his timely warning obviously saved many lives. The earth movement that caused the land subsidence was partly due to tectonic disturbances (about 0.03 ft. per year) and partly from oil field activities. The total subsidence was believed to be about 0.2 ft. per year. The Board expressed the need for a more comprehsnive study to determine with reasonable accuracy, the contribution of the two factors to the total subsidence.(30)

There are many lessons to learn from this failure. It was designed to store water from the Owens River and the Colorado River aquaducts for distribution to the southwest Los Angeles area. The inflow could be regulated by a valve in the inlet tunnel. Thus, the reservoir and the dam were designed not to be overtopped by floods.

It may be noted that the dam was built on defective foundation. The earth movements and the weak and the erodible nature of the foundation were considered when the dam was designed and the geological investigations were thorough. The designers did consider the erodible nature of the foundation when subjected to water pressure. This resulted in the creation of the impervious blanket. But, the possibility of shearing of the membrane on account of fault movement was not thought likely.

The City of Los Angeles, and its Department of Water and Power sued the oil companies operating in the Inglewood Oil field on the west side of the reservoir. The oil field activities were known to the designers of the Baldwin Hills, and could perhaps have been projected into the future with the assistance of the oil companies.

The reservoir was kept under strict surveillance and the maintenance operations were good.(30) The last annual maintenance inspection of the dam by the State Supervision of Dam-Safety Office was made on April 3, 1963, about nine months before the failure, and it was found to be satisfactory.

The Baldwin Hills dam failure thus clearly showed that the conventional methods of safety supervision are quite inadequate. Other activities surrounding the dam may have serious consequences on the safety of the structure. Also whenever a certain structural provision is made for protecting a dam, the operation of such provisions be vigilantly monitored and materials under stress be sampled to determine deterioration. An undetected deterioration terminates in a sudden failure with catastrophic consequences.

Few opinions have been expressed as to whether the failure could have been avoided. Based on available evidence a conflict of opinion is quite obvious.

i. *Koyna Dam*

The Koyna Dam, built in 1962, on the River Koyna about 3 miles upstream of an abrupt eastward bend, was located in a region of peninsular India with little or no history of seismic activity. However, earth tremors were reported during the first filling of the reservoir, and, gradually, the magnitude and intensity of the tremors increased with time. A major earthquake occurred on December 11, 1967, having a magnitude of 6.5 R.F., which shook the dam and the neighbouring Koynanager town. Some 180 people lost their lives and another 2200 were injured in the ensuing catastrophe.

Filling of the Koyna reservoir started during the monsoons of 1962; earth tremors were reported in the vicinity. Later in 1963 increased frequency of tremors was observed with rumbling sound. Prior to September 13, 1967, a total of about 96 shocks were recorded with magnitude ranging between 1.45 to 3.57 and the focal depth between 3 to 5 km. However, on September 13, shock magnitude increased to 5.5 and then on December 11, 1967 the records showed magnitudes as high as 6.5 with numerous fore and after shocks. The focal depth of this shock was between 8 to 20 km, epicenter 3 km. (south of the dam), radius of perceptibility about 600 km and peak acceleration more than 50 percent gravity for 5 seconds.

It is rather surprising to note that no serious damage occurred to the dam in spite of the reservoir being full and the severity of the earthquake. Even though the dam was designed for a seismic acceleration equivalent to 0.05g., it withstood a peak horizontal acceleration of 0.5g. without collapsing. This clearly indicated inadequacy of present knowledge to predict seismic behaviour of dams. The horizontal earthquake acceleration alone does not fully define the seismic regime. Clearly a more satisfactory measuring index is required, and this should include more representative parameters in addition to the horizontal acceleration. Ambraseys has pointed out that neither an earthquake can be specified solely by its maximum acceleration nor the structure and its foundation can be assumed to be absolutely rigid (31). Studies carried out in Japan show that 76 dams have failed due to earthquakes of average magnitudes (32). Thus, the conventional equivalent static design methods are not realistic (33).

The Koyna dam thus survived a catastrophic ground acceleration of 0.5g suffering only minor bruises, which included damage to: (i) 50 ft.-high tower over the elevator shaft; (ii) gate control chamber; (iii) supports of road bridge over tallest spillway section (due to relative motion between adjacent monoliths); (iv) horizontal crack on the upstream face at elevation 2,060 ft. above datum; (v) faint seepage at the downstream face; and (vi) electric generators below the dam.

Remedial measures were carried out by prestressing the monoliths, while other necessary repairs could restore the dam to normal working condition.

A dynamic analysis was carried out (50), taking into account horizontal acceleration, dead weight and water pressure on the dam. It indicated a peak tension of 250 psi at elevation 2,060. The accelerograms also showed twice the number of zero crossings compared to those observed in California where seismic accelerations as high as 50 percent gravity has been reported by Jansen (51). Besides, the seismogram analysis also indicated a slip along a fault, whereas no surface evidence of faulting was observed.

The highest hydrostatic pressure at the reservoir bottom is calculated as 100 psi. This pressure is comparitively much smaller than the dilational stresses (compressive) at depths of 2 to 5 miles, where the hypocenters of the shocks are normally located. However, the shear stresses at these depths vary between 5 to 2 psi and thus appear to be most significant in causing stress failures that result in fault slip. The shear can, therefore, be considered as a triggering stress. Apparently the earth's crust beneath Koyna Reservoir was already in a critical state of shearing stress before the dam had been constructed.

Based on the experiences of Northwest Territories of Canada, Healy and Jackson (34) show that zones having historical very low seismicity could have residual stress in the earth's crust which could cause swarms of micro-earthquakes. The shocks are caused by stress failure or slip over a fault area.

Percolation of water in the rocks due to rising reservoir level may also adversely affect the strength of the rock, and thus trigger an earthquake. No evidence could be found at Koyna to justify such a phenomena.

A team of United Nations experts found the cause of earthquake at Koyna to be of tectonic origin, folding and faulting. (5) However, it has also been suggested that to blame the reservoir for the earth shocks was "to give too much credit to too small a load on all counts." Based on observations at seven man-made reservoirs (Boulder Dam (Colorado), Monteyard Dam (France), Grandval Dam (France), Kremasta Earth Dam (Greece), Kariba Dam (Rhodesia and Zambia), and Embalse de Canelles Dam (Spain) and Koyna Dam (India)) throughout the world, Rothe (35) has shown that tremors and shocks have occurrred in their neighbourhood where almost no seismic activity had been observed prior to their construction. The exact mechanism of these tremors is yet to be fully understood.

Oberti and Lauletta (32) reveal numerous difficulties in simulating the behaviour of concrete with a lead alloy, and dynamic earthquake loads with a special impact generator. Also, the modern methods of theoretical analyses are suitable for very simple structures within the elastic range. Non-uniformity of cross-section, structural discontinuities, cracks, non-homogeneity, etc., invariably complicate theoretical analyses.

The Koyna disaster points out two of the basic shortcomings for more effective design of dams:

(i) Understanding of dynamic behaviour of dams;

(ii) Causes and effects of various triggering mechanism for earth shocks.

It may be possible to learn many more lessons from the Koyna dam failure if systematic research and investigation is carried out to resolve the problems it has posed.

j. *Smith Mountain Dam*

The Smith Mountain dam, located 35-miles southeast of Roanoke, Virginia in United States, and spanning a gorge of Roanoke River, is a concrete arch dam. The dam was built to create a storage reservoir as a part of a pumped-storage system. Water is pumped into this reservoir from Leesville dam pump storage, located 18 miles downstream. Actually, the inflowing water had such a high debris content that it created a serious problem. A similar situation at the Oroville dam in California also presented a difficult debris problem. Table V briefly summarises some details of the dam. Four penstocks with pumps, installed to raise water from the Leesville dam to the Smith Mountain reservoir, had a trash-rack each at the upstream of the pipe ends. In January, 1965, a routine inspection by divers revealed serious damage to the trash-rack bars. Also, some holes had formed on the screen. It was thus concluded that screen panels were too light, and the unsupported length of the panel was sufficiently large to be damaged by vibrations generated from contact with the folowing water.(55) As a result of these observations the bar assembly was strengthened by providing bars of $\frac{1}{2}$-inch size in place of 3/8-inch on the panel screen, with horizontal bracings at 18-inches spacing.

The damage to the trash-rack screens was not expected to cause a catastrophic failure of the dam as such; however, the deteriorating screens, if left unrepaired for a long time could have caused an appreciable damage to the turbines. Timely detection of the defect was possible only on account of the underwater inspection and investigation that are now carried out as a matter of routine on dams. Underwater vigilance and supervision of dams and reservoirs have also been stressed in some recent submissions to the ICOLD.(56)

TABLE V

Name of Dam	:	Smith Mountain Dam
Type	:	Concrete arch dam
Height	:	227 feet (above river bed)
Crest length	:	816 feet
Reservoir capacity	:	1.1 million acre-feet (40 mile long lake)
Purpose	:	storage dam for a pump-storage system and power generation
Turbines	:	4 plants installed, out of which 2 are of reversible type

Again in October 1966 the divers found that though the new bar assembly had withstood the gush of water too well, the trash-rack frames were however torn apart leaving the penstock open. An engineering analysis revealed that though the frames were safe against all loads including the dynamic water loads, the damage occurred partly due to Von Karman effect. The fluid moving around a solid shape sets up alternating vortices of high and low pressures on opposite sides that create cyclical reversal of stresses. Besides, it was observed that pumping causes a flow velocity of 18 to 20 fps in the racks as against 4 to 5 fps in the conventional units, and also the 20-feet diameter penstock jet impinged only on the end panel while almost no water passed through the sides and top of the racks. Thus these considerations required two major changes in the design:

(1) The horizontal members, beams and angles of the trash rack were streamlined to allow smother flow of water around them. This was done by fitting curved nosings in addition to welding plates between the beam flanges and angle legs. These changes not only minimized the Von Karman effect, which caused cyclical damage, but also almost doubled the section modulus of the supporting frame.

(2) The end panels were made movable to be raised above the penstock during the pumping stage.

Whereas these measures were aimed at eliminating observed adverse effects, the effectiveness of design features will depend on the test of constant field observation as well as model analysis. It is worth remembering that these critical design features are often ignored by designers in the planning stage, even though the knowledge of complex plysical phenomena is available to guard against such defects. It is, therefore, evident that designers must have before them a comprehensive listing of all possible failure mechanisms known

against which the design features must be tested to guard against repitition of unfortunate events and avoidable losses that such inadvertent defects could inflict. For every design task one must lay down specifically all possible modes of failure prior to testing of satisfactory safety performance of the design.

The lack of knowledge on sound and reliable techniques for trash racks design is highlighted by the fact that "the racks were unable to stand up either as originally designed or as modified in 1965.(55) The difficulties arose on account of the use of the turbines as pumps to drive water into the reservoir through the penstocks. Two significant problems that created troubles need elaborate investigation and study:

(i) damage to upstream trash racks; and

(ii) danger to reversible turbines from impounded water.

It is interesting to note that the new trash racks, that replaced the damaged ones, have been provided with necessary instrumentation to study pressure, velocity, and stresses induced in the rack screens due to the flow caused by the working of reversible turbines. Analysis and interpretation of the data collected would contribute significantly to the theory and practice of trash rack design and also provide the necessary understanding of the physical phenomena that occurs.

It may also be worth noting that nearly all design innovations create difficulties in the beginning and hence require a thorough investigation prior to incorporation into the actual system. An extra measure of safety factor should thus be provided at earlier stages of introduction of a new technique, even though much investigated and tested.

Failures should by no means condemn the economy-oriented philosophy of modern design. This design technique has, in fact, introduced modern science to many new physical phenomena hitherto quite unknown or not understood at all. The introduction of new technology with a thorough and continuing investigation provides the necessary tools for safe and economic design .

k. *American Falls Dam*

The 43-year old American Falls Dam, built in 1927, across the Snake River in the southern Idaho, was a combination of earth embankment and concrete gravity dam. The dam spanned a length of 5227 feet, of which the concrete gravity section was 3,500 feet long, its crest being about 95 feet above the lowest foundation level. United States Bureau of Reclamation (BuRec) was the owner of the dam.

Fifteen years later in 1942, the first signs of serious concrete deterioration in the dam became evident to the BuRec investigators.(52) They also observed that:

(i) the concrete deterioration was due to the interaction of alkaline aggregates with cement;

(ii) poor bond existed at horizontal joints in the concrete gravity section;

(iii) upper portion of the dam, constituting a roadway and piers between radial gates, suffered considerable deterioration.

Obviously, the dam was faced with a complex safety problem. An important question before the owner was to determine the state of deterioration of the structure and the rate at which the structural degradation was expected to progress. It was also necessary to assess the safety implications of the aforesaid observations. With these aims in view, an engineering investigation, carried out by BuRec, came out with the following recommendations:

(a) The dam was in no danger of collapse at the present time; however, it did not meet the prescribed safety standards;

(b) It was necessary to lower the storage level to protect the dam against excessive ice pressures;

(c) A replacement dam costing $15 million should be built immediately down-stream.

A Chicago consultant also made investigations on the dam. He found that "the degree of deterioration of concrete is comparatively slight and that the deterioration stopped many years ago". It is, therefore, clear that while there was a good deal of agreement between both investigating parties regarding the state of deterioration then, the view-points on standards of safety and future progression of concrete deterioration remained a matter of controversy and inaccurate guesswork. Whether to replace a deteriorating dam or to use it with certain precautions is often a difficult decision to make. While the use of cuation and judgement in extending the useful life-span of such dams bring greater economic benefits and allows better utilization of existing resources, a disaster would mean a much greater loss. A comparative evaluation of these aspects should be taken into consideration in addition to the reliable projection of future deterioration rate while making a critical decision on the safety of the dams.

Apparently, our immediate need for research can be identified in:

(i) developing sound, scientific, and socio-economic criteria for safety standards; and

(ii) finding satisfactory techniques to predict concrete deterioration in dams.

It is true that much remains to be done in attempting to meet these research needs. However, it is desirable in such circumstances to build a downstream dam and operate the two structures in a manner as to ensure sufficient safety to threatened life and property.

1. *Mequinenza Dam*

Mequinenza Dam, a concrete gravity structure with a highest depth of 260 feet spanned a 1550 feet long gorge of Elbro River, in Spain. Leakage appeared at the left bank in 1964 soon after the dam was completed. Both abutments started leaking thereafter, thus draining out more water than 1,000 cu. ft. per second. A thorough investigation showed that cracking of concrete was responsible for the serious leakage.(53) Several measures undertaken to ensure the safety of the structure as well as the threatened villages downstream are stated below:

(i) Heaving and dumping of earth and rock on the upstream face of the dam was started;

(ii) Concrete was injected into the cracks on the downstream side; and

(iii) Reservoir was drained, eventhough the resulting loss was $50,000 a day.

It was amply clear that noncompliance of construction specifications caused cracking and deterioration of concrete. The measures taken for the safety of the structure is, however, commendable when contrasted with the earlier mistake. In view of the increasing number of dams suffering from concrete deterioration, it is worthwhile that serious research be directed to investigate the basic causes of this phenomena and to find a feasible remedy for the problem.

m. *Drum Afterbay Dam*

The old Drum Afterbay Dam, built in 1924 by the Pacific Gas and Electric Company across the Bear River near Colfax, California, about 140 miles northeast of San Francisco, was a concrete arch dam. The structural details of the dam is summarized in Table VI. Soon after the dam was constructed, it continued to be used as a down-river re-regulator for a power plant till, in 1943, it also became a diversion structure. Its diversion role was further expanded in 1965.

Almost two decades later, it became necessary to investigate the disintegrating effects observed on the downstream face of the dam. Leakage of water through the dam was also noted. As a result of the investigation, it was shown that poor concrete and unsatisfactory horizontal joint between lifts were responsible for the defects. The concrete had poorly graded aggregates with high water-cement ratio, which, eventhough fulfilled the strength requirements, were not sufficiently frost resistant. Thus, due to frequent and severe freeze-thaw cycles, the concrete on the surface increasingly deteriodated. A remedial measure was therefore initiated to prevent this progressive disintegration. The deteriorated concrete was chipped off the surface and cavities filled with gunite (pneumatically applied mortar).

Serious disintegration was once again observed in the early 1960's. At many places the gunite fill loosened out, the deterioration of concrete extended to depths varying between 3 inches to 8 inches, the spillway surface had abraded, and lime deposits on the downstream face became prevalent. The raison d'etre for the defects was once again

TABLE VI

1.	Name of dam	The Old Drum Afterbay Dam
2.	Height	95 feet
3.	Crest length	325 feet
4.	Thickness at the top	7 feet
5.	Thickness at the bottom	21 feet
6.	Volume of concrete	6,000 cu. yrds.
	Spillway	Gated spillway with a 25 feet high toe dam forming a plunge pool at its foot
7.	Maximum Allowable design stress	365 psi
8.	Design formulae	Cylinder Formula
9.	Reservoir capacity	275 acre-feet
10.	Drainage area	12 square miles
11.	Upstream structure	Drum Power Plant $\frac{1}{4}$ miles upriver

located in poor grading of concrete and frost action. Despite the visible effects of lime deposit on the downstream face, the possible leeching and cement-aggregate reactivity effects were ignored on account of relatively small magnitude of such deposits. Later in 1965 when the concrete surface was chipped for the purposes of repair, it became quite evident that the "depth to sound concrete was considerable in some areas and indeterminate in others".(54) It was, thus obvious that frost action alone failed to explain the actual concrete deterioration in the dam.

An interesting series of investigations, reported by Pirtz and others(54), were carried out on the dam over a few years. These were:

(1) Concrete Coring Program,

(2) Soniscope Test Program,

(3) Additional Core Testing Program,

(4) Petrographic Examination, and

(5) Stress Analysis

The result of these investigations revealed that the deterioration of the dam was significantly affected by the extremely deleterious and destructive interaction between siliceous aggregates and alkali in portland cement, leeching of cement paste matrix due to water percolation, and the soluble sulphate ion (present in the aggregates) attack. These processes are seldom recognized even at an advanced stage of progression and have been claimed to be extremely slow in dry concrete and progress much more rapidly in saturated zones with increasing percolation of water.

Pirtz et al (54) also demonstrated the effective utility of soniscope tests and petragraphic examination used with ample supplemental data from the additional tests in determining the changes occurring in the quality of concrete. The examination of test data and stress analysis indicated an advanced state of deterioration and a lowered safety factor of the dam. Furthermore, considerations of benefits and costs in the light of increased risk to safety showed that an optimum remedial measure would be to build another concrete arch dam, immediately downstream of the extant dam, as a replacement structure.

As a result of these findings, the owners of the dam built a replacement structure, a similar concrete arch dam, immediately downriver that went into operation in the early 1968. Thereupon the old dam was partially demolished; and a new one with better safety assurance was placed in operation at the crucial location of the old dam.

The lessons of the Drum Afterbay experience are invaluable for engineers responsible to develop means for safety investigation and supervision of dams. It is an excellent example of initiating remedial measures before it is too late. Quite evidently a catastrophic disaster was averted by taking timely measure to replace a dam that deteriorated to a potentially dangerous degree prior to actual failure. There is little doubt that the dam would have failed in reservoir full condition, had the deterioration been allowed to progress to a dangerous degree. The investigation and replacement of Drum Afterbay Dam, therefore, must be considered a fruit of our novel safety-oriented outlook on dams and should provide the necessary foundation for the development of this growing outlook throughout the world.

o. *Conclusions*

One of the most outstanding observations that emerges from the study of some of the typical dam impairments, accidents, failures and disasters, as discussed in this chapter, is the serious conflict of technical viewpoints and complexities involved in resolving the implications of the various possible defects and weaknesses leading to impairment or failure. The conflict has been responsible for many contradictory conclusions such as "the failure was the result of utter negligence of duty or deliberate inaction inspite of alarming information" as against "the failure could not have been avoided or was quite unpredictable." The confusion arising from such conflicts have very often camouflaged the main issue, that is, lack of any knowledge about the true behaviour of dams. Obviously, these case studies clearly indicate directions of research that may be productive and require re-examination and re-evaluation. Thus significant knowledge and benefit can be obtained from the study of dam failures.

There are often several factors which together combine to bring about the final and sudden collapse of a dam. The present theory and practice in dam engineering is highly inadequate for accurate and reliable prediction of the synergistic effects of these complex factors. For this reason it is often necessary to resort to model studies for design, analysis and development of suitable theory and practice in dams. Unfortunately the use of models for these purposes is not only a complicated task but also is an expensive and time consuming proposition. Such diffficulties have often been respnsible for catastropic dam-disasters.

At the root of all dam-failures lie ignorance and lack of knowledge. Thus every accident, failure and disaster of dam have some useful lessons to offer. Such lessons could be of benefit in either avoiding or rectifying errors, already made, in future. Yet there are instances when similar mistakes are made and authorities express regret that the lessons of earlier failure had not been properly diffused. Effective diffusion of information on dam-failures is, therefore, a highly important task. In fact, the information inventory and diffusion systems for failed and deteriorating dams is highly unsatisfactory in its present state. Thus, the present study serves to focus attention on this problem.

No specialist education and research program covering a broad field of dam-safety has so far been made available. Owing to this fact, hardly any comprehensive and broad-based study of the basic problems in dam engineering has been made possible. This may probably explain the former relatively slow progress in this field and also comparatively higher emphasis placed on it recently. Perhaps all this has been possible due to the public pressure brought to bear on the authorities as a result of numerous catastrophic dam-disasters throughout the world.

We have probably not learnt to act even after several catastrophic disasters have come to pass. A moot question, however, is: have we attained an action program compatible with the present problems and in keeping with future requirements? The answer is probably a big no.

CHAPTER IV

A GENERAL OVERVIEW

About 300 dam failures were studied in this investigation and the data have been analysed in this chapter. It is quite clear that at the present state-of-the-art, all the different types of dams are subject to failure. In fact, majority of the dams studied have failed catastrophically in two ways:

(1) overtopping of the dam by a sudden flood, and

(2) undermining of defective foundation, abutment and/or environment.

