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SYNOPSIS Fill dams of height 50 meters or more are increasingly being planned, designed and constructed in highly seismic areas of the world. These dams incorporate various earthquake design measures. The design measures have been critically examined. Design guides with respect to free board, core base width, crest width, type and material of core and material of shells have been reviewed. Some examples of actual changes made in the design, for the seismicity of the region have been given (Table II). Finally, a table of pertinent data of over 50 high dams located in highly seismic areas of the world is included (Table I).

INTRODUCTION

A fill dam under earthquake loading can fail under the following possible modes.

1. Loss of free-board due to differential tectonic ground activity, slope failure and/or soil compaction due to ground shaking
2. Slope failure, crest settlement, cracking, concentrated leaks and piping in embankment
3. Major fault movement, shear failure, development of concentrated leaks and piping in foundation
4. Overtopping of dam due to slides, rockfalls and seiches in the reservoir
5. Failure of spillway and/or outlet works.

The psuedostatic earthquake analysis of a fill dam, performed by application of a static horizontal force expressed as the seismic coefficient times the weight of the structure, is a very dangerous engineering practice, in that it gives a false sense of security. A rational and straight forward approach which enables the engineer to develop technical judgement is still in the early stages of development. The finite element method, though a very useful tool, also requires simplifying assumptions with respect to geometry and material properties to make the problem adaptable to mathematical solution. The experience with performance of dams during earthquakes is still not sufficiently extensive to provide general guidance and precedence. Extensive research being conducted all over the world gives us a qualitative understanding of behavior of fill dams under earthquake loading.

This qualitative understanding suggests some basic concepts to improve the stability against a failure induced by seismic forces. These basic concepts, in general can be grouped under the following categories.

1. A conservatively selected free board to accommodate crest settlement, tectonic uplift of reservoir bottom and waves generated by seiches and reservoir rim slides.
2. Proper embankment zoning with respect to material and geometry.
   - A. A wide plastic central vertical core and well compacted free draining shells or a homogeneous clayey embankment.
   - B. Conservatively designed (gradation and thickness) transition zones (leak stopper), filters, vertical chimney drain and filter and/or drain blankets.
3. A well designed foundation-embankment contact, consisting of rock or clayey soil foundation, good foundation treatment for bearing capacity and piping and increased contact width at abutments.
4. A wide and adequately armored crest.
5. A conservatively designed reservoir lowering outlet discharge capacity.

This paper discusses different earthquake design measures and critically reviews prevelant practices. Suggestions to further improve the performance of the dams under earthquake loading are included. An illustration of a dam constructed of a dispersive soil core and river alluvium shells, located in a highly seismic area of Argentina, is given.

FREE-BOARD

Free-board for fill dams is always increased in seismically active areas. In general, the

