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Recent developments of upstream membranes for rockfill dams

by

W. G. KEARSEY

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1. SUMMARY

This report is intended to provide guidance to the practising engineer who wishes to evaluate the feasability of using an impervious upstream membrane for a rockfill dam. The early history and development of impervious membranes is presented in Section 2. The major advantages and disadvantages of the various types of membrane are presented in Section 3. The use of membranes for dams founded on alluvial deposits is discussed. While these arguments are of necessity of a general nature it can be seen that upstream membranes have special advantages that no other form of dam construction has.

Some aspects of construction techniques are briefly discussed in Section 4. These comments are intended to highlight some advantages and limitations of construction plant or techniques. The points regarding construction of rockfill are especially important as the performance of the membrane will depend solely on the amount of deformation of the rockfill. As a complementary section several case histories of each type of membrane are presented in Section 5.

There are, unfortunately, dams that develop excessive leakage through the upstream membrane. Repairs to several dams have been reported in the literature. Some case histories of repair are presented in Section 6, together with probable reasons for the membrane's poor performance. The method of repair depends very much on the particular case. The leaks have usually responded to treatment and at a fraction of the cost of replacing the dam.

Little information is available to assist in the design of dams located where extremely cold weather can be expected. The most relevant case histories, presented in Section 7, relate to asphaltic concrete dams located at high altitudes. They may form the basis for further studies. The good detailing of the perimetral and crest joints as well as any construction or contraction joints is of great importance in ensuring a watertight membrane. Descriptions and examples of joints are given with the case histories and the design of joints is reviewed in Section 8. The design engineer may well have other ideas that will work equally well.

Over a hundred years of experience in designing modern dams have resulted in many improvements. The trends in construction have been highlighted in Section 9. The inclusion of Rollcrete in this section is, in the opinion of the author, a logical step forward

to improve the performance and production of impervious membranes. Rollcrete has advantages when used alone and in combination with both cement and asphaltic concrete membranes.

The design of asphaltic concrete and cement concrete membranes is discussed in Section 10. The absence of simple rigorous solutions is noted, and empirical methods with proven success are presented. The empirical methods are updated in the light of recent experience and are recommended for design. Future use of upstream membranes is considered and outline design methods suggested.

2. INTRODUCTION

To meet the increasing demands for water for domestic, irrigation and power generation purposes the number of dams and reservoirs throughout the world is steadily growing. In the past the most favourable sites have been chosen. The preferred site for power generation, for example, has been a narrow deep rocky valley. Suitable sites are becoming increasingly more scarce. Dams are of necessity being constructed at locations where the subsoil conditions are less than ideal or where the valleys are wide and shallow. At these locations earth and rockfill dams are attractive alternatives as they impose lower stresses on the subsoils and can be built relatively cheaply. Where settlements in the subsoils occur, the earth and rockfill dams possess sufficient flexibility to accommodate considerable deformation without rupture. Dams are also needed in areas of seismic risk so the deformation characteristics of earth and rockfill are assets in these areas. Stiff structures such as arch or concrete gravity dams are theoretically more prone to severe earthquake damage.

The first large dam to have a reinforced concrete face was completed in 1910. Table 1 lists some concrete faced dams. Concrete upstream membranes were initially used on dumped rockfill embankments. The post-construction movements or the movements on first filling were often sufficient to rupture the joints. Subsequent settlements, due to the leakage water washing out fines, rearranging fill to a denser state or reducing the strength of the fill by wetting, opened the gaps more. One example of the magnitude of the settlements due to wetting is illustrated by the Cogswell Dam. Built in 1935, 85 m high, 1.8 m of settlement occurred in a day, after heavy rain. Later settlements reached a staggering 4.1 m. Washing with water for several months increased the settlements to 5.3 m. Poor performance of several other dams about this time included Guadalupe Dam in Mexico. This dam, 28 m high, was completed in 1943 as a concrete faced rockfill dam but difficulty was experienced in pouring the upstream slab due to severe settlements, Ref. 1. During the first filling the crest settled 2.1 m. The dam was subsequently left unfilled until 1947 when filling to full height produced leakage of 4 cumec. Upon emptying, the slab at the toe was discovered to be unsupported and a 40 cm wide cavity had been eroded. A 4 l/s flow was still occurring in the toe drain and subsequently a stream was found to have been incompletely diverted. It is interesting to

Table 1

Dams with upstream concrete membranes.

Dam	Country	D.O.C.[1]	Height m	Membrane Maximum Thickness cm	Membrane Minimum Thickness cm	Max.Th. / Height %	Design Equation	Remarks
Relief	U.S.A.	1910	42					
Main Strawberry	U.S.A.	1916	145	38	22	0.85		
Dix River	U.S.A.	1925	84.2	46	20	0.55		
McKay	U.S.A.	1925	48	32	20	0.66		85 l/sec
Fordyce	U.S.A.	1927	42	45	30	1.07		
Medow Lake	U.S.A.	1930	22	10	5[2]	0.45		
Don Martin	Mexico	1930	30	30	20	1.00		
Bonita	U.S.A.	1931	31	30	20	0.97		
Salt Springs	U.S.A.	1931	101	90	30	0.89		
Ladce	Czechoslovakia	1936	14	15	-	1.07	Unreinforced	
Cogoti	Chile	1939	75	40	20[2]	0.53		
Madeco	Mexico	1939	47	42	20	0.90		
Guadalupe	Mexico	1943	28.5	50	20	1.94		4 cumec leakage. Reconstructed.
Lower Bear I	U.S.A.	1952	70	77	30	1.10		
Lower Bear II	U.S.A.	1952	45	59	30	1.30		
Upper Bear	U.S.A.	1953	24	12.5	7.5[2]	0.52		

Dam	Country	D.O.C.[1]	Height m	Membrane Maximum Thickness cm	Membrane Minimum Thickness cm	Max.Th. / Height %	Design Equation	Remarks
Ishibuchi	Japan	1953	53	60	40	1.13		
Lemolo	U.S.A.	1954	36	48	30	1.33		
Nozori	Japan	1956	44	66	–	1.50		
Pinzanes	Mexico	1956	54	55	30	1.02		
Quioch	Great Britain	1956	34	38	30	1.12		
Leichhardt	Australia	1957	26	12.5	7.5[2]	0.48		
Corella	Australia	1957	23	10	7.5	0.48		
Courtright	U.S.A.	1958	97	85	30	0.88		1.26 cumec. Repaired
Wishon	U.S.A.	1958	88	76	30	0.86		
Paradela	Portugal	1958	118	110	30	0.98		
Sassiere	France	1959	30	30	30	1.00		
Taum Sauk	U.S.A.	1963	35	25	25	0.71		
Des Fades	France	1966	68	60	35	0.88		
Skalka	Czechoslovakia	1965	15	25	–	1.67	Unreinforced	
Miksova II	Czechoslovakia	1965	22	28	–	1.27	Unreinforced	
Canes	France	1966	40	50	40	1.25		
Karaoun	Lebanon	1966	60	50	30	0.83		
Cabin Creek	U.S.A.	1966	75	45	30	0.60		
New Exchequer	U.S.A.	1967	148[3]	85	45	0.92		13.9 cumec. leakage
Piedras	Spain	1967	39	25	25	0.64		
Mackey	U.S.A.	1967	48	31	31	0.64		

Dam	Country	D.O.C.[1]	Height m	Membrane Maximum Thickness cm	Membrane Minimum Thickness cm	$\frac{\text{Max.Th.}}{\text{Height}}$ %	Design Equation	Remarks
Wilmot	Australia	1968	33.5	25	25	0.76		
Pindari	Australia	1969	46+30	75	48	1.00		
Nyrsko	Czechoslovakia	1969	36.5	45	30	1.23		
Kangaroo Creek	Australia	1969	59	60	30	1.00	$\frac{H\,0.305}{60.9} + .305$	8 l/sec
Hunico	Peru	1970	15	30	30	2.0		0.4 cumec.
Palona	Australia	1970	40	25	25	0.64		
Serpentine	Australia	1971	38	25	25	0.67		
Cethana	Australia	1971	110	52	30	0.47	0.3+0.002H	35 l/sec
Alto Anchicaya	Colombia	1974	140	70	30	0.50		1800 l/sec leakage reduced to 180 l/sec
Foz Do Areia	Brazil	1980	160	80	30	0.50	0.3+0.0357H	165 l/sec
Outardes 2	Canada		55	30	30	0.55		
Pozo De La Ramos	Spain		97	70	35[2]	0.72		
Yacambu	Venezuela		162	76	30	0.71	1+0.0028H	in ft.
Villagudin	Spain	1981	33	30	30	0.91		

[1]Date of Completion.
[2]Membrane of Gunite or laminated concrete.
[3]A rockfill dam, which heightens one of a gravity arch, 148m high. The height of the membrane is 92m.

note that the reconstructed dam also leaked and a third dam was eventually built and completed in 1968.

Design details have evolved empirically from older dams, constructed monolithically without expansion joints, that have performed satisfactorily. Bucks Creek Dam, Ref. 2, in California, and Mckay Dam, Ref. 3, in Oregon, are two examples presented by Sherrard et al., Ref. 4. Much leakage has been diagnosed as being caused by torn waterstops. Sherrard questioned whether less trouble might have been experienced if the slabs had been built without the joints. The use of compacted, and compacted and wetted rockfill has reduced settlements within the fill to acceptable values. Coupled with the use of slipforms the concrete facing slab is now increasing in popularity.

The advent of heavy, high output construction equipment has enabled the construction of larger and larger dams. Higher densities in the fill and subsequently smaller post–construction settlements are becoming common. These factors have enabled the recent successful use of concrete upstream membranes. They have been used on slopes averaging about 1:1.3 with vertical height up to 160 m. Most dams with upstream concrete membranes have been constructed where rock was very close to the surface. See Table 2 listing some of the larger recent concrete faced dams.

Asphaltic concrete has been used for upstream impervious membranes since 1934 and up to 1968 some 61 large and small dams had been completed successfully. Tables 3 and 4 list dams with asphaltic concrete upstream membranes completed before 1968. Prior to 1934 asphaltic concrete was used as erosion protection to some dams, utilising the high modulus values of bitumen and rock when subjected to wave impact loading. The protection took the form of binding rocks together by pouring hot bitumen between them. Since about 1950 asphaltic concrete has been used on large dams as an impervious membrane. Construction machinery specially built for operation on sloping surfaces has replaced the road paver adapted for operation on sloping surfaces by the addition of a winch.

The first completed dam using asphaltic concrete was Amecker in Germany in 1934, where the asphaltic concrete was used to reseal an existing clay cored dam. The pioneering use of asphaltic concrete started in 1926 with the planning of El Ghrib in Algeria. This dam was not completed until 1937. The shape of the dam was modelled on

Table 2

Some recent concrete faced dams.

Dam	Country	Height m	D.O.C.[1]	Foundation
Cethana	Australia	110	1971	Rock
Alto Anchicaya	Colombia	140	1974	Rock
R.B.Bailey	U.S.A.	110	1978	3 m of alluvium
Chusa	Colombia	130	1978	
Yacambu	Venezuela	160	1980	Rock
Foz do Areia	Brazil	160	1980	Excavation to sound rock.
Mackintosh	Australia	78	1981	
Sugarloaf	Australia	90	1979	Deeply weathered rock.
Jamrani	India	160		

[1]Date of Construction.

Table 3

Dams with asphaltic concrete membranes.

(After Visser, et al., 1970.)