Hence, a recent document prepared by the United States Committee on Large Dams on model laws for dam-safety lays special emphasis on these two aspects, and especially on a comprehensive investigation of dam foundations.

Five types of dams are generally identified in the statistics of dam failures. They are: (1) Earth-fill dams, (2) Masonry dams, (3) Coffer dams, (4) steel dams, and (5) Timber dams. The last two kinds have not been discussed in this study since they are no longer considered satisfactory or feasible at any location, for any purpose whatsoever. The first three types are treated in considerable detail in the following sections.

It is encouraging, however, to note that the number of dam-failures in the last two decades has been quite small compared to the total number of dams actually built. This becomes particularly clear with reference to the data on large dams. Details of some of the dam failures studied are discussed in the following sections.

EARTH-FILL DAMS

Statistics show that the majority of the dams built in the past have been earth-fill structures. Mermel(1)* reported that out of 925 large dams constructed during the period 1966 to 1969 throughout the world, 700 were earth and rock-fill dams. It is thus also possible to predict that earth and rock-fill dams will continue to enjoy greater popularity on account of the following reasons:(2)

(i) Soil mechanics, which provides scientific basis for their design, is developing quite rapidly;

(ii) The technique for prediction and control of flood flows is being improved;

(iii) Only weaker sites suitable for earth dams have been left for future exploitation;

(iv) whereas masonry construction costs are on the increase, cost of earthwork is almost constant.

Obviously, the safety of earth-fill dams will be of much greater importance. The following factors were found to affect the security of earth-fill dams:

(i) spillway capacity
(ii) freeboard (to prevent overtopping)
(iii) leakage through or under the dam
(iv) upstream and downstream slope stability
(v) slope protection against external damages
(vi) sedimentation of reservoir
(vii) maintenance of existing facilities at the design standards
(viii) sliding tendency at the downstream toe of homogeneous clay-textured dams due to water saturation

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* See references at the end of the report

(ix) unsatisfactory performance of cores in the absence of drainage filters

etc. etc.

Some examples of excellent earth-fill sections of Trinity dam (California), Swift Dam (Washington), and Santa Felicia Dam (Peru), especially tailored from the point of view of safety and economy, is presented in the Encyclopedia Britannica (1968).

Adequate compaction of earth fill dams is also an important safety requirement. Density of the fill has direct bearing on stability and watertightness.

It is found that the popularity of hydraulic-fill dams have dwindled in recent times. The reasons(2) are:

(i) several failures occurred in hydraulic fills, particularly during construction, and

(ii) improvement in the rolled-fill methods have made this type quite popular.

It is thus obvious that rolled earth-fill dams will continue to increase in number with improved quality of construction.

However, statistics show that majority of failure occurred to earth dams; and this proves the inadequancy of design methods from the standpoint of the factors (mentioned above) which affect its safety.

Overtopping

Handling of flood waters during construction is an important safety operation for earth dams. Quite a few accidents and failures have occurred due to overtopping of incomplete structures. It is, therefore, necessary that flood diversion works must be such as to eliminate all possibilities of overtopping of incomplete structures. A careful planning and design of by-pass facilities is therefore quite important.

The most common single cause of failure of earth dams is overtopping. Of the 300 dam failures studied, 84 were due to this cause. Overtopping is almost invariably fatal to an earth dam. It is therefore very necesary that complete hydrologic investigation of the streams dammed be made to ensure that spillway capacity is sufficient to take care of any flood that can be expected. In calculating spillway capacity, it should be realized that the possibility of settlement in the dam would reduce the freeboard, and in effect reduce the capacity of the spillway. The Johnstown Dam in Pennsylvania(3) failed in 1889, partly because settlement at the centre of the dam reduced the freeboard from a designed size of seven feet to about four and a half feet. An extreme flood destroyed the dam after it was overtopped at the central low point. About 2,300 people were drowned to death in the resulting catastrophe. Similar in cause, though less spectacular in its effects, was the overtopping of a small dam in Horton, Kansas.(4) Prompt action on the part of the local citizens prevented a complete failure in this case. Whether or not these dams would have failed if settlement had not occurred is quite unknown. Undoubtedly the settlement was a contributing factor. The settlement can generally be avoided in modern dams, if each layer of earth is thoroughly compacted before the following layer is placed.

Other dams which failed due to overtopping were the Melzingah Dams of the Fishkill and Mattawean Water Company,(5) two dams in Providence, R.I.,(6) the Oakford Park and Fort Pitt Dams in Pennsylvania,(7) the Dells and Hatfield Dams in Wisconsin,(8) the Puddingstone Dame,(9) Horsecreek Dam in Colorado,(10) the Elk City Dam in Oklahoma,(11) the East Pittsford Dam in Vermont,(12) the Frenchman Creek Dam in Montana,(13) and undoubtedly many others of lesser importance.

Several of the dams failed as a result of failures of upstream dams. The spillways, perhaps normally of sufficient capacity, were unable to cope with the sudden floods released by the upstream dam. The second Melzingah Dam, the lower dam at Providence, R.I., and the Hatfield Dam in Wisconsin, all failed in this manner. In the case of the Frenchman Creek Dam, the dam itself did not fail, although the spillway on the natural ground was washed out when a flood of about double its designed capacity flowed through it. The Elk City Dam failed because the construction engineers without consulting the designers, placed a 2½-foot concrete wall on top of the spillway to increase the reservoir storage. The spillway flow capacity was also reduced by more than 50 percent.

The Puddingstone Dam had no spillway; its small outlet tunnel was clogged with debris during a flood, and the dam was with consequent failure.

An earth dam at Erindale, Ontario(14) failed not because of inadequate spillway capacity but because of inadequate protection from the discharge of the spillway. The only outlet, an eight- by six-foot conduit near the crest, discharged its flow directly on to the downstream slope of the dam. Although it was faced with riprap, the protection washed out completely during the overtopping flood. Soon the entire earth-fill was fully destroyed.

It has been observed that embankments of small heights have been more frequently overtopped and destroyed. Several such examples are available in the statistics. Whereas earth dams as high as 130 feet have been overtopped though less frequently. One such structure was the Seefield Dam in Utah. In fact, taller dams with higher disaster potential are better equipped to handle floods of unexpected magnitude. It is thus clear that elaborate and more effective devices for handling floods can be created. It is merely a question of economics. However, it is necessary to recognize that design economics must take full account of the improved safety requirements.

Piping

Piping is defined as the progressive deterioration of concentrated leaks in the dam or its foundation caused by erosive forces that carry away soil particles with the seeping water. The erosive forces are equal and opposite to the drag which is a natural resistance to flow through porous earth-fill. Piping occurs when erosive force is greater than erosion resisting force which depends on interlocking soil characteristics like cohesion and weight, and also on choking effect of the filter. Concentration of seepage in certain areas and gradual decay of embankment or foundation material also tends to increase the erosive forces. It has been reported that piping has caused a greater number of catastrophic failures than any other mode of failure except overtopping of embankment.(16)

Failure of Schofield Dam in Utah (1928) occurred on account of internal piping of fine-grained soil zone placed directly against a zone of quarried rock. However, no failure is so far known to have occurred on account of migration of particles from fine soil zone to a zone of coarse soil consisting of sand or gravel.

A study at several piping failures and accidents show that development of piping has occurred in many possible ways:

(i) Leaks appeared on the very first filling;

(ii) Leaks appear after many years of operation;

(iii) Leakage in the form of clear water seepage for many years increased gradually until rapid failure;

(iv) Heavy muddy leak appears a few days or hours before failure.

It should, however, be noted that a temporary, short-lived muddy leakage during first filling of a reservoir may not be quite dangerous if due to minor mudfilled cracks in the bedrock.

There are several reasons for the formation of leaks in the embankment that usually lead to piping failure: -

(i) Inadequate compaction of embankment,
(ii) Insufficient pervious layers in the embankment,
(iii) Inferior compaction adjacent to masonry outlet pipes, etc.,
(iv) Poor compaction and bond between embankment and foundation or abutments,
(v) Differential settlement cracks,
(vi) Cracking of outlet pipes due to foundation settlement, or spreading of dam base, or deterioration of pipe,
(vii) Drying cracks,
(viii) Animal burrows,

Leakage through the natural foundation soil is more common and potentially dangerous. In the case of sandy pervious surface layer on the foundation, sand boils may be formed in the downstream. These boils, if left unattended may lead to complete failure by piping. On the first filling of the Fort Peck dam on Missouri River, a boil (25 ft dia. and 35 ft deep) developed up the side of the casing of a test well.(17) Similar boils occurred at the Pentenwell Project and was treated satisfactorily, on the basis experienced gained at the Fort Peck dam.

Piping has been effectively controlled in a number of actual cases by use of graded filters. Density of the embankment is also an important factor in reducing leaks likely to cause piping. Sherard(18) demonstrated that clay with a plasticity index greater than 15 has highest piping resistance while embankments of fine uniform cohesionless sand had the lowest resistance. Thus, two important needs have been identified by Sherard for improvement in piping resistant design for dams:

(i) Research to extend the knowledge of the relationship of type and density of soil to piping resistance of embankments;

(ii) Considerations of piping resistance in the design of embankment, and choice of core materials should receive special attention.

It is not known if these considerations would substantially reduce the rate of piping failures which is about 10 percent at present. It should anyhow be a significant step in gaining more knowledge about the means of reducing such defects.

The Water-Works Dam at Utica, N.Y.(19) was poorly constructed, of light, sandy loam and was provided with no corewall. The earth was placed from wheelbarrows without rolling and with very little wetting. Seepage through the unconsolidated loam developed piping and resulted in a failure of the upper one-third of the dam.

Five years later, in 1907, the Lake Francis Dam(20) failed. This was another example of poor construction. Although the bottom two-thirds of the dam was built carefully in the specified six to eight inch layers, moistened and rolled, the remainder was built hurriedly with no layers and no wetting and even the sides of the banks were not cleared of stumps and roots. A settlement crack appeared suddenly in the upper section, which developed piping and finally wash out a 98-foot wide gap through the embankment.

The Lyman Dam in Colorado(21) was carefully constructed of finest quality materials, but it failed in 1915 because the reservoir level rose rapidly as the dam was being built. On account of improper drainage the dam was not properly consolidated. It thus developed piping in the embankment near the base that finally washed out 350-feet long gap.

The porous foundation of the Greenlick Dam at Scottdale, Pa.(22) was responsible for the failure of the dam. Several feet of settlement and cracks at the crest gave evidence of the poor consolidation during construction. Finally, water piped through the seams of the rock at the end of the dam, rather than through the embankment itself. The leak rapidly enlarged and a large portion of the dam was washed out.

The French Landing Dam in Michigan(23) was undermined when water seeped through the sandy foundation material. The piping ultimately caused a 135 feet wide breach through the dam.

Sloughing or Raveling

One of the dangerous modes of piping failure has been progressive backward erosion of concentrated leaks through or under the dam starting at a point where the seepage water is discharged out.(16) This type of failure occurred in a few older homogeneous dams, beginning with a small miniature slide and erosion at the downstream toe forming a relatively steep face which, when saturated by seepage from the reservoir, slumps again forming a higher and more unstable face. This process continues till the reservoir breaks through the dam with a sudden breach. Leakage all along may remain quite small.

Sinker Creek Dam, Idaho (1943) failed by rapid backward sloughing on account of excessive seepage saturation. It occurred as the reservoir had been filling for a longer time than usual. And within eight hours of the first indication of sloughing, the water broke through the dam. In fact, very little is known about this kind of failure, and hardly any significant research has so far been carried out in relating the various design features of earth dams with the progression of such defects.

Seepage along conduits

Pressure pipes or conduits passing through earth dams have been a frequent source of trouble. Water from the reservoir has a tendency to follow along the surface of such pipes or conduits, if no cutoff collars are provided. Even if collars are provided, the possibility of opening of the pipe joints by unequal settlement is always present.

Two types of failures are noted. In some cases no cutoffs were used with the conduits to break the unobstructed path of the water along their outside. Others were due to break in the conduit that allowed water under full reservoir pressure to escape into the earth-fill which was then washed out.

The Hatchtown dam failure in Utah(24) was due to seepage along the outside of a five foot by eight foot concrete conduit that washed out some earth and eventually pushed a 30-foot section of the dam downstream. The Davis Reservoir embankment in California, which failed in 1914(25) and the Weisse-Desse River Dam in Austria(26) failed for the same reason, that is seepage along outlet conduits with no cutoff collars. In both these cases, poor construction methods were partly to blame for the failures, and mainly it was due to poor compaction. In the Weisse-Desse dam, for example, in addition to a poor type of material, the placing was done in 16-inch layers. Later examination showed that there was hardly any beyond four or five inches into each layer.

Another failure partly due to poor construction was that of the Horse Creek dam(27) in 1914. It was reported that the upper courses of the dam had been placed in layers of almost four feet thick without any rolling. A settlement of seven feet in the first year gave evidence of the poor compaction obtained. The ultimate cause of failure was the seepage of water from the underlying conduit which cracked as a result of settlement of the gate structure.

The Table Rock Cove dam(28) failed in 1929 even though the construction methods were quite satisfactory. A 42-inch conduit, which passed through the dam were found broken to pieces at every joint. As the valve had been placed at the downstream end of the conduit, the reservoir pressure forced water through the cracks in the joints and cut a deep, narrow trench in the embankment and caused a slump of the downstream slope.

Embankment and Foundation Slides

Slides occur in the embankment and the foundation whenever the average stress along any potential sliding surface exceeds the average strength of the material and the mechanics of failure is similar to that of the natural landslides. Methods of stability analysis has been developing with the aid of knowledge gained from actual slide failures. The reliability of analytical procedures have been improved over the years by extensive study and research. Vaiont slide, however, reveals that much remains to be known about the exact development, triggering and behaviour of the actual sliding phenomena.(29) It should also be noted in this context that prediction of slide failure of an artificial embankment or foundation of smaller size has been possible to a certain degree of reliability. Development of reliable procedures for stability analysis resulted in the reduction of the number of slide failures after 1940, as will be revealed from Middlebrooks' statistics in Table I. Table II, however, shows that chances of slide is greater in the earlier years of the life of the structure and almost 50% of the slide failures occurred within 5 years of embankment construction; and around 40 years of satisfactory performance may be considered an effective test of time, so far as slides are concerned.

It is, therefore, clear that slide failure of embankments and foundations have reasonably been brought under control. Only a few "poorly constructed dams with very weak foundations" have failed in recent years.(16)

TABLE I

Chronological Distribution of slide Failures (In Percentage of Total Number of Dams on Which Slides Occurred) Source: Reproduced from Sherard (Ref. 16)

Calendar Year (During Which Slide Occurred)	Percentage of the Total Number of Slides Recorded
1850-1860	0
1960-1870	0
1870-1880	0
1880-1890	3
1890-1900	3
1900-1910	16
1910-1920	23
1920-1930	26
1930-1940	23
1940-1950	3
1950-	3

TABLE II

Relation of Occurrence of Slide to Age of Dam (In Percentage of Total Number of Dams on Which Slides Occurred) Source: Reproduced from Sherard (Ref. 16)

No. of Years After Completion	Percentage of Slides Occurring
0-1	29
1-5	24
5-10	12
10-20	12
20-30	12
30-40	11
40-50	0
50-100	0

Slides during construction have been relatively few with no major damage or loss. It may, however, damage the relationship between the engineer and the client. The engineer should, therefore, always make sure that the client is aware of the calculated risk if and when taken for the sake of economic design. In fact, there can be two types of construction slides: (i) slow slides, and (ii) rapid slides. The difference in the two types of slide probably depends on the characteristic of foundation clay. Rapid slides occur when foundation clay contains bedding planes, lenses, or layers of coarse silt or fine sand which are sensitive (i.e. they lose strength appreciably under shearing movement) and abruptly transfers much of the weight to the pore pressure spread over the horizontal layers. On the contrary slow slides occur when foundation clay is insensitive.

Several earth dams, being constructed by the hydraulic-fill method, were damaged to a greater or lesser degree by slides occurring during the construction operations. A dam in the Lea Valley in Essex, England (30) designed as 34 feet high failed when it had been completed up to 26 feet. The downstream slope moved outward over a length of 300 feet, sliding in a circular arc which passed through a part of the foundation made of yellow clay. The Snake Ravine Dam in California (20) was built on a site of two previous construction failures. The dam failed during construction largely due to the fact that it was built on top of wreckage of the first two, and no foundation explorations were made.

The Fort Peck dam (31) in Montana failed when the hydraulic fill slid rapidly during construction due to the liquifaction of loosely compacted sand thus killing eight people. Since the rolled-fill is unlikely to be in such a loose state, this lesson may not be applicable to it. The Waco dam (32,33) in Texas, which was about 90 ft. high with relatively flat side slopes, failed on account of weaknesses in the rock foundation. The slide surface passed through a soft, brittle and sensitive clay about 20 ft. below the horizontally bedded shale formation.

The Tappan and Clendening dams, (34) in Ohio, were considered to have failed by slow slide on account of overcompaction of the embankments which caused the underlying soft clay foundation to expand and push the toes upstream. The incompatible strain characteristics of the embankment and the foundation is the most probable cause of a slow slide at these two dams.

The North Ridge dam (35) in Canada failed by slow slide during construction in 1953 when almost complete. The foundation clay, hich had high plasticity index (51%) and water content (36%), was monitored with piezometers and construction was recommended only when excessive pore pressure had dissappeared. It is however interesting to note the visible manifestations that were actually observed at the North Ridge dam before the slide actually occurred. They were: (i) appearance of cracks; (ii) slight bulging of the fill; and (iii) overthrust at the downstream toe. Observation of these symptoms could be useful in future for monitoring of earthdams. Later the dam was completed to the proposed height without excessive movement.

Failure of the Chingford dam (1937) in England and the Lafayette dam (1928) in California are also typical examples of slow slides. At LafayetteeDam no attempt was made to complete the embankment to a height originally planned. Besides, the slopes were also considerably flattened. Yet the slow and uniform movement continued during the 25 years after completion.

The Muirhead dam slide (1940) in Scotland is another typical example of slide failure during construction resulting from excessive pore pressure in clay.(37) Banks(38) considers Knockendon and Muirhead as early examples of the application of Soil Mechanics to dams in U.K.. Konckendon was the first place where the pore pressures in boulder clay were recorded during construction.(36) This led to changes in the design criteria, and the actual designs of the two dams, simultaneously under construction, were altered to make them safe against sliding. In fact, both the dams had a similar design and even same height (90 feet), and therefore suffered from the same defect.

Marshall Creek Dam(39) (1937) failed in Kansas also during construction by a typical rapid slide when the embankment was 10 ft. short of its proposed height. The major movement covered a period of 15 to 20 minutes and was accompanied by a rumbling noise and dense clouds of dust.(40) The dam was founded on a 30-feet bed of non-continuous layers of soil capable of transmitting high pore-pressures.

There are several examples of downstream slope slide earth-dams during reservoir operation. They have been primarily of two types: (i) deep slides passing through clay foundation; and (ii) shallow surface slides.

Deep slides are caused by internal pore-pressures built up due to almost reservoir full seepage through and/or under the embankment.

The slide frequently extend to the upstream slope below the full-reservoir water level thus reducing the freeboard. The unstable vertical slide scarp left standing often sloughs or slides due to the existing pore-pressure and may ultimately breach the dam releasing a huge volume of flood. Such failures can, however, be saved by emergency action and if not at least its disastrous consequences can be prevented. The movement of these slides are almost same as slow slides, around 3 to 4 ft. per day in the early stages.(16)

In the Fruit Growers dam (1910), in Colorado, the downstream slide buried the outlet conduit. It thus necessitated the use of a bull-dozer to cut through the embankment and natural abutment in order to empty the reservoir. A second slide due to wetting of downstream slope could therefore be averted.

While the Great Western dam (1958), of Colorado suffered a downstream slide when water reached and stayed at the maximum elevation for about two weeks. The dam was however raised and strengthened in the previous year. The slide had the following characteristics:

(i) the slide surface passed into natural clay foundation,

(ii) it opened on the upstream side below the maximum water level allowing free access of water into the slide surface,

(iii) it had cut through the steel outlet conduit, thus crushing and choking the outlet.

(iv) the rate of slide movement was 1 foot per day.

A syphon pipe was therefore used at the crest to lower the water level at a rate of 1 foot per day in order to maintain a safe freeboard. However, sand-bagging of the crest to increase freeboard was ruled out as it would have accelerated the slide movement. The dam was thus saved and subsequently rebuilt.

The Gunnison Dam near Austin, Colorado (43) failed by sliding of the downstream portion of the dam in 1937, when the entire bank became saturated. The lowering of the effective weight of soil and reduction disastrous failure by cutting a trench through the dam and allowing the water in the reservoir to flow out.

The Sinker Creek Dam in Idaho (44) failed similarly when sustained full reservoir levels had saturated the embankment from bedrock to crest. A three hundred foot breach opened when the entire height of the dam slumped.

Shallow slides do not occur due to deep percolation of impounded water through the dam. They are, however, due to saturation of the downstream face by shallow percolation of excessive precipitation. This is exactly what happened at the Costilla dam, New Mexico (1920) when an 80 feet long mass of earth from downstream slope slid down from above the heavy rock riprap covering the lower portion of the slope. It is often difficult to determine by visual examination whether the slide is merely superficial. Test pit observations carried out at the Costilla dam revealed that the slide was only a few feet thick. (45)

The shallow slides may occur near the poorly drained downstream berms, roads and rock-riprap layers anytime after construction as a result of rainstorms. Minor slips could occur during the first rainy season due to poor compaction of surface layers. All this may involve quite expensive remedial measures and hence should be taken care of in the construction phase.