TABLE I - DAMS LOCATED IN HIGHLY SEISMIC AREAS

PROJECT				EARTHQUAKE		DAM					IMPERVIOUS CORE					FILTER		TRANSITION ZONE				FOUNDATION			SPILLWAY		
													MATERIAL									UNDER CORE					
NUMBER	NAME	PURPOSE	COUNTRY	SEISMIC ACTIVITY	SEISMIC COEFFICIENT	TYPE	HEIGHT (m)	LENGTH (m)	FREE-BOARD (m)	CREST WIDTH (m)	CORE TYPE	CORE WIDTH AT BASE	UNIFIED CLASSIFICATION	LIQUID LIMIT	PLASTICITY INDEX	U/S THICKNESS (m)	D/S THICKNESS (m)	U/S THICKNESS (m)	D/S THICKNESS (m)	D/S CHIMNEY DRAIN (m)	D/S BLANKET (m)	TYPE	TREATMENT	UNDER SHELLS	TYPE	CAPACITY (m^3/sec)	EMERGENCY (m^3/sec)
1	CHICOOSEN DAM	–	MEXICO	H	.16	ER	264.0	500	10.00	25.0	CV	100	CL	40	20.0	7.5	7.5	7.5–50.0	7.5–50.0	–	–	CB	CG	A/UD	T	17,000	–
2	NETZAHUALCOYOTL DAM	HF	MEXICO	H	–	ER	138.0	478	5.40	15.0	CV	60	ML/MH	50	20.0	–	4.0	4.0–10.0	4.0–10.0	–	–	R	CG	A/U	C	11,000	10,650
3	EL INFIERNILLA DAM	H	MEXICO	H	–	R	148.0	335	7.60	12.0	CV	50	CL	49	25.0	2.5	2.5	5.0–10.0	5.0–10.0	–	–	R	CG	A/UD	T	13,400	–
4	TSENGWEN DAM	M	TAIWAN	H	.25	ER	133.0	440	10.00	10.0	CV	125	SM/GM	22	8.0	–	–	–	–	–	FB	R	CG	R/UD	–	9,470	–
5	OCHAR–EL–OUED DAM	HI	N. AFRICA	M	–	R	103.0	370	6.00	10.0	CV	37	CL	43	20.0	3.0	3.0	3.0	–	3.0	FB	R	CG	A/UD	–	–	–
6	PALO QUEMADO DAM	H	S. AMERICA	H	–	R	160.0	370	8.00	12.0	CV	90	ML	33	7.5	4.0	4.0	6.0	–	6.0	F&DB	R	CG & Gunite	R/UD	–	–	–
7	PUEBLO VIEJO DAM	H	C. AMERICA	H	–	R	133.0	250	15.00	13.0	CV	46	CL	41	19.0	3.0 – 7.0	3.0–7.0	3.0–12.0	–	3.0–12.0	DB	R	CG	A/UD	–	–	–
8	ORDVILLE DAM	M	USA	L–M	–	E	235.0	1707	6.70	24.4	S	80	GC	–	–	–	–	12.0–40.0	12.0–60.0	–	–	–	CB	–	C	–	–
9	CAYGOREN DAM	I	TURKEY	H	–	E	55.0	655	6.25	10.0	CV	68	CL	–	–	–	5.0	–	–	–	–	R	CB	A/UD	–	–	–
10	AYVACIK DAM	H	TURKEY	M–H	.12–.15	R	179.0	425	5.00	15.0	C	60	CL	–	–	15.0	15.0	5.0–15.0	5.0–55.0	–	–	R	CB	R/UD	C/G	11,000	–
11	AVSAR DAM	–	TURKEY	H	.15	E	43.5	–	8.25	10.0	CV	60	CL	–	–	–	2.0 – 7.0	2.0–18.0	–	–	–	A	–	A/UD	–	–	–
12	KARTALKAYA DAM	I	TURKEY	H	.15	ER	57.0	205	6.00	12.0	S	50	CL	–	–	3.0	3.0	–	–	–	–	–	–	A/UD	–	–	–
13	HASANLAR DAM	IF	TURKEY	H	.15	R	72.0	310	17.30	12.0	CV	80	CL	–	–	4.0	4.0	–	3.0 – 8.0	–	–	R	–	R/UD	–	–	–
14	ALMUS DAM	M	TURKEY	H	.15	ER	93.5	370	6.20	12.0	CV	165	CL	–	–	–	–	–	–	–	–	R	–	A/UD	C	1,550	–
15	BULDAN DAM	IF	TURKEY	H	.15	ER	63.0	295	7.50	10.0	CV	60	CL	–	–	2.5–18.0	2.5–18.0	–	–	–	–	R	–	R/UD	–	–	–
16	RAMGANGA MAIN DAM	H	INDIA	M–H	.12	E	126.0	–	6.70	12.0	CV	60	CL	–	–	30.0	24.0	4.5	4.0	3.5	DB	R	–	A/UD	C	–	YES
17	RAMGANGA SADDLE DAM	H	INDIA	M–H	.12	E	72.0	–	8.00	12.0	C	20	CL	–	–	12.0	40.0	4.5	2.4	2.0	F&DB	R	–	A/UD	C	–	YES
18	BEAS DAM	M	INDIA	M–H	.12	E	132.6	1950	9.20	13.7	CV	40	CL	30	12.0	6.0	6.0	–	–	–	–	R	–	A/UD	C	12,400	–
19	BAD DAM	H	DOMINICAN REPUBLIC	H	.15	E	118.5	400	7.40	8.0	CV	56	CL	40	20.0	6.0	6.0	–	5.0	3.0	–	R	–	CB & CG	C	–	YES
20	SAN LORENZO DAM	H	EL SALVADOR	H	.20	R	46.0	635	11.50	12.0	S	25	CL	40	15.0	8.0	8.0	–	–	2.0	F&DB	R	CG	R/UD	C/G	10,300	–
21	GURI DAM	H	VENEZUELA	L	.05	R	110.0	600	6.00	11.0	CV	55	ML	50	15.0	1.5	2.0	1.5	2.0	–	–	R	CG	R/UD	C/G	18,000	–
22	TARBELA DAM	M	PAKISTAN	M–H	–	ER	143.0	2740	5.50	12.0	S	80	GW/SM	–	–	–	–	3.5 - 6.0	4.0 - 8.0	6.0	F&DB	A	–	A/UD	C/G	18,400	23,800
23	MANGLA DAM	HI	PAKISTAN	H	.15	E	138.0	1036	9.80	12.5	S	70	CL	–	–	–	4.0	50.0	–	–	F&DB	R	CG	A/UD	C/G	25,500	6,500
24	DERBENDI KHAN DAM	HI	IRAQ	M	.10	R	135.0	445	10.00	17.0	CV	100	CH/CL	50	26.0	6.0	9.0	–	–	–	–	R	–	R/UD	C	–	–
25	ALICURA DAM	H	ARGENTINA	H	.20	E	130.0	800	5.00	12.0	CV	84	CL	35	15.0	3.0	3.0	–	–	3.0	–	R	–	A/UD	C	3,000	–