No.	Name and year of completion		Country	Storage capacity 10^6 m^3	Maximum height m	Area of revetment m^2	Slope	Construction details	Remarks
1	Amecke	1934	Germany	1,0	12	2.500	1:2	Stone 10/15, Gravel 20/40, ST, 6 cm AC, SC	Resealing of existing clay-core dam
2	Turawa	1937	Germany		13	8.000	1:3	ST, 5 cm AC, SC	
3	El Ghrib	1937	Algeria	280	58	13.000	1:1-1:0,67	8 cm ND, 2x6 cm AC, 10-12 cm CC-slabs	CC slabs 1954 partly slipped down
4	Bou Hanifia	1938	Algeria	73	54	23.000	1:0,95-1:0,8	12-20 cm ND, 12 cm AC with reinforcement, CC slabs	
5	Schevelinger	1940	Germany	0,3	15	3.500	1:1,75	8 cm ND, Stones 5/60, 3 cm BC, 6 cm AC, SC	
6	Dreilägerbach	1950	W-Germany	4,28		700	1:2,5	2,5 cm BC, 6 cm AC, SC	Wingdam
7	Genkel	1952	W-Germany	9,75	43	11.000	1:2,25	CS, 6 cm BC, 6 cm AC, 12 cm BD, 6 cm BC, 9 cm AC, SC	DS
8	Oued Sarno	1952	Algeria	70	36	11.000	1:2,5-1:2	Clay-core, 10 cm BD, 8 cm AC, SC	DS
9	Glen-Anne	1953	USA		30	18.000	1:4	Earthdam, 4x7.5 cm AC protective layer	Wingdams 1:2 slope
10	Perlenbach	1954	W-Germany	0,85	18	2.600	1:1,75	2,5 cm BC, 6 cm AC, SC	
11	Iril Emda	1954	Algeria	160	75	65.000	1:1,6	CS, 12 cm ND, 12 cm AC, 8-15 cm CC	ND drain tubes every 10 m
12	Henne	1955	W-Germany	39,0	58	28.000	1:2,15-1:2,07	CS, 6 cm BC, 7 cm AC, 10 cm BD, ES, 10 cm AC, SC	DS extra core[1]
13	Maria al Lago	1955	Italy		18	7.700	1:2,25-1:2	5 cm rein. CC, 12 cm porous CC, 10 cm AC, 12 cm CC-slabs	Part of La Fedaia scheme
14	Riveris	1956	W-Germany	5,0	45	12.000	1:2	CS, 6 cm BC, 4 cm AC, 15 cm BD, 8 cm AC, SC	DS up till half way the slope
15	Wahnbach	1956	W-Germany	43,2	48	25.000	1:1,6	CS, 6 cm BC, 4 cm AC, 11 cm BD, 9 cm AC, SC	DS
16	Montgomery	1957	USA	6,3	35	22.000	1:1,7	3-7 cm BC, 10+9+7,5 cm AC	Crest level + 3.315 m
17	Radoina	1959	Yugoslavia		42	6.000	1:0,89-1:0,74	15 cm porous CC, 9 cm AC, CC without joints	
18	Hardap	1961	S.W. Africa	252	36	40.000	1:1,7	CS, BC, 10 cm AC 2-layers, SC	
19	Venemo	1963	Norway	20	64	12.000	1:1,7	BC, 15 cm AC 3-layers, SC	
20	Bigge	1964	W-Germany	140	55	46.000	1:1,75	ES, 4 cm BC, 6 cm AC, 11 cm BD, ES, 12 cm AC, SC	DS, extra core[1]
21	Steinbach	1964	W-Germany	4,9	35	16.000	1:1,75	ES, 5 cm BC, 4 cm AC, 7 cm BD, 8 cm AC, SC	DS
22	Diessbach	1964	Austria	4,8	36	8.000	1:1,7	ES, 6 cm BC, 8 cm AC, SC	
23	Zoccolo	1964	Italy	33,1	66	42.500	1:2,5-1:2	7 cm BC, 5+4+4 cm AC, SC	
24	Silbergrund	1964	E-Germany	0,19	12	2.000	1:1,7	5 cm BC, 2x3 cm AC, 10 cm BD, 5 cm AC, SC	Secondary dam to Ohra scheme, DS
25	Kessenhamm	1964	W-Germany	0,3	18	3.800	1:2	3 cm ES, 3 cm BC, 2x4 cm AC, SC	Secondary dam to Bigge scheme
26	Kruth-Wildenstein	1964	France	12	35	13.000	1:1,5	12,5 cm CC, 2x3,5 cm AC, 10 cm CC	
27	Ulmbach	1965	W-Germany	0,8	20	6.300	1:1,8	7 cm BC, 2x4 cm AC, SC	
28	Morávka	64/66	Czechoslovakia	11,2	38	25.000	1:1,75	6-7 cm BC, 5+2x4 cm AC, 4+4 cm AC, SC	Up till 2x4 cm AC 1964, Rest 1966
29	Ohra	1966	E-Germany	19,6	59	22.300	1:2	5 cm BC, 4 cm AC, 10 cm BC, 2x4 cm AC, SC	DS
30	Innerste	1966	W-Germany	20	35	38.500	1:1,75	ES, 8 cm BC, 5 cm AC, 10 cm BC, 2x4 cm AC, SC	DS
31	Ste-Cecile-d'Andorge	1966	France	12	45	8.000	1:1,7	10 cm BC, 2x6 cm AC, SC	
32	Upper Blue River	1966	USA	2,6	22	6.200	1:1,7	25 cm AC in 3 layers	Crest level + 3.582 m
33	Homestake	1967	USA	55	69	52.000	1:1,6	BC, 35-17,5 cm AC in layers of 9 cm	Crest level + 3.131 m
34	Rönkhausen	1967	W-Germany	1,3	27	9.000	1:1,8	ES, 4 cm BC, 7 cm AC, SC	Lower dam PS
35	Nagold	1967	W-Germany	5,5	31	8.000	1:2	ES, 6 cm BC, 4 cm AC, 7 cm BD, 2x4 cm AC, SC	DS
36	Kindaruma	1967	Kenya	16,7	28	14.700	1:1,7	ES, 6 cm BC, 4 cm AC, 8 cm BD, 2x5 cm AC, SC	DS

Table 3 continued.

No.	Name and year of completion		Country	Storage capacity $10^6\ m^3$	Maximum height m	Area of revetment m^2	Slope	Construction details	Remarks
37	Trapan	1967	France	1,3	24	6.000	1:2,5	8 cm BC, 10 cm BD, 2x5 cm AC, SC	
38	Magosawa	1967	Japan	0,9	13	8.000	1:3-1:2	5 cm AC, 5 cm BC, 6 cm BD, 10 cm protective stone	DS Irrigation
39	Otsumata	1968	Japan	1,8	52	11.000	1:1,8	3 cm BC, 4 cm AC, 13 cm BD, 2x5 cm AC, SC	DS
40	Villarino	1968	Spain	2,475	23	51.000	1:1,75	ES, 4 cm AC, 6 cm BD, 7 cm AC, SC	DS Wingdams to CC gravity dam
41	Salagou	1969	France	170	52	20.000	1:1,5	above + 102 m : 10 cm BC, 2x6 cm AC, 10 cm AC(2)	below + 102 m sloping asphalt core
42	Legadadi	1969	Ethiopia	38	21	12.700	1:1,55	12 cm gravel 20/40 5 cm ES, 3 cm BC, 2x6 cm AC, SC	Secondary dam
43	Pedu	1969	W-Malaysia	880	60	15.000	1:1,7	ES, 5 cm BC, 2x5 cm AC, SC	
44	Man zanares el Real	1969	Spain	40	40	23.100	1:1,75	ES, 3 cm BC, 5 cm AC, 8 cm BD, 5+6 cm AC, SC	DS(3)
45	Grane	1969	W-Germany	45	67	38.500	1:1,75	8 cm BC, 2x6 cm AC, SC	
46	Alesani	1969	France	11,3	65	13.000	1:1,7-1:1,6	10 cm BC, 2x6 cm AC, SC	(Corse)
47	Dungonnell	1969	N. Ireland	1,1	17	4.200	1:1,7	ES, 8 cm AC, 13 cm BD, 2x5 cm AC, SC	DS
48	Coo-Trois Ponts	1969	Belgium	8	20	9.000	1:2	above ground water : 1-5 cm BC, 6 cm BD, 6 cm AC, SC	Upper dam lower reservoir PS
49	Coo-Trois Ponts	1969	Belgium		25	12.000	1:2	below ground water : 1-5 cm BC, 4 cm AC, 6 cm BD, 6 cm AC, SC	Lower dam lower reservoir PS
50	Gijón	1969	Spain	2,5	15	14.500	1:2,35-1:1,95	BC, 6 cm AC, 12 cm BD, 12 cm AC, SC	DS
51	Aboño	1969	Spain		17	14.500	1:2,35	4 cm ES, 3 cm BC, 6 cm AC, 10 cm BD, 2x5 cm AC, SC	DS
52	Diga di Saretto	1969	Italy	0,25	13	1.500	1:2	25 cm Reno-matresses, grouted with 120+80 kg/m² sandmastic	Sealing of craked cc revetment
53	Ninokura	1969	Japan	28	37	7.000	1:2	4 cm AC, 5 cm AC, 10 cm BD, 2x4 cm AC, SC	DS
54	Poza Honda	1970	Ecuador	98	40(4)	25.000	1:2,5	5 cm ES, 5 cm AC, 8 cm BD, 2x5 cm AC, SC	DS(4)
55	Ponte Liscione	1970	Italy		60	50.000	1:2	6 cm BC, 6 cm AC, 10 cm BD, 2x6 cm AC, SC	DS
56	Ry de Rome	1970	Belgium		22	4.000	1:1,85	5 cm BC, 6 cm AC, 8 cm BD, 2x6 cm AC, SC	DS
57	Nidda	1970	W-Germany	7	33	16.000	1:1,6	4 cm ES, 3,5 cm BC, 2x4,5 cm AC, SC	
58	Vallon d'Ol	1971	France	2,8	45	16.000	1:2	10 cm BC, 2x6 cm AC, SC	
59	Obernaud	1971	W-Germany	14,9	60	28.000	1:1,93	6 cm BC, 4 cm AC, 10 cm BD, ES, 2x4 cm AC, SC	DS
60	Schömbach	1971	E-Germany		14	30.000	1:2,5	8 cm BC, 4 cm AC, 8 cm BD, 2x4 cm AC, SC	DS
61	Miyama	1971	Japan	22,0	67	41.000	1:1,85	ES, 15 cm AC, 20 cm BD, 2x6 cm AC, SC	DS, Irrigation

(1) For extra safety an internal core with reduced permeability was constructed.
(2) The upper layer of 10 cm AC has a protective function.
(3) This rockfill dam was built immediately downstream of an old concrete gravity dam.
(4) Toe of revetment is connected to top of bit. core, acting as cut off wall. Total height of sealing construction 60 m.

AC = Dense asphaltic concrete.
BC = Binder and/or levelling course.
BD = Bituminous drainage layer.
ND = Non-bituminous drainage layer.
ES = "Einstreu" (blinding with coated chippings).
SC = Bituminous seal coat.

ST = Surface treatment.
RT = Reflective surface treatment.
CC = Cement concrete.
CS = Cement stabilisation.
DS = Seepage detection system.
PS = Pumped storage scheme.

Table 4

Pumped storage reservoirs with asphaltic concrete membranes.

(After Visser, et al., 1970.)

No.	Name and year of completion		Country	Storage Capacity 10^6 m^3	Height of embankment m	Slope	Area of sealing in m^2 Slope	Area of sealing in m^2 Bottom	Construction details (S = Slope, B = Bottom)	Remarks (S = Slope, B = Bottom)
1	Reisach-Rabenleithe	1953	W-Germany	1,5	16	1 : 2	CC-slabs	8.800	B : reinforced bitumen seal + 12 mm asphalt mastic	B : 1 : 5 and 1 : 0
2	Geesthacht	1957	W-Germany	3,3	17-26	1 : 2,5	80.000	220.000	5 cm SA ; S : 7 cm Ac, SC ; B. 6 cm AC, SC	
3	Schwarzach	1958	Austria	1,5	26-34	1 : 1,75	40.000	35.000	ND ; S : 5 cm CS, 6 cm BC, 12 cm AC, SC B : ES, 6 cm BC, 12 cm AC, SC	
4	Leitzach	1960	W-Germany	0,6	6.5	1 : 1,75	CC-slabs	108.500	B : 1.5 cm SA, 3 cm coated chippings, 2 x ST	S : 20 cm CC-slabs
5	Vianden I	1962	Luxembourg	3,1	19	1 : 1,75	73.000	147.000	S : 9 cm BD, 3 cm ES, 7 cm AC, SC	
	Vianden II	1963	"	3,9	19	1 : 1,75	96.000	195.000	B : 16 cm granulated slag, 1 cm ES, 3 cm BD, 6 cm AC, SC	
6	Hieflau	1963	Austria	1,8	12	1 : 1,75	40.000	145.000	S : 5 cm BD, 6 cm AC, SC : B : 3 cm SA, 5 cm AC, SC	AC in one course
7	Taum Sank	1963	U.S.A.	5,3	33		CC-slabs	158.000	B : 2 x 5 cm AC	
8	Erzhausen	1964	W-Germany	1,5	17	1 : 2	65.000	105.000	S : 3 cm AC, 10 cm BD, 6 cm AC, SC ; B : 6 + 3 cm AC, 10 cm ND, ES, 6 cm AC, SC	DS
9	Glems	1964	W-Germany	0,8	21	1 : 1,75	37.000	33.000	S : 4 cm BC, 5 cm AC, 6 cm BD, 7 cm AC, SC ; B : 4 cm BC, 5 cm AC, R, 7 cm BD, 6 cm AC, SC	DS ; R
10	Eggberg	1966	W-Germany	2,0	25.5	1 : 1,75	63.000	70.000	S : 1-5 cm BC, 4 + 6 cm AC, SC ; B : 3-5 cm BC, 5 cm AC	
11	Rönkhausen	1967	W-Germany	1,0	18	1 : 1,8	35.000	70.000	S and B : 3 cm ES, 3 cm BC, 6 cm AC, SC	
12	Seneca	1968	U.S.A.	3,3	22	1 : 2	104.000	111.000	S : 7.5 cm BC, 2 x 4 cm AC, SC ; B : 2 x 4 cm AC	120,000 ton asphalt mix is used
13	Coo-Trois Ponts I	1969	Belgium	4,0	33-50	1 : 2	100,000	110.000	S : 10 cm BD, 5 cm BC, 6 cm AC, SC	
	Coo-Trois Ponts II	1972	"	4,0		1 : 2			B : 4 cm BC, R, 5 cm AC	
14	Turlough Hill	1973	Eire	2,1	22	1 : 1,75	66.600	86.600	S : 3-6 cm BC, 6 cm AC, SC ; B : 3-5 cm BC, 5 cm AC	
15	Ludington	1973	U.S.A.	102	40	1 : 2,5	600.000	Clay	S : 5 cm SA, 45 cm ND, 7,5 cm BC, 2 x 5 cm AC, SC	Slope lining reaches 7 m into bottom

AC = Dense asphaltic concrete.
BC = Binder and/or levelling course.
BD = Bituminous drainage layer.
ND = Non-bituminous drainage layer.
ES = "Einstreu" (blinding with coated chippings).
SC = Bituminous seal coat.
ST = Surface treatment.
RT = Reflective surface treatment.