Upstream slope slides could occur during the reservoir drawdown. It is not quite impossible for upstream slides to take place in reservoir full condition, although almost all cases known have occurred following reservoir drawdown. Some of the observed characteristics of these slides are stated below:

(i) They are seldom the cause of complete catastrophic failure because of low water level and complete absence of a tendency for continued sloughing and sliding. There is a complete dissipation of pore-pressure on account of upstream slide which is quite the contrary to downstream slide.

(ii) Majority of drawdown slides occur between maximum water surface and mid-height of the dam at an average rate ranging between 0.3 to 0.5 feet per day, mostly during the first lowering and sometimes after many years of operation.

(iii) Majority of the slides had a sliding surface passing deep into the clay foundation, and at the top normally extending up to the upstream edge of the crest and sometimes as far as middle of the crest while only a few reaching downstream edge of the crest.

(iv) The rate of movement is same as that of deep slow downstream slides.

(v) None of these slides have caused sinking of the crest and thus freeboard remained within safe design limits.

(vi) At Belle Fourche Dam,(41) in Montana, a shallow upstream slide occurred in 1931 (after about 20 years of operation) on account of an unprecedented rate of drawdown. This movement was similar to a shallow downstream slide arising from heavy rainstorms.

(vii) In some hydroelectric power dams, where riprap wave protection was placed in the range of elevation over which reservoir was to operate, progressive shallow surface slides developed during the first filling below the riprap due to erosive wave action as the slope was steep. Similar slides occurred when the embankment was made of gravel. The effect was further aggravated when the slope surface was undressed or steeper haul roads were left in place.

Obviously, a proper grading and dressing of upstream slope is essential for its stability. Some general conclusions drawn by Sherard et al(16) on slide failures of earthdams may be worth noting:

(i) Rolled-earth dams have not failed by sliding unless the embankment or the foundation consisted of fine-grained solids.

(ii) There appears to be a strong correlation between the deep slides and the fine-grained, highly plastic soil in the embankment and clayey foundation with high plasticity and natural water content.

Spontaneous liquifaction have been found to be the cause of many flow slides in earth dams. It has been reported (16,42,46) that the Calaveras dam, a hydraulic fill under construction in California, failed in 1915 by a flow slide occurring due to the liquefaction of outer shell of the embankment. As a result of hydraulic deposition the outer shell was in a loose state. Thus the shear strain, produced by the upstream fluid pressure, triggered the flow slide of the loose embankment.

Similar slides were reported at the Necaxa dam, (47) in Mexico (1909) and the Alexander dam, (48) in Hawaii Island (1930). The Sheffield dam, in California, failed by flow slides caused by spontaneous liquefaction of the loose lower portion of the embankment due to earthquake shocks. It has been discussed earlier in great detail along with other case studies. Also the failure of the Red Mountain dam, near San Diego, California in 1949, was due to a flow slide of liquified material. The internal downstream sand drain, consisting of 3 ft. thick uncompacted layer of sand, got chocked up by waste stripping material following the first filling of the reservoir. Consequently the liquified zone of the embankment and sand drain was pushed out as a flow slide. In spite of several such failure, it is unfortunate that the mechanism of liquefaction and flow slides have hardly been studied in a comprehensive manner.

Cracking in Earthdams

Significance of cracking has been well recognized by dam-builders only in the last decade when considerable amount of attention has been paid to it. Sherard et al (16) in 1963 pointed out the astonishing dearth of literature and experience on earth dam cracking even though they have been responsible for a large number of leaks and piping failures in embankments. Such failures have been observed even in well constructed embankments. In fact, Arthur Casagrande (49) and some other specialists of soil mechanics are now of the opinion that these cracks may develop as a result of natural shrinkage and settlement of earth-fills. Poor design and construction can make the situation still worse.

There has been some deliberate efforts in the past to conceal serious defects arising from cracking of embankments. It was considered to be a mere consequence of unsatisfactory construction practice and hence damaging to the professional pride. There appears to be a gradual change in this attitude as more and more published experiences, discussions and research on prediction of cracking in dams are being made available. A recent informal discussion on cracking of clay cores held by the British Geotechnical Society in January, 1970 has aptly concluded that no reliable quantitative analysis to predict the formation of cracks in embankments could be developed so far. (54) It is, therefore, clear that much research would be required to rationalize this aspect of dam-design.

Differential settlement has been considered an important cause of cracking. When an embankment is subjected to such a settlement, it may develop tensile strains resulting in cracking patternsn of different types which depend on the geometry, and the relative compressibility of foundation, embankment and abutments.

Various types of cracks have been observed in the earth embankments. They are: (i) transverse cracks; (ii) longitudinal cracks; (iii) localized cracks; (iv) continuous cracks; (v) open cracks; and (vi) interior cracks. There is much ambiguity regarding the cause and formation of these cracks. However, a comprehensive discussion of the available information on this phenomena is presented by Sherard et al(16).

Transverse cracks have generally been found to develop in the embankment piping. Failure of 112-ft. high Apishapa Dam (1923) in Colorado is known to have occurred due to horizontal transverse cracks which formed by arching of the upper portion of the embankment that prevented the crest from settling as much as the foundation. The foundation consisted of compressible soil while the abutments were made of steep hard rock. The transverse cracks were undermined by piping till the water broke through the enlarged cracks in the embankment within a few hours of filling.(41,50,51) Also, Rao(52) pointed out that bad quality earth and its poor compaction was responsible for the cracks and ultimate failure.

Compression of embankment, differential settlement in the vicinity of a haul road left on the abutment, differential settlement near the outlet conduit, variable compressibility of foundation material, different compressibility of the core and shell of the embankment, presence of more compressible clay lens or less compressible interior masonry structures in the embankment have all been responsible for cracking in various dams. Poor quality of embankment materials is also an important factor.

Although there are no reliable field and laboratory methods for estimating maximum settlement required to cause cracking, Sherard (41) and Narain (53) provide useful guides to predicting cracking potential based on soil gradation, plasticity, water content and soil compaction. Further research would be required to correlate these factors with actual occurrence of cracking and this may provide the necessary knowledge to minimize cracking in dams. A relaible prediction of cracking in earth dams should thus proceed hand in hand with the attempts to evolve a satisfactory crack resistent embankment design.

Of late, some attention has been paid to the formation of cracks in the clay core of dams and embankments. An objective analysis shows that cracking of cores is the result of interaction between three factors:

(1) Differential settlement between a core and its supporting shoulders causing a net vertical stress;

(2) Overburden pressure; and

(3) Pore pressure set up by the reservoir head.

At some level in the embankment the resultant stress may be rendered zero. And this would cause the cracks to open. Reduction of pore pressure by lowering reservoir level will cause the roof of the crack to collapse and in non-cohesive moraine soil the seal will be re-established, while in cohesive materials a stable crack may be formed due to erosion of clay fraction leaving the coarser particles within.

The swelling of cores due to reduction in effective stress have also been observed. It is found that cracking is most likely in soils of low swelling coefficient or low plasticity index. As cracks are formed due to low resultant earth pressures, they can be prevented by increasing earth pressure by grout injection. In fact, this has been practically proved at the Balderhead dam.(54)

The Balderhead dam, in England, showed signs of trouble in 1968 when a 100 mm diameter hole developed at the filter. At once investigations were started to determine the causes of the trouble. Twenty trial pit were dug to monitor subsidence and resultant earth pressures in the embankment. The test piezometers were found reasonably accurate and economic device for detection of cracking. However, use of earth pressure cells could be quite reliable but expensive.

The leakage was quite high, about 60 litres per second, during reservoir full condition. As the reservoir level was lowered, the dam crest settled by 125 mm and the leakage rate decreased substantially. The test pits also showed the formation of a smaller hole right across the dam from upstream to the downstream end of the core. However, the horizontal cracks closed when the reservoir level was lowered.

A clay-cement grout injection was made into the core of the dam. A seriously damaged core was reinforced by a plastic concrete wall inserted in a slurry trench to meet the cut-off wall.

From the Balderhead experience it became quite clear that the investigation of cracking in earth-dams was not only difficult and complex but also quite time consuming. The investigations of Balderhead dam clay core could be completed in more than one year.

Another 10 m high embankment with sloping clay core took considerable time for a thorough investigation when leakage appeared at the toe. Eventually, it was found that the actual pipe hole use to drain water during construction had been left in the clay core.

The swelling of shale shoulders produced a lifting effect on the core and reduced resultant stresses. The vertical stress was also affected by different properties of the adjoining zones of the shoulder and the core as well as the changing core section. The leakage through the cracks formed due to these effects and resultant erosion could have caused the failure of Baldwin Hills and Belo Horazonte dams.(54)

Observations on Lovewell Dam, in Nebraska have shown that drying cracks can even form in a homogeneous embankment of clay, with low plasticity index, if allowed to dry in the sun for an appreciable length of time during construction. The work on the dam was suspended half way for three weeks. Drilling and boring explorations revealed that these cracks could absorb large amounts of water and closed on account of swelling of the embankment soil in presence of water. This experience led U.S. Bureau of Reclamation to specify that "contractor (should) protect completed portions of the embankment against drying out by sprinkling or covering with loose earth." Thus, Creager (55) as early as 1939 also pointed out the necessity "in extreme cases" for continuous wetting of embankment with water from a ditch on the crest to prevent the formation of shrinkage cracks. Sherard et al, (16) however, finds that a combination of factors such as (1) hot, dry, windy weather with long periods of empty reservoir condition; and (2) highly plastic, poorly compacted, and low density embankment material could cause high tensile stresses that form excessive shrinkage cracks. These cracks can develop near the top of the dam parallel to the crest and in other areas on the surface causing constant maintenance problems and crack erosion which may require expensive and complete reconstruction of embankments.

Wave Action

Wave erosion of upstream slopes is caused by wind generated waves. These waves depend on bad storms that are quite uncertain and last normally for a short time. However, only a few poorly constructed dams with inadequate freeboard and excessively steep unprotected upstream slopes have been threatened with complete failure. In most cases the embankment slopes were badly eroded and damaged. Thus it was necessary to carry out expensive emergency measures or heavy repairs for the safety of the dam.

Different materials are used for the protection of earth slopes against wave action. They are, in decreasing order of preference, as follows: (1) dumped rock riprap; (2) hand placed rock riprap; (3) articulated concrete pavement slabs; (4) monolothic reinforced concrete pavement; (5) steel plate facing; (6) asphaltic concrete layer with floating log-booms protection; and (7) soil-cement facing. The preferences have been evolved as a mere thumbrule and little,if any.scientific study of the comparative performance of these materials have been made. However, Bertram(56) presents a very useful study based on the data collected from 100 dams in the United States. His conclusions are worth noting:

1. Forty of the dams studied had layers of dumped rock riprap varying in thickness between 9 and 96 in. as upstream slope protection. In only two of these, or 5% of the dams protected with dumped riprap, were there failures which resulted in displacement of the rock and loss of slope protection. The average rock size on the two dams which failed was only 5 in., and the estimated wave heights were 2.5 and 4.0 ft., respectively. In addition to the two dams with displaced riprap, loss of the underlying gravel filter layer caused damage to three others in this group.

2. Twenty of the dams had hand-placed rock riprap with thickness between 12 and 48 in. In six of these, or 30% of the total, displacement of the rock occurred. The damaged dams had riprap blankets between 12 and 36 in. thick, and the maximum wave heights were estimated to be between 1.5 and 8.0 ft. In four of the six dams which failed, the primary damage was attributed to floating trees or ice.
3. Concrete slabs varying in thickness between 4 and 12 in. were used on 14 of the dams, and five of these were damaged from wave action. Two dams had 8-in.-thick monolithic reinforced concrete slabs, which were not damaged.

It is quite clear from Bertram's conclusions that dumped rock riprap is an excellent slope protection material. On the other hand, no conclusion can be drawn about the behavious of reinforced concrete pavement for which sufficient was not available. It is true that reinforced concrete pavements would be superior to the articulated slabs, which are not quite satisfactory. Further investigation would be required to extend this study as well as verify the conclusions over a more representative cross-section data.

The Point-Of-Rocks dam,(16) in Colorado, nearly escaped a catastrophic failure in 1927 as a result of wave erosion. It was also established that a 4-inch thick reinforced concrete slab on a 1.5 to 1 upstream slope is quite inadequate to protect a dam 90 feet high against severe wave action. The usual practice of making steeper upstream slopes in earth dams was therefore given up.

While the Belle Fourche dam,(57,58) in South Dakota, suffered a minor damage due to wave action on the upstream facing, which consisted of precast concrete slabs 6 feet x 5 feet in plan with a ½-in. spacing between them for drainage and a maximum of 2 feet gravel filter underneath. It occurred only four years after the completion of the dam. In fact, this event could have served as a warning against the potential weakness.

Nevertheless it was ignored, and a few years later a major storm dislocated large number of slabs which required heavy and expensive repairs. Since then the problem has become of a continuous nature, every major storm causes damage to the facing and it is particularly serious when the waves strike the facing obliquely. During the wave recessions two important phenomena were noticed: (1) spouting of water jet from the facing joints, and (2) some slabs were charged out about 10 ft from the dam facing. It was thus obvious that seepage of water through the joints was responsible for the serious head difference or pore pressure on the facing slabs during recession. It thus became very necessary to take account of seepage in the design of earth dam facings on account of its great significance from the point of view of both safety and economy.

Use of tongue and groove joints, probably of inadequate design, for connecting the facing slabs became a source of perpetual trouble at the Minatare dam, Nebraska, built in 1915. The filter beneath the slab was often washed out and the resulting settlement caused cracking of the slabs. As a remedy, cement grouting was used to fill the cavities under the slabs. However, it was noted by the maintenance crew that the grouted joints formed a monolithic structure of the slabs and was quite effective in preventing future trouble. A thorough investigation of such a process would be useful to devise a safe and economic concrete slab facing for earth dams. However, it is also important to note the severe settlement and cracking of concrete facing slabs during a storm, with gale winds lasting for three days, at the Harold dam, California after 30 years of so-called satisfactory performance. The report of the State Engineer(59) also emphasized that this condition took many years to develop to the point where slight settlement and cracking of the slabs allowed an increased rate of erosion. Obviously it is very necessary to exercise continuous vigilance over the performance of concrete slab facings and also to investigate means to minimize, if not eliminate, the usually observed defect of gradual sucking out of filter material through the weep holes in the concrete.

The Kingsley dam, in Nebraska had a somewhat different type of slope protection consisting of concrete slabs bonded with iron rods. Even in this case a similar type of undermining occurred as observed at the Harold dam, which required replacement of the facing by dumped rock riprap at an estimated cost of $3 million.

Sherard,(16) however, shows that monolithic reinforced concrete pavement provides the most successful wave protection for the upstream slopes. Though no failure has so far occurred, there exists a possibility of bulging and cracking of concrete due to ice action. In addition to it the "wave breakers", that are incorporated to prevent excessive wave run up on smooth concrete facing, generated heavy water spray that created several difficult problems such as: (1) wetting of the downstream slope with possible harmful effects; (2) appreciable loss of reservoir water; and (3) neighbouring unpaved roads became muddy. Curved wave walls have, however, been designed as a partial solution to these problems.

Soil-cement facing has been tried by the U.S. Bureau of Reclamation engineers on a test section at Bonny Dam, Colorado and this was adopted in the design of Merritt Dam, constructed in 1963. Such a facing appears to be highly promising from the point of view of safety and economy too.

Damage Due to Water Soluble Materials

Presence of certain water soluble ingredients like gypsum, etc., in the materials of embankment, foundation and abutment, subjected to seeping water, can cause serious troubles. Leaching of soluble gypsum causing appreciable loss of solids from the abutment and foundation material has been observed on two Californian dams: the Dry Canyon and the Buena Vista. The leaching gradually increased leakage and loss of solids in both dams. Whereas grouting in the abutment had to be resorted to after 24 years of operation in the former case, the latter settled 2½ ft on account of an earthquake. It was thus evident that weakening of the structure due to leaching though gradual was quite appreciable to cause sufficient concern about the safety of dams. The dam becomes like a diabetic patient gradually losing weight and strength and the danger of complete failure gradually increases with time unless vigilance is maintained and remedial measures (corrective and preventive) are taken in time. Timely remedy is the only was to save such inadvertantly constructed dams.

In addition to weakening of earth dams by cavity and flow channel formation due to leaching, these soluble materials in the seeping water may be deposited once again in the pervious filters thus choking the drain. This happened at the Vermillion dam,(60) in California. It is also likely that the seeping water with soluble salts may appear at the downstream face where the drying effects of wind and temperature cause deposition of these soluble solids thus choking the seepage pores of the embankment, foundation and/or abutment. Choking of these seepage paths causes high pore-pressure to develop to dangerous levels at the downstream face with consequent sliding or sloughing.

According to Sherard et al(16) there has been no failure or damage on account of leaching of embankment material for the following reasons:

(i) Mostly the river water is saturated with the salts of the region through which the river flows. As embankments are built of soil containing similar soluble ingredients, they dissolve only slightly.

(ii) In most modern construction practices water is added to the borrow material which also dissolves the soluble salts.

(iii) The percentage of soluble ingredients is normally not very high. Obviously Sherard et al disagrees with the viewpoint of Field(61) that Apishapa earth dam failed due to dissolution of salts from the embankment material. However, it is generally accepted that poor material was definitely responsible for the formation of cracks and occurrence of piping in the embankment of the Apishapa dam.(52) It is, therefore, quite obvious that dissolution of soluble salts must have been one of the contributing factors in the deterioration of the embankment which finally gave way.

Burrowing Rodent Damages

There are two types of burrowing animals that may attack earth embankments: (1) muskrats and (2) ground squirrels. The muskrat normally cuts through embankments from the upstream side to the downstream in a small dam with tail water at the toe. The mouth of the holes are usually hidden under water or in the bushes growing at the water-line. Whereas the ground squirrels dig only in dry soil and stop away from seepage lines. They burrow only above the high water level and are known to cut through embankments during prolonged periods of drought.

The destructive influence of these rodents on small earth dams is well known in certain areas while they normally do not penetrate great depths in major dams.(16) Floating of creosote in the reservoirs or compaction of outer layer of the earth fill with creosote may reduce muskrat hazard in earth embankments.(62) At the Throttle dam,(63,64) in New Mexico, a 12-gauge galvanized iron was core was installed to full height and width of the dam in order to prevent frequent rodent attacks. The Heart Lake dam,(16) in Colorado, had an asbestos-bonded corrugated sheet metal core which was sufficient to save the dam from such animals.

It should, however, be noted that none of these methods, used with some success, have been put to a longer test of time. Obviously, it would be necessary to keep a close watch on dams and embankments that are threatened by burrowing animals even if preventive measures are utilized. Constant vigilance over even minor embankments in areas infested by muskrats or ground squirrels is highly desirable.

Earthquake Failures

Sherard et al(16) documents 12 earthquake failures and Ambraseys(65) describes seismic behaviour of 58 earthdams in exhaustive detail. No individual dam failure due to seismic shocks will be discussed here. Some important conclusions are, however, summarized below:

(i) Available information concerning seismic behaviour of earth dams is quite inadequate to improve the design and construction methods. Some efforts have been made in this direction by Ambraseys and others. However, it is often difficult to ascertain the reliability of the collected data.

(ii) One of the recent dams that suffered severe earth shocks is the Koyna dam in India. The event is highly significant as it initiated a serious rethinking about the validity of existing concepts on seismic behaviour of dams. Full details can be obtained in Chapter III wherein the Koyna dam has been specifically discussed in an elaborate manner. However, it may be noted that the Koyna dam is a concrete gravity structure; and no well-constructed modern earth dam has as yet been subjected to similar severe seismic shocks. The Hebgen dam which was subjected to a severe earthquake in 1959 was built as early as 1914. Thus, little, if any, information is available on the seismic behaviour of earth dams that are built with the benefit of modern design and construction techniques. Yet, it is generally believed that earth dams have greater resistance to earthquakes as compared to masonry dams. Nevertheless, it may be noted that some of the recent concrete dams like the Koyna dam, and the Blackbrook dam also suffered very little damage even though they were subjected to a seismic acceleration that far exceeded the design value.

(iii) Longitudinal cracks at the top of the embankment and crest settlement were observed in the majority of earth dams subjected to severe shocks. Crest settlement were comparatively small.

(iv) Horizontal component of crest movement was much greater than that of the foundation.(66) This was responsible for greater damage to the thinner top of the dam.

(v) Very few earth dam slopes slid during earthquakes.

(vi) Low frequency earth-shocks caused greater damage to earth dams than to the buildings. Ono Dam in Japan located 60 miles from the epicenter was severely damaged while many dams located very near to the epicenter were scarcely damaged.

(vii) Earth dams with concrete core walls suffered greater damage and cracking. The assessment of the St. Mary and the Hebgen dams revealed that the concrete core wall and the embankment did not vibrate in unison during the earthquake. Probably a similar phenomena also occurred in Ono and Murayama dams with central clay cores. Non-homogeneity of the dam body is considered responsible for aggravating cracking within the body of the embankment.

(viii) New knowledge of earthquake resistant design for earth dams has been recently used by the Bechtel Corporation of San Francisco, California. The firm has designed an earth dam to be located on the San Andreas Fault. The design is expected to absorb movements as high as 30 ft horizontal and 5 ft vertical. It has an upstream impervious core, protected by riprap, supported by wide zones of clean sand, gravel and rock designed to be self-healing.(67)

ROCK-FILL DAMS

There are two types of rock-fill dams: (1) dumped fills and (2) rolled fills. One of the important defects in the dumped rock-fill dams is the settlement of the fill that cause cracking of upstream membrane thus endangering safety of the dam. Sluicing of rocks during dumping however reduces post-construction settlement. Also, careful placing of upstream rubble layers may partially prevent cracking of the impervious membrane.

It may be noted that use of graded filters in earth and rock-fill dams to prevent piping (migration of fine-core materials) has been an unique achievement in the dam-building science from safety point of view.