26	ULLUM DAM	HI	ARGENTINA	H	.15	E	67.0	400	9.00	12.0	C	45	ML/MH	40	15.0	5.0	5.0	–	4.0	3.0	–	R	CB	A/UD	C	2,500	–
27	CERRON GRANDE DAM	H	SAN SALVADOR	M	.10	R	92.0	800	8.00	10.0	S	38	MH	51	20.5	15.0	20.0–60.0	–	–	–	–	R	–	A/UD	C/G	2,200	–
28	YURI DAM	H	HONDURAS	M	.10	ER	60.0	186	7.00	8.0	CV	31	MH	–	–	2.0	2.0	–	–	–	DB	R	CG	R/UD	C	–	–
29	NAMIOKA DAM	I	JAPAN	H	.12–.20	R	52.0	305	3.40	8.0	CV	27	–	–	–	–	2.0	70.0	65.0	–	–	R	–	R/UD	C	172	–
30	TAISETSU DAM	M	JAPAN	M–H	.10–.15	R	86.5	440	4.50	12.0	CV	50	–	–	–	4.0	4.0	–	–	–	–	R	CG	R/UD	C/G	1,500	–
31	NIIKAPPU DAM	H	JAPAN	M–H	.10–.15	R	102.8	326	4.00	11.0	CV	38	–	–	–	3.0 – 7.0	3.0 – 7.0	–	–	–	–	R	CG	R/U-A/D	C/G	1,050	–
32	MIZUKUBO DAM	I	JAPAN	H	.12–.20	E	62.0	205	1.50	10.0	CV	39	–	–	–	2.0	4.0	0 – 40.0	0 – 40.0	–	–	R	CG	R/UD	C/G	495	–
33	TERAUCHI DAM	FIW	JAPAN	H	.15–.25	R	83.0	420	4.50	10.0	CV	76	–	–	–	8.0	4.0–12.0	–	–	–	–	R	CG	R/UD	C/G	1,080	–
34	HIROSE DAM	M	JAPAN	H	.12–.20	R	75.0	255	6.00	10.0	CV	50	–	–	–	–	3.0–15.0	3.0–20.0	--	–	–	R	–	R/UD	C/G	1,380	–
35	KUROKAWA DAM	H	JAPAN	H	.12–.20	R	98.0	325	4.00	11.0	C	33	–	–	–	2.0 – 7.0	2.0 – 6.0	–	2.0–14.0	2.0–12.0	–	R	–	R/UD	C/G	175	PS
36	SHIMOKOTORI DAM	H	JAPAN	M–H	.10–.15	R	119.0	279	4.00	11.0	C	46	–	–	–	2.0–12.0	2.0–12.0	2.0–12.0	2.0–12.0	–	–	–	–	R/UD	C/G	1,920	–
37	NABARA DAM	H	JAPAN	M–H	.10–.15	R	85.5	305	3.50	10.0	CV	41	–	–	–	3.0–14.5	3.0–17.5	–	–	–	–	R	–	R/UD	C/G	475	–
38	UCHITANI DAM	H	JAPAN	M–H	.10–.15	R	64.0	200	4.80	10.0	CV	30	–	–	–	3.0–12.4	3.0–13.4	–	–	–	–	R	–	R/UD	C	135	–
39	ABURATANI DAM	H	JAPAN	M–H	.10–.15	R	82.0	189	3.50	10.0	CV	37	–	–	–	3.0–16.0	3.0–17.5	–	–	–	–	R	–	R/UD	C/G	570	–
40	FUKUJI DAM	FIW	JAPAN	M–H	.10–.15	R	91.5	260	9.20	12.2	CV	36	–	–	–	4.0	8.0	120.0	120.0	–	–	R	–	R/UD	C	1,600	–
41	KASSA DAM	H	JAPAN	H	.12–.20	R	90.0	504	4.00	10.0	CV	40	–	–	–	2.0 – 8.0	2.0 – 8.0	–	–	–	–	R	CG	R/UD	T	116	PS
42	FUTAI DAM	[illegible]	JAPAN	H	.12–.20	R	87.0	286	4.00	10.0	CV	50	–	–	–	3.0–10.0	3.0–10.0	–	–	–	–	R	CG	R/UD	C/G	1,950	PS
43	TAKASE DAM	H	JAPAN	M–H	.10–.15	R	176.0	365	5.00	14.0	CV	70	–	–	–	–	2.0–15.0	5.0–35.0	3.0–15.0	–	–	R	–	A/UD	C/G	1,700	PS
44	NANAKURA DAM	H	JAPAN	M–H	.10–.15	R	125.0	250	5.00	12.0	CV	70	–	–	–	2.0 – 6.0	2.0–15.0	2.0 – 6.0	3.0–15.0	–	–	R	–	A/UD	C/G	1,950	PS
45	IWAYA DAM	M	JAPAN	H	.12–.20	R	126.0	366	19.00	10.0	C	42	–	–	–	3.0–10.0	3.0–20.0	–	–	–	–	R	–	R/UD	C/G	3,500	–
46	TEDORIGAWA DAM	M	JAPAN	M–H	.10–.15	R	153.0	420	4.00	12.0	CV	48	–	–	–	8.0	8.0	0 – 60.0	0–60.0	–	–	R	–	A/UD	C/G	2,900	–
47	MYOJIN DAM	H	JAPAN	M–H	.10–.15	R	89.0	402	4.00	10.0	CV	48	–	–	–	3.0–17.0	3.0–25.0	–	–	–	–	R	–	R/UD	C	80	PS
48	TOKACHI DAM	HF	JAPAN	M–H	.10–.15	R	84.0	443	6.00	12.0	CV	48	–	–	–	6.0	6.0	–	–	–	–	–	–		C/G	2,600	–
49	GOSHO DAM	M	JAPAN	H	.12–.20	R	53.0	225	4.50	10.0	CV	22	–	–	–	5.0	7.0	7.0	10.0	–	FB/D	R	CG	A/UD	C/G	4,200	–
50	SASAGAMINE DAM	I	JAPAN	M–H	.10–.15	R	49.0	319	3.30	10.0	CV	22	–	–	–	2.0	2.0	3.0	3.0	–	FB/D	R	CG–CB	R/UD	C/G	980	–
51	OUCHI DAM	H	JAPAN	M–H	.10–.15	R	102.0	340	4.50	10.0	CV	60	–	–	–	2.0–14.0	2.0–14.0	3.0–12.0	3.0–12.0	–	–	–	CG	Tuff & Breccia /UD	C	176	–
52	TAMAHARA DAM	H	JAPAN	H	.12–.20	R	116.0	600	4.00	12.0	CV	50	–	–	–	2.0–24.0	2.0–24.0	–	–	–	–	R	CG	R/UD	C/G	160	–
53	MIHO DAM	M	JAPAN	H	.12–.20	R	95.0	588	4.50	15.0	C	40	–	–	–	4.0–15.0	4.0–12.0	–	–	4.0	–	–	CG	R/U-A/U	C/G	3,100	–
54	SETO DAM	H	JAPAN	H	.12–.20	R	111.0	343	3.80	11.0	C	32	–	–	–	1.0–12.0	1.0–12.0	2.0–20.0	2.0–20.0	–	–	R	–	R/UD	C/G	190	PS