CC = Cement concrete.
CS = Cement stabilisation.
SA = Sandasphalt.
R = Wire netting (e.g. polyester) reinforcement where differential settlements are expected.
DS = Seepage detection system.
PS = Pumped storage scheme.

concrete gravity dams of that time with the steep, for asphaltic concrete, slopes of 1:1 at the toe, steepening to 1:0.7 at the crest. The dam was constructed of rockfill faced with hand laid masonry. See Ref. 5. A porous cement drainage layer 8 cm thick was used to smooth off the face before the application of two 6 cm thick layers of asphaltic concrete. As protection against thermal and physical damage the asphaltic concrete was covered with a 10 cm thick facing of porous cement concrete, reinforced with wire mesh. The facing was suspended from a capping beam at the crest. The dam performed satisfactorily until 1953, when the corrosion of the wire mesh caused the porous facing to fail. The remaining facing was removed and the dam painted with white reflecting paint as a thermal protection. The white paint controlled the temperature to 16°C lower than if the surface remained black. This surface performed adequately, despite no maintenance during the Algerian uprisings, until 1970 when a new facing was designed in cement concrete. The original porous drainage layer was retained together with the lower unaged portion of the asphaltic concrete. El Ghrib was ahead of dams in Europe by 18 years. Two other dams, Bou Hanifa and Il Emda, 54 and 75 m high respectively, were constructed in Algeria in the meantime.

The first major German dam, Genkel, 1952, 43 m high, was compacted mainly by hand. The asphaltic concrete was spread by machine and compacted by vibrating heated plates. The machinery was developed further for Henne dam, 1955, 58 m high. A heavy stamper attached in front of a spreader by a beam compacted the asphaltic concrete. The spaces between the large stones of Genkel and Henne dams were filled with porous cement concrete to provide a smooth surface for the asphaltic concrete. Bigge dam, 1965, 55 m high, had a levelling layer 50 to 150 mm thick of crushed limestone compacted by an 8 t grid roller. Table 5, gives some details of early German dams of comparable size and shows the development of the asphaltic concrete design.

Asphaltic concrete has been used where the foundations are not directly onto rock. Current practices may include one or more of the following design innovations.

1. As an added precaution against leakage into the dam fill, membranes have been constructed in sandwich form, using an additional drainage layer over the first impermeable layer. The drainage layer is then covered with a second layer of asphaltic concrete.

Table 5

Early German asphaltic concrete dams.

Dam	Genke Dam 1952	Henne Dam 1955	Bigge Dam 1964
Height	43 m	58m	55m
Slope	1:2.25	1:2.07	1:1.75
Mastic	5 kg/m^2	2 x 3 kg/m^2	3 kg/m^2
Refined asphaltic concrete	3 x 3 cm	3 x 3 cm	2 x 6 cm
Asphaltic binder	2 cm	–	
Precoated chippings	–	Unknown	1 cm
Precoated stones	10cm	10 cm	11 cm
Asphaltic concrete	2 x 3 cm	2 x 3.5 cm	6 cm
Asphaltic binder	6 cm	6 cm	4 cm
Levelling cement or asphalt course	Cement	Cement	150 mm crushed limestone with bituminous binder.
Bitumen content concrete	8.1%	8 – 8.4%	8 – 8.4%
Voids	3%	3%	0.2 – 2%
Largest grain in aggregate	8 mm	8 mm	12 mm
Filler content	18%	14.2%	13.5%

2. The membrane has been laid as two layers of dense asphaltic concrete. However, blisters caused by the separation of the two layers are eliminated by the use of one thick course of asphaltic concrete.
3. The sandwiched drainage layer may be separated into strips by watertight partitions at intervals. The partitions are formed at the edges of the asphaltic concrete strips as they are laid. The separated drainage layer then can be connected by ducts to a toe drain or drainage gallery so that any leak through the outer membrane can be located.
4. Due to the success of asphaltic concrete in being completely watertight on a number of dams, there has been a tendency to leave out the drainage layers and, provided the dam is suitably protected, no other measures have been taken.

The first use of thin sheets of plastic for forming an impermeable element of a dam was probably for the 61 m high, Terzaghi Dam, Ref. 6, in 1960. The sheets were only laid over part of the dam as a secondary defence against piping. The first dam with an upstream thin sheet membrane was the 10 m high, Dobsina dam in Czechoslovakia. Hobst, 1961, Ref. 7, describes its satisfactory performance after 14 years, Ref. 8. The ICOLD Committee on Materials has produced Bulletin No. 38, which lists thin membrane types and their use on fill dams. The use of thin membranes is only superficially examined in the rest of this report as their use has been mainly confined to low water retaining structures.

Complete failures of upstream membrane faced dams have been uncommon. Five failures are reported by ICOLD, Ref. 9, and have been summarised in Table 6. All the cases gave warning of failure and the reasons for failure are not entirely due to defects in the membrane. Two of the dams leaked significantly but were repaired to give adequate service.

Other materials for impervious membranes, steel, P.V.C., plastic, rubber and combinations, have been used with some success. Steel has the longest history and has been used on a number of dams. Sherrard, Ref. 4, describes eight that have performed satisfactorily. Table 7 summarises the essential details. Steel and fabrication costs have risen faster in proportion to asphalt and concrete with the result that this form of dam has fallen out of favour. One arch dam in Italy, Ref. 10, has used steel as a complete facing as a remedial measure. One recent example of the use of steel for a membrane is included in

Table 6

Failures of dams with upstream membranes.

Dam	Country	D.O.C.[1]	D.O.F.[2]	Membrane Details	Mode of Failure
Beaver Park	U.S.A.	1914	1914	Reinforced Concrete	3.7 cumec leak.
Swift	U.S.A.	1914	1964	6in. to 2ft. of reinforced concrete	Overtopped
Cogswell	U.S.A.	1934	1934	6in. of concrete	Settlement on wetting, 4.3 m. 3.5 cumec leakage.
Nhzhne Tulomskaya	U.S.S.R.	1938	1938	Asphaltic concrete	While under construction the slopes slid during hot weather.
Baldwin Hills	U.S.A.	1951	1963	6in. Asphaltic concrete	Fault movement ruptured the membrane and the drains failed. Dam breached.

[1]Date of Construction.
[2]Date of Failure.

Table 7

Steel faced dams as reported by Sherrard. (1963)

Dam	Height m	D.O.C.[1]	Plate details	Slope	Remarks
Ash Fork, Arizona.	14	1898	3/8 in. riveted		In a very dry area.
Skaguay, Colorado.	23	1900	1/2 to 3/8 in. at mid height	60°	Two cleanings, chippings and painting up to 1963.
El Valdo, New Mexico.	53	1943	1/4 in.	1:1.5	Rolled gravel dam. Some wrinkling and buckling at the abutment situated on old slide debris.
Crystal Creek, Colorado.	28	1936	1/4 in. copper bearing		Minor rust pitting.
South Catamount, Colorado.	27	1936	1/4 in. copper bearing		Minor rust pitting.
Krahn, U.S.S.R.	34	1935	6 mm	1:1.5	Curved up expansion joint. Emptied at least once a year.
Salazar, Portugal	64	1948	5/16 in.lower 1/3 1/4 in. above	1:1.25	Dumped rockfill in 2.75 m lifts. Crest settled 405 mm in 10 years with 355 mm downstream movement. 11 l/sec leakage initially.
Rio Lagaritijo, Venezuela.	24	1958	1/4 in.		Large settlements were expected. Cathodic protection.

[1] Date of Construction.

the section, Case Histories.

In this report asphaltic concrete is taken as being a material composed of natural aggregates with a bituminous binder derived from oil. This definition is, however, at variance with the publication, Bulletin No. 32, Bituminous concrete facings for earth and rockfill dams, produced by the ICOLD Committee on Materials. In this publication the term bituminous concrete was adopted to have the above definition. The General Reporter to Q.42 at the 11th ICOLD, 1973, Madrid, used a similar definition to that of this author.

3. ADVANTAGES AND DISADVANTAGES

The requirements for adequate performance of an impervious element of a dam are that:

1. It must be watertight against the maximum water pressure that may develop.
2. It must be non-erodable if a leak should develop or in the case of internal elements, must be protected from piping.
3. The physical properties of the membrane must be able to meet the imposed stresses and deformations occurring under the working or construction conditions without rupture.
4. It must be able to be applied under the given construction conditions.
5. It must retain the above properties for the working life of the dam.

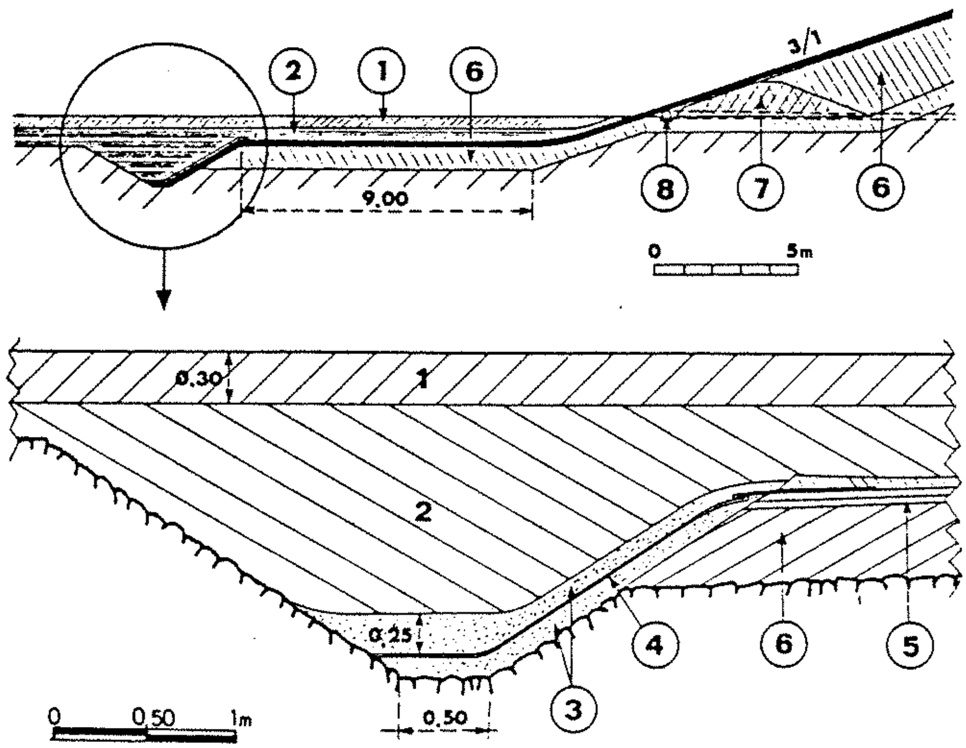
It will be seen from the case histories in Section 5 that asphaltic concrete, concrete and thin membranes all meet conditions 1 and 2. Most recent designs are able to satisfy Condition 3 although it is the inability of the membrane to follow the deflections of the rockfill that has caused many of the reported leakages. Condition 5, durability, as will be seen in Section 5, has not been a great problem and the majority of membranes perform adequately. See Table 6 summarising the failures reported by ICOLD. Failures due to membrane rupture occurred almost immediately after construction.

The locations where upstream membranes could be favourably considered are where:

1. There exists an impermeable foundation within reach of current technology.
2. Suitable fine, impervious materials are not locally available for the construction of internal impervious zones.
3. Construction is required to be continued in periods of wet or cold weather or when rapid progress can be made in short periods of good weather.
4. Differential and total settlements of the rockfill and the foundation are within the acceptable limits of current practice for upstream membranes.
5. The savings in cost of the auxiliary structures are maximised by the use of upstream membranes.