Besides, safety and stability of rock-fill dams are also dependent on the following factors: (1) spillway capacity; (2) freeboard; (3) quality and durability of rock material; (4) settlement of fill; (5) settlement and piping in the foundation; (6) strength and stability of the fill; (7) stability of upstream and downstream slopes; and (8) grout curtain.

An outstanding hazard to the safety of rock-fill dams is overtopping. Few rock-fill dams have withstood the erosive forces of overflowing water, and the most frequent cause of failure is the provision of inadequate spillway capacity.

The Castlewood dam near Denver, Colorado(68) failed in 1933 when the foundation material at the toe was carried away by overflowing water. It might be noted that this dam was severely criticized for several defects in its design, yet it stood with minor repairs for about 45 years until it was overtopped to a depth of one foot.

The Lower Otay dam in California(69) was rock-fill structure 130 feet high, with a rivetted steel diaphragm in its centre acting as the watertight membrane. Although the upstream face was protected against erosion by a log boom, the downstream face had no protection. Obviously the rock fill downstream of the diaphragm was quickly washed away as a result of overtopping of the dam in 1916 and the diaphragm tore open like "a pair of gates".

The Bluewater Dam in New Mexico,(70) a smaller rock-fill structure also succumbed to overtopping in 1909, when an unprecedented rainfall caused the reservoir level to rise above the crest.

Three other dams, of composite type rather than pure rock fill, having an earth-filled upstream portion however failed for other reasons. The Pleasant Valley Dam(71) failed when shrinkage cracks opened in the earth-fill section, and the fine material washed out through the rock-fill portion with resultant caving of the earth above. The Lake Avalon Dam in New Mexico(72) of similar construction, failed in almost a similar manner. It is however suggested that holes made by the burrowing animals in the earth-fill portion allowed seepage paths to develop through the structure. Also, undermining of the foundation caused the failure of the Zuni Dam in New Mexico(73) when water passed under a cap of lava rock flanking the dam and extending under the spillway. The spillway fell down almost seven feet vertically and some of the earth fill was washed away while the rock-fill portion of the dam was practically undamaged.

Rock-fill construction generally produces a very satisfactory and stable structure, and provided overtopping can be prevented, there is relatively little danger of failure. Large vertical settlements have occurred in some rock-fill dams, and in some cases it was more than 5% of the height. The excessive vertical settlement accompanied by horizontal displacement can cause dangerous cracking of impervious diaphragm. This has occurred in many cases, although a complete failure of the dam does not usually result.(74,75) Such a phenomena may be regarded as a downstream deflection of rock-fill dams.

It was estimated that the crest of Crane Valley Dam,(16) in California moved about 9 ft horizontally in the downstream direction in four years at a decreasing rate since the filling of the reservoir in 1910. Thereafter, measurements show that a movement of 5 ft occurred in 38 years (up to 1952). The movement was, however, retarded by dumping large quantities of additional rock on the crest which resulted in a crest width of 100 ft and downstream slope of 1.3 to 1. Exploratory shafts sunk upstream and downstream of the dam revealed the following details:

(i) The bottom of concrete core wall did not move and remained keyed to the bedrock.

(ii) The core wall had badly cracked horizontally and vertically almost 6 ft above the bedrock level and changed the slope of the wall.

(iii) Leakage and seepage was however prevented by the sealing effect of hydraulic fill upstream.

(iv) The material under the upstream slope in contact with the core wall was in a state of liquid mud and exerted lateral pressure on the core wall.

It was thus clear that the high lateral pressure was a significant factor in the crest movement in addition to the steep slope of the dumped rock-fill. It may however be noted that no theoretical methods are as yet available for the analysis and prediction of rock-fill dam behaviour and estimating critical downstream slopes to minimize such movements.

Similar fate befell the Oued Kebir Dam(16) in North Africa (1929) where the central concrete core, about 30 feet thick, was sheared off and moved downstream by 5 inches while the top moved almost 3 feet. The core was supported by an embankment of partly dumped unsluiced quarried rock (half of it was hand-placed) at a downstream slope of 1:1. This event once again proved beyond doubt that although rock-fill dams did not develop slides on steeper slopes, they did, however, suffer from creeping downstream movements, which could be dangerous on the long run.

MASONRY DAM

Dams built of stone masonry or concrete are called masonry dams. These are of several types: (1) gravity, (2) buttress, (3) arch, (4) arch-gravity, (5) cofferdams, and (6) brick dams.

The Almanza Dam, built in the early 16th century, is the oldest masonry dam in service for last 400 years.(1) The Alicante Dam has also survived for the last 300 years without any evidence of serious damage. Obviously, the life span of masonry dams can be quite long if properly designed, constructed and operated with due supervision.

Some of the important factors relevant to the safety of masonry dams are: (1) water pressure, (2) silt pressure, (3) wave generated forces, (4) ice pressure, (5) earthquake forces, and (6) seepage effects on the dam and the foundation. It is, therefore, very necessary to consider the extreme effects of these forces in order to arrive at a safe and economic design. Such an opportunity is provided by a study of dam failures.

Various mechanisms of failure of masonry and concrete dams have been discussed in the following sections. It is, however, pointed out that the design of dams are normally checked for stability against overturning and sliding, for structural strength of the dam body and the foundation, for underseepage (or uplift) and downstream degradation, and last but not least, for handling of flood water (spillway and freeboard). The overall safety of the structure depends on how well each of these aspects have been taken care of.

In the discussions to follow the term "masonry" would be loosely used to distinguish dams of stone laid in mortar from the much more common concrete dams. However, with the increasing construction costs, masonry dams may become less common in North America, unless technologic development renders it otherwise.

A list of 153 masonry and concrete dams, that failed, is tabulated in Appendix I. Also, a statistical summary according to dam types is given in Table III.

The statistics in Table III shows that out of a total of 153 dam failures studied about 70 were rubble, stone and other types of masonry dams. The majority were of gravity type while only two of them were buttress structures.

Another 50 dams that failed were of concrete gravity type. These casualties were quite comparable to the total number of failure of masonry dams. In fact, the major bulk of failures fell into the category of masonry and concrete gravity type dams with both almost equally to blame. The primary reasons for the failures were improper design and unsatisfactory site conditions.

Failure of other types of dams like concrete arch, multiple arches with buttresses, arch gravity, buttress type, ambursen type, R.C. cofferdams and brick dams were comparatively quite few. The main reasons probably were the strength of concrete shells and also comparatively small number of these structures, built in the past.

The legendary invulnerability of arch dams has been shattered long back. However, it is diffucult to ignore the safe and economic performance of these structures, which have been demonstrated behond any doubt. Hence, they offer much greater promise for the future at suitable locations.

TABLE III

A Statistical Summary[a] of Failures and Accidents of Masonry and Concrete Dams

	Type of Dams	Number of Failures and Accidents
(1)	Masonry Gravity	68
(2)	Concrete Gravity	50
(3)	Masonry with Buttresses	2
(4)	Concrete Slab with Buttresses (Ambursen)	12
(5)	Concrete Arch	8
(6)	Multiple Arch with Buttresses	8
(7)	Arch Gravity	3
(8)	R.C. Cofferdam	1
(9)	Brick Dam	1
	Total	153

(a)The actual data is discussed and tabulated in appendix I.

Overtopping

Overtopping of masonry and concrete dams have been quite frequent. This situation not only points a finger to the fundamental inadequacy of our reservoir system design from hydrologic considerations but also establishes the basic uncertainty of meteorologic factors responsible for flash flood runoff generation of unpredictable magnitude. No satisfactory means have so far been developed to safely drain out such unpredictably large volume of flood waters. Inadequate spillway has therefore been an important cause of failure. In addition to this difficulty the water retaining structures have been overtopped due to excessive sedimentation in the storage basin beyond a safe limit. However, it may be remembered that the idea of safe storage volume is at best a probabilistic concept and no absolutely safe storage volume is known to exist vis-à-vis economic considerations of spillway and the capacity of the storage valley. Faulty operation may lead to choking of the outlet or careless filling of the reservoir, etc., which could cause disastrous overtopping failures.

Overtopping may thus be considered a system failure and much research is in progress to minimize such failures. These researches attempt to relate the system safety with the economics of propsed measures. Overtopping of a few dams, however, reveal that it may not at all cause any structural failure whatsoever. Rincon de Bonette hydroelectric dam(76) on the Rio Negro River in Uruguay, designed for a flood, with a probability of occurrence of once in thousand years, was overtopped after 14 years in April, 1959, by a flood with an estimated probability of once in half a million years as a result of a unique meteorologic condition called Sudestada Prolongada. The possibility of even greater floods have now been predicted by the meteorologists. Despite being overtopped by such a large volume of flood the dam did not fail, although a subsidiary dam breached to provide some relief to the structure.

Vaiont dam in Italy which was also overtopped by a sudden wave of water generated by a massive land-slide is known to have escaped complete structural failure. While it is generally agreed that the structure was subjected to a tremendous force due to rock-slide and consequent overtopping by a 330-feet high flood wave, Schnitter(77) believes that the wave generated by the slide jumped mostly over the dam without actually straining it too heavily. However, it must be accepted that the condition created by the overtopping flood had neither been imagined by the designers of the structure nor provided for in its structural design. Yet the dam withstood the adverse conditions remarkably well.

Besides, there have been many overtopping of dams which caused minor damages due to inherent defectiveness in design, construction and maintenance. Such damages apparently can be eliminated by better supervision, and improved techniques of design and construction. The accidental failure as a result of inadequate spillway occurred to the Fergoug Dam,(78) in Wadi Oued, Algeria, the flash flood overtopped the dam and washed away the weak crest to a depth of 33 ft. The weakness of the crest was, of course, the cause of the structural failure which could have been avoided. Later the dam was, however, rendered superfluous by Bou-Hanifa Dam.(79) The Santa Catalina Dam, in Durango Province, Mexico, also suffered almost no damage but inflicted a serious loss of property and life.

Several dams such as the Sella Zerbino dam (Molare No. 2) in Italy, the Sedjar dam in Syria, the Elmali dam in Turkey, the Killingworth, the Prosser Creed, the Sweetwater and the Two Medicine dams in United States were all overtopped due to inadequate spillway resulting in little or no damage whatsoever. The Two Medicine dam, however, had the earth embankment washed away and spillway slightly tilted due to undermining. Investigation of these cases offer evidence that it may be possible to design masonry and concrete dams safe against structural failure due to overtopping.

The Moyie River Dam in Idaho(87) was overtopped with little damage to the concrete dam itself, but a spillway channel through a ridge at the side of the dam was washed out, and the abutment of the dam was cut away. The Merril concrete gravity dam,(80,81) in Wisconsin, lost its earth wings due to the overtopping floods, whereas the riprap protection was only washed away from the Moose Jaw River Dam,(82) in Saskatchewan, Canada. Perhaps, these failures also could have been eliminated by suitable design and construction methods. The inadequacy of the design becomes quite apparent in case of the Del Gasco masonry gravity dam,(83) which consisted of two vertical walls filled with rock and clay. The dam failed during construction by overtopping of the incomplete structure as the arrangement for diversion of flood water was quite inadequate. The percolating water caused swelling of clay that pushed out the front wall.

It is unfortunate that complete absence of scientific principles of dam design and inadequate realization of the power of flowing water in the past led to the construction of such masonry dams that were completely destroyed by the overtopping flood. One of the ancient failure occurred in Egypt around 2800 B.C. when the Saad El Kafara Dam(84) was washed away by the first heavy rains in its 72-square miles catchment area. Some curious features of the dam were: (1) no mortar used in construction (2) no spillway provided (3) inadequate reservoir capacity and (4) no other drainage outlet. Meteorologic considerations showed that the dam could be overtopped and washed away during any average heavy rains and absence of sediments in reservoir proved that this occurred soon after the dam was completed. Later, the Gasco Dam (1780) in Spain, the Columbua Dam (1901) in Georgia, the Rasul Dam (1929) in Pakistan, and the Clinton Dam (1938) in Connecticut were also overtopped, breeched and washed away by extreme floods. These examples, however, show that in fact very little was really learnt from some of the past failures. Even the lessons of

Saad El Kafara were not known to the builders of Marib dam, in Yemen, who committed a more disastrous blunder that completely destroyed a rich prosperous valley of Sheba. Use of suitable masonry and satisfactory provision for drainage of flood waters are probably the most important lessons of these failures.

Overtopping of masonry dams could cause several other types of failure, such as: (i) undermining of the foundation and subsequent wash out, (ii) erosion of the foundation in the downstream side, and (iii) sliding at the dam base. It should however be noted that no overturning really occurred due to overtopping floods. The overturning tendency, probably, affects the stability and upthrust in the foundation which in turn either favours undermining and wash out below the dam or even sliding at the base due to increased upthrust.

The Cheurfas dam,(79,85) built across Sig River in Algeria, was overtopped in 1885 three years after construction and the foundation was badly eroded making it quite unserviceable. The dam was however restored to service in 1935 by Coyne prestressing. Also, the Dayton reinforced concrete cofferdam, in Ohio, was undermined below the base and washed away twice. This was, in fact, a temporary structure. It is, therefore, quite clear that even though the undermining and erosion of masonry dam foundation have not been very frequent due to overtopping, such a possibility should be borne in mind by the designers.

A few other masonry dams such as the El Habra Dam (1927) in Algeria, the Bric Zerbino Dam (Malare No. 1, 1935) in Italy, and the Austin Dam (1900) in Texas have failed by sliding on the foundation when overtopped by high floods.(85,88) All these dams were built prior to 1895 when the "middle third" rule was "altogether unknown".(85) This rule was, however, propounded by Maurice Levy in his communication to the French Academy of Science four months after the collapse of Bouzey dam in 1095. The performance of the dams so far reveal that the 'middle third' rule has been very effective and probably oversafe in preventing overturning and sliding of dams. The provision also eliminates tensile stresses in the upstream toe and hence no water can percolate below the dam that could increase upthrust and sliding tendency.

Again, burrowing rodents were responsible for weakening of the foundation under the Manchester dam(111) in Connecticut. The dam, however, failed by sliding when overtopped by a flood.

A typical case of breeching of the Khadakwasla masonry dam near a step in the foundation has almost shaken the most established principles in dam technology. This dam, according to the existing static design criteria, should have failed when overtopped in 1961 by (i) sliding, (ii) overturning, and (iii) tension failure at the tallest section. But none of these failures really occurred showing that the "middle third" rule was highly conservative and oversafe. It was also shown that the conventional static analysis could provide highly erroneous conclusion near a step in the foundation.

The above analysis makes it amply clear that overtopping of masonry and concrete dams presents a potential danger to dam-safety. The various problems posed need a thorough and comprehensive investigation in order to evolve a satisfactory design criteria for these dams.

Slips and Slides in Foundation, Abutment and Environment

Landslides at the reservoir site during construction and after has been responsible for complete abandonment of dams. The Ruck-a-Chucky concrete arch dam (1940), in California, had to be abandoned as a result of extensive slips and slides in the reservoir environment.(89) The Zardezas concrete gravity dam in Algeria was seriously endangered by large volume of slides and required construction of retaining walls to stabilize the slopes.(79,90)

Probably the largest known slide in a reservoir environment occurred at the Vaiont Dam, in Italy, in 1963. The slide is often described as floating of a whole mountain by a high head of water that crashed into the reservoir. It is thus recognized that the water level in the reservoir was an important factor that triggered such a massive slide. It is generally believed that the mode in which the slide actually occurred could not be predicted and the velocity of the mass was an unusual factor that still remains unexplained. The measurement of the slide velocity has, therefore, acquired great importance and have been carried out for the first time in an Austrian reservoir rock-slide.(91)

In spite of the realisation that dams should be designed safe against sliding, the Tigra masonry dam(93)(1917) in India and the Ashley Brook Ambursen dam(92)(1909) in the United States failed by sliding on the foundation on account of the uplift that developed but could not be anticipated by the designers. Inadequacy of cut-off wall resulted in percolation of water beneath the foundation causing a reduction in the net downward force as well as friction factors acting on the dam. These failures have, therefore, directed special attention of the modern dam-designers to the design of satisfactory cut-off walls, grout curtains and foundation drainage facilities.

The Saint Anthony Falls Upper Dam, in Minnesota, a masonry gravity dam built in 1894, was weakened due to the ice-pressure that tended to rotate it about the toe, and later in 1899 it slid down on the foundation by the force of a flood wave.(94,95) Whereas the Lincoln Pond gravity dam,(96) in New York, slid on a smooth rock foundation inflicting considerable loss of life. Both the cases are glaring examples of inadequate design and construction of the dam and its foundation. These experiences has, however, improved our knowledge of dam-building and it has been possible to detect in advance and thus eliminate such failures to a reasonable extent.

The Grois Bois Dam(97)(1838), one of the old French dams, slid about 2 inches during first filling and was therefore reinforced with eleven buttresses to prevent further movement and catastrophic sliding of the dam. However, later on sliding failures most often occurred due to seepage undermining of the foundation and pore pressure that resulted from inadequate foundation drainage. The Danville concrete gravity dam,(98) in Illinois is considered to have completely collapsed on account of such a condition in 1930. Seepage in shale stratum under the Austin dam,(99) in Pennsylvania, caused foundation settlement and sliding of the dam. The concrete in the foundation also cracked showing that the concrete base had been weakened due to settlement and sliding. Similar fate befell the North Fork Dam in Illinois, and the Roxbury Dam, in Vermont.(85)

An important mode of foundation failure observed at the Nashville Dam in Tennessee (1912), Malpasset Dam in France (1959), and Wheeler Dam in Alabama (1961) have created considerable concern to the dam builders of modern times. The Nashville dam failure, that occurred by a slide on a saturated clay seam, could not draw sufficient attention probably on account of its relatively insignificant height (34 ft). Besides, Soil Mechanics had not by then found its way to application in dams. The situation is perhaps also explained by the relative lack of concern about this failure even though it had been observed that a section of the limestone foundation remained attached to the masonry foundation after the slide. Unpredictability of such slides in the Malpasset and the Wheeler(100,101) dams, which were much higher structures and resulted in extremely catastrophic consequences, immediately became of grave concern and much technical intercourse and research is presently underway to produce means and know-how to control these failures.

Erosion, Piping and Seepage

Besides slips and slides, undermining due to mere erosion and piping in the foundation has also been quite frequent. It is a common observation that a large majority of failures occurred during the period of construction and in the early years of operation. Length of such a period may be about four to six years after the completion and full stage operation of the dam. Undermining normally starts at points of weakness from the very earliest stage of loading of the structure, and, therefore, all such points need to be kept under strict surveillance during the early period of operation.

Poor foundation material, inadequate cut-off, lack of drainage of percolating water into the foundation, and ice thrust that create severe tensile stresses at the heel causing undermining by water penetration are the numerous causes of piping. Erosion of the foundation material on the downstream side has occurred on account of inadequate protection against the erosive action of the spillway discharge. Seepage of downstream water into the foundation must be checked as it can really undermine the foundation beneath the apron and the dam.

Ice thrust was the cause of failure of gravel foundation below the Bow River Dam in Alberta.(102) Erosion of the poor foundation material was also observed in the Namaka concrete gravity weir in Alberta,(103) the Komoro buttress dam in Japan, the Ansonia Mill Concrete retaining wall in Connecticut. The Lake Waco concrete gravity dam,(104) in Texas, failed in

1947 on account of erosion of shale foundation below spillway apron and overflow section. The overflow section, the apron and a part of the dam was badly damaged. The dam was, however, subsequently repaired. Piping and erosion has thus caused complete overturning, sliding and destruction of many masonry dams. Numerous dams failed by piping during construction, initial filling or a few years after completion. The Puentes dam(105), in the Murcia Province of Spain, failed after 11 years of operation by undermining of the wooden pile foundation when the water level in the reservoir rose for the first time to a height of 154 feet. A very large number of people were drowned and property destroyed when water gushed out catastrophically through the wash-out beneath the dam. The remaining structure, however, remaining propped on the abutments. Similar failures occurred to the Sacedon and the Vega de Armijo dams in Spain, and the Macdonalton, the Port Angeles, the Lake Leigh, the Elwha, and the Janesville dams etc. in the United States. The Dansville dam, in New York, also has the deck slab snapped due to the erosion of the foundation. Obviously, foundation undermining has been one of the major causes of masonry and concrete dam failures.

Undermining of abutments has caused complete failure of some concrete gravity dam in reservoir full condition. This occurred to the St. Francis dam, in California, which marked the beginning of a comprehensive safety supervision of dams in that state. The Lake Lanier dam, in North Carolina, and the Moyie River dam, in Idaho were both concrete arches that failed by abutment wash out. Soft or poor foundation rock was probably responsible in both these cases for abutment undermining.

It is also quite clear that seepage of water has been a dangerous factor in the safety of foundations. Inadequate cut-off had resulted in excessive uplift under the apron of the Sarda Masonry Weir. The magnitude of the uplift was almost twice the weight of the apron.(85)

Faulty foundation design from the point of view of seepage has caused many failures and, therefore, should deserve special attention and investigation of the behaviour of cut-off and drainage devices in the foundation.

The West Brook dam,(99) of reinforced concrete Ambursen type, in New York State, failed in 1916 due to seepage beneath the foundation. Water percolation had been observed at the toe during filling but unfortunately ignored because no engineer was posted on the job during the construction. Thus six bays were washed away as a result of undermining of the foundation.

Overturning

A few dams have failed by overturning about the toe. The Conodogainet Creek dam (1912) in Pennsylvania, and the Rockport dam (1912) in New York overturned due to ice pressure. It appears that the designers had ignored the overturning forces due to ice pressure that should have been taken into account for application of "middle third" rule. The Housatonic masonry dam, in Connecticut, however, failed due to loose foundation rocks that seriously disturbed the stability of the dam. It is, thus, clear that even deformable foundation could also cause overturning of solid masonry dams.