LEGEND

PROJECT
M = MULTIPURPOSE
H = HYDROELECTRIC
F = FLOOD CONTROL
I = IRRIGATION
W = WATER SUPPLY

EARTHQUAKE
H = HIGH
M = MEDIUM
L = LOW

DAM
ER = EARTH A ROCKFILL
R = ROCKFILL
E = EARTHFILL

IMPERVIOUS CORE
CV = CENTRAL VERTICAL
C = CENTRAL VERY SLIGHTLY SLOPED
S = SLOPING

TRANSITION ZONE
F = FILTER
D = DRAIN
B = BLANKET

FOUNDATION
R = ROCK
A = ALLUVIUM
D = DOWNSTREAM
U = UPSTREAM
CB = CONCRETE BLOCK OVER ROCK
CG = CONSOLIDATED GROUTING

SPILLWAY
T = TUNNEL
C = OPEN CHANNEL
G = CONTROLLED

EMERGENCY
PS = PUMPED STORAGE

greater the distance between the normal reservoir water level and the top of the dam, the safer the dam will be against earthquake failure. The actual increase depends on:

1. Watershed hydrology, reservoir capability to generate waves and seiches, and its capacity to store an abnormal flood without raising excessively the normal reservoir level.

2. Geology of the reservoir rim with respect to landslides and rockfalls.

3. Capacity to lower reservoir level in an emergency.

4. Dam material properties with respect to crest settlement under earthquake shaking.

5. Geographical location of the dam with respect to population, and extent of damage and loss of life it could cause in case of failure.

We have no technical means of calculating the exact individual effect of the above factors. In many cases the crest of the dam, designed for probable maximum flood (PMF) is considered just adequate for seismic events. But it is always a sound engineering judgement to be conservative and add an extra few meters on top of PMF. The degree of conservatism considered justified depends to a great extent on the dam height (Sherard, 1967). Fig. 1 shows a relationship between free-board and the height of the dam obtained from data given in Table 1.

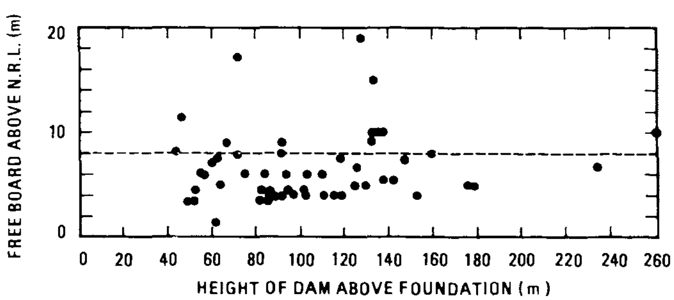

Figure 1. Free Board-Height Relation

It is suggested that either a minumum freeboard of 8 meters above the normal reservoir level or 5 meters above the reservoir level obtained during probable maximum flood (PMF), be selected for high dams located in highly seismic areas.

EMBANKMENT ZONING

Except for the evaluation and treatment of foundation soil or weak rock, the most important design decisions, from the standpoint of increasing the safety against earthquake hazard, are those which concern the zoning of the embankment (Sherard, 1967).

The embankment zoning provides resistance to concentrated leaks and should include placing the strongest, erosion resistant and high shaking strength material where it will do the most good. It must incorporate a leak resistant impervious core, adequately compacted shells, well designed transition zones followed by filter and vertical chimney drain, and a filter and/or drain blanket under shells.

Impervious Core

Stability of a fill dam under earthquake loading is closely related to mechanical properties of core and shell material; placement water contents and compaction, and type and position of the impervious element. The core material must be dense, plastic, and impervious. A plastic core, in addition to being highly impermeable, can undergo measurable deformations and is resistant to concentrated leaks. A soft or plastic core can also prominantly transfer stresses to stiff shells, and therefore develop mostly hydrostatic stresses with a minimum of shear stresses. In the case of sand gravel and fines mixture, leak resistance depends on plasticity of fine fractions, and the gradation and maximum size of coarse fraction. Whereas for clay, leak resistance depends on the plasticity of fine grain soils. Fig. 2 shows the two types of core materials that have been widely used in seismic areas.