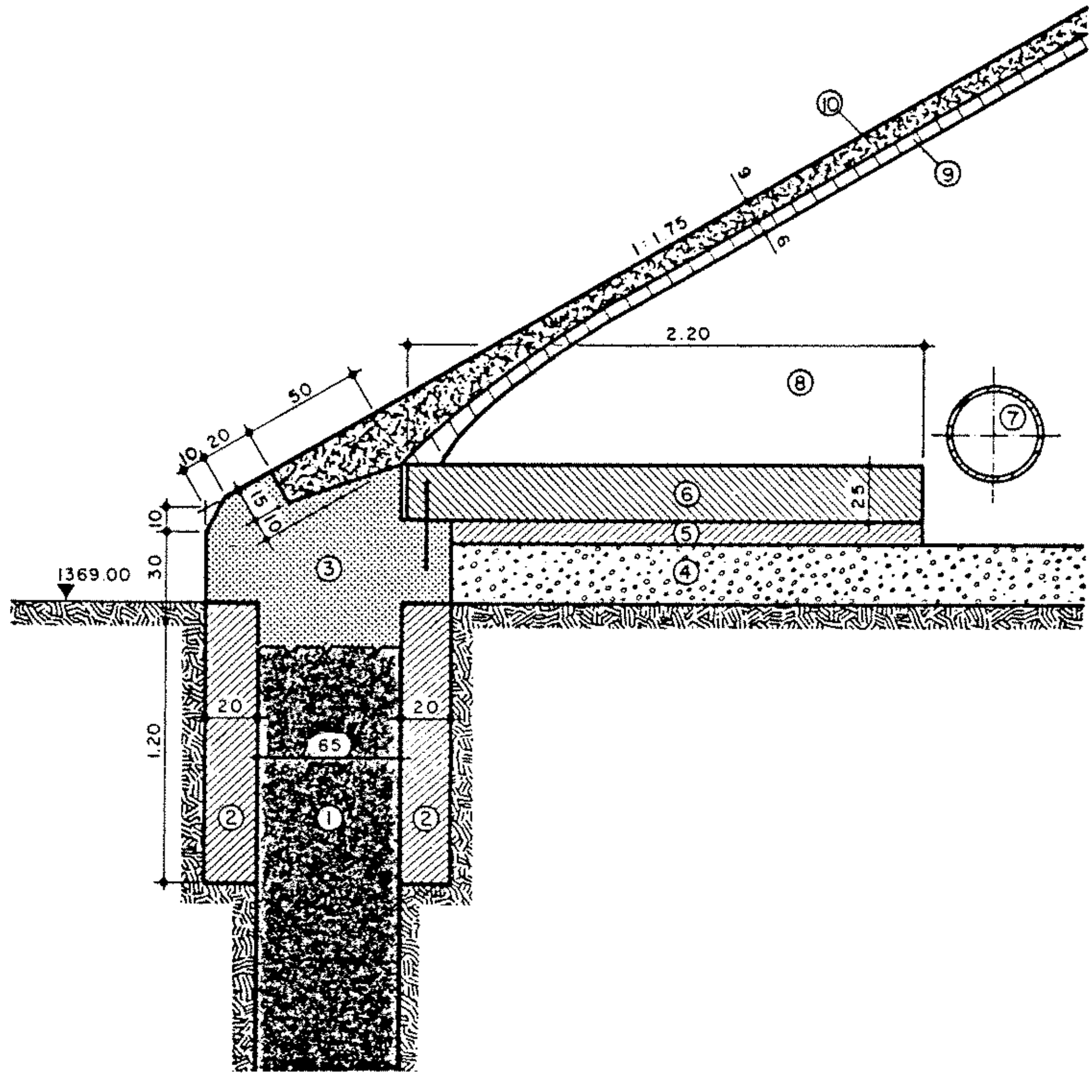
Condition 1 is necessary to enable a suitable cutoff system against underseepage, to be constructed at the upstream perimeter of the dam. Hard rock is most suitable for the foundations of the perimetral plinth. However, upstream membranes have been used in situations where alluvial material forms the foundation. In this situation a positive cutoff located at the upstream toe of the dam is essential to control seepage and prevent uplift. Cutoff structures have been constructed by, slurry trench walling, excavating for a cast in place concrete wall, sheet piling and the variations of piling, and grouting. The cutoff structure then forms a solid base for the plinth. The designer will have to ensure that the settlements of the compressible foundation and the relatively incompressible cutoff are compatible. However the situation is eased by the fact that small settlements, compared to the centre of the dam, occur at the perimeter. Foundation spreading causing shear failure of the cutoff could be a problem. Concrete faced dams have rarely been used where the depth of the alluvium is greater than a few metres. Asphaltic concrete facings have been used on alluvial foundations with appropriate design. Examples are Godey, Switzerland with 20 to 30 m of alluvium and Zoccolo, Italy with up to 100 m of alluvial material. See Section 5.2.2 for details of the foundation conditions at Zoccolo dam and the special design features at the cutoff. It may be advantageous to situate the cutoff at a distance upstream of the embankment toe and then continue the membrane to the cutoff. See Figure 1 as an example of this layout. Additional movement can be accommodated with an intermediate slab as on Figure 29. An arrangement made at Godey dam is shown on Figure 2.

The rockfill is required not to deform significant amounts under the action of the water loads. This is generally achievable by compacting small lifts on the upstream side, and where the deformations are not so critical thicker lifts are allowed. Construction during wet weather does not affect the degree of compaction obtained in lifts not containing large amounts of fine material. The upstream zone or the bedding zone to the membrane generally has a greater proportion of fines so that the placing of this material might have to stop in periods of heavy rain. The high permeability of rockfill ensures that ponding of water does not occur. Experience has shown that dry rockfill can be placed even in cold weather. Ref. 11 and 12. The fill may be placed in the less sensitive downstream zones thus avoiding any building of snow or ice into the upstream zone. The construction of the membrane can be scheduled to make best use of summer or dry conditions.



1 Protection layer of schists
2 Impervious fill
3 Clay layer
4 Butyl sheet pinched between the two impervious layers (DBC)*
5 Filter (bituminous concrete)
6 Shell of schistous rocks
7 Drain
8 Drainage pipe ∅ 200 mm

Figure 1. Revin, Connection between facing and reservoir bottom.
(After ICOLD Bulletin No. 39, 1981.)



Connection between the slurry trench and the bituminous facing.

(1) *Slurry trench.*
(2) *Guide walls.*
(3) *Cap wall.*
(4) *Levelling fill.*
(5) *Levelling concrete.*
(6) *Transition slab.*
(7) *Drainage pipe, ⌀ 400 mm.*
(8) *Drainage layer.*
(9) *Pervious bituminous concrete.*
(10) *Impervious layer.*

Figure 2. Godey, Connection between the slurry trench and the bituminous facing.

(After Schenk, 1979.)

The prediction of the deformations of the rockfill and foundations is still unreliable despite the use of Finite Element Methods (F.E.M.). It is, therefore, difficult to design joints in the membrane and design is still largely empirical. Condition 4 is most easily satisfied by compacted rockfill constructed directly onto a rock foundation. In this case the inevitable differential movements between the plinth and the rockfill can be accommodated by the perimetric joint and remain watertight. With a decrease in stiffness of either the foundation or the rockfill the design of the perimetric joint becomes more difficult as tensile and shear forces are developed across the joint.

The slopes of membrane faced dams are generally steeper than those of central core dams. Thus the length of the auxiliary structures such as diversion tunnels, spillways or riparian outlets are reduced. The cost savings satisfy Condition 5.

Apart from the problems of dissipating wave energy and wave runup, membrane faced dams do not need extra wave protection. The selection, placing and maintenance of the correctly sized rocks for riprap is expensive and savings here may well offset any additional costs of a membrane faced dam.

Unlike earth cores which pre-stress the shoulders of the dam by horizontal pressure greater than those from the reservoir during filling, Penman, Ref. 13 and Ref. 14, upstream membranes transfer load to the fill only as the reservoir impounds. This fundamental difference in action gives the upstream membrane advantages over an impervious core. These advantages are that:

1. It gives the dam greater stability against shear failure by providing an additional downward component from the water forces.
2. The greatest possible mass resists the water pressure compared to the core dam where only half of the dam's mass resists horizontal water forces.
3. There is greater resistance to seismic loading. Having the reservoir pressure upstream of the total mass of the dam appears to be beneficial.
4. The upstream shoulder is not saturated with water. This reduces its earthquake liquefaction potential and ensures a higher operating effective friction angle.

The advantages and disadvantages of an upstream membrane compared to a zoned dam, some of which have already been mentioned, are listed below:

Advantages	Disadvantages
Rockfill can be placed in most weather conditions. The membrane can be added later in good weather.	Impounding cannot start until the membrane is complete. The economics of water control may require completion in stages.
The membrane prevents seepage from entering the dam. The dry strength of rockfill is greater than the wet strength.	Seepage if it occurs can cause additional settlement.
Drains can be provided to intercept seepage.	Drain malfunction could cause stability problems.
The membrane is accessible for repairs.	It is susceptible to mechanical damage and damage by ice or terrorism.
It is flexible enough to cope with normal deflections of rockfill without rupture.	It requires good constructional control of the rockfill, bedding and joint construction.
It is not subject to erosion if leaks occur.	Initial deflection of the dam upon impounding can cause leaks.
Location of any leaks can be pinpointed if appropriately designed drains are used.	
It may have a self-healing leak capacity. Asphalt and concrete are both able to heal small leaks.	It cannot be relied on.
	Foundation spreading can cause shear failure of the cutoff.
	After rapid drawdown or at low operating levels uplift forces may be generated under the membrane.
With higher dams the cost of specialist equipment for use on the facing is small compared to the total cost.	Specialist, expensive equipment is required to place the asphalt and concrete membranes.
The crest may be made narrower and the overall width may be less than an equivalent cored dam.	
Auxiliary works may be shorter or cost less as the overall width of the dam is reduced.	

Less site preparation is required, i.e. only the plinth foundation need be cleaned off to sound rock. The need for key trenches and prepared areas for blanket or finger drains is removed.	
	Shorter working life especially with rubber, some asphaltic concrete mixes and steel.
Wave protection is a secondary function of the membrane.	Riprap is visually more pleasing.
	It could suffer from poor workmanship while core dams can suffer some abuses without their performance being adversely affected.
Creep can relieve stresses in the membrane.	It can suffer from long term creep of the membrane or rockfill or both.
	The slopes cannot be locally flattened over poor foundation conditions.
Rockfill can act as its own cofferdam in combination with a low upstream cofferdam to permit riverbed toe slab construction. The partially completed rockfill with a semi-pervious upstream zone but without its upstream membrane can be used to store flood water temporarily.	

3.1 Cement concrete membranes.

This type of construction often enables the use of steeper slopes than those possible with asphaltic concrete. Asphaltic concrete is subject to long term creep deformations which probably precludes its use on the steeper slopes. Concrete slipform pavers are able to operate on almost vertical slopes. Asphaltic concrete pavers need to work and compact the asphaltic concrete before it is rolled and as a consequence require flatter slopes to operate. In addition the efficiency of rollers is severely reduced on the steeper slopes.

Relief dam built in 1910 with a reinforced concrete facing is still performing satisfactorily. (See Table 1). Other dams as reported by Sherrard, Ref. 4, have their membranes in excellent condition after many years. The addition of an air entraining agent will enhance the durability. Sulphate attack is not considered to be a major problem as reservoir water rarely contains significant amounts of sulphates. The cracking of the concrete exposing steel to attack by water and air is a constant threat. Thermal cracking, mechanical damage and cracks due to tensile and bending forces can be predicted. There is a considerable depth of experience to help the designer. Concrete is generally resistant to damage by falling rocks, floating ice or debris without special provisions.

Cement concrete construction has the advantage over asphaltic concrete in that only one pass of the paver is required to complete the membrane. Preparation work required before the membrane can be concreted includes, compacting and levelling the bedding, blinding the surface with concrete or bitumen, laying the reinforcing steel, setting the sideforms and waterstop, setting the paver rails to the correct line and level. Construction of the asphaltic concrete membrane also includes compacting the levelling layer and blinding the surface, but then the sequence of work is, lay the impervious layer, lay porous asphaltic concrete as drainage, lay one or two layers of impervious asphaltic concrete and then the surface is sealed usually with a sand bitumen coating. Asphaltic concrete thus requies a lot more work on the slope of the dam, and must cost relatively more.

Concrete membranes are able to accommodate some deflection of the fill without cracking. For example a 500 mm deep slab 10 m wide can deform 5 mm in the centre without cracks developing. Consideration of the mode of deformation of the membrane on a rockfill dam under the action of the water forces will show that the cracks would form on the downstream side of the membrane. See Figure 3 for the deflected membrane shape of a 44 m high dam with a 525 mm membrane tapering to 300 mm on a rockfill dam with a modulus of deformation of 120 MPa. The upstream side of the membrane will everywhere be in compression. If cracks on the downstream side of the membrane are permitted and if the impermeability of the concrete is sufficient then larger deformations are allowable. For the same slab, 10 mm of deflection could occur before the cracks would have penetrated 200 mm into the slab. Taking the concrete to the limit of its compressive strength, say 25 MPa., a deflection of 60 mm in the centre of the slab would occur before failure. Provided no shear deformations occurred the concrete could still remain watertight as the water face would be in compression. Cracking of the

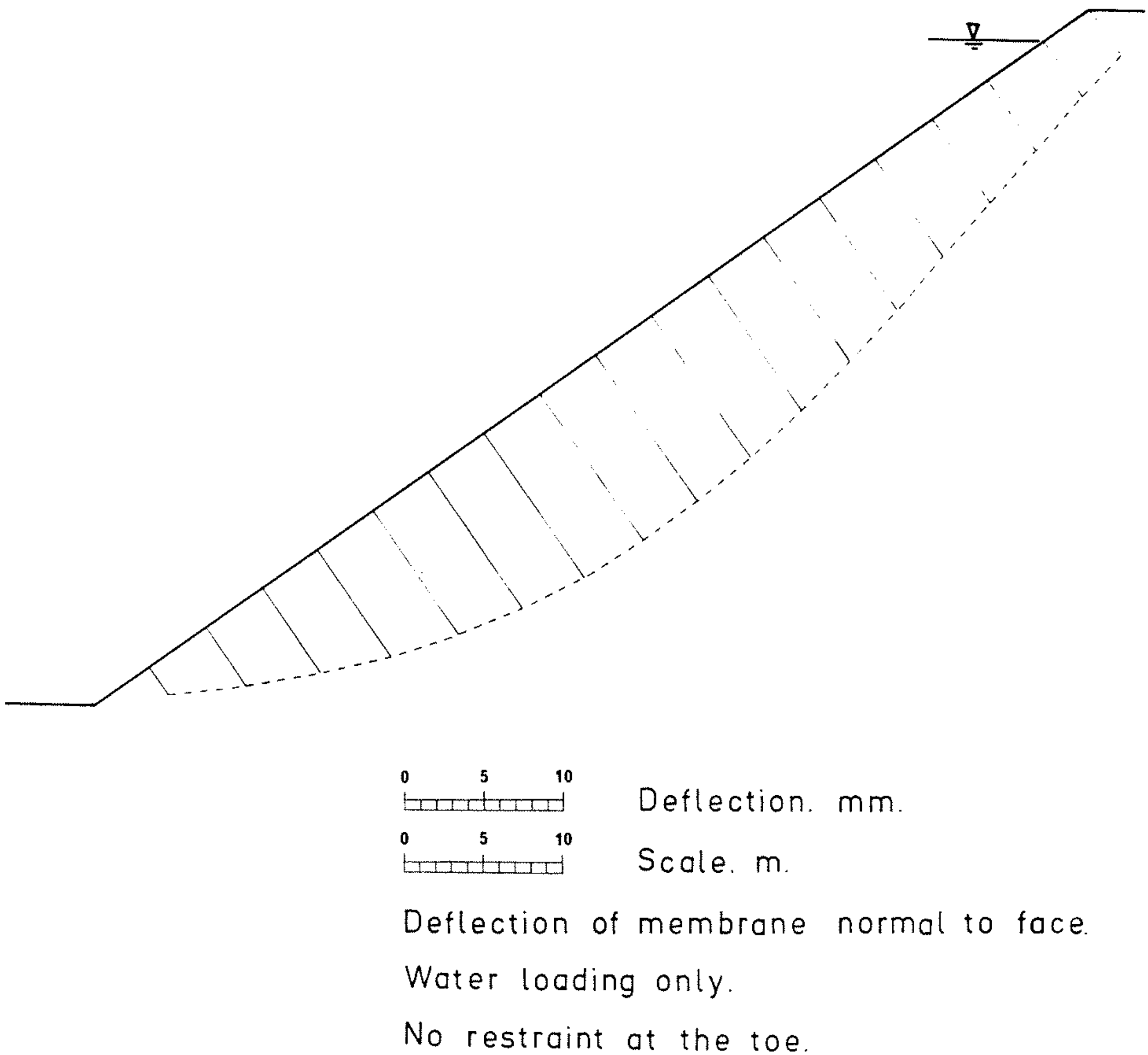


Figure 3. Deflection of an upstream membrane under the action of water forces.