It should, however, be recognized that though the dam designers prior to 1850 were well aware of the necessity of checking the design for dams against sliding and tilting, but the full realization of safe requirements for both these factors came only after the failure of the Bouzey Dam in France in 1895 when Maurice Levy communicated the 'middle third rule' and explained that under such design condition water would not penetrate the dam body or the foundation due to the overturning tendencies.

Foundation Movement

Movement in the foundation and abutments was responsible for the failure of the Ljusne Stommar concrete gravity dam (1949) in Sweden, and the Horonai dam (1941) in Japan. Minor foundation movement, probably does not affect masonry dams very much and as such failures are few and far between. However, the possiblity of foundation movement should be examined and taken full account of in the design of masonry dams. It is likely that the fate of the Baldwin Hills Dam may some day befall any masonry dam too.

Highly tangential nature of the abutment is a very dangerous situation for arch dams. The Vaiont dam in Italy and the El Frayl dam in Peru suffers from such defects.(77) In fact, the tangential forces could be extremely dangerous at sites with movement potential. The designer should therefore guard against such errors. Though no masonry dam is known to have failed so far in this manner, it is important to remember this mechanism particularly when designing high dams.

Leakage Through The Masonry Dam

In most concrete dams, built earlier to 1920, laminations were commonly observed in the monoliths.(106) This was a result of preparing concrete mix by hand and tossing them into the blocks from scaffolds about 2 meters in height. Leakage through the interlayer surfaces of these laminations progressively increased due to leaching and disintegration of minerals. The disintegrated products were carried in suspension and solution with the seepage water to the downstream face. Whenever the flow was small and ambient temperature sufficiently high, the evaporation of water caused deposition and incrustation in the pores near the downstream face. With higher seepage rate, however, the material in suspension also flows out. In the former case the upstream pores get choked up causing high pore pressure on the face leading to a progressive slide failure, while on the contrary in the latter case the concrete is weakened due to gradual loss of material.

Frost action has a serious effect on the concrete and masonry water retaining structures. Cracking of concrete due to frost action, ice pressure, seepage, and poor design of concrete mix could all be responsible for leakage in masonry dams. Leakage of concrete dams like the Mequinenza Dam in Spain, and the Griffin dam in Pennsylvania is well-known. Much study and research is in progress of late to prevent leakage of masonry dams, which can seriously impair the safety of these structures.

Cracking and Spalling of Concrete

The study of a number of failures due to cracking and spalling of concrete in dams have shown that this kind of impairment may occur on account of various causes: (i) By frost action and ice pressure, (ii) Temperature or shrinkage cracking, (iii) Cracks and spalling due to floods, and (iv) Concrete disintegration.

Frost action is harmful to the concrete because of the brittleness caused by such low temperature. The surface concrete, therefore, should be specially designed to resist frost action. Frosting caused a serious spalling of the Allard dam, a cracked multiple arch concrete structure in Quebec. The surfaces were, however, repaired with gunite. A similar fate befell the Dell Mountain multiple arch dam in the United States. The Des Moines dam, in Iowas, however, failed due to concrete deterioration initiated by ice pressure. Cracks were also noticed in the piers of the Lake Hodges dam in California, and the Lake Pleasant dam in Arizona. It is quite important to note that the visible signs of cracking proved to be a safety alarm to the caretakers of these two structures who were obliged to take suitable measures that ultimately helped to avert the imminent collapse. In the latter case however the spillway was reduced in height to hold water 41 ft below the high water level. Obviously, it was an important step to save the dam from a catastrophic failure.

While cracking and spalling of concrete create serious maintenance problems and require expensive repairs, it is not likely to cause disastrous failures unless left unheeded for a long time that allow progressive deterioration and weakening of the structure. Also now-a-days, it is considered very important to make underwater investigations of concrete and stone masonry structures at definite intervals in order to keep track of their deterioration which cannot be detected by any other conventional inspection procedures. The Tennessee Valley Authority has found tremie concrete, dry bagged concrete, and preplaced-aggregate concrete very useful in underwater repairs and they show excellent resistance to erosion.

It is also of great importance to note that temperature cracks can create very serious problems particularly in concrete arches. The Tolla concrete arch dam, in Corsica, severely cracked due to unequal temperature stresses caused in the body of the dam. Most modern dams, therefore, have provision for the measurement of temperature in concrete gravity and arch dams. Numerous papers presented at ICOLD Congress (1970) in Montreal describes the instrumentation which helps in drawing isothermal lines within the body of the dam to analyse the stresses due to such temperature differentials. The experience gained from these measurements must be fed back to improve the conventional methods of design so that safety of dams against such failures may be ensured.

Disintegration and deterioration of concrete due to various causes such as: (i) Alkali aggregate reaction, (ii) Sulphate ion attack, and (iii) Leaching of cement paste matrix are now quite well known. The deterioration is further aggravated by fluctuations of stresses, severe loading stresses, and poor bond at the concrete joints. This is amply demonstrated by the American Falls Dam, in Idaho, for which the U.S. Bureau of Reclamation recommended lowering of storage level to reduce deterioration. An early replacement of the dam was also suggested in order to satisfy the revised safety standards.

The Drum Afterbay(108) concrete arch dam, in California, had to be replaced on account of serious concrete deterioration due to the above mentioned causes. The investigations on this dam revealed the value of soniscope tests, petragraphic examination and other additional tests in determining the strength of progressively deteriorating concrete. Similar disintegration occurred at the Gem Lake(109) multiple arch concrete dam in California due to low freezing temperature and the Manitou concrete arch dam in Colorado. Thus both the structures were later converted to gravity section in order to protect them against further deterioration and complete failure. The Matilija arch-gravity dam also suffered expansive disintegration and was ordered to be lowered and subsequently replaced. The dam had a poor foundation too.

It is thus quite obvious that cracking and spalling of concrete could be very dangerous and highly catastrophic if treated indifference. In general, however, it may have minor safety implications but might involve sufficiently expensive maintenance problems.

Faulty Design, Construction and Operation

In fact, all failures can be considered a result of poor design and construction of a dam, its foundation and the environment, or faulty operation of the structure. Various cases of failure involving poor design and construction have therefore been discussed in earlier sections, devoted to failures due to other causes identified earlier. However, it is difficult to draw a definite line between dam failures that occurred due to lack of knowledge and those due to ignorance of the methods already available.

It may be noted that De Sazilly (1853) and Delocre (1866) were the first to independently point out the necessity to design the gravity dam dimensions on the basis of strength of materials i.e., internal stresses.(105) They also emphasized that these internal stresses must be kept within safe limits in addition to the statical stability of dams checked against the criteria for sliding and tilting. They further suggested the requirements to design dams safe for reservoir full and empty conditions, for trapezoidal distribution of stresses under the dam base, and for vault effect of arch dams.

Over the years these considerations have developed into sophisticated design criteria and safety standards. To these considerations, many more have been added by way of study and analyses of various dam failures. The actual and model studies of dams in operation also contributed to this knowledge.

Poor construction joints between concrete and masonry have been the cause of failure in several dams, and, therefore, need the special attention of the designers. One of the outstanding examples of this type is found in the failure of the Veg de Tera Dam, in Zamora Province in Spain.

The joint between concrete and masonry was weakened and inadequate construction resulted on account of suspension of work during winter. The buttresses collapsed in reservoir full condition only two years after the completion of the structure. The flood released caused extensive destruction downstream. Similar failure occurred to the Colonial and the Portland dams in the United States.

Other reasons like absence of proper supervision over design, construction, operation, and maintenance of dam by trained specialists, improper construction during the war, use of lean mortar and poor masonry, application of ice pressure, failure of power tunnel gates, flood damage to the green masonry, and lastly cracking caused by grouting operations have all been responsible for some dam failures. Such defects though not frequent must be guarded against by designers and construction planners. Ignorance of engineering principles have been behind most causes of failures, hence they can be remedied by imparting an adequate education and training on the actual behaviour and failure of dams to the personnels responsible for supervision and management of reservoirs. As at present, such an education program is quite non-existent.

The Bouzey Dam, in France,(59) 1732 feet long and 66 feet high, failed because of a combination of several items. Its designers had allowed a tension stress of 3000 lbs. per square inch at the upstream face of the dam. While this may have been safe for the cement mortar specified, a lime mortar was substituted; and later examination showed defective workmanship in placing the stone blocks in the mortar. A partial failure, shortly after completion of the dam, gave a warning that all was not well; but went quite unheeded. The cracks were repaired, some additional masonry was placed, but the general structure was not altered. Five years later, in 1895, the dam collapsed completely.

It is unlikely at the present time that any designer would permit tension stresses to occur at the upstream face of a masonry dam; but another lesson to be learned from this failure is, that careful inspection during construction must be maintained to ensure that all specifications are adhered to, and that workmanship is at all times of the highest quality. In addition, a partial failure should immediately require a complete re-examination of the entire design, and certainly no refilling of the reservoir should be permitted until it is quite certain that the causes of the partial failure have been completely removed.

The St. Anthony Falls Dam(110) was a granite masonry dam 366 feet long across the Mississippi River at Minneapolis, built in 1897. Construction schedules were planned on the basis of an estimated date for the spring break-up. An early thaw caused the spring floods to arrive sooner than expected, and the mortar was still green and a planned protective apron had not been constructed. The resulting battering by logs and ice removed several courses of masonry and necessitated rebuilding part of the dam. This failure sounds a warning that ample safety factors should be used in planning of construction dates in conjunction with changes in season and possible high flows, or that auxiliary protection of some kind be provided to guard against unexpected floods.

Faulty operation of dam has at times been the cause of disastrous accidents and failures. It was the floor of the gate chamber in the right diversion tunnel that failed at the Bhakra Dam on August 21, 1959, in India. The failure caused a serious leakage draining out water at the rate of 9000 cusecs. It took more than four months to control the leak by dumping clay upstream. The collapse of the chamber floor actually resulted from the vibrations caused by use of a tunnel gate for flow regulation which is highly improper. In fact, a partially opened gate can thus be very dangerous. Also, penstock vibrations due to water hammer in hydroelectric projects can be of serious consequence. Such a phenomena was responsible for the cracks that developed in the arch structure of the El Fraile dam (1963), in Peru.

The closure of drain-pipe by city officials of a small concrete gravity dam, in Minnesota, resulted in the failure of in all five small dams downstream. Obviously such irrational action on the part of dam operators can be highly catastrophic.

In case of a few dams, unfortunately, no cause of failure has been reported. In fact, for many others the actual cause reported may not be the correct one. The reasons for the incorrect report is not difficult to discern. Only a few failures of the past have been adequately investigated and most of them have been incorrectly interpreted due to lack of knowledge or insufficiency of the data collected. Of late, however, most developed nations have realized the significance of investigating dam failures and have allowed several investigating parties to collect and interpret available pre-and post-disaster information. This is an important shift from the past and has proved an effective way of harnessing knowledge about the safety of dams and reservoirs. However, the increased tendency to frame charges of manslaughter and so on against engineers responsible may have a highly degradable effect on the profession and hence on the safety of dams. Engineers, therefore, must devote sufficient thought to the problem in order to eliminate this state of insecurity which can seriously affect their activities in the interest of the profession and the people.

FAILURE OF COFFERDAMS

There has been some catastrophic failures of cofferdams during construction. Basically there are two types of cofferdams used: (i) earth or rock-fill structures, and (ii) sheeted cofferdams.

An earth fill cofferdam constructed on the Snake River at the Lower Monumental Lock and Dam, in Idaho, failed in 1964 due to overtopping floods, inflicting heavy damage of \$500,000.(112) Another earth-fill cofferdam at the Green Peter dam site, in Oregon, also failed by overtopping in 1964.

The Dayton cofferdam, built in Ohio in 1918, was, however, a reinforced concrete structure. The flood water overtopped the dam, when partially constructed, and undermined the foundation of the completed structure.

It is also noted(113) that several catastrophic failures of cofferdams served as a reason for investigating action of sheeting and distortion produced when driving them into the ground. The study revealed that the tendency for inter-lock distortion depends on: (i) size of the cofferdam, and (ii) density of the soil. The investigation also showed that the model study helps the development of a more realistic design criteria for safety of cofferdams. After all, cofferdams are primarily meant to provide protection to construction works during high floods. Frequent failure of cofferdams would thus be a negation of its fundamental purpose. Obviously, the safety of these temporary structures becomes of prime importance.

TABLE IV

DAM FAILURES IN UNITED STATES (1950-59)

S.N.	Name of Dam	Name of State	Type	Height in Feet	Year of Failure	Causes of Failure
1.	Croton	New York	-	255	1955	Cracks after flood.
2.	Frenchman Creek*	Montana	-	40	1952	Overtopped.
3.	Gallinas River*	New Mexico	-	-	1957	-
4.	Goodrich Creek*	Oregon	-	-	1956	-
5.	Hebgen	Montana	Earth	123	1959	Earthquake subsidence and cracking but no actual failure.
6.	Jackson Bluff*	Florida	Earth	-	1957	-
7.	Madison Canyon	Montana	Earth	197	1959	Earthquake caused mountain slide, created natural dam.
8.	Masterson	Oregon	Earth	60	1951	Piping thru' the fill
9.	Stockton Creek	California	Earth	95	1950	Failed at abutment, probably along contact or crack.
10.	Terrace	Colorado	Earth	168	1957	Leak thru' dam, sloughing.
11.	Wister	Oklahoma	Earth	99	1951	Piping.
12.	Worster (Eaton)	Colorado	Earth	68	1951	Concentrated Seepage.
13.	Yuba*	California	Earth	25	1951	Seepage Slide.

Note: The dams below 15 meters in height are marked by asterisk(*). Eight dams are of 15 meters in height or above while only five are below.

TABLE V

DAM FAILURES IN UNITED STATES (1960-69)

S.N.	Name of Dam	Name of State	Type	Height in Feet	Year of Failure	Causes of Failure
1.	Baldwin Hills	California	Earth	160	1963	Movement beneath foundation
2.	Bynum dam*		-	-	1964	Not known (Diversion dam during construction)
3.	Daguerre Point*	Yuba River	Earth	-	1963	Flood washed out one abutment
4.	Eklutna*	Alaska	Earth	26	1964	Earthquake
5.	Fontenelle dam	Wyoming	Earth	139	1965	Leak through abutment caused washout downstream
6.	Green Peter	Oregon	Gravity (Cofferdam)	341	1964	Overtopped
7.	Hell Hole	California	Rock-fill	410	1964	Overtopped during construction
8.	Latonka*	Pennsylvania	Earth	43	1966	Water piped through embankment under concrete spillway which collapsed
9.	Little Deer Creek	Utah	Earth	86	1963	Water seeped through earthfill and breached
10.	Lower Monumental*	Washington	Gravity & Earth	-	1964	Cofferdam overtopped by flood
11.	Matilija	California	Concrete gravity & arch combined	163	1965	Bad foundation and expansive concrete deterioration
12.	Mayfield	Washington	Earth	-	1965	Erosion below dam
13.	Meadow Lake*	Oregon	Earth & Rock	30	1962	Rain completely washed out
14.	Middletown*	Connecticut	Earth & Stone	35	1961	Leaking dam collapsed
15.	Mill Brook*	Massachussetts	Buttress	40	1966	Crudely patched buttress dam collapsed
16.	Newell Creek	California	Earth & Rock	215	1961	Earth slide during grout curtain
17.	Oxbow	Oregon	Earth & Rockfill	154	1961	Minor damage
18.	Pen Forest	Pennsylvania	Earth	145	1960	Piping - partial failure
19.	Reservoir No. 1*	Utah	Earth	Unknown	1961	Piping along outlet caused wash out
20.	Spaulding Pond*	Connecticut	Earth & Rock	20	1963	Leak under the dam
21.	Steinaker	Utah	Earth	140	1962,1965	Seepage through earthfill and abutment caused sinkhole
22.	Swift (No. 2)	Montana	Earth	189	1964	Overtopped and washed out
23.	Two Medicine*	Montana	Earth	37	1964	Overtopped, washed out
24.	Waco	Texas	Earth	140	1961	Failure during construction due to settlement and slide
25.	West Branch	Ohio	Earth	92	1965	Settlement in foundation due to pore pressure
26.	Wesley E. Seale	Texas	Earth	83	1965	Dam gates damaged
27.	Wheeler	Alabama	Concrete gravity	72	1961	Slide in the foundation

Note: The dams below 15 meters in height are marked by asterisk (*). Remaining sixteen dams have a height of 15 meters or above in this table.

Conclusion

Various failure mechanisms in dams and embankments have been discussed in some detail. It is quite obvious that some mechanisms are well understood and their preventive as well as corrective measures have been found out through research and practical observation of dams. Whereas, there are quite a few failure mechanisms which are difficult to reconstruct or interpret on the basis of data collected by various investigators. They pose complex unsolvable problems in the present state of knowledge. A comprehensive research and investigation program is thus required to tackle the problems identified. Table IV shows the dams that failed in the United States between 1950 and 1959. Table V shows a similar data for the next decade, 1960 to 1969. It is clear that only about 8 large dams failed in 1950-59, whereas almost double that number, that is 16, failed in 1960-69. The total number of dams at the end of each of the two decades is about 2400 and 3500 respectively. Obviously, there has been an increase in the number of dam failures which indicate the need for greater concern and attention to the safety of dams and reservoirs.

In Canada, however, the dam failures have been relatively few. Besides, the documentation of the failures has also been quite unsatisfactory. Smith(114) described the failure of a poorly designed and constructed dam in Canada without mentioning its name and the correct location of the dam. Thus, very little importance is really attached to the safety of dams in this country. This report is, therefore, intended to focus on these problems the attention of Canadian authorities, builders, consultants, engineers and owners concerned with dams and reservoirs. Evidently, the study provides sufficient material for further research and development in this field in Canada or elsewhere.

CHAPTER V

SAFETY OF RESERVOIRS

Reservoir failures are discussed in detail in the preceding pages. However, it can be shown that the frequency of reservoir failure is quite small compared to the frequency of reservoir construction. Schnitter(1)* demonstrates this on the basis of data on dams of more than 15 meters in height built in Western Europe and the United States (See Table I). From the statistics it is clear that during 1900 to 1909 about 9 per cent of dams in the United States failed, when the total number of such dams was 100. Since then the safety records improved considerably over the years and between 1950 and 1959 the rate of dam failures reduced to about 0.4%. The improving safety records indicate that efforts toward reservoir safety have been quite a success.

It was also noted that the total loss of lives from dam failures between 1900 and 1909 was around 100; while between 1920 and 1929 it increased to 1,010; and again between 1950 and 1959 it reduced to about 570. The figures show that the risk to life and property from failure of dams and reservoirs is still quite high. Interestingly enough, some attention has been paid of late to this aspect of dam safety, and the concern expressed in the various submissions to the Tenth International Commission on Large Dams (1970) at Montreal is an important step in the evolution of comprehensive safety measures for design, construction and maintenance of reservoirs.

It may be quite relevant to ask at this stage: what is really

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*See references at the end of the report

TABLE I

FAILURES OF DAMS OF MORE THAN 15M HEIGHT BUILT IN WESTERN EUROPE AND THE USA SINCE 1900 (ONLY FAILURES LEADING TO STORAGE RELEASES, BUT EXCLUDING ACTS OF WAR)

YEAR OF COMPLETION	TOTAL NO. OF DAMS	FAILED DAMS		NAME OF FAILED DAM (YEAR OF FAILURE)	DEATHS (FAILURES)(2
		NUMBER	PERCENTAGE		
1900-1909	190/100(3)	9/9(3)	4.7/9.0(3)	Scottdale(1904), Hauser(1908), Zuni(1909) Jumbo West(1910), Austin(1911), Hatchtown (1914), Sepulveda(1914), Long Tom(1916) Lake Toxaway(1916)	100 (6)
1910-1919	280/220	12/12	4.3/5.5	Stony River(1914), Horse Creek(1914), Hebron (1914 and 1942), Lyman(1915), Plattsburg (1916), Mammoth(1917), Schaeffer(1921), Bully Creek(1925), Wagner(1938), Sinker Creek(1943) Swift(1964)	10 (3)
1920-1929	430/280	8/6	1.9/2.1	Apishapa(1923), Gleno(1923), Mayie(1925) Lake Lanier(1926), Diandi(1926), St. Francis (1928), Balsam(1929), Sella Zerbino(1935)	1,010 (5)
1930-1939	450/280	1/1	.2/.4	La Fruta(1930)	0 (1)
1940-1949	390/240	0	0	None	0
1950-1959	960/530	4/2	.4/.4	Stockton Creek(1950), Vega de Tera (1959), Malpasset(1959), Baldwin Hills(1963)	570 (3)
Total (60 years)	2,700/1,650(4)	34/30	1.3/1.8	(23 earth and 11 concrete dams)	1,690 (18)

NOTES:
(1) Excluding Scandinavia
(2) No. of failures for which data are available
(3) Second figures apply to USA alone
(4) Among which 1,260/1,040 earth dams
(5) 410 deaths in 14 failures in USA alone

comprehensive reservoir safety? According to Huggenberger(2) "the question of overall safety does not only relate to the construction itself but must also include part of the storage basin and its shores". Therefore, in addition to the structural safety of the dam and the foundation, safety of the reservoir environment(3) is also a major concern in the evaluation of the comprehensive safety. A recent commentator(4) has found ample evidence to the effect that the branch of engineering science relating to dams put as their primary objective the overall safety not only of the structure but also of the public directly affected by the existence of the dams and reservoirs. Clearly, a comprehensive safety approach must ensure:

(i) safety and stability of dams, embankments and reservoir bottom;

(ii) safety of reservoir environment; and

(iii) safety of life and property potentially threatened.