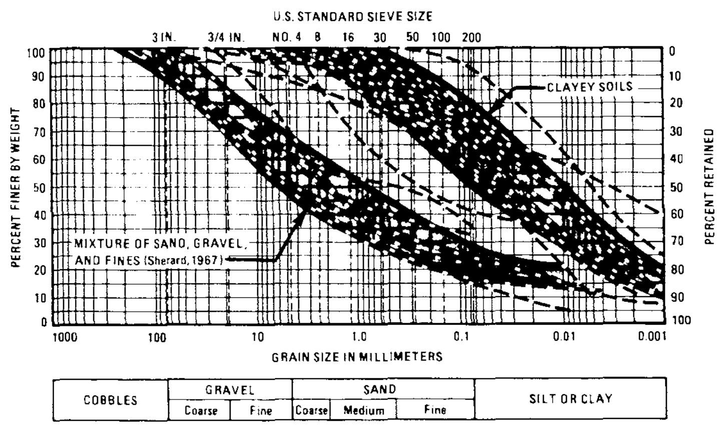

Figure 2. Two Typical Core Materials

A wide core of well graded mixture of sand, gravel and fines, or plastic soils is good insurance against concentrated leaks. Futhermore, the core should be widened at its abutments by about 25 to 50 percent. Sometimes, a 2 to 5 meter thick, highly plastic and impervious pad of 25 to 50 percent extra width, is placed under the impervious core. Fig. 3 depicts a relationship between the width of the impervious core at its base and the height of the dam. The relationship is obtained by studying data of the dams given in Table 1.

Impervious material in contact with the abutments and the foundation should be placed 2 to 3 percentage points wet of optimum moisture contents to obtain additional plasticity.

A central vertical core imparts greater seismic stability than a steeply inclined or sloping core (Finn, 1967 and Takahashi & Nakayam, 1973). There is a greater possibility of tension developing in the upper part of the dam. The intensity and zones of tensile stresses are dependent on the material and type of core, and are greatest in sloping core and least in central vertical core. The strength loss in

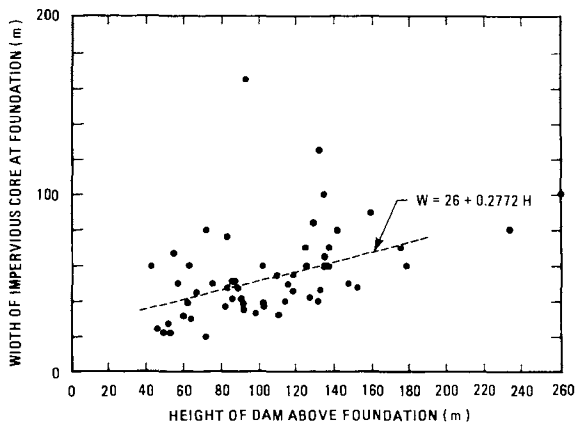

Figure 3. Impervious Core Width-Height Relation

the core material due to stress cycling during earthquake, combined with inadequate restraint at the top, results in greater deformation in sloping core dams, especially when the core is near the face of the dam. In the interior of the core, the direction of shear stresses coincides with the inclination of the sloping core, and thus, introduces inner sliding failure.

Table No. 1 shows that most of the dams constructed in seismic areas have a central vertical core.

Shells

Soils considerably improve in their ability to resist deformation if the density is increased by just a few percentage points. Our limited experience shows that a well compacted embankment, even when saturated, is basically insensitive to deformation caused by pulsating shear stress and does not experience excessive deformation.

Free draining soils are less susceptible to saturation, pore pressure development and hence, strength loss during earthquake shaking. Thus, well compacted free draining shells offer better resistivity against earthquake.

Transition Zones, Filters and Drains

Concentrated leaks through a dam, foundation and abutments under earthquake loading can be caused by embankment slumping, foundation failure in vertical and horizontal planes, and abutment sliding. Moreover, the fact that a number of dams have failed in a period upto a day after earthquake, suggests that piping through cracks from earthquake shaking may be responsible for failure. These facts suggest use of adequate measures against leaks, piping and erosion. The measures must include relatively impervious wide transition zones, thick cohesionless well graded upstream and downstream filters, vertical chimney drain and filter and/or drain blanket under the downstream shell. In case of severe damage to the core, a relatively impervious downstream transition zone, along with filter and chimney drain will prevent damaging seepage and piping. The chimney drain will carry incoming seepage to the blanket drain and will prevent pore pressure build-up in the downstream shell. Pore pressure build-up in the shell of rockfill is not as severe as it is in well compacted sandfill, alluvial fill and earthfill.

The filter blanket prevents piping of upstream shell into foundations and of foundation in turn into downstream shell. The number, type, thickness and gradation of transition zones, filters and chimney drains is dictated by the plasticity and gradation of core material and gradation and strength characteristic of available shell material.