downstream face should not be a problem if the fill is essentially free draining or if no leaks occur. Cracks penetrating as far as the steel reinforcement could eventually cause failure of the slab by spalling of the downstream face due to the corrosion pressures of the steel. The pressure from the spalling and corrosion could eventually cause cracks to appear in the upstream face. However, leaking cracks are not necessarily disastrous if the fill does not settle further on wetting or erode.

Differential movements within the fill are potentially disastrous for the integrity of a concrete membrane. The Scotts Peak dam in Tasmania, Ref. 15, developed severe leakage when differential movements occurred. The repair is described in Section 6. The differential movements occurred between the gravel of the first stage and the main body of the dam.

The cost of materials for a cement concrete membrane is probably greater than that for asphaltic concrete due to the differences in the volumes of each material used. The total cost of the dam will depend on such outside factors as the availability of bitumen or cement, distance from the supply, availability of suitable aggregates and the availability of specialist contractors. An economic assessment of each damsite is needed and the final choice of membrane may not entirely depend on the relative costs of cement or asphaltic concrete.

3.2 Asphaltic concrete.

This type of construction has been used extensively for reservoir lining. The lining can be laid of the same material at the same time with the same plant as the dam facing. One of the main advantages that asphaltic concrete has over cement concrete is that large amounts of asphaltic concrete can be laid relatively quickly. A large body of experience in the use of asphaltic concrete has been built up by specialist asphaltic concrete contractors, particularly in Germany. The experience has been obtained from the use of asphaltic concrete on roads where laying up to 5 miles of pavement per day is common.

Asphaltic concrete is more flexible than cement concrete and should be able to follow the deformations of the rockfill without rupture. It can be seen from Table 8, summarising the results of a simple parametric study, that decreasing the stiffness of the membrane reduces the stresses. The advantage of lower stresses in the membrane is to

Table 8

Summary of Finite Element Parametric Study.

Variation of Stiffness.

Water Loads Only.

Membrane Stiffness MPa	Bedding Stiffness MPa	Rockfill Stiffness MPa	Maximum Horizontal Deflection mm	Maximum Vertical Deflection mm	Maximum Stress in Membrane MPa
20700	180	120	5.03	0.69	4.578
	120		5.12	0.64	4.694
	60		5.06	0.55	5.022
10000	180	120	5.96	0.21	3.026
	120		6.03	0.14	3.106
	60		6.11	0.00	3.314
5000	180	120	6.65	0.01	1.973
	120		6.74	0.08	2.012
	60		6.89	2.40	2.185
1000	180	120	7.14	0.15	0.522
	120		7.20	0.18	0.564
	60		7.29	0.26	0.663
500	180	120	7.19	0.17	0.216
	120		7.24	0.19	0.286
	60		7.29	0.24	0.350
20700	240	240	3.04	0.09	3.169
20700	60	60	8.23	2.69	7.047

Dam modelled as 44.5 m high, slopes 1:1.6 upstream, 1:1.5 downstream.
Membrane 525 mm thick tapering to 300 mm at crest.
Water level 2 m below crest level.
Base of membrane restrained horizontally and vertically.
Linear elastic F.E.M.

reduce the possibility of cracking, creep and fatigue related failure from cyclic water loadings.

A further advantage of the flexibility of asphaltic concrete is its ability to self heal leaks. Although cement concrete possesses this ability to a slight extent the sealing of leaks by asphaltic concrete is remarkable. The NGI tested pieces of asphaltic concrete, cut from a test dam, in a permeameter under a water head of 60 m. The permeability dropped from 1×10^{-6} cm/sec to 1×10^{-9} cm/sec during the six months of the test. Other evidence comes from the engineers designing Iril Emda in Algeria. Ref. 16. Various sized holes in the asphaltic concrete were made and under pressure the permeability dropped. At a head of about 50 m all the holes were nearly completely self healing.

Repairs have been relatively easy to make with the new section being bonded to the old mat by heating the old surface with infra–red prior to the placing of the new layer. However the surfaces must be dry. One common repair has been to replace the material above the water level where the black surface heats up excessively causing rapid ageing. Ageing reduces flexibility, increases shearing strength and compressive strength making the material brittle. The greatest damage occurs where the asphaltic concrete has not been well compacted, especially where the paver has not been able to work close to the toe or parapet.

There have been several cases where the formation of blisters on the face of the membrane has caused concern. The blisters have formed at the interface between the two surface layers of asphaltic concrete. Water vapour, or spilt fuel, penetrates the asphaltic concrete from below and becomes trapped at the boundary of the layers. A rise in temperature, perhaps the result of the lowering of the water level, causes the liquid or vapour to expand. The pressure separates the layers. Steffan, Ref. 17, claims that on a 1:2 slope the pressure can lift a 9 cm thick layer. The blisters are difficult to repair as the problem will affect the whole dam. Replacing the whole of the impervious surface with one layer is the only long term solution.

An efficient monitoring system, separating the pervious drainage layer into sections, has proved effective in pinpointing leaks. Few concrete faced dams have been reported with such accurate monitoring systems.

Joint detailing between strips is easier than for cement concrete. Asphaltic concrete will seal to the adjoining strip and assisted with heat or a bitumen tack coat, the joint is easily watertight. Joints have also been bevelled to aid in adhesion. Staggering the joints in the layers is also effective in reducing the potential for leaking. The transition between ground and slope should be of such a radius that the paver may adequately place and tamp the asphaltic concrete or that the rollers may compact the asphaltic concrete. The design of the top of the slope should ensure that both the paver and the rollers can operate efficiently. See Bulletin No. 39, published by ICOLD, for details of some of these connections at recent dams. Layout of the strips becomes difficult in steep sided valleys. The space needed to start the paver may necessitate uneconomic excavation.

3.3 Thin film membranes.

Thin film membranes generally have been used on dams less than 25 m high, although there are some dams up to 60 m high using them. Their application is especially suited for low dams where the expense of sophisticated heavy machinery is not justified. The advantages and disadvantages are noted below:

No expensive, heavy machinery is required.	Care is needed to prepare the surface for the membrane to avoid any projections that could cause punctures.
Welds or joints can be made by unskilled people with a minimum of equipment.	Joints are weak spots and are prone to leak.
Simple methods can be used to fasten the membrane to the slope. Concrete blocks or bolted plates are common methods.	They require protection from rock, weather and drifting objects. They are easily damaged during construction.
	They age rapidly when exposed to sunlight or heat.
They can be manhandled into place.	High winds cause difficulty in laying. Flat slopes are required for the safety of the workmen.
Defects can be spotted by eye. Repairs are relatively easy to make.	They require reservoir lowering for inspection and welding or vulcanising of repairs.

They have much more flexibility to follow
the deflections of the embankment.

The cost of thin film membranes may be a fraction of the cost of the project compared to that of asphaltic concrete or cement concrete. However, the total cost taking into account the preparation and the disadvantages may have precluded their use on higher dams. They are used as a repair membrane for other forms of dam construction, and for temporary dams.

4. CONSTRUCTION PROCEDURES

4.1 Construction of concrete membranes.

Originally concrete facings were placed by hand in relatively small bays often separated by timber partitions which were left in place. The early use of concrete facings is described by Galloway, Ref. 2. The introduction of slipforming the concrete membranes occurred about the same time as the change to compacted rockfill from dumped rockfill.

A slipform paver requires a guide system for both line and level. This is usually provided by rails either bolted to the adjacent completed slab or set in concrete blocks cast in the rockfill. The slipform is moved up the slope by double acting jacks pulling on steel bars or cables anchored at the crest. The use of fully automated hydraulic jacks enables a constant rate of travel to be maintained. The paver is ballasted and provides working space above the concrete for placing and vibrating the concrete. Below the paver, provision is usually made for concrete finishing to be carried out. Water is often used as a curing agent and is sprayed on the concrete until the reservoir is impounded. Surface hardeners can also be applied from the working platform.

The slow progress of the paver means that large volumes of concrete do not need to be batched. The output of a small mixing plant close to the crest of the dam is satisfactory. A backup supply should, be provided and one of lesser capacity will often be sufficient. The delivery of the concrete to the paver is best done by a skip running on the same rails as the paver. Concrete has been delivered by pump or "elephants trunk" ducting. Both of these methods suffer from the possibility of segregating the concrete or loosing grout. Pumps are also not very efficient at the low rates of supply required for slipforming.

The whole sequence of compacting the bedding, levelling, laying rails, building stop ends, laying reinforcement, concreting and moving the paver needs to be carefully planned to avoid congestion. The use of a transfer gantry, itself running on rails on the crest of the dam, enables the paver to be transferred sideways without the use of a crane. In addition the gantry enables the paver to lay the facing right up to the crest of the dam. A gantry for the placing of the sheets of reinforcement was used at Cethana dam, Australia, removing the need for building the reinforcement cages on the slope. Reinforcement cages were made up off the dam in a less congested working area. Ref. 18.

Vertical joints often have two waterstops, one copper at the base of the slab and one of rubber at mid-depth of the slab. Some dams have had horizontal steel passing through the vertical joints. Joint fillers have also been used. The vertical joints can become very congested areas, involving a lot of labour and time in setting up. Simple detailing will ensure that the construction is relatively easy and less liable to error or faulty workmanship.

Odd sized panels are needed around the perimeter of the face to connect to the plinth. The use of a small slipform paver speeds up progress here. The layout of the panels will depend on the particular geometry of the site.

The plinth is usually constructed ahead of the rockfill so that the grouting operations can continue using the plinth as a working platform. Cutoffs to control leakage through the foundations are also constructed ahead of the membrane so that connection can be made when the membrane is started.

4.2 Construction of asphaltic concrete membranes.

Asphaltic concrete was originally laid by hand but the extensive use of asphaltic concrete for roadworks necessitated the development of sophisticated machinery. The asphalt paver in one pass is able to spread a 2 to 3 m wide strip of uniform thickness. The thickness of the layer is controlled by automatic sensing devices sliding on either the adjacent strip and the underlying layer or on wires laid to the correct profile. The spreader consists of a hopper from which the mix is conveyed by screw conveyor and distributed evenly across the strip. Levelling and smoothing is done by a heated vibrating screed. Provision is usually made for heating the edge of the adjacent strip prior to placing the asphaltic concrete. The paver is winched from an operating platform at the crest of the dam at a rate of about 1 m/min. The platform is also equipped to enable the paver to continue to place asphaltic concrete right up to the crest. The platform also transports the paver sideways from one strip to the next.

Compaction is generally done by at least two rollers winched either from the spreader or from the crest. Additional rollers operating on the completed strip complete the required number of passes. Vibrating rollers are frequently used for compaction. Vibration should only be applied while the roller is going uphill to avoid creating waves of asphaltic concrete in front of the roller. Details of the weights of rollers, mixes used and placing temperatures have been tabulated by ICOLD's Bulletin No. 32, for a large number of dams. Table 9 presents these details for a selection of dams. The number of passes required for proper compaction depends on the composition of the mix and roller weight. The temperature and consistency of the sub-grade also affects the degree of compaction obtained. A fluid is often sprayed onto the drums of the rollers to prevent adherance of the asphaltic concrete to the drum. One of the cheapest and most effective fluids is diesel fuel. This causes softening and deterioration of the asphaltic concrete and can assist in the formation of blisters. It should not be used.

To improve the watertightness of the vertical joints they can be preheated. Other additional steps can be taken. These are:

1. Profiling the joint to increase the surface contact area.
2. Spraying a tack coat of hot bitumen or bitumen emulsion.
3. Staggering construction joints between the layers.
4. Reheating and recompacting a strip 30 to 50 cm wide along the joint. This can be done by hand held heaters and compactors.

The hopper capacity has been made large in an attempt to hold sufficient material to complete one strip with one filling. However, as dams have got larger continuous methods of refilling have been developed. These range from the simple winching of a dump truck down the slope to insulated containers controlled from the crest.