What is required is a balanced optimization of these factors with consequent maximization of benefits. This will imply reduction of the frequency of failure as well as the total resulting damage to a minimum possible value. This concept has its origin in the following facts:

(i) In the present state-of-the-art complete elimination of reservoir failure is quite impossible;

(ii) So long as the reservoirs are subject to a calculated risk of failure(5) it is perfectly justified to minimize the disaster potential (the total expected capacity for destruction of life and property in the event of failure) of all reservoirs within reasonable economic limits. This approach has seldom been used by the reservoir designers to optimize safety.

(iii) Safety and economy in reservoir design have conflicting objectives. Absolute safety would require unlimited costs to satisfy various requirements, while absolute economy entails a very small but dangerous investment. Probably the most feasible method for a balanced design of the conflict system is to be found in Systems Analysis.

Owing to the aforesaid realization, a comprehensive reservoir safety approach would constitute a broad-based provision of safety alternatives that are enumerated below.

Basically there are two aspects of safety: (1) Engineering-Economic aspects, and (2) Human dimensions. It should be noted that use of technologic solutions without regard to the changing pattern of human requirements could be of little use. At times such application could be highly disastrous.

The engineering-economic approach provides corrective safety measures that should minimize failures within economic limits. These measures are:

(i) Supervision of planning, design, construction, operation and maintenance of all primary structures, appurtenant works and all environmental factors related to safety by a specialist or a team of specialists,

(ii) Establishment and revision of rules for supervision and outlining a sound procedure for their effective application,

(iii) Study and investigation of dam failures to understand the behaviour of dams in extreme situations and thus improve upon the subjective (or judgement) and objective aspects of safety supervision, and

(iv) Study of dams under construction and operation in order to diffuse information beneficial to future activities concerning similar structures.

(v) Mission-oriented research in various disciplines to tackle specific problems of dam-safety or creation of a new field of endeavour (to be called Dam-Safety Engineering) to expedite development of the requisite tools for effective supervision of design, construction and operation of dams.

Study of human dimensions of dam-safety reveal that some preventive measures are very necessary to limit the loss of life and property in the event of a dam-disaster. Substantial socio-economic benefits are expected from the operation of some of these measures. These are:

(i) Regulation of potential disaster plain by zoning;

(ii) Protection of such area by secondary structures;

(iii) Use of disaster warning and alarm systems;

(iv) Control of socio-economic subsistenance pattern and rehabilitation in the potential disaster area by use of mass media;

(v) Effective use and refinement of the above mentioned corrective and protective resources through adaptive research and investigation of new technology;

(vi) Instituting reservoir safety insurance to provide protection against risk of disaster to life and property and to create a system for discouraging incompatible economic growth in the potential disaster area and yet making an optimum use of the resources of the region.

(vii) Legislation for effective public control and efficient use of all available resources for safety of reservoirs. Laws must require strigent supervision and application of rules without really discouraging technologic innovation. The priorities in the human dimensions of safety should be made mandatory for application to the reservoirs.

Various technologic aspects of safety supervision in reservoir engineering will be discussed in detail in this chapter. Safety regulation of potential disaster plain from the standpoint of human implications will be discussed in some detail in the succeeding chapter. A balanced consideration of all factors is the prime objective of comprehensive safety approach in dams. The whole idea is relatively quite new to the existing institutions charged with the administration of reservoir safety. Thus,we are often confronted with complex economic difficulties and lack of public support which prevent the implementation of a suitable comprehensive safety program. Lack of knowledge is probably the most important reason for the continuance of this obstacle.

SUPERVISION OF DAMS AND RESERVOIRS

Supervision of dams and reservoirs is an important question. It is carried out in the various phases of activities like planning, design, construction, operation and maintenance of these water retaining artificial and natural structures.

Most specialists believe that, statistically speaking, it is quite desirable to have a higher safety factor for structures when developing a new technique. This is often ignored and result in the failure of dams. Yet the temptations of economy continue to dictate such forgetfulness. Many dam failures classified as poor design can be put in this catagory. It is thus a necessary criteria for supervision of dams and reservoirs.

The general behaviour of dams and embankments is an important knowledge for supervision. Takase(6) shows that the factor of safety of earth dams tends to increase with time if undisturbed in the early stages of its life-span. Davies,(7) however, points out that this increase is only for a short period of time and after the factor of safety has reached a peak value, there is a gradual decay of the embankment leading to ultimate termination of the life-span. Similar behaviour has also been noted with masonry and concrete dams. All activities in planning, construction, operation of dams must take account of this characteristic if safety of the structure is to be ensured.

Supervision encompasses a general overview of work meditated or done, collection of data and information, recording pertinent observations, analytical assessment and critical review of every possible safety aspect of the design, construction and operation. The task requires an effective organizational structure and proceedure for action, suitable documentation and record-keeping methodology and development of analytical tools for accomplishment of the desired objectives.

Supervision Organizations and Procedures.

Dickerson(4) briefly reviews the organization structure and procedure for supervision of dams and reservoirs in some of leading countries throughout the world. He shows that in most countries some measure of state control on safety of reservoirs is well-recognized. Yet each of these systems vary considerably in their actual approach.

TABLE II

SOME EXISTING DAM SAFETY REGULATIONS

S.N.	Country	Year	Name of the Regulation
1	France	1896	Law of 8 April 1896 for Civil Works.
2		1919	Law of 16 October 1919 covering Works for the Utilization of Hydraulic Energy.
3		1925	Circular of 21 December 1925, Ministry of Agriculture.
4		1927	Interministerial Circular of 20 July 1927.
5		1960	Decree no. 60-619 of 20 June 1960.
6		1966	Decree of 13 June 1966.
7	Federal Republic of Germany	1965	Deutsche Normen, DIN 19,700 Stauanlagen, November 1965.
8	Italy	1965	Regolamento per la Compilazione dei Progretti, la Costruzione e l'Esercizio delle Dighe di Rittenuta, ottobre 1965.
9	Japan	1964	River Law.
10		1965	River Law Enforcement Ordinance.
11		1965	River Law Enforcement Regulations.
12		1964	Electric Utility Law.
13		1965	Electric Utility Law Enforcement Regulations.
14	Norway	1917	Preliminary Law on Concession.
15		1959	Preliminary Law on Concession.
16		1940	Law on River Basin.
17	Spain	1967	Instruccion para proyecto, construccion y explotacion de grandes presas. Ministerio de Obras Publicas. Direccion General de Obras Hidraulicas.
18	Switzerland	1953	Loi fédérale complétant celle qui concerne la police des eaux.
19		1957	Reglement d'exécution de l'article 3bis de la loi concernant la police des eaux, du 22 juin 1877.

TABLE II (Continued)

S.N.	Country	Year	Name of the Regulation
20	Union of Soviet Socialist Republics, Poland, Czechoslovakia.	1957	Instruction of Determining Maximum Flood Discharge. Values for Design of Hydraulic Structures.
21		1961	Instructions on Topographic Survey for Hydraulic Structures.
22		1962	State Building Codes and Regulations.
23		1962	Building Codes and Specifications. on calculation of foundation of hydraulic strucutres; on design of earth fill and rock fill dams; on design of hydraulic fill dams; on design of concrete dams on rock foundation; on design of concrete and reinforced concrete dams on soft soils.
24	United Kingdom	1930	Reservoirs (Safety Provisions) Act.
25		1966	Report on Reservoir Safety (Ad hoc Committee Report, Institution of Civil Engineers, London).
26	United States of America		Federal Power Commission (with authority over licensed power projects only).
27		1963	Federal Power Act (revised).
28		1964	Regulations under the Federal Power Act.
29		1965	Order no. 315.
30		1965	License Form L-6
31		1966	State of California Statutes and Regulations pertaining to Supervision of Dams and Reservoirs in California.
32		1970	Model Law on Dam Sefety (U.S. Task Committee on Large Dams).
33	UNESCO	1966	Recommendation for Reservoirs

Note: - Copies of these documents are available from UNESCO for purposes of research and review of national regulations on dam safety. Photocopies can also be had from the personal library of Mr. E. Gruner, Consulting Engineer, Basle, Switzerland. (Personal Communication, 1970).

Hardly any comparative study is available to assess the relative merits of each of these systems. Thus every country or state must evaluate for itself the arrangments that best suit its objectives and requirements.

Table II shows a list of various regulations that have been developed in some leading nations who enjoy an advanced status in dam building technology. These regulations could be very useful in developing a new set of regulations adapted to a country which does not possess one for its own use. A high degree of convergence between the regulations and the organization structure is essential for effective task accomplishment.

In the United States various federal organizations such as the Federal Power Commission, the United States Bureau of Reclamation, the Tennessee Valley Authority, the Corps of Engineers, and the Department of Agriculture have all quite well developed programs for safety supervision of reservoirs that fall within their jurisdiction. Most states in the United States have their own institutional arrangement to exercise safety over dams built within their purview. An excellent state program in dam-safety is administered by the Division of Dam Safety in the State of California's Resources Agency.

The activities of Federal Power Commission, vested with necessary statutory authority to prescribe the enforce standards of safety for construction, operation and maintenance of dams and appurtenant structures, have been described in detail by Thomas et al(6). The program includes project review and inspection which begins long before any construction is started and continues throughout the life of the project. The various steps in this procedure are:

(1) Issue of preliminary permit,

(2) Review of new project,

(3) Review of existing project,

(4) Inspection during construction:

  (a) Vigilance over permit-holders program,

  (b) Collaboration with other state and Federal agencies for joint action,

  (c) Hiring of consultants for safety study during construction,

(5) Inspection and review of operating projects, and

(6) Inspection by independent consultants every five years.

Interestingly enough, the inspection-in-depth program of the Federal Power Commission has enhanced the assurance of safe operation of major structures by continued surveillance and more exacting performance testing through instrumentation.

Among the states, a remarkable program in dam safety is administered by the Resources Agency of California through its Division of Dam Safety. The activities of the predecessor division, since the enactment of legislation in 1929 after the failure of the St. Francis Dam, have produced excellent results and safety traditions in the State(3). As a result of the catastrophic failure of the Baldwin Hills Dam, in California, and other spectacular foreign failures like Malpasset, Vaiont, Veg De Terra, etc., more stringent regulations were enacted with regard to safety measures and classification of structures, subject to the jurisdiction of the state supervision(9). Special emphasis on the protection of life and property due to failure and/or release of water from dams and reservoirs has been envisaged in the powers(10) delegated to the department. The statutes are constantly reviewed and modified in light of experiences gained through continuous comprehensive evaluation of dam-safety. Cortright(11) documents some interesting case studies on the investigation and analyses of deteriorating dams. Taylor(9) has also done a similar thing. In fact, Cortright goes further to show that this procedure of safety assessment followed at the Division of Dam Safety is responsible for the development of many new powerful analytical and computational tools which could ultimately be useful to the designers of dams.

Generally speaking, most organizations entrusted with the task of vigilance over reservoirs carry out two types of inspection for the purpose of safety. They are:

(1) Continuous inspection and supervision

(2) Major inspection and supervision

It would be seen that both types of supervision are equally important for safety. It is not possible to offset one by the other as they are highly complementary in nature.

Continuous Inspection and Supervision

This type of continual vigilance by the site engineer, maintenance supervisors, and trained personnel in the employment of the owners and operators of the reservoir is found quite effective for ensuring safety. The procedure is followed in almost all countries and is found useful in keeping a close and constant watch on the reservoir thus enabling detection of serious problems at earlier stages when remedial measures could be less expensive.

Many problems involving safety of dams require a long period of observation before a definite specialist opinion can be sought on its safe operation. A continuous inspection and supervision of dams is thus very useful in collecting all relevant data required for a specialist study if ever determined necessary. In fact, it is only through constant vigilance that need for additional study can be identified. Problems like critical foundation condition, embankment stability, reservoir siltation, underwater deterioration etc. all require special study by consultants, but their need can be determined only by the continuous observation and analysis of collected data.

Another interesting natural phenomena has been reported by various observers. A recent report is due to Murti(12), who shows that climatic changes may occur on account of the creation of a large artificial reservoir. A climatic change may lead to changes in the meteorologic characteristics and hence the precipitation pattern in the catchment area. Thus, such observation may require a complete hydrologic redesign of the reservoir. It is thus obvious that constant observation and inspection of dams and reservoirs is highly desirable.

Major Inspection and Supervision

In addition to the continuous supervision and inspection of reservoirs, it is necessary that highly experienced engineers and specialists review the routine inspections at certain intervals in order to assess any extraordinary safety needs. Also a qualified and independent engineer is required to inspect the reservoir at intervals varying between 2 to 10 years. In the United Kingdom, Reservoir (Safety Provisions) Act 1930 specifys a thorough inspection by an approved engineer every 10 years; whereas the United States Federal Power Commission lays down added security as an advantage for inspection by independent consultants every 5-years over and above the routine

TABLE III

LIST OF OBSERVATIONS ON DAMS IN OPERATION

(1) *Hydrologic and meteorologic measurements*

(1) Rainfall
(2) Duration of sunshine
(3) Air temperature
(4) Reservoir volume
(5) Gauge for head flow
(6) Gauge for tail flow
(7) Reservoir water temperature
(8) Tail flow temperature
(9) Chemical and biological water quality

(2) *Control by visual instruments*

(1) Visual observation and inspection
(2) Trigonometrical measurements
(3) Precision levelling
(4) Hydrostatic levelling
(5) Telescope gauge
(6) Radiosonde gauge
(7) Embankment pressure
(8) Percolating water from bituminous concrete facing
(9) Percolating water in inspection gallery
(10) Underground water level in inspection gallery
(11) Underground water level downstream dam
(12) Uplift pressures
(13) Temperature inside the gravity dam
(14) Groundwater level in control wells in the neighbourhood of reservoir
(15) Discharge of neighbourhood creeks
(16) Stability of Valley Slopes

(3) *Functional control of operating installations*

(1) Testing safety valves of bottom outlets, penstocks and other outlets
(2) Testing seepage water pumps
(3) Automatic emergency doors in control galleries are tested.

inspections; besides the Swedish practice(13) justifys three to four years as a reasonable interval and the State Supervision Branch in Norway(14) specifies two to three years. It is, however, generally agreed that five years is a reasonably safe interval and maybe adequate, unless special and unfavourable conditions require an earlier inspection.

Suggestions for Effective Supervision

Examination of the engineering-economic approach to corrective safety measures in reservoirs found that supervision was a very broad area of endeavour and, in fact, many technical factors, identified, eventually contributed to the improvement of supervision. The specialist supervision, rules for supervision, study of failures, study of operating dams, mission-oriented research and all contributed to the improvement of supervision which in turn assured a greater safety for these structures. An interesting monogram by Pugsley(31) suggests several items of caution. A comprehensive listing of measurements and investigations, to be routinely carried out for functional control of operating installations in order to ensure the safety of dams and reservoirs, is presented by Koniег and Waelter(25) and reproduced in Table III. In actuality, a large volume of useful material can be derived from the literature surveyed in this study in order to construct an effective model for dam safety supervision and control.

It is thus evident from this brief review of literature on dam safety, that in the investigation of dam disasters, failures, accidents and deterioration lies the elixir for never failing dams. Governments must wake up to their responsibility to support research in the field of dam-safety. There is a great dearth of published material on failure of dams; and this lack of information is responsible for repetition of replicate failures of dams. Much of the safety provisions, that developed in the past, have their origin in some dam-failures. Some provisions were developed only after several failures had rendered any further loss quite intolerable. Thus the whole approach has been more or less quite a negative one.

A positive approach to the problems of dam-safety would be to study present and past failures to identify the true behaviour of dams. Investigation of dams under construction and operation would provide the data on the existing practices and standards. Study of existing status of dam-safety in the framework of past failures may provide the necessary tools of prediction of dam behaviour. There has been very little effort to couple these two important fields of endeavour, and hence the lag in the real growth of a dam safety science. Obviously, concerted efforts to improve the state of affairs in dam building is possible only by creation of a new science of "Dam-Safety Engineering" which may fulfill the task that remains so far ignored.

CHAPTER VI

RESERVOIR SAFETY: HUMAN DIMENSIONS

Introduction

Dams are built to serve the needs of man and his environment that together comprise the human society. In order to build dams and reservoirs beneficial to the growth of the society, it is necessary to identify the sociological priorities and characteristics that influences safe design of structures. An approach to safety cannot be merely economic, it should be subjected to socio-economic analyses in order to rationalise and improve the present criteria for optimal design. It is anticipated that this approach will evolve design criteria which might considerably reduce the failure probability by new design innovations. It may also reduce the existing social antagonism towards dam-failures.

It is rather unfortunate that the dam builders have rarely recognized the importance of this approach. Oftentimes they are too overwhelmed by economic factors and technological handicaps that influence the process of decision-making today. A brief discussion of the behavioural aspects of dam-disasters should be of considerable interest to all dam designers.

Failure of dams and reservoirs, which cause loss of lives and damage to properties invariably attract public attention.(1) Hence, it motivates the government to take advance safety measures and support research projects to investigate corrective and protective action.(2) Table I shows the estimated cost of property damages resulting from a few select dam failures. The dam disasters involving loss of private and public properties only concern the owner of the dam and the parties damaged. They are of serious public concern when the owner has no means to compensate the losses. Losses due to dam failures, as shown in Table I, have proved to be a substantial blow to the highly developed nations, and could well prove disastrous for the developing nations.

TABLE I

ESTIMATED DAMAGES OF SOME DAM DISASTERS

Dam	River	Country	Year Failed	Damages in Million $US
Mill River	Mill	U.S.A.	1874	1.0
Lynde Brook	Lynde Brook	U.S.A.	1876	1.0
Johnstown	Little Conemaugh	U.S.A.	1889	100.0
Brokaw 2	Wisconsin	U.S.A.	1938	0.7
Malpasset	Le Reyan	France	1959	68.0
Bab-i-yar	Dneiper	U.S.S.R.	1961	4.0
Baldwin Hills	Owens	U.S.A.	1963	50.0
Mayfield	Cowhitz	U.S.A.	1965	2.5
Wyoming	Sybille Creek	U.S.A.	1969	1.5
Pardo	Seco de Frias	Argentina	1970	20.0

Loss of lives due to some select major dam failures is shown in Table II. The catastrophic loss of lives and damages to properties become a direct concern to the public, and hence to the government. The role of the government in overall supervision of reservoir safety has therefore been well recognized in the modern times.

Water Resources and Dam-disasters

Water Resource development, through construction of dams, dykes, embankments, and reservoirs is probably as old as civilization itself. The full impact of this development upon human culture from early times to the present day is yet to be well understood.(3) Far more unsatisfactory, however, is our understanding of the socio-cultural consequences and implications of dam disasters. While water has been worshipped as god, it has also been hated as an evil force whenever it unleashed forces of devastation and destruction.

On an average, major dam disasters have occurred once or twice a year somewhere in the world.(4) In 1963, however, the number of major dam disasters exceeded two, and the highest casualty of 3,000 dead were recorded at the Vaiont.

The advantages of an adequate warning system with prior planning of rescue, evacuation and other post-disaster operations was clearly demonstrated by the Baldwin Hills disaster in Los Angeles. Only three lives were lost even though the total property damage approached $50 million. The Oros dam disaster is also reminiscent of high death tolls with comparatively much less property damage. The reasons for this can be found in the inadequacy and unsatisfactory nature of the warning, evacuation, and rescue systems. At Vaiont, the telecommunication lines were destroyed by the impact of overtopping floods. Unfortunately, it was not foreseen by the authorities, and, hence, the first warning came only

TABLE II

DEATH CASUALTIES IN SOME MAJOR DAM FAILURES

Dam	Country	Date of Disaster	Loss of Lives
Baldwin Hills	California	December 14, 1963	3
South Fork (Johnstown)	Pennsylvania	May 31, 1889	2,200
Saint Francis	California	March 13, 1929	450
Veg De Terra	Spain	January 10, 1959	144
Malpasset	France	December, 1959	421
Oros	Brazil	March 25, 1960	1,000
Babii Yar	U.S.S.R.	March, 1961	145
Hyokiri	Korea	July, 1961	250
Quebrada la Chapa	Colombia	April, 1963	250
Vaiont	Italy	October 9, 1963	3,000
Khadakwasla	India	July 12, 1961	400
Pardo	Argentina	January 6, 1970	25
Puentes	Spain	April 30, 1802	600

after the flood water had travelled a few miles downstream to be noticed by a policeman. Other warning signals, for example, minor earth slides, uneasiness of some local animals, etc., were, however, ignored by the authorities. They were quite satisfied with their own engineering analyses that made them confident about the stability of the earth mass which actually slid with catastrophic consequences. At Oros, on the other hand, advance warnings were issued which enabled large numbers of people to escape.(5) Even though approximately sixteen thousand persons escaped, the limiting number of casualties were about 1,000. It therefore showed that the design and operation of the warning and related systems left much to be desired.

Garb and Evelyn Eng(4) in their disaster handbook draw significant conclusions from a comprehensive study of different kinds of disasters including dam failures. They suggest:

(i) studies of major disasters reveal that about 95% of the casualties could have been prevented;

(ii) effective disaster operations depend on how well each individual and organization do their own job, as well as allow others to do their's;

(iii) very little opportunity is available to test, improve and innovate methods of handling disaster problems.

Little, if any, attention has been paid by engineers responsible for design, construction and operation of dams to devise means to meet the risk they create. Our knowledge of casualty prevention vis-à-vis dam-disaster is highly inadequate. There is an acute scarcity of empirical data and research effort on the individual and organizational behavioural systems during these disasters. And this overwhelming neglect of research in human dimensions of reservoir safety has eventually offered hardly any opportunity to tackle problems relating to dam-safety and dam-disasters.