FOUNDATION EMBANKMENT CONTACT

The maximum induced dynamic stress depends on the fundamental period of a dam which increases with height; soft core (very slightly) and soft foundation. A resonance effect occurs when the fundamental period corresponds to the predominant frequency of the earthquake. A strong earthquake shaking could cause shear failure in many foundation soils except in stiff, insensitive and dense cohesive soils. Dams constructed of clay soils on clay or rock foundation have withstood extremely strong shaking from 0.35 to 0.8g from a magnitude 8.25 earthquake with no apparent damage (Seed et al., 1978). The probability of a dam failure increases with decreasing ability of a foundation to withstand safely concentrated leaks and with decreasing strength under earthquake shaking. Even alluvial deposits on the top of rock foundation can considerably increase earthquake wave amplitude and vibration time.

Therefore, a dam embankment must either be founded on sound rock, or tough clay, or the rock must be treated for bearing capacity and ability to resist concentrated leaks. A concrete slab is sometime constructed under the impervious core to prevent concentrated leaks and piping. Consolidation and shallow grouting should invariably be provided under the core. If a minor amount of alluvium is left in place, its density must be greater than that of the overlying embankment.

CREST

Earthquake loading induces maximum acceleration at the top of the dam with considerable amplification of the ground acceleration. Therefore, the greatest stress changes occur near the top and shallow parts of the slope, thus, subjecting shell, core and outside slopes to tensile stresses. The intensity and zone of tensile stresses depends on the material properties and arrangement of the impervious core.

Even a soft foundation, can cause cracks in the crest under earthquake shaking, but, in this case, cracks originate from the base and progress to the crest. Moreover, at the upper part of the dam, confining pressures are small and the direction of maximum shear stress is parallel to the slope which tends to induce local slides. Qualitatively, it has been observed, that regardless of material properties and location of impervious core, the top 1/5th of the height of a dam, experiences a prominent seismic response, large displacements and transverse cracks.

Because of the possibility of earthquake induced transverse cracks, local slides and slumping in the crest region of the dam, leading to erosion of material along the direct seepage path and eventual piping, and because of increasing probability of dam failure with decreasing resistance of the dam crest to erosion from overtopping, careful design and construction of the crest is most important. The following design measures are suggested for designing a crest for a dam located in a seismic area.

1. The crest of the dam should be constructed with a well compacted sand and gravel mixture, well graded from coarse gravel size to number 200 seive size because, there is less available room at the top to construct thicker filters and transition zones.

2. The crest should be designed wider than a crest required for dams in non-seismic areas. A relationship between crest width and dam height for various dams located in seismic areas is presented in Fig. 4. The relationship suggests that a crest width of 12 meters (minimum), should be provided in seismic areas. Since experience has indicated that vibration forces increase with height, and because, in a mass system decreasing with height, the abrupt increase in mass due to additional crest width induces change of inertia at the top, the practice of steepening the upper slope near the crest of the dam to obtain a wider crest, though widely used for economic reasons should be discouraged.

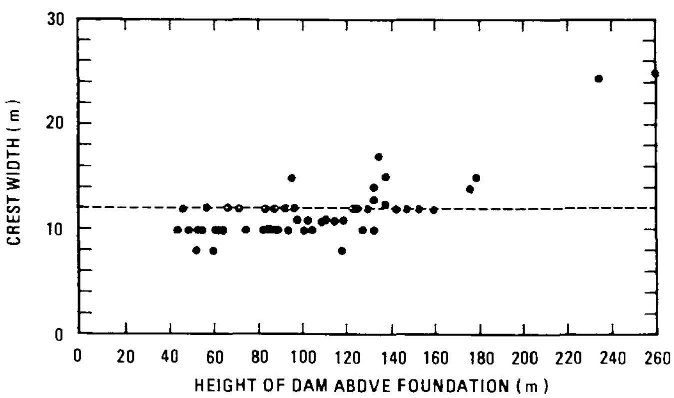

Figure 4. Crest Width-Height Relation

OUTLET DISCHARGE CAPACITY

The probability of failure under earthquake loading increases with the decreased discharge capacity of the outlet structure through which the reservoir level could be lowered with speed. The outlet structures consist of irrigation outlet, flood outlet, service spillway and emergency spillway.

The service spillway should invariably be an open channel spillway excavated in sound rock with flatter rock slopes to minimize occurrence of any rock slides into the spillway chute. It is always easier to repair an open channel spillway or its blockage than it is a tunnel spillway. Moreover a tunnel spillway is worst affected by tectonic movements. A spillway founded on soil should be avoided because of its foundation susceptibility to land slides and erosion. An uncontrolled spillway should be considered if possible to avoid gate opening problems due to misalignment caused by earthquake shaking. A smaller concrete spillway, which can be constructed as one monolithic structure, can stand earthquake shaking better than a wider structure, in which spillway walls and chute slab act independently.

Another consideration which should seriously be considered in the spillway design is that it might be necessary to spill before repairing the structural damage to the spillway. Therefore, it is suggested that a wide emergency spillway should be provided with a well designed fuse plug. A typical design of a fuse plug is given in Fig. 5. An emergency spillway is often formed by utilizing a saddle or depression along the reservoir rim, or by excavating a channel through a ridge or an abutment. The exit channel of the emergency spillway should either be directed to a different watershed or it should be a sufficient distance away from the dam to avoid damage to the embankment or other appurtenant structures. The fuse plug can be designed with its top one-half meter to a meter lower than the crest of the main dam.