A non-destructive testing program for the asphaltic concrete facing using nuclear methods is advisable. At Henne dam the taking of cores for testing from the asphaltic concrete has led to leakage of the patches. Ref. 19.

4.3 Rockfill construction.

The original method of construction, end dumping, produced large settlements on inundation. Using large volumes of water for sluicing reduced the settlements. The use of thin, well compacted rockfill is more successful in producing a stiff fill. Fines are left in the rockfill during this construction method and reduce the contact stresses between the

Table 9

Construction details of some asphaltic concrete membranes.

Dam	Tackcoat or Stabilisation cm	Blinding Levelling cm	Secondary Impervious Course DBC cm	Drainage Layer cm	Binder Course cm	Primary Impervious Course DBC cm	Surface Protection cm
Sarno				10		2 x 4	RT
Zoccolo	Pen BE			8 - 9		5+4+4	SC
Bigge		BC 3 - 7	6	11 Pen B	ES	2 x 6	0.5 BMx
Dungonnel		BC	7.5	12.5		2 x 5	SC
Ogliastro				8		6	SC
Ninokura	M 4		5	10		2 x 4	SC
Pesti		4.5		3		2 x 5	SC
Luddington		SB 5	7.5	ND 45	6	2 x 6	SC
Miyama	M	ES 3.5	6	8		2 x 6	SC
Godey	Pen B				6	6 - 9	
Horch-wurten					8	8 - 12	
Pla de Soulcem					10	2 x 6	
Le Verney					10	2 x 6	

BC	Binder course	B Mx	Bituminous mastic
DBC	Dense bituminous concrete	ES	Blinding with coated chippings
M	Macadam	ND	Non bituminous
Pen BE	Penetration with bituminous emulsion	Pen B	Penetration with bitumen
RT	Reflective treatment		
SB	Sand bitumen	SC	Bituminous sealing coat

rocks. The processing of the rockfill to remove the fines only to have the rock crush under the action of the rollers to produce its own fines is clearly a waste of time. The strength of the rockfill is reduced by wetting and by increases in confining pressure. Charles and Watts, 1980, Ref. 20, have shown that the shear strength of rockfill, τ, can be described by a power law.

$$\tau = A(\sigma')^{b}$$

where A and b are constants,

and σ' is the effective stress.

This relation is used to define the factor of safety of an infinite slope:–

$$F = \frac{A}{\gamma^{(1-b)}} \frac{\cos\theta^{(2b-1)}}{\sin\theta} \frac{1}{z^{(1-b)}}$$

where γ is the bulk density

θ is the angle of the slope

z is the depth of the slide surface

For a finite slope the factor of safety is higher indicating that steeper slopes could safely be used for compacted rockfill. De Mello, 1977, Ref. 21, remarks that early dams built of dumped rockfill in the U.S.A. generally had much steeper slopes than those considered necessary for later dams built of compacted rockfill. This is surprising considering that there are no records of instability of dumped rockfill dams, compacted rockfill has superior strength compared to dumped rockfill, and the stability of a dam on a firm foundation depends solely on the shear strength parameters of the rockfill.

Although 50 to 90 m high dams have been formed in the U.S.S.R. directly by blasting, the most modern accepted procedure is with large earthmoving equipment and vibrating rollers. Rockfill material containing plenty of fines is brought to site by large dumping trucks. It is then spread by bulldozer, pushing the rock over the edge of the advancing lift. Larger fragments are broken down within the layer thickness and a relatively smooth surface is then left for the compaction equipment. Compaction is invariably by heavy vibrating roller. The development of the vibrating roller has been of great significance in the production of stiff rockfill. The roller is usually smooth, vibrating at about 25 Hz. See Table 10 for lift thickness, compaction equipment and number of passes for a selection of dams. The maximum rock size allowed is usually slightly smaller than the lift thickness. The thickness of the lift will depend on the rock and the ability of the roller to compact adequately the base of the layer. Field trials are commonly carried out to determine the optimum lift thickness and the number of passes of the roller. A method specification then can be used for compaction control. Nuclear densometer or insitu density tests give a better appreciation of the density than the U.S.B.R. relative density test. The relative density test is inappropriate for the common sizes of rockfill in use today.

The compacted surface is kind to haulage vehicles and is stable during periods of heavy rain. Larger fragments of the next lift indent into the surface producing a rockfill free of discernable layering. An optimum placement water content for rockfill will vary from rocktype to rocktype, but for some sandstones and limestones it appears to be at least 6%. Because of the free draining properties of most rockfills it is safe to err on the wet side. The use of sluicing water to assist in the compaction has often been used. Sluicing may not be necessary when the lifts are small and well compacted, but it does reduce the possibility of additional settlement occurring from the flow of leakage water.

4.4 Construction of thin membranes.

Thin membranes are especially suited to low earth dam construction where the use of dry material, ensured by the membrane, enables steeper slopes to be used. The placing of thin membranes such as P.V.C. or butyl rubber is essentially a labour intensive method. A high standard of cleanliness is required above and below the membrane to avoid puncture. Protection of the membrane from environmental attacks is often accomplished with precast concrete blocks. The membrane is often simply anchored by burying its ends in a trench and filling the trench with concrete. Thickening of the membrane above water level is common to minimise the ageing effects.

Table 10

Smmary of rockfill construction details.

Dam	ROCKFILL				BEDDING			
	Lift Thickness m	Maximum Aggregate Size cm	Compaction by	No. of Passes	Lift Thickness m	Maximum Aggregate Size cm	Compacted by	No. of Passes
Villagudin	1.0 S	100	10 t V	4 M	0.5	40	5.5 t V	15
Cethana	0.9	60	10 t V	4	0.45	22.5	10 t V	4
Rama	1.5 S	120	10.5 t V	8–10	0.7	35		
Nyrsko	1.5 S				0.8	4–15		
New Exchequer	1.2 S	120	10 t V		0.6	37.5	10 t V	
Hunico	0.5		8 t V					
Alto Anchicaya	0.6 S	60	10 t V		0.5	30	10 t V 10 t V	4 8 upslope
Foz do Areia	0.8 S	70	10 t V	4	0.4	15	10 t V	4
Outardes 2	0.9	75	10 t V	4			10 t V	6 upslope
Yacambu	0.6	60	10 t V	6	0.3	7.5	10 t V	6

S – Sluiced
M – Minimum number
V – Vibrating

5. CASE HISTORIES

5.1 Concrete Membranes.

The case histories of a number of dams with concrete upstream membranes are reviewed. The principles of the design method used for determining the thickness of the membrane has rarely been reported. This may be because an empirical approach has been used or because the thickness of the membrane was determined solely on practical limitations of the site. The first published rational design was that by Wilkins, Ref. 22, for the Cethana dam, Australia. Many authors have quoted this article since, so that it can be assumed that this dam design has influenced the design of many later dams. The main conclusions of the designers are presented with the case history as the design evolves.

5.1.1 Cethana, Australia.

Two methods were used to forecast deflection of the fill and the strains in the concrete membrane. One by Wilkins, Ref. 23, was a semi–empirical method, the other was based on the finite element work of Broughton, Ref. 24. In this analysis Young's modulus and Poisson's ratio could be made stress dependent. The conclusions reached were as follows:–

1. Large movements of the slab would occur if the slab were built at the same time as the rockfill.
2. Strains were largely independent of the slab thickness and dependent on the fill movement.
3. Strains would not be excessive if the rockfill were well compacted and well constructed.
4. Most of the slab would be in compression and from this the designers deduced that soft material in the joints was not to be used. Soft material in the joints would, in fact, be detrimental to the performance of the slab.
5. The critical area for the analysis and design was around the periphery of the face where there would be tension in the slab.

The design points resulting from the conclusions of the study were as follows:

1. The rockfill was to be compacted in 0.9 m layers and close to the abutments half of

this value.

2. The rockfill was to be completed before starting the construction of the slab.
3. The slab was to be reduced in thickness to the minimum required for impermeability or to the minimum for long life.
4. Plain butt joints were to be used in the membrane.
5. In the perimetral slab the reinforcement quantity was to be increased, the joint spacing was to be reduced to enable the slab to move over the fill as differential movements occurred and avoid tension cracking. Extra waterstops were detailed in these areas.

The dam was zoned into areas requiring different compaction and grading. The most important were:

Zone 2 – Directly under the slab, consisting of well graded rockfill with maximum size of aggregate 225 mm placed in 0.45 m layers. The zone was intended to act as a semipervious barrier and be easily trimmed to the correct profile. Specified compaction was 100% relative density obtainable with four passes of a 10 t vibrating roller.

Zone 3A – Composing most of the main body of the dam, about two-thirds of the total volume. It consisted of a well graded rockfill, maximum size of aggregate 600 mm compacted in 0.9 m lifts. Specified compaction was again 100% relative density obtainable with four passes of a 10 t vibrating roller. After four passes the material began to breakdown. The specification was changed to require two passes parallel and two passes at right-angles to the dam axis where space permitted.

Zone 3B – Downstream third of the dam. Layer thickness 1.35 m with relaxed grading limits. Compaction was as before.

All zones were to be sprayed with water, not less than 15% of the volume of the rock, before and during compaction.

A special zone 2 compacted in 120 mm layers by hand held tampers was required adjacent to the plinth. Rock was to have a minus 100 mm grading. A transition filter zone was placed between zones 2 and 3A of maximum rock size 375 mm in 0.45 m layers 3 m into zone 3A.

The preparation of the face ready for the concrete included:

1. Trimming to line.

2. Compacting the rockfill again.
3. Protecting the prepared face from the weather and the movement of men and materials.

This was achieved by trimming the face to profile while construction proceeded. Compaction of the face was by four passes of a 10 t roller without the vibration as it was found that with vibration material was dislodged from the face and rolled down the slope. Further trimming was followed by four passes of the roller with half vibration. A bitumen emulsion sealing was applied in three layers followed by a hand spreading of 3 mm crushed rock screedings. To complete the preparation eight passes of the roller with full vibration was used the next day.

The plinth was founded on sound rock. Open joints in the rock were grouted after completion of the plinth and prior to the consolidation grouting. Design features of the plinth are:

1. Minimum length of contact between the rock and the plinth is
   1/20th of the head where founded on sound rock.
   1/10th of the head where founded on poor rock, but never less than 3 m.
2. Top surface of the plinth is in the same plane as the slab to ease formwork requirements.
3. The minimum thickness of the plinth, normal to the foundation, is 0.5 m at the upstream side, and on the downstream side the thickness is such that there is not less than 1 m of rockfill below the slab at all locations.

Contraction joints are provided in the plinth at 7 to 10 m spacing. Longitudinal reinforcement is specified as 0.5% of the plinth area. Curing of the plinth concrete was to be for 14 days with a water spray. All contraction joints were sealed with copper waterstops. To withstand consolidation grouting pressures the plinth was dowelled into the rock. Consolidation grouting was to extend 8 m below the plinth and a grout curtain was injected to a depth equal to half the head.

The slab was considered to be uniformly supported against the normal water load and as the slab must follow the deflections of the rockfill, moment resistance of the slab was not relevant. In the absence of any other criteria, judgement was used to ensure watertightness and long term durability. A design equation of $0.3 + 0.002H$, where H is the head in meters, was adopted as this had worked previously. The maximum over tolerance thickness due to the unevenness of the face was taken as 125 mm.

The details of the joints are sketched on Figure 4. The perimetric joint between the plinth and the slab is protected by two waterstops, one rubber and one copper. The joint also contains a 12 mm thick pine timber filler to allow for rotation, shear and possible compression without spalling of the joint.