Saventhem and Muller(17) cite interesting statistics to emphasize the magnitude of losses due to dam-disasters. Well over 50 dams and embankments have failed in the last 20 years and the resultant loss of lives have been more than 5000 and the estimated aggregate damage to property ranged between $250 million to $750 million. With more than 10,000 dams presently in operation, Saventhem and Muller suggest, that comprehensive insurance for dams enters the realm of economic feasibility as soon as it is seen in terms of a 'common need'. The purpose of the insurance is to spread the cost of a single dam failure over the whole community of dam owners. Yet there has been little headway to provide such a disaster compensation program in reality. On the contrary, an ICOLD Committee, set up to investigate the dam safety insurance proposals, have been suspended (1970) on account of lack of funds.(10)

Community vs. Dam Disasters

It has been said that disasters provide the context in which significant human dramas are revealed.(13) Yutzy considers disasters a collective phenomena that upset patterned social arrangements of communities.(14) It has been found that most communities organize themselves around traditional functions such as participation in production-distribution-consumption, socialization, social participation, social control and mutual support that are relevant to living in a particular location. Disaster threatens the achievement of valued ends of these functions.(15) The adaptive response pattern called "the emergency social system" swings into action with new set of priorities on account of (1) high demand for certain goods and services, and (2) incapability of the affected community to produce all or some of the good and services. Thus a typical ordered set of priorities evolve as the organizational goal during disaster emergency:

(1) preservation of life;

(2) restoration and maintenance of essential services (transportation and communication for disaster operations);

(3) maintenance of public order (safeguard of property and control of crowds);

(4) community use of all property; and

(5) maintenance of community morale through mass media, and emergency communication systems.

The organization of this priority system is however dependent on the two characteristics: (a) the disaster agent, and (b) the community.

There is little doubt that the communities which go through prior planning and preparation for disaster will develop an effective emergency action more rapidly than others. Yet research and study seeking relationship between these variables in catastrophic reservoir disasters have long remained a neglected science.

The disorganizing effects of disasters are well-known. Dynes,(16) however, explains how disaster tends to organize a community. The organizing effects of the dam-disasters have been observed, and as a result increased attention has been directed to reservoir safety supervision of late. But yet little, if any, research has been carried out on warning alarm and rescue operation systems. Finding an economically feasible method of cost sharing for major reservoir disasters, supporting safety research for dams, and enforcing powerful institutional constraints, which would prevent building and operation of unsafe dams remain as yet an unaccomplished task on account of many social, cultural and human obstacles.

It is therefore astonishing to note that though serious and much dreaded consequences of reservoir-disaster were recognized long before the modern era, social science research heretofore been quite unknown and ignored in this field. Only studies of preliminary nature such as those of Vaiont and Baldwin Hills disasters have come only recently from the Ohio Disaster Research Centre.(7) More scientific and quantitative studies of human dimensions are thus highly desirable and must be taken up soon.

Complete absence of public participation in the choice and selection of safety criteria for reservoirs may have contributed to this anomalous state of affairs. No suitable political system or institutional arrangement exist to rectify the situation. Also the International Commission on Large Dams have made some efforts to establish a common international standard for evaluation of dam failures and accidents and thereby establish criteria for comprehensive evaluation of dam-safety.(8) Unfortunately, due to diverse natural, social and economic background of different countries, it may not be possible to achieve an appreciable measure of success in a short time.(9) However, the remarks of Dickerson is interesting to note:

> "It will be evident ..... that all engineers concerned with the branch of engineering science relating to dams put as their primary objective the overall safety not only of the structure but also of the public directly affected by the existence of the dam and reservoir."(10)

Dickerson also suggests that some form of control at a governmental level should be universal for dams and reservoirs. It is, therefore, explicit that an effective public participation for controlling reservoir safety would be a great leap forward in the realization of a comprehensive reservoir safety approach.

Education of public opinion is thus quite important for an effective participation to support essential safety measures. A safety-conscious public will not only demand a safe reservoir but also will be prepared to pay for the measures that would strengthen safety. With the public support, it is quite likely that the present criteria of safety and economy of reservoirs may substantially change, giving rise to new needs in planning and supervision of design, construction and operation of dams and reservoirs. This aspect has seldom been realized by those responsible for reservoir management, and much less by the policy makers and legislators entrusted with decision-making. It is rather unfortunate that engineers in this field have not responded sufficiently to the public needs.

Some Examples of Public Reaction

Following the disaster of the Veg de Tera , a Spanish dam, on January 1959, civil and criminal proceedings were initiated against ten engineers, and probably four of them were convicted. Whereas this might have been a surprise to many,(19) it can hardly be denied that public abhorrence of the event was largely responsible for such an extreme measure. Later, in the same year Malpasset dam-disaster led the authorities to search for a scapegoat to satisfy public wrath. The Chief Engineer, Dargeous, was charged for homicide and injuries through negligence in investigation, construction and operation of the dam, though the court found him not guilty.(29) The verdict, however, does not clear the designer Andre Coyne and his office of the responsibility for the disaster simply because the dam was built under their directions. Despite a serious difference of opinion among experts about conflicting failure hypothesis (one considering failure to be unforeseeable and the other as a result of sheer negligence), the community response to the whole affair was one of extreme abhorrence and disappointment. Similar public reaction was voiced after the Vaiont and Baldwin Hills disasters. Yet there has hardly been any study whatsoever to assess the full implications of the changing community response to the dam-disasters. Improvement in the safety standards so far made it the result of arbitrary subjective appreciation of the public concern. Unless a definitive study is made, it would be difficult to initiate a comprehensive safety approach in dam-safety commensurate with the public needs and desires.

It has been pointed out by some consultants that their clientele do not respond favourably to improved safety standards, particularly when cost considerations show striking differences. Some of them are led to believe that active pursuit of the cause of safety cannot take place without a concommitant loss of clientele. The truth underlying these claims cannot be entirely ignored. Sherard et al(20) have also discussed this difficulty and finds it quite necessary to emphasize the optimal safety measures in dams for the benefit of clients at all design, construction and operational phases. Evidently, a definitive study of the problem is urgently called for. An assessment of consultant-client relationship vis-à-vis safety requirements, and adaptation of the consultant market to the changing and improving safety criteria as a result of technical development and public demand for safety should, therefore, be a useful mission-oriented research.

Existing Discrepancies

Unfortunately none of the aforementioned studies have so far been undertaken. This has clearly demonstrated the claim that much remains to be done to study and adapt the technical developments in dam-safety to the public demand.

Whereas the cost implications of improved safety provisions are clear, the benefits are difficult to calculate. The data available is not sufficient for a reliable assessment of reduction in the frequency of dam-failures. Also in the absence of sufficient data and techniques of quantification for the socio-cultural impacts of dam-disasters, these benefits are considered secondary in benefit-cost terms. Nevertheless it must be remembered that in human terms they are primary and direct. The reservoir safety should therefore be controlled in a manner that significant regional populations "are not left overwhelmed and disenchanted". Public participation in safety decision-making and in sharing of benefits and costs will surely improve the present not so logical situation. Besides, ignoring of probable disaster costs, however difficult to assess, in the benefit-cost analysis of reservoir projects is not a satisfactory practice. The seriousness of these costs have seldom been realized due to lack of efforts to collect relevant data and assess the probable disaster costs. There is little doubt that their assessment will provide a novel approach for seeking more acceptable solution to problems of reservoir-safety. Little, if any, research has been directed to this end. The purpose of this research is to point out the need for such investigations during the planning phase of reservoir projects.

It is true that planning phase frequently takes years and is not always undertaken for purely economic reasons.(11) Reservoir safety investigation and research with special emphasis on socio-economic aspects must form an important part of the planning phase. Thereafter the management should continue to evaluate these factors during subsequent construction and operation, phases. It must be realized that the resulting economic and cultural growth, initiated by the creation of a large reservoir, is likely to increase the demand for higher standards of safety. Reservoir managers must recognize this need and act accordingly in all phases of their activities. The dynamic nature of the economy calls for a dynamic safety model as a framework for the establishment and review of the safety standards for the reservoirs. The lack of recognition of this dynamic nature is responsible for the phase lag between demand and supply; and this may explain the cause of the existing discrepancy.

Public Perception of Dam Safety

It can be emphatically stated that the reservoir managers have made almost no efforts to gain insight into the domain of public perception of reservoir safety and the social consequences of dam-disasters. No relationship is yet known between public perception and the risk in the light of technical measures for its effective minimization. Dearth of systematic study in this field is a real reason for the absence of suitable design criteria of safety, compatible with technical knowledge and public perception. Evaluation of social factors related to the safety of reservoirs is urgently needed for optimum reservoir design, which must be based on problem perception and consequent public demand for a remedy. These in turn would depend substantially upon the efforts of the dam managers and experts to educate public opinion,(21) which has already shown grave concern for reservoir disasters.

The primary purpose, therefore, is to muster public support for a program that would minimize the risk of reservoir failure and its adverse consequences. A successful operation of such a scheme is likely to have a feed-back effect thereby further advncing the overall safety program. The complete lack of public participation in this field so far has been due to the past authoritarian decisions that jeopardised safety for achieving what may be called an irrational economy.

Ibsen and Ballweg have focussed attention on specific methodologies to study public perception of water problems.(12) The risk and occurrence of reservoir disasters is a very important water management problem. Slow realization of its true significance by the engineers and, complete neglect of it by the social scientists, have so far rendered reservoir safety measures lop-sided and ineffective. Research on socio-cultural systems and interest group strategy vis-à-vis reservoir safety is expected to have considerable influence on the development of realistic technical criteria for safety and economy. Thus the change of approach would certainly be to the advantage of dam builders, owners and the public; and shall at least eliminate disasters which are on account of irrational economy the like of which occurred at Schoelkoff. It will also help to eliminate the public distrust and professional humiliation that resulted from failure of the Baldwin Hills, Khadakwasla, Malpasset, Vaiont and Veg de Tera and other dams.

Some available disaster information on a few dams that failed in the past indicates the value of such data to engineers committed to safe management of such structures. Much remains to be done towards a thorough investigation of the dam disasters as such, however.

Rayner(6) prepared a bibliography of about eighty publications that attempts to conceptualize or formulate disaster models or theories. These models could be utilized to evolve a systematic logical framework for rational analysis, design, and subsequent alteration of the reservoir safety criteria. As no inventory of empirical data on dam-disasters is available, it would be worth while to utilize relevant experiences from other types of disasters to reinforce the information already gathered. Due to the absence of any comparative study and research based on different types of disasters, it is, however, difficult to determine as to how effective such knowledge would be in the design and operation of reservoir systems so as to minimize human sufferings and also maximize economic benefits. A socially acceptable solution to these problems is, therefore, a matter of great urgency.

It is, therefore, necessary to initiate programs to determine public concern for reservoir disasters in an effort to define a solution that would merit the best public support. However, the truth is that the best public support is a highly controversial criterion and is a relative term. The safety criteria would therefore certainly differ with location, time, cultural status, and other pertinent variables. A more comprehensive study of the problem is possible only if considerable field data are collected, and empirical studies are made to increase our understanding of the problem.

Dynes(18) also suggests a comparative study of community response to stresses like flood disasters and water pollution in order to identify priorities, perceptions and structural adaptations in differing stress situations and community structures. It would also be necessary to assess the value of such investigations in the field of dam-disasters, wherein disaster-planning is totally non-existent.

Safety Hypotheses

In the present state-of-the-art, dams are liable to fail. While many dams have failed with adequate advance warning and little damage or loss, some have suffered sudden and unexpected disasters with considerable loss of life and property. Observations of this nature have inspired considerable thinking by experts and researchers on the basic philosophy of reservoir failures.

One view aptly propounded by Gruner and advocated by Eberhardt,(41) is best expressed in the following quotation:

> "Every dam which impounds water presents a potential danger which should neither be under- nor over-estimated. The risk of a sudden disaster is forever inescapable, and while knowledge and vigilance may reduce such a risk, it can never be entirely banished. All man-made works carry within them the seeds of their own decay and eventual destruction."(42)

The hypothesis is probably an extension of a broader concept of "force majeur", the act of God, which conceives all earthly objects as perishable. It is a derivative of fatalism, which perceives the final goal of all earthly phenomena as "decay and eventual destruction." This phenomenon may be considered an integral part of an overall process of "change" that is known to occur throughout the universe.

The hypothesis that "a sudden disaster if forever inescapable"(42) is also a probabilistic concept based on the past observations. However, the term "sudden", applicable to certain events in the present-state-of-knowledge, may not be valid for a similar occurrence with the advancement and improvement in the methods of observation and control of dams.

In a Water Power commentary on Gruner's paper to the Institution of Civil Engineers, London in 1963 it was stated:

> "Mr. Gruner concluded ...... that every dam that impounds water presents a potential danger and that the risk of disaster can never be entirely banished. This would be a dangerous statement to trust to the tender mercies of the popular daily press, but it is at the heart of the thinking of the born engineer."(29)

It is unfortunate that such a fear is expressed on the revelation of a realistic scientific fact. Public reaction to this exposition may, in fact, mean the creation of new guidelines for future improvement of the criteria on dam-safety. The engineers' purpose is not only to construct dams. He should also devise means for better public perception of the basic engineering problems thereby develop favourable public response to measures of improvement required in face of these problems which probably have no immediate solution. The truth is that past engineering-economic approach ignored the welfare of the people while maximizing economic efficiency. Today, the concern for the safety of people is dominant in the minds of those responsible for design, construction and maintenance of reservoirs.(10) However, much remains to be done to strengthen and assure safety to life and property against man-made risks posed by the reservoirs.

This investigator finds good reasons to believe that it is more harmful to the profession as well as to the development of dam-building science to keep this simple probabilistic fact about the risk of reservoir failure a secret. It should, therefore, be made public. A true public perception of these problems will enlist: (1) a greater sympathy for the engineers who are hoodwinked by the complex conditions at an apparently safe dam-site that later on fails, (2) an increased public support for safety measures by funding proposals hitherto neglected or considered economically not feasible, (3) support for intensive research in reservoir safety thus permitting advanced preparation for failures and disasters that might occur in future, (4) pressure and incentive for timely management decisions which hitherto were made only when a disaster would occur, [It is interesting to note the remarks of Shirley-Smith in this context. He stated: "it is good to know that since the recent floods, the government announced that it would support the Institution's project for further investigation of flooding in Great Britain".(2)](5) a political support for action in harmonizing peoples opposition to dam construction,by providing effective and acceptable measures leading to full satisfaction of safety demands, (6) fund support for disaster planning in efficient management of normal and post-disaster operations.

Consequently, the revelation of the risk of dam failures to the public should be the greatest service to the profession of dam building. It may even be the beginning of a new look and initiation of a novel approach to safety of reservoirs. Efforts have now begun to assess the implications of calculated failure-risks of dams and the necessary counter measures required to minimize, if not eliminate, the hazardous consequences.

That there always exists a calculated risk of failure for the hydrologic design of a reservoir has been demonstrated by Yen based on probability theory.(43) This risk, which is the result of uncertainty in the natural hydrologic cycle, can never be made zero unless full regulation and control of weather becomes a possibility for man. Besides, there are other structural uncertainties which also cause some degree of failure risk. It is difficult to visualize if these numerous risk factors can be controlled in a manner as to render the failure probability to zero. Obviously therefore, one can generally accept that the risk of failure may not be completely eliminated.

Another viewpoint, directly opposed to the above hypothesis, have been put forward by another Swiss expert Huggenberger,(8) a Zurich consulting Engineer. His view is appropriately described in his own concluding remarks:

> "Based on 45 years of experience in the design and development of mechanical and electrical measuring instruments, of remote control installations with indicators and recorders, of consulting for placing instruments in more than 130 dams and on the widespread experiences in the field, I am convinced that (dam) failures and catastrophies can be avoided."(8)

This conclusion has its roots in the scientific theory of "cause" and "effect". Huggenberger suggests comprehensive assessment of actual safety of dams based on suitable methods of computation, model tests, and constant surveillance which include technical observations (made with measuring instruments, concrete, foundations, abutments, basin slopes, seepage water analysis, etc.) and scientific analyses based on investigations of creep, actual stress-strain condition, autogenous growth, plastic behaviour and other fundamental observations appropriately organized and critically evaluated to interpret the security of the structure and devise suitable measures to avoid serious damage or destruction. He has expressed confidence that the reservoir failures could be eliminated when the theory and practice of design, construction and supervision attains the necessary perfection. Serafim and Guerreiro(44) also show that such a perfection can possibly be attained. It is thus possible to successfully monitor dams with skilled experts and full knowledge of dam behaviour.

It is well known that precise prediction of the behaviour of hydraulic structures, particularly dams, have not been possible so far. It is also difficult to predict as to whether such a thing would ever be made possible. Yet experts like Huggenburger and Serafim emphasize the possibility of eliminating failures and disasters.

A major break-through has been made in the recent times with the development of reliable instrumentation for dams. There has also been of late a shift in the economic approach to safety. Thus, it may be possible to operate these structures in a manner that sudden and catastrophic failures may be avoided.

It is the opinion of this author that the two aforesaid viewpoints should not be compared. Both may be retained to serve different purposes. While Gruner has conceptualized an existing real-world situation based on historical observations, Huggenberger talks of a normative model serving as a motive force to improve the theory and practice of reservoir safety to a superior perfection. In fact both views are at the two extremes. The truth lies somewhere in between. With the gorwth of science and technology we shall certainly reduce the uncertainty to a considerable degree.

So numerous are the factors that affect the safety of a reservoir that it remains quite undetermined if this uncertainty can be completely eliminated.

While both extreme viewpoints have been held by concerned experts of reservoir safety, this author feels that the present knowledge, study and research on dams is highly inadequate to pronounce a rational objective judgement on this controversial issue. A recent development of sophisticated, reliable and comparatively less expensive instrumentation for the various components of reservoir structure has enabled us to better our understanding and control over these systems; and efforts for their safe monitoring no more appear to be a wild goose chase. On the other hand, excessive emphasis on efforts to fully eliminate all possibilities or risks of failure in the present state-of-the-art might lead to a blind alley. Both directions at present is quite unproductive beyond a certain extent. It is, therefore, necessary to apply some form of systems approach to obtain a rational safety criteria for reservoir systems. The design, construction and maintenance procedures for these systems should be tested and improved on the basis of these criteria. With the development of knowledge through research and supervision of existing and new structures, these criteria shall have to be modified in the light of new developments. Thus step by step as we advance in the realm of safer reservoir systems, it may be possible to assess the value of the two antagonistic hypotheses in order to ascertain the directions which are productive. The present knowledge may be quite inadequate to sit in judgement on this controversial issue.

However, the immediate future reservoir plans must be a balance of the two extreme hypotheses in order to formulate realistic and reasonable programs. Such a balance is possible only by means of system analysis. The comprehensive systems planning approach has seldom been applied to the field of reservoir safety. A suitable approach is, therefore, required to harmonize the conflict system and evolve a set of rational criteria on which a realistic reservoir safety plan may be based.

The conclusions of Huggenburger seem to have greater promise in the light of recent development of instruments and techniques for safety supervision of dams in the last decade. Refinement and greater improvement of these equipments and techniques for interpretation of observed data will provide the means to accurately trace (i) structural deterioration, and (ii) distress during operation which may result in a failure. The accurate predictions based on observed data will help in the following safety decisions:

(i) timely retirement of dams and reservoir; and

(ii) advance regulation of reservoir operation to prevent possible occurrence of distress situations leading to ultimate failure.

Both these safety practices have been increasingly used in the recent times. However, regulation and control of the probable disaster area could also effectively minimize disaster losses in the event of a failure. So long as the goal of Huggenburger remains unachieved, the disaster area planning will have to be given a greater emphasis and considerations. The present objective is, therefore, to completely eliminate dam-disasters if not dam failures. Thus man-made disasters should not be tolerated, and as far as possible should be avoided on all counts. A moot question in the management of a sound reservoir safety program should, therefore, be "What is your disaster plan?"(45) In fact, most existing reservoirs do not have any planning for probable disasters at all. And generally the owners believe that the possibility of a disaster is quite remote.

Changes in Safety Standards

A change becomes a necessity when the supply does not keep pace with the demand of the people. When the gradient of the supply curve is less than that of the demand, the supply lags behind the demand more and more with the passage of time. For a particular society there is a maximum value of this lag which it can no longer tolerate and that is considered a limit for revolution, i.e., a change. Tremendous public resentment expressed after the major dam-disasters like Baldwin Hills, Khadakwasla, Malpasset, Vaiont, Veg de Terra, etc., represented that safety standards lagged far behind the aspirations and demands of the people in a progressive modern civilization. A revision of conservative criteria and reformulation of structural and non-structural safety measures were, therefore, required. The change in outlook thus constituted a revolution in reservoir safety engineering wherein the emphasis was shifted to the novel concept of "elimination of disasters". However, the short memory of the disasters, that come to pass, was responsible for inadequate realization by the policy-makers of the significance of investment in safety. Also the dearth of programs for educating and influencing public opinion in favour of safety did not allow the changes to be as effective as it otherwise could be. Eventually the modern dam-safety engineering should pay greater attention to public education in order to devise an effective safety program.

TABLE III

DAM DISASTER PREVENTION

(I) CORRECTIVE MEASURES	(II) PREVENTIVE MEASURES
(a) Dam Safety supervision of	(a) Disaster Plain Regulation
(i) Planning	(i) Zoning Ordinances
(ii) Design	(ii) Subdivision Regulations
(iii) Construction	(iii) Building Codes
(iv) First Filling	(iv) Others
(v) Operation	
(b) Flood Forecasting	(b) Warning signs
(c) Watershed Treatment	(c) Public Education and mass media systems
(d) Urban Redevelopment (including permanent evacuation, flood proofing, etc.)	(d) Insurance
(e) others	(e) Taxation
	(f) others

On the basis of the philosophy outlined in this section earlier, it would be recognized that four important measures pertaining to human dimensions require an urgent mandate for the dam builders. They are: (i) Zoning, (ii) Dam Disaster Warning, (iii) Reservoir Safety Insurance, and (iv) Legislation. These four topics have been dealt with in detail elsewhere and may not be elaborated here. However, a comprehensive dam disaster prevention system has been outlined in Table III, following a similar scheme offered by Jensen(46) for flood-plain management.