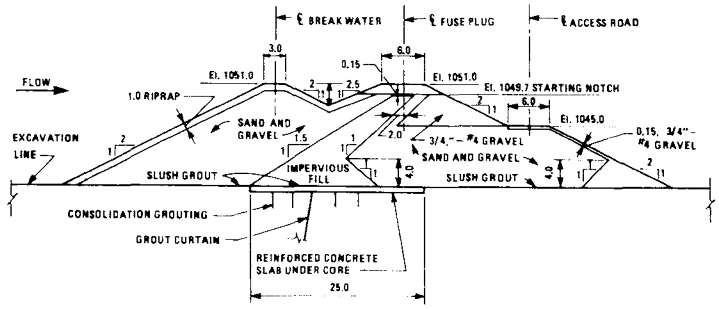

Figure 5. Typical Fuse Plug

DAM WITH DISPERSIVE SOIL CORE

The Ullum Dam is 67 meters high, and is located in a seismically active area of Argentina. The dam is located about 20 km upstream of the capital of the province. The dam consists of an impervious core of dispersive soils and shells of river alluvium. It is founded on a sedimentary rock formation containing claystone, siltstone and a very pervious conglomerate bed and many structural discontinuities. The following seismic measures were included in its construction (Fig. 6).

1. A very conservatively selected free-board of 9 meters.

2. A 2 meter thick pad of non-dispersive soil at the bottom of a dispersive soil core and well compacted shells.

3. Thick upstream and downstream transition and filter and a drain blanket under downstream shell.

4. Increased width of impervious core and transition zone at the abutment.

5. A concrete slab between the foundation and impervious core. A 1 meter deep filter trench both upstream and downstream of the concrete slab.

6. Increased crest width (12.0m) consisting of a well graded mixture of sand, gravel and fines.

7. An upstream wrap around of the abutments by the upstream shell.

8. An open channel spillway with intermediate and low level outlets.

9. An existing river diversion control structure located 2 to 3 km downstream of the dam.

10. Drain holes in the abutments.

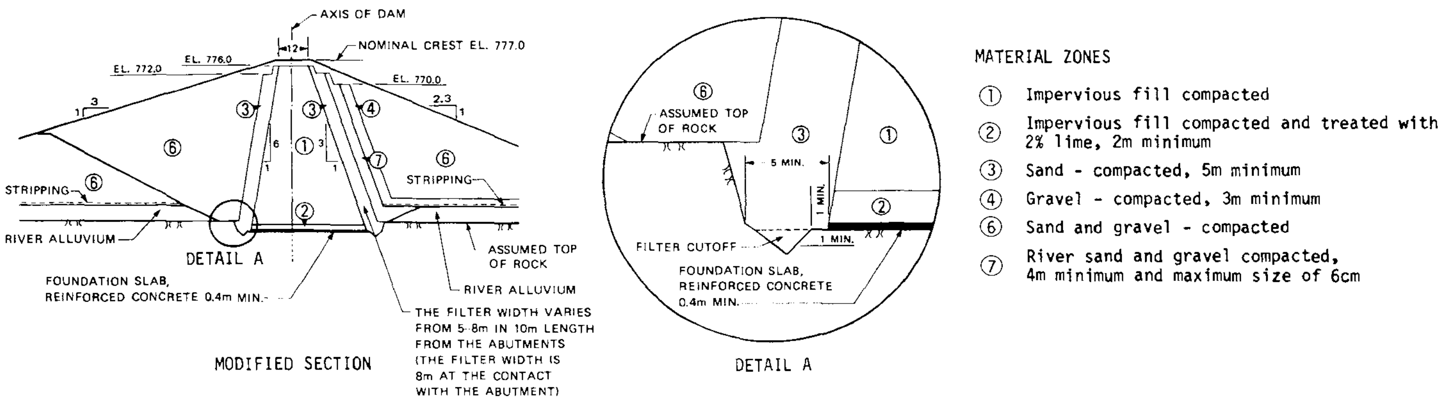

Figure 6. Ullum Dam - Seismic Design Measures

TABLE II

ACTUAL DESIGN CHANGES MADE FOR THE SEISMICITY OF THE REGION FOR HARZA ENGINEERING COMPANY PROJECTS

NO.	DESIGN FEATURE	PROJECT (Country)	CHANGES
1	Freeboard	All	Most dams design for PMF. This gives adequate free board for seismic events. Guri (Venezuela) designed for 5 m above PMF.
2.	Filter, transition, vertical chimney, and filter and drain blanket	Guri (Venezuela)	- filter thickness increased
		Bao (Dominican Republic)	- added fine filter as crack stopper, and chimney drain
		Ullum (Argentina)	- filter thickness increased at abutment, provided multistage filter and chimney drain
3.	Wide plastic core and well compacted free draining shells, flatten slopes and stabilizing berms	Cerron Grande (El Salvador)	- flatten embankment slopes and increased compaction of sand shells
		San Lorenzo (El Salvador)	- flatten embankment slopes
		Bao (Dominican Republic)	- flatten embankment slopes and used only free draining material in shells
		Yuri (Honduras)	- flatten embankment slopes
		Maqarin (Jordan)	- rockfill instead of sand shells
		Guri (Venezuela)	- berms added
		Yacyreta (Argentina)	- berms added
4.	Well designed foundation embankment contact	Cerron Grande (El Salvador	- sand removed from the foundation
		Maqarin (Jordan)	- sand removed from the foundation
		San Lorenzo (El Salvador	- added adits and drain holes in abutment
		Guri (Venezuela)	- core width increase in core trench
		Maqarin (Jordan)	- dam site moved to avoid seismically unstable material in foundation
		Rio Lindo (Honduras)	- dam site moved due to old landslide
		San Lorenzo (El Salvador	- dam site moved due to old landslide
5.	Adequately armored crest	Cerron Grande (El Salvador	- crest width increased
		San Lorenzo (El Salvador	- crest width increased
		Guri (Venezuela)	- crest width increased
		Bath Co. (USA)	- crest width increased