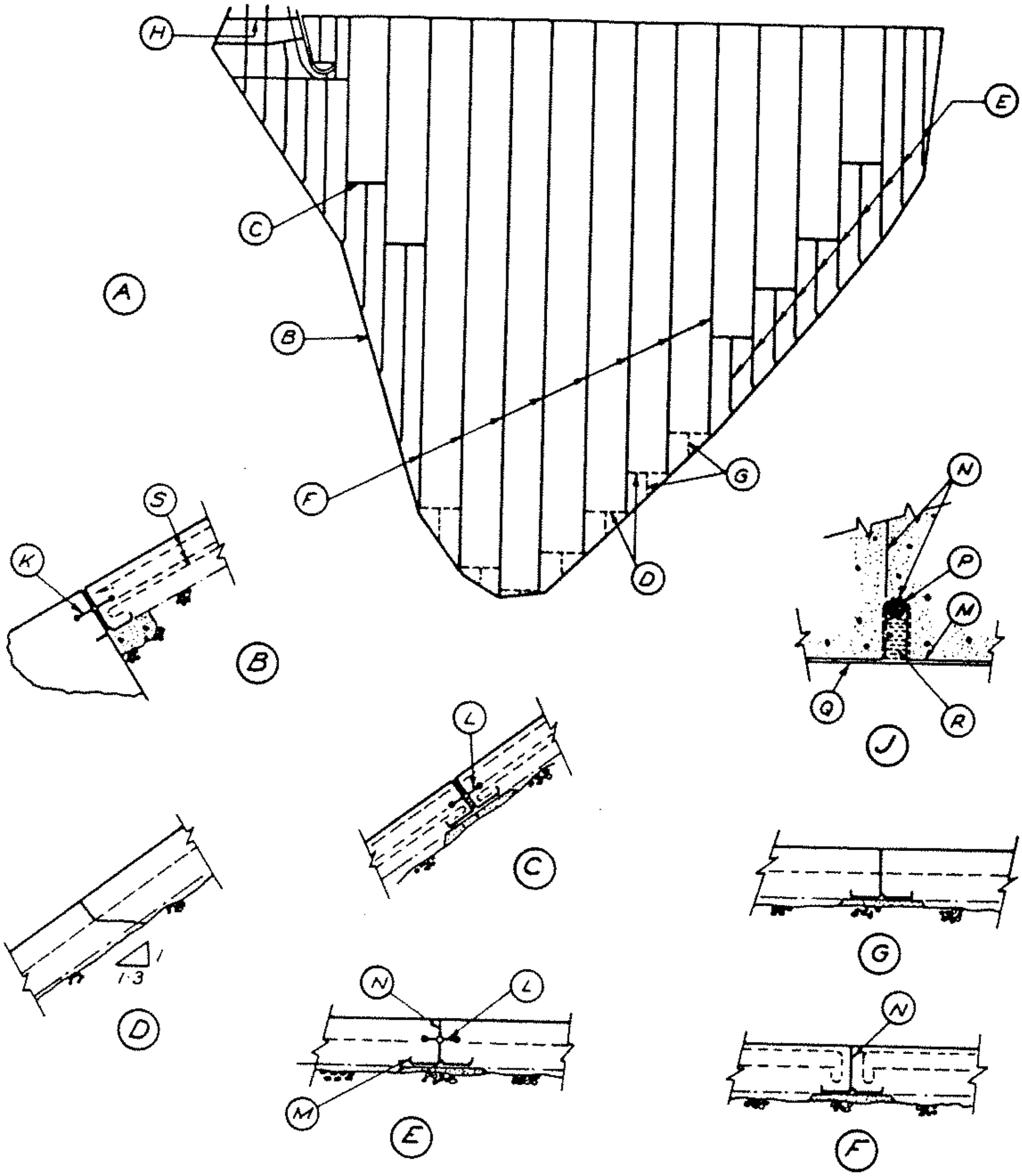
The vertical joints have the surface of the previous slab painted with bitumen but with no filler. A copper waterstop was provided at the base of the joint. Joints are bent to lie normal to the plinth for the last 0.6 m. Joints close to the perimeter where tension is likely to develop also have a rubber waterstop at mid-slab thickness.

Horizontal joints were provided to terminate the joints between the reduced width slabs used at the perimeter. Their construction was similar to the vertical joints but a 12 mm pine timber filler was inserted to allow for shear movement along the joint line.

The copper waterstop was of W form with a neoprene rubber insert retained by polyethylene foam in the centre rib to prevent collapse of the rib under water pressure. The waterstops were bedded on a mortar pad at the correct line and level. A single layer of bituminous felt was placed between the pad and the waterstop. The rubber waterstop is robust and deformable. Vulcanised field joints can be made to produce the T shaped intersections.

As the slab face was determined to be in compression over most of its area the reinforcement only has the duty of controlling thermal movements and shrinkage after construction. For this purpose 0.5% of the area of the slab, plus an over thickness tolerance of 100 mm, was considered adequate. In the tension zones the horizontal reinforcement was increased to control tension cracking of the concrete. A high yield deformed bar was used. Additional reinforcement was placed around the perimetric joints and all horizontal and vertical joints not under tension, to prevent possible spalling of the slab corners.

Curing of the slab with water continued until first filling. Cracks still developed. They were horizontal and through the full depth of the slab corresponding to the spacing of the horizontal reinforcement. They were attributed to a rapid temperature drop after concreting. No remedial action was taken as they were observed to close as autogenous



JOINT DETAILS

(A) View normal to face.
(B) Perimetric joint.
(C) Horizontal contraction joint.
(D) Horizontal construction joint.
(E) Vertical contraction joint (Type 1).
(F) Vertical contraction joint (Type 2).
(G) Vertical construction joint.
(H) Spillway.
(J) Detail for copper waterstop.
(K), (L) 305 mm, 230 mm rubber waterstop.
(M) Copper waterstop.
(N) Surface painted with bitumen.
(P) 12 mm. dia. noeprene.
(Q) Bituminous felt strip.
(R) 16 mm × 32 mm closed cell polyethylene foam.
(S) Reinforcement.

Figure 4. Cethana, Joint details.

(After Wilkins, et al., 1973.)

healing took place.

The performance of the dam is described by Fitzpatrick, et al. Ref. 25. To monitor the performance the instrumentation consisted of:

1. Four hydrostatic settlement gauges within the body of the dam at approximately a third and half height.
2. A total of 33 surface movement stations located on the crest and downstream slope, co-ordinated by precise surveying.
3. A total of 23 underwater membrane movement stations, primarily located on three cross-sections. Vertical movement was measured by a cable attached to the station and to a float at reservoir level. The cable was tensioned and graduated marks levelled from the shore. The movement in the plane of the slope was measured by graduated cable attached to the station and passing over a surveyed mark on the crest.
4. Inclinometer tubes attached to the face of the dam at the same three sections to measure the normal deflection of the slab.
5. Measurement of the movement of the perimetric joint between the membrane and the plinth, in the plane of the membrane, at eight locations.
6. 2 m long gauges measuring strains in the membrane at 32 locations. The gauges were arranged in 45° or 90° rosettes. Temperature was also measured by the gauges. Check 250 mm long gauges were used alongside the 2 m long gauges to assess the effect of the reinforcement. Five of the 250 mm gauges were embedded in concrete and suspended down the face of the dam to determine thermal stresses and autogenous growth.
7. Leakage was measured by a V notch weir. In periods of dry weather infiltration through the dam and abutments was assumed to be zero and the resulting flow in the drains of 35 l/s was attributed to flow through the membrane or under the dam.

Performance of the dam is briefly described. Within the embankment, settlement at mid-height was 425 mm before impounding. A further 100 mm occurred during and after impounding. From the results of the four settlement cells a modulus of deformation Er can be calculated.

$$Er = \frac{9.8\ D\ H}{S}$$

S is the settlement of the cell, and D is the density of the fill.
The moduli to Feb 4th are reported on Table 11. which includes first filling. Er is stress dependent and corresponds approximately to the values of the secant modulus obtained from large diameter triaxial tests. This supports the relationship used by Wilkins, Ref. 23, for the design analysis.

Crest settlements' maximum values were 69 mm vertically and 41 mm horizontally downstream, 18 and 8 mm towards the centre of the crest on the left and right sides respectively. Midway along the crest, deflection normal to the face was 79 mm. On the exposed portion of the membrane above water level the transverse deflections opened the vertical perimetric joint. The sum of the openings agreed closely with the crest transverse movements.

Down slope movement as measured by the wire survey and by integrating the strains did not correspond exactly. The integrated strains are in agreement with the surveyed slope deflection at the crest and the measured joint opening at the bottom of the slab.

Normal deflections of the slab are reproduced on Figure 5. Neglecting the horizontal movements of the slab measured by the wire survey, the vertical movement measured by the wire survey and the inclinometer are in close agreement. See Figure 5. The maximum deflections of 117 mm and 115 mm by the wire and inclinometer respectively are close and commensurate with a modulus of deformation for the rockfill of 204 MPa.

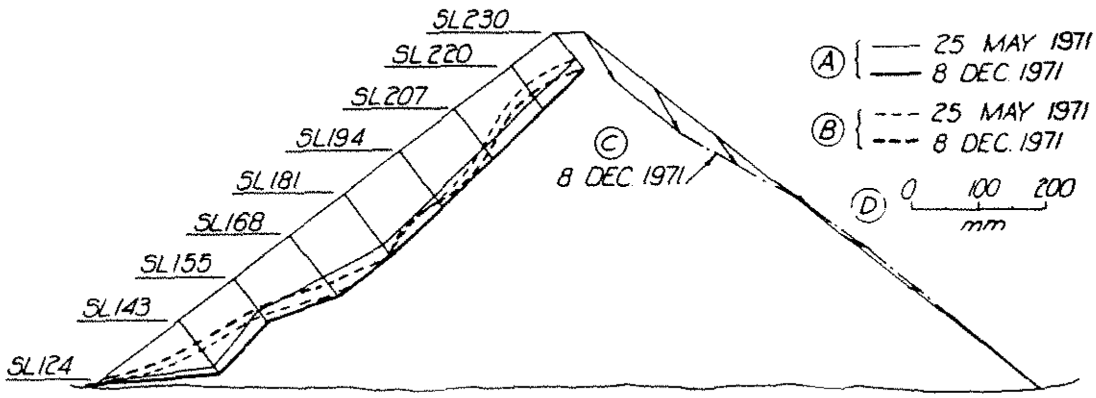
Perimetric joint openings after first filling were a maximum of 11.5 mm at the toe reducing to 1.5 mm at two thirds of the way up the abutment. The movement parallel to the joint on the abutments was generally much less than the joint opening. Strains, prior to filling, were generally compressive of the order of 100×10^{-6}. After first filling all strains were compressive with a maximum of 290×10^{-6}. Some tensile strains developed during filling. Stresses calculated from the change in strain were compressive except for some regions adjacent to the perimetric joints. The maximum compressive strain was not more than 10% of the failure strain of concrete.

Some conclusions were reached on the performance of the instrumentation. Wires for the measurement of downslope deflection would be omitted. Additional floats

Table 11

Cethana dam, deformation moduli from observed settlements.

Zone m	Density t/m^3	Depth of Fill Below or Between Cells m	Height of Fill Above Cell m	Settlement up to 4th Feb. 1971 mm	Modulus of Deformation MPa
120.5 – 151.5	2.07	30.5	79.0	266	185
120.5 – 181.5	2.07	61.0	48.5	249	135
151.0 – 181.5	2.07	30.5	48.5	272	112



Membrane normal deflection and deflection of downstream face.
Through slab 9 F (8 th. Dec. 1971).

(A) Wire measurements.
(B) Inclinometer measurements.
(C) Deflection of downstream face.
(D) Scale of deflections.

Figure 5. Cethana, Membrane normal deflection.

(After Wilkins, et al., 1973.)

and wires would be attached to the plinth for reference. The measurement of the normal deflection of the slab at the perimetric joint would have been useful as would the measurement of joint movements on vertical slab joints adjacent to the abutments. The 250 mm gauge length results were compatible with the 2 m long gauges so the shorter gauge could be used to save costs.

5.1.2 New Exchequer, U.S.A.

The new dam abuts onto the existing gravity dam. The height requiring facing was 92 m at a slope of 1:1.4. See Figure 6. The rockfill was placed quickly with four zones of differing specifications:

Zone	Max. Size	Lift Height	Compaction
1	375 mm	600 mm	10 t vib. roller
2	1.2 m	1.2 m	10 t vib. roller
3	1.2 m	3 m	Hauling and grading equipment
4	50% over 300 mm		Dumped from 18 m. No compaction

Sluicing with water was done for Zones 2,3,4. The facing was concreted in 18 m long 6.4 to 15 m wide bays as the construction of the rockfill proceeded. Copper waterstops were built in between the bays. The joint between the old dam and the new concrete was given special attention. See the detail of Figure 6. The joint was designed to be flexible to accommodate movements at the different operating water levels.

During the first filling the crest settled 457 mm and moved downstream 122 mm. Leakage started at 280 l/s building up to 340 l/s. The reservoir was lowered and some spalling of the concrete at the joints was repaired. Leakage increased to 13.9 cumec. Settlement increased and was observed to be causing the face to be dishing, pulling the slabs away from the perimetric joints.

Remedial work consisted of filling the notch between the old and the new dams and some way up the slope with a mixture of sand, gravel, clay and bentonite (10–20, 25, 55–65, 1.5 % respectively). The mixture was placed underwater by a special skip winched up and down the concrete facing. Leakage eventually reduced to 85 l/s. Ref. 26.