Conclusion

Human dimensions of dam safety is a question of great relevance to the future activities in this field. The conservatism and constraint to development imposed by a number of court proceedings, that followed some of the recent major dam failures, have seldom been assessed. The engineers, however, are themselves responsible for lack of sufficient attention paid to the public perception of dam failures and the shifting demand for standards of safety. Though safety standards have been improved over the years, the adverse public reaction to some of the major catastrophic failures provide evidence to the fact that standards have not been modified and adjusted to serve the public needs satisfactorily. There is, therefore, a gap between demand and supply in the field of dam safety. Sociological research should, therefore, proceed hand in hand with all dam building activities in order to identify the dynamic public attitudes and make necessary adjustment - technical, social and hence administrative - to harmonize the increasing public discontentment. This discontent finds its expression in the burst of public wrath in the post-disaster situation. It is also felt that greater feed-backs from public hearings and diffusion of relevant information on dam safety will substantially improve the situation. A cooperative management of safety of dams and the downstream probable disaster area appears to be the most comprehensive approach to the problem. Absence of such programs is, probably, the cause of present anomaly; yet there is hardly any proposal put forward that encompasses fully these beneficial objectives.

CHAPTER VII

SUMMARY CONCLUSIONS

This study originated with an intention to investigate the major dimensions of dam failures in as comprehensive a fashion as possible. The conclusions arrived at in the preceeding pages are summarized herein explicitly, without any reference to their earlier treatment. The primary methodology of this dissertation is a "case study" approach undertaken in conjunction with a review of most major literature in the relevant field. The endeavour has been revealing and highly illuminating in as much as it helps to put various critical factors concerning dam safety in a rational perspective. In short, the findings are enumerated below:

1. The comprehensive study of dam safety is, at present, highly under-rated since its benefits are only vaguely understood.

2. Dam disasters are an increasing problem, in magnitude as well as in dimension.

3. Complex technical, economic and social-psychological considerations are required insofar as dam safety is concerned; their progmatic treatment involves in-depth research and comprehensive inspection programs with differing vigilance levels.

4. A formalism of disaster planning is almost non-existent insofar as reservoirs are concerned. A scheme of action has been suggested for the institutionalization of disaster alarm system, rescue-evacuation systems, urban redevelopment, disaster plain regulation, insurance, information systems and taxation, in addition to safety supervision of dams, as effective means of task accomplishment.

5. The probable disaster area and the human dimensions related thereto are major concerns of this thesis.
6. Model studies provide limited insight into the marginal phenomena of structural failure; analysis of data on prototype failures or extreme performances are the sole means of obtaining valid insights into structural behaviour.
7. Case studies of dam failures are like case laws in the legal profession. They are of utmost value to designers, and managers of these structures. It is, therefore, essential that they form an important part of such specialist training in improving professional practice. Notwithstanding this realization, there is as yet no systematic training or curriculum available for this purpose.
8. Some major conclusive evidences have been presented that are indicative of our limited knowledge of the behaviour of dams. Existing conflicts and contradictions in expert opinions emphasize the necessity for further study and research on dam failures; and these issues represent pregnant research areas.
9. An increasing trend in the safety demand has been identified. It derives from social factors like education, public awareness, affluence and other factors. This trend is beside the increasing disaster potential of modern dams.
10. Study of dam failures are handicapped by the existing attitudes: (a) reluctance to permit impartial investigation, (b) apathy after the disaster and (c) intangible nature of study benefits. Thus, research on safety has been damped and tunnel-visioned economic approach has prevailed over a long-term rational policy.

11. Some of the major efforts in investigating dam failures have been described briefly and chronologically.
12. Useful failure statistics by type of failure mechanisms and kind of dams have been developed.
13. Thirteen detailed case studies have been presented, critically analyzing the data on the basis of available analytical resources.
14. About 300 dam failures have been analyzed in the chapter on general overview, classifying all possible kinds of dam failures. The data presented would prove useful to designers, reservoir managers as well as researchers.
15. An increasing trend in dam failures has been identified.
16. Public participation and public input into the safety decision-making process in dams is found highly desirable and suggested for future implementation and testing.
17. A new trend in safety supervision is reflected in the increasing use and invention of measuring devices that are installed in most modern large dams. Successful exploitation of this trend holds much promise for the future.
18. And finally, the dissertation is proposed as a nucleus for the development of a suitable study curriculum to train specialists with broad perspectives of critical safety aspects in reservoir management.

In the nutshell, the study is intended to be a useful and effective contribution for the development of a suitable dam safety management system in Canada.

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APPENDIX

Based on the data available from the catalog of dam failures, accidents and disasters by Babb and Mermel (Ref. 85 of Chapter IV) , a tabulation has been constructed for masonry dams country-wise showing the name of the dams that failed, type of the structure, height of the dam, the year in which constructed, the year of failure and the causes of the dam failure. A similar tabulation for earthdams is also available in a paper by Middlebrooks (Ref. 14 of Chapter II). This table was prepared in the 1950s and hence has been updated by providing tables IV and V in Chapter IV.

The tabulated data compiled for our study is presented in this appendix. The compilation puts the whole spectrum of actual failures of masonry dams in perspective. In fact, no such presentation on the masonry dams is known to this author. As pointed out earlier, such tabulation has a precedence only in the studies of earth dam failures. Obviously, the data would be of considerable benefit to the research workers and other professionals concerned with dam-safety.

STATISTICS OF MASONRY AND CONCRETE DAM FAILURES

	Name of dam	Type	Height (ft)	Year Constructed	Year failed	Details of failure
	ALGERIA					
1.	Cheurfas	Rubble Masonry Gravity	131	1882	1885	Overtopped; foundation erosion
2.	El Habra No. 1	Rubble Masonry Gravity	110	1856	1871	Poor masonry; failed during flood
3.	El Habra No. 2	Masonry Gravity	115	1865	1871	Badly leaked when filled; overtopped and slide.
					1927	overtopped.
4.	Fergong	Stone Masonry Gravity	141	1870	1927	Overtopped
5.	Sig River	Masonry Gravity	69	1845	1885	Overtopped and undermined.
6.	Zardezas	Concrete Gravity	210	1938	1932	Landslides during construction
	BULGARIA					
1.	Vasil Kolarov	Stone Masonry	157	1949	1951	Deterioration of masonry.
	CANADA					
1.	Allard	Multiple Arch	43		1928	Spalling of concrete due to frost action.
2.	Bow River	Concrete Gravity			1912	undermining of gravel foundation; fracture due to ice thrust.

	Name of dam	Type	Height (ft)	Year Constructed	Year failed	Details of failure
3.	Montreal	Masonry Wall with backing			1896	Leakage due to ice pressure.
4.	Moose Jaw	Concrete gravity			1916	Overtopped, rip rap washed
5.	Namaka	Concrete gravity weir	20		1915	Foundation, not on rock, eroded.
6.	Name Unknown	Concrete gravity & buttress	25		One after the completion	Poor design and construction directed by a town electrician
	CHINA					
1.	TA-FUNG-MAN	Concrete Gravity	300	1943	-	Poor design and construction during war.
	EGYPT					
1.	Saad El Kafara	Masonry Gravity	33			Overtopped
2.	Mohammed Ali	Masonry Gravity		1835	1867	Foundation piping and blow-out
	FINLAND					
1.	Soininkoski	Concrete Gravity	30	1922	1954	Erosion in foundation during initial filling.

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..	Name of dam	Type	Height (ft)	Year Constructed	Year failed	Details of failure
	FRANCE					
1.	Bouzey	Masonry Gravity	60	1791	1804	Piping
				1870	1884	Wall cracked and slid forward.
					1895	Dam weakened due to water penetration, cracked horizontally and broke open.
2.	Grois Bois	Masonry Gravity	94	1838	1838	Slid 2 inches during first filling.
3.	Malpasset	Concrete thin Arch	218	1954	1959	Failure of left abutment flood destroyed the whole dam.
4.	Tolla	Concrete Arch	295	1961	1962	Cracks due to unequal temperature.
	GERMANY					
1.	Edertal (Eder)	Masonry Gravity	157	1914	1943	War breaching.
2.	Moehnetal Sperre (Mohne)	Gravity	131	1913	1943	Breached by enemy action.
	GREAT BRITAIN					
1.	Eigiau	Concrete Gravity	35	1908	1925	Footings not deep enough in clay foundation that scoured a 30 ft opening and poor concrete.

	Name of dam	Type	Height (ft)	Year Constructed	Year failed	Details of failure
	INDIA					
1.	Bhakra	Concrete Gravity	300		1959	Gatehoist chamber floor failed due to partial opening in the gates.
2.	Kaddam	Masonry Spillway	74	1958	1958	Overtopped and breeched as gates could not be opened timely by hand in the absence of electricity.
3.	Khadakwasla	Rubble Masonry	107	1875	1961	Overtopped by floods and breached near a step in the foundation.
4.	Krisha	Masonry Weir		1854	1952	Poor design, piping failure.
5.	Kundli	Masonry Gravity	148	1924	1924	Unexpected flood washed away green masonry.
6.	Mahanadi	Masonry Weir			1886	Breached a deep scour hole near the center sluice, cause never explained (Merriman)
7.	Pagara	Masonry Gravity	100		1923	Overtopped
8.	Sarda	Masonry Weir		1930	1956	Faulty design, uplift under the apron.
9.	Shirawate	Masonry	125	1912	1920	Leakage thru the dam.
10.	Takenpur	Masonry		1895	1908	Leakage thru foundation
11.	Thorkarmadi	Masonry Gravity	195	1922	1962	Sweating and leakage on downstream face.
12.	Tigra	Masonry	92		1917	Sliding due to uplift

	Name of dam	Type	Height (ft)	Year Constructed	Year failed	Details of failure
13.	Upper Colerum	Masonry Gravity	8	1836	1837	Leakage undermined foundation, the weir partialy washed away.
14.	Walwhan (Maharastra)	Masonry Gravity		1916	1961	Sweating and leakage of downstream face.
	IRAQ					
	All the 6 dams are primitive masonry structures known to have been built at a time when neither the engineering skill and knowledge of dams was sufficiently advanced nor was the documentation as scientific and reliable as in the modern times. Thus little is known about them.					
	ITALY					
1.	Bric Zerbino (Molare No 1)	Concrete Arch-Gravity	262		1935	Overtopped by flood; slid on foundation
2.	Gleno	Multiple Arch Buttress (gravity tangents)		1923	1923	Poor masonry in arch foundation caused collapse of 9 arches.
3.	Sella Zerbino (Molare No. 2)	Masonry Gravity	46		1935	Overtopped, insufficient spillway.
4.	Vaiont	Concrete Thin Arch	858	1961	1963	Overtopped by a flood as a result of massive landslide in the reservoir environment; dam did not collapse.
	JAPAN					
1.	Horonai	Concrete Gravity	46	1940	1941	Foundation movement.
2.	Komoro	Buttress	53	1927	1928	Foundation erosion, dam abandoned.

	Name of dam	Type	Height (ft)	Year Constructed	Year failed	Details of failure
1.	Komuro	Concrete Gravity	197			Not known
	KOREA					
1.	Hwachon	Concrete	267	1943	1957	Enemy action
	MEXICO					
1.	Santa Catalina	Rubble Masonry Gravity	49		1906	Overtopped, inadequate spillway.
	NORWAY					
1.	Tunhovd	Concrete Gravity	121	1920		Damage due to frost action.
	PAKISTAN					
1.	Rasul	Rubble Masonry Weir		1902	1929	Washed out by flood
	PERU					
1.	El Fraile	Concrete Arch			1963	Cracking due to vibrations of penstocks; tangential abutments.
	SOUTH AFRICA					
1.	Churchill	Multiple Arch Buttress		1947	1965	Cracks in the buttress

	Name of dam	Type	Height (ft)	Year Constructed	Year failed	Details of failure
	SPAIN					
1.	Alcantarilla	Masonry Gravity	46	100 A.D.		Washed away by floods
2.	Brguillo	Concrete Gravity	295	1931	1937	Hostile action during the civil war
3.	Cubillas	Masonry with buttress		100 A.D.		Not known
4.	Del Gasco	Masonry Gravity	187		1799	Overtopped during construction.
5.	Elche	Rubble Masonry	76		1836	Not known
6.	Esparragalejo	Masonry with buttress		100 A.D.		Not known
7.	Gasco	Masonry	187		1780	Washed away by flood during construction.
8.	Mequinenza	Concrete Gravity	243	1964		Severe leads due to concrete cracking
9.	Ordunte	Concrete Gravity	182	1934	1937	Enemy action; overtopped in 1964.
10.	Puentes	Rubble Masonry	164	1791	1802	Undermining and wash out of wooden pile foundation when first filled up to 154 ft.
11.	Sacedon	Concrete Gravity				Piping in the foundation.
12.	Torrejon	Concrete Gravity				Failure of power tunnel gates.
13.	Vega de Armijo	Concrete Gravity				Piping below the dam.
14.	Vega de Terra	Ambursen type	112	1957	1959	Poor construction and also unsatisfactory design.

	Name of dam	Type	Height (ft)	Year Constructed	Year failed	Details of failure
	SUDAN					
1.	Kashem El Ghirba	Concrete Gravity			1964	Undermining of foundation and wash-out.
	SWEDEN					
1.	Forshuvudforsen	Concrete Gravity				Frost damage
2.	Ljusne Strommar	Concrete Gravity	72		1949	Foundation movement during operation
3.	Porjns	Rock Gravity	60	1916	1916	Damaged by frost action.
	SWITZERLAND					
1.	Albigna	Concrete buttress	380	1959	1962	Cracks on the corner of one monolith due to test grouting.
	SYRIA					
1.	Sedjar	Masonry Gravity	20			Overtopped
	TURKEY					
1.	Cavdarhisa (Istamboul)	Masonry (clay core)	23			Not known
2.	Ebmali (Istamboul)	Masonry (earth wings)	60	1892	1916	Overtopped
3.	Orukaya (Ankara	Masonry (clay core)	52			Not known

	Name of dam	Type	Height (ft)	Year Constructed	Year failed	Details of failure
	URUGUAY					
1.	Rincon de Bonette	Concrete gravity	120	1946	1959	Dam overtopped due to unexpected precipitation, subsidiary dam breached as relief measure.
	UNITED STATES					
1.	Ansonia Mill	Concrete Wall	35	1911	1912	Foundation undermining, wall displaced and partially overturned.
2.	Ashley Brook	Ambursen type	58	1908	1909	Undermining due to inadequate cutoff walls, sliding at the base.
3.	Austin	Concrete gravity	50	1909	1911	Undermined; settlement, slide and seepage in shale stratum.
4.	Austin	Masonry gravity	68	1893	1900 1915	Overtopped and slid downstream. Damaged by floods.
5.	City	Concrete gravity on clay	28	1908	1909	Inadvertant closure of drainpipe by city officials.
6.	Clinton	Masonry Gravity	12		1938	Overtopped and washed away.
7.	Colonial	Concrete Gravity	31	1906	1912	Poor construction joints.
8.	Columbus	Masonry Gravity	39	1901	1901	Overtopped and breached
9.	Conodogainet Creek	Concrete Gravity	11	1911	1912	Overturned by ice pressure

	Name of dam	Type	Height (ft)	Year Constructed	Year failed	Details of failure
10.	Coon Rapids	Concrete Gravity	21	1914	1917	Washout of pile foundation.
11.	Croton	Masonry Gravity	229	1905	1955	Cracks after flood
12.	Dansville	Ambursen type	15		1909	Undermined during flood, deck slab snapped.
13.	Danville	Concrete Gravity	10	1903	1930	Poor concrete, no foundation drains, slid due to uplift.
14.	Dayton	R.C. Coffer-dam				Overtopped and undermined twice.
15.	Dell Mountain	Multiple Arch	98	1916	1919	Frost damage to porous concrete
16.	Des Moines	Concrete Gravity			1893	Faulty construction and ice pressure.
17.	Elwha	Concrete gravity	110	1912	1912	Foundation washed out, dam left hanging in the gorge.
18.	Fall River	Concrete gravity			1908	Slid on the base
19.	Fertile	Concrete Gravity			1910	Poor design, foundation not deep enough.
20.	Gem Lake	Multiple Arch	75	1917	1925	Disintegrating concrete structure converted to gravity section.
21.	Griffin	Concrete	62			Leakage in concrete section.
22.	Housatonic	Masonry Gravity	40	1869	1891	Dam overturned by of foundation.
23.	Janesville	Ambursen type			1912	Undermining of faulty foundation.
24.	Killingworth	Masonry Gravity	18		1938	Overtopped by unexpected rainfall.

	Name of dam	Type	Height (ft)	Year Constructed	Year failed	Details of failure
25.	Lake Hodges	Multiple Arch	136	1918	1918	Cracks developed in piers, no collapse.
26.	Lake Lanier	Concrete Arch	62	1925	1926	Abutment washed out by undermining.
27.	Lake Leigh	Ambursen type	32	1906	1911	Leakage beneath a faulty foundation.
28.	Lake Pleasant	Multiple Arch		1927	1926	Buttress cracked during construction, spillway cut down to hold water 41 ft. below top.
29.	Lake Waco	Concrete Gravity	69	1930	1947	Erosion of foundation below spillway apron and overflow section.
30.	Lincoln Pond	Masonry Gravity	25	1910	1912	Slid on smooth rock foundation.
31.	Little Rock	Rubble Masonry Wall	36		1887	Dividing wall of the reservoir collapse, cause unknown.
32.	Lower Shelton	Stone Masonry Gravity	25		1903	Overtopped
33.	Lower Tallassee	Masonry Gravity	30	1901	1901	Tangent sections washed out when overtopped.
34.	Lynx Creek	Masonry Gravity	50		1891	Poor construction, mortar too lean, failed during construction.
35.	Macdonalton	Concrete Gravity	16	1910	1911	Clay foundation undermined by piping, poor material and construction.

	Name of dam	Type	Height (ft)	Year Constructed	Year failed	Details of failure
36.	Manchester	Masonry Gravity	30		1902	Overtopped and slid on foundation, muskrats brrowed under foundation.
37.	Manitou	Concrete Arch	50		1924	Partial failure by concrete disintegration, converted to gravel fill.
38.	Matilija	Gravity Arch	163	1949	1965	Poor foundation and concrete deterioration.
39.	Merrill	Concrete Gravity (earth wings)			1912	Earth wings gave way due to overtopping
40.	Mill Brook	Ambusen type	40	1909	1966	Buttress collapsed near a crudely patched hole.
41.	Moyie River	Thin concrete arch	53		1926	Spillway undermined abutment washed out by flood.
42.	Nashville	Masonry Wall	34	1889	1912	Washed out with a section of limestone foundation; slide on water saturated clay seam.
43.	North Fork	Concrete Gravity	13	1903	1930	Cracking of concrete and sliding on foundation.
44.	Oswego River	Masonry Gravity	14	1870	1912	Undermined by leakage.
45.	Owasco Lake	Masonry Gravity	10	1860	1912	Undermined by leakage, failed due to lack of repairs.

	Name of dam	Type	Height (ft)	Year Constructed	Year failed	Details of failure
46.	Port Angeles	Masonry Gravity	108		1912	Piping thru foundation
47.	Portland	Masonry Gravity	17		1891	Poor construction bond between courses.
48.	Portman Shoals	Masonry Gravity	44	1897	1901	Overtopped, and the dam breached
49.	Prosser Creek	Concrete Gravity	35		1928	Overtopped, no damage.
50.	Puscotatuck River	Concrete Wall	8	1909	1910	Faulty design
51.	Riverside	Concrete Gravity			1912	Wing wall scoured by spillway water.
52.	Rockport	Masonry Gravity			1912	Overturned by ice pressure
53.	Roxbury	Masonry Gravity	24	1870	1903	Poor gravel foundation, downstream bulging, leakage and slide.
54.	Ruck-a-Chucky	Concrete Arch			1940	A slide resulted in abandonment of dam site.
55.	St. Anthony Falls	Sandstone Masonry Gravity	18	1894	1899	Dam went out sliding during flood; damaged earlier by ice pressure
56.	St. Francis	Concrete Gravity	205	1926	1928	Abutment failure in reservoir full condition
57.	Saranac River	Masonry Gravity	45	1895	1912	Partial failure due to ice pressure
58.	Scranton	Masonry Gravity		1885	1895	Poor design and workmanship.
59.	Stony River	Ambursen type	51	1913	1914	Foundation undermined, no damage.
			70	Rebuilt	1915	Undermined due to inadequate cut-off.

	Name of dam	Type	Height (ft)	Year Constructed	Year failed	Details of failure
60.	Sweet water	Concrete Gravity	113		1916	Overtopped, inadequate spillway, partial failure.
61.	Tampa	Masonry Gravity Weir	22	1898	1898	Dynamited by persons opposed to its construction.
62.	The Angels	Rubble Masonry Gravity	52		1895	Poor foundation, undermined during flood.
63.	Traverse City	Concrete Gravity		1904	1907	Scouring of foundation (backwash)
64.	Two Medicine	Ambursen	27	1913	1964	Overtopped and washed out, spillway tilted.
65.	Vernon Heights	Asphalt Concrete Wall	8		1896	Poor design and construction of foundation caused failure.
66.	West Brook	Ambursen type	35	1915	1916	Foundation undermined, six bays washed out, no city engineer during construction.
67.	Wheeler	Concrete Gravity	72	1936	1961	Sliding on the foundation clay seam.
68.	Whiting Street	Masonry Gravity	21		1902	Ice pressure caused leakage at the base.
69.	Winston No.1	Brick dam	74	1884	1904	Poor design, dam overturned.
70.	Woodward	Concrete Gravity	35	1913	1918	Poor design and construction, foundation undermined by leakage, no engineer supervision.
	U.S.S.R.					
1.	Dnieper	Concrete Gravity	203	1932	1941 1943	Destroyed during the war.