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REFERENCE

1. Balissat, M. and Bossoney, C. (1979). Dynamic Behavior of Three Rockfill Dams, Proc. 13th International Conference on Large Dams. New Delhi, India, Vol. 2, pp. 1115-1145.

2. Blanc, G.J. (1963). The Derbendi Khan Project. Water Power Oct: pp. 423 and Nov: pp. 447.

3. Comision Federal De Electricidad, (1979). Investigation on the Design and Performance During Construction of Chicoasen Dam, Mexico. Contribution to the 13th International Conference on Large Dams, New Delhi, India.

4. Comision Federal De Electricdad, (1976). Behavior of Dams Built in Mexico. Contribution to the 12th International Conference on Large Dams, Mexico.

5. Davis, Calvin V. (1958). The Derbendi Khan Dam. Symposium on Rockfill Dams, Journal of Power Division, Proc. of the ASCE. pp. 1741, 1-23.

6. Finn Liam, W.D. (1967). Behavior of Earth Dams During Earthquakes. Proc., 9th International Conference on Large Dams Istanbul, Turkey, Vol. 6, pp. 355-367.

7. Golze, Alfred R., Seed, H. Bolton and Gordon, Bernard B. (1967). Earthquake Resistant Design of Oroville Dam. Proc., 9th International Conference on Large Dams. Istanbul, Turkey, Vol. 4, pp. 281-299

8. Goyal, K.C., Srivastava, J.N. and Joshi, V.K. (1973). Impervious Material and Slope Protection at Ramganga Dams. Proc., 11th International Conference on Large Dams. Madrid, Spain. Vol. 3. pp. 719-741.

9. Japan National Committee on Large Dams. (1973, 1976 and 1979). Dams in Japan. Contributions to International Conferences on Large Dams.

10. LOGANI, K.L. and Lhez, H.H. (1979). Technique Developed During Foundation Treatment of the Ullum Dam Constructed of Dispersive Soils. Proc., 13th International Conference on Large Dams New Delhi, India. Vol. 1. pp. 729-748.

11. Palta, B.R., Malhotra, J.C. and Agarwal, M.L. (1973). Impervious Core and Slope Protection of Beas Dam. Proc., 11th International Conference on Large Dams. Madrid, Spain Vol. 3. pp. 743-768.

12. Seed, H.B. (1967). Earthquake Resistant Design of Earth Dams. Canadian Geotechnical Journal, Vol. 4, No. 1. pp. 1-26.

13. Seed, H.B., Makdisi, F.I. and De Alba, P. (1978) Performance of Earth Dams During Earthquakes. Journal of the Geotechnical Engineering Division. Proc. of the ASCE, GT 7. pp. 967-994.

14. Sherard, J.L. (1967). Earthquake Consideration in Earth Dam Design. Journal of the Soil Mechanics and Foundation Division, Proc. of the ASCE. SM4, pp. 377-400.

15. Sherard, J.L. (1975). Seismic Design of Embankment Dams, Proc., 6th Pan American Conference on Soil Mechanics and Foundation Engineering Buenos Aires, Argentina.

16. Takahashi, M. and Nakayama, K. (1973). The Effect of Regional Conditions in Japan on Design and Contruction of Impervious Element of Rockfill Dams. Proc., 11th International Conference on Large Dams, Madrid, Spain, Vol., pp. 501-524.

17. Turkish National Committee on Large Dams (1967). Contribution to the 9th International Conference on Large Dams. Istanbul, Turkey.

18. United States Department of the Interior, Bureau of Reclamation, (1975). Dynamic Analysis of Tsengwen Dam. Taiwan, Republic of China.

19. Ural, Orhan M. (1967). Design and Construction of Earthquake Resistant Dams in Turkey. Proc., 9th International Conference on Large Dams. Istanbul, Turkey, Vol. 4, pp. 311-325.

20. Varde, Oscar A. and Medici, J.C. (1975). Alicura Hydroelectric Project-Material Investigation. Proc., 5th Pan American Conference on Soil Mechanics and Foundation Engineering. Buenos Aires, Argentina. Vol. 2. pp. 219-241.

21. Verde, Oscar, A. and Federico Ortiz de Zarate (1975). Alicura Hydroelectric Power Project. Proc., 5th Pan American Conference on Soil Mechanics and Foundation Engineering. Buenos Aires, Argentina. Vol. 2. pp. 243-254.