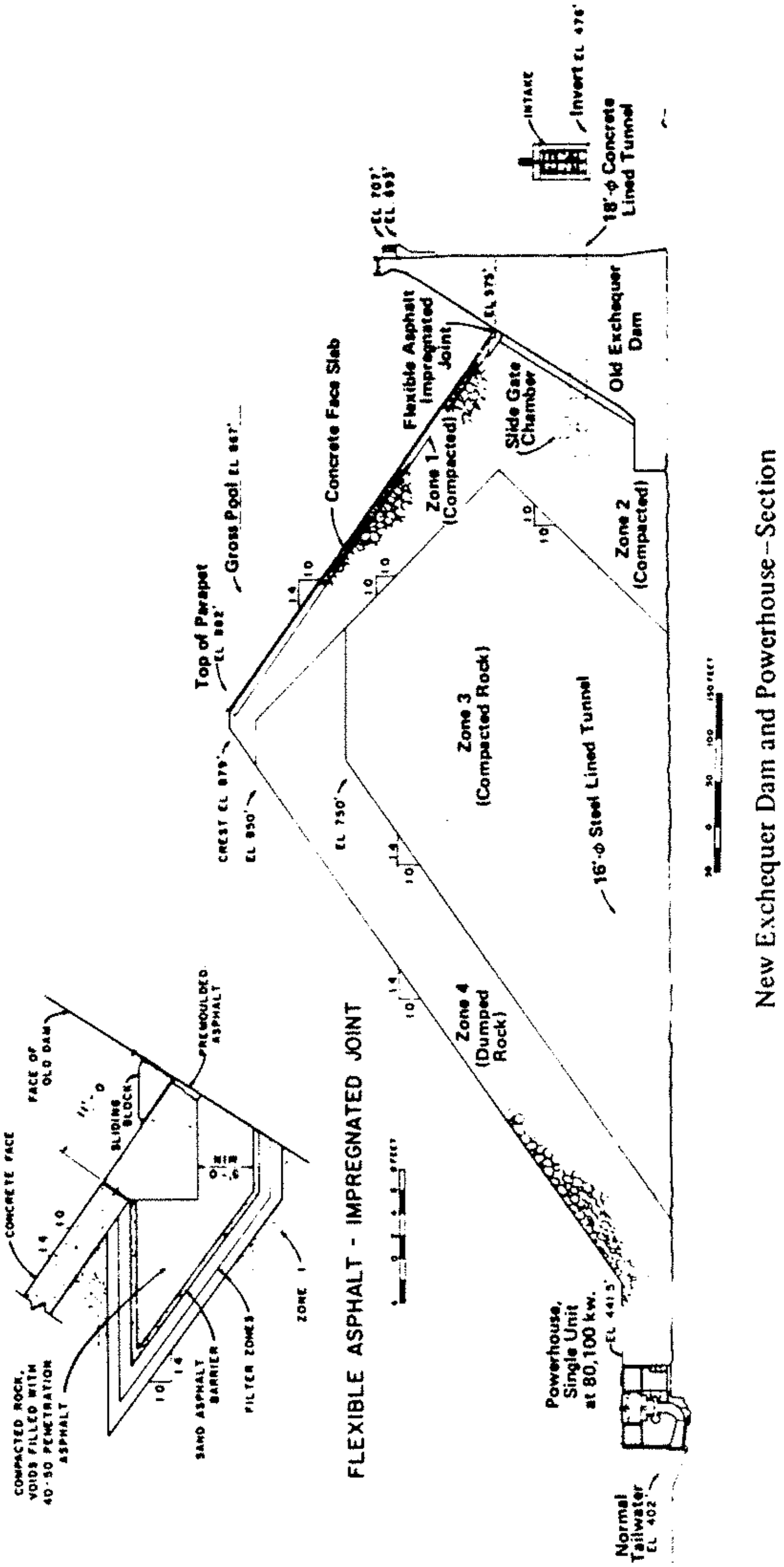


Figure 6. New Exchequer dam and powerhouse, Section.

(After Reitter, 1970.)

5.1.3 Nyrsko, Czechoslovakia.

Rockfill was of mica schist placed in 1.5 m layers assisted by water jetting with 200 - 300 l/m^3. Bedding material for the membrane was 80 cm of 4 - 15 mm gravel. Concrete binding 5 - 15 cm thick was used on top. Concrete was usually placed in 12 m by 12 m panels with joints protected by a rubber waterstop and filled with an oak board and mastic. Panels more than 40 cm thick have two layers of reinforcement. Ref. 27.

5.1.4 Kangaroo Creek, Australia.

Steeply dipping intersecting clay seams in the left abutment had ruled out an arch dam. The dam was situated in a high risk seismic area, 8 on the Modified Mercalli, and three miles west of the dam is the active Eden fault. The schist available for rockfill was expected to lose strength on wetting and possibly deteriorate from sulphate attack from disseminated pyrite. The rock was used and expected to break down under the action of rolling. The fill was to be placed wet in a zoned embankment using a volume of water equal to the volume of the rockfill. Hard durable imported rock was specified for the filters.

Downstream slope protection was to be by chain link wire mesh anchored to the foundation. However, the mesh used was too thin. The bedding layer for the membrane was to be compacted with 3 passes of a vibrating roller in the upward direction. A mortar pad was prepared along the line of the vertical joints and surfaced with asphaltic concrete. Besides providing an even surface for the joint forms and screed rails they were designed to prevent that portion of the slab adjacent to the joint from being restrained by the rockfill. No horizontal joints, except for the construction joints, were used. During the placement of the reinforcing mat some unravelling of the face occurred. The average vertical settlement of the face before concreting was 150 mm. The thickness of the slab T was determined from,

$$\frac{H \times 0.305}{60.96} + 0.305$$

where H is the vertical distance from the grout cap level.

The slabs were 12.2 m wide, a small size, due to the seasonal temperature variation of 45°C. Reinforcement at the rate of 0.5% of the area of the slab was placed in each direction at the centre of the slab. At the perimeter an extra 0.1% of steel was provided in addition to the anti-spalling steel provided above and below the waterstops at all joints. The anti-spalling steel was welded to prevent any tendency for bursting of the slab edges. Joints were sealed by a 240 mm P.V.C. waterstop. At the joint between the face slab and the plinth a cork filler was used to stop any tendency of the concrete to cut the waterstop if the joint rotated. A cork strip was also provided above the waterstop in the face slab joints. Movements of the slab were expected because of the settlements expected from the poor quality rockfill.

The plinth was dowelled into the rock to resist the grouting pressure of 48 kPa.

The cement content of the concrete was high at 328 kg/m^3 to assist in autogenous healing of cracks. The maximum temperature rise at the centre of the slab was only 6^{0}C and no major cracks were observed.

Instrumentation consisted of settlement pins each side of vertical joints and electrical jointmeters. Small settlements were observed during filling. The maximum vertical settlement normal to the face, 52 mm, was at about mid-height near the centre of the dam. The maximum vertical settlement on the face, 43 mm, occurred at about 10 m below Full Supply Level (F.S.L.). The maximum joint opening was 10 mm in a vertical joint in the centre of the dam 12 m below the crest. Joint rotations also occurred where a joint settled relative to an adjacent joint but no evidence of concrete spalling was found. The maximum opening of the plinth to slab joint was 3 mm. Leakage settled at 500 l/s.
Ref. 28.

5.1.5 Hunico, Peru.

This dam was originally provided with a 3 mm steel facing, as storage capacity was urgently required. Rockfill in 50 cm layers was compacted by 8 t vibrating rollers. The final concrete was reinforced with steel, 0.55% of the area, in each direction. Strips were 10 m wide and 30 cm thick. No horizontal joints were used. Reinforcement does not pass through the vertical joints but steel dowels, 50 cm long, were provided at 1.5 m centres. Joints were made watertight by rubber waterstops.

Leakage was about 0.5 cumec reducing to about 0.4 cumec in 2 years.

An earthquake of 7.75 on the Richter scale occurred about 300 miles away. The seepage water became turbid for about half an hour after the shock. A small horizontal crack 1 mm wide was found in the centre of the drainage gallery in the cutoff at the toe. No other damage was observed. Ref. 29.

5.1.6 Alto Anchicaya, Colombia.

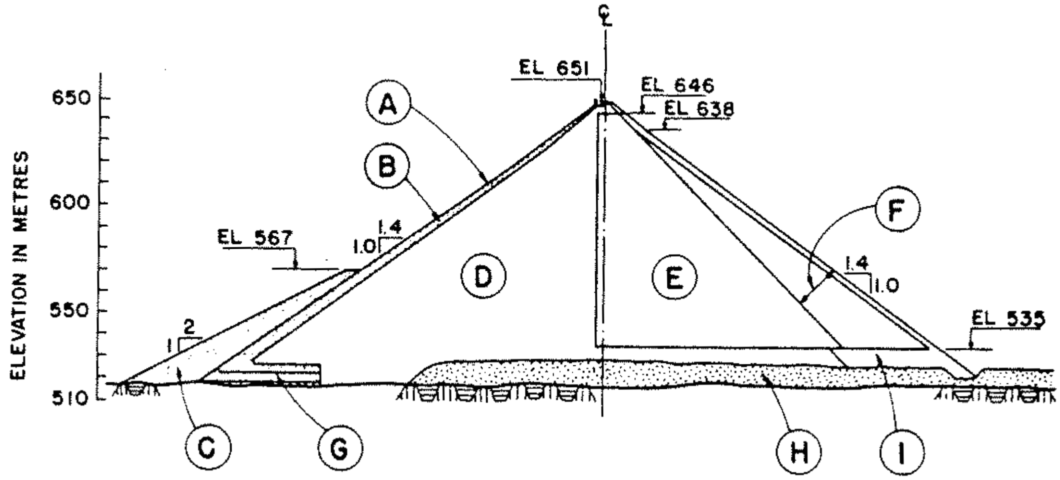
The details of the rockfill zones and the compaction details are given on Figure 7. The membrane was built to approximately one quarter height simultaneously with the rockfill. An impervious cover was then provided for this area. When the rockfill reached 30 m below the level of the crest slipforming the face restarted. There were six slipforms in operation at once concreting the 15 m wide strips. The layout of the strips is shown on Figure 8. The perimetric joints are protected with rubber waterstops with a timber filler below the waterstop. Vertical joints and normal joints to the plinth are similar but with timber fillers above and below the waterstop. The concrete surfaces of the vertical joints are painted with bitumen. It may be noted that the perimetral hinge slabs are large.

Instrumentation provided consisted of 22 joint meters of the vibrating wire type to monitor joint movements in three directions. Sixty strain gauges embedded in the slab were to investigate strain deformations. The locations of the groups of joint meters are shown on Figure 9.

Upon filling the movements were in agreement with the predictions using the moduli obtained from settlement cells within the fill. Leakage reached 1.8 cumec. The reservoir was lowered and the membrane inspected. No large cracks, ruptures or spalling were discovered. On the left abutment all the perimetric joints were found open to about 30 m below the crest. The leakage zone #1 to #2 was considered to have been caused by the rotation of the hinge slab away from the plinth where an abrupt change in geometry occurred. The joint between the hinge slab and the rest of the face was closed.

On the right abutment the perimetric joint was again open. The hinge slab to the main face joint was open in the lower section. Perpendicular joints were also open. See Figure 9. One leak was found, #6, in the centre of the slab and no remedial action was taken for this leak.

Some of the joints were dug out to expose the waterstop. It was found that the concrete had not penetrated fully around the waterstop and leakage had occurred around



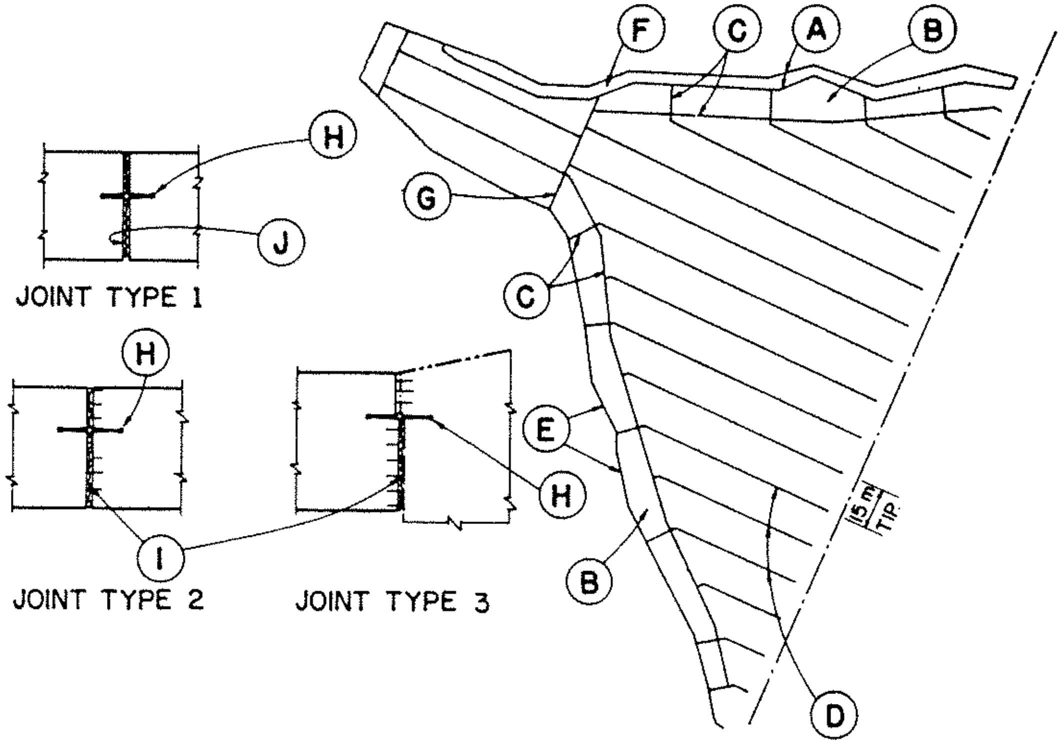
ROCKFILL ZONES

ZONE	TYPE	SPECIFICATION
B	C I	WELL GRADED, 0.30m MAXIMUM SIZE, COMPACTED BY 4 PASSES 10 TON VIBRAT. ROLLER-LAYERS - 0.50m 8 PASSES OF THE 10 TON. VIB. ROLLER UPSLOPE DIRECTION.
I	2 A	1m - MAXIMUN SIZE - COMPACTED BY 4 PASSES 10 TON - VIB. ROLLER - 1.0 m LAYER
D	3 B	BEST MATERIAL - WELL GRADED - 0.60m MAXIMUN SIZE, COMPACTED BY 4 PASSES - 10 TON VIB. ROLLER 0.60m LAYERS-WATERED BY 200 L/m3.
E	4 B	SIMILAR TO 3B BUT WITH MORE FINES. COMPACTION AND LAYERS AS FOR 3B.
FILTERS		SAND AND GRAVEL 3/4" TO PROVIDE PROTECTION FOR EROSION OF THE ABUTMENTS AND RIVER BED.

(A) Concrete face membrane.
(B) Transition zone - Type C 1.
(C) Impervious cover.
(D) Rockfill - Zone 3 B.
(E) Rockfill - Zone 4 B.
(F) Oversize rockfill.
(G) Filters.
(H) Riverbed alluvium.
(I) Rockfill - Zone 2 A.

Figure 7. Alto Anchicaya, Cross section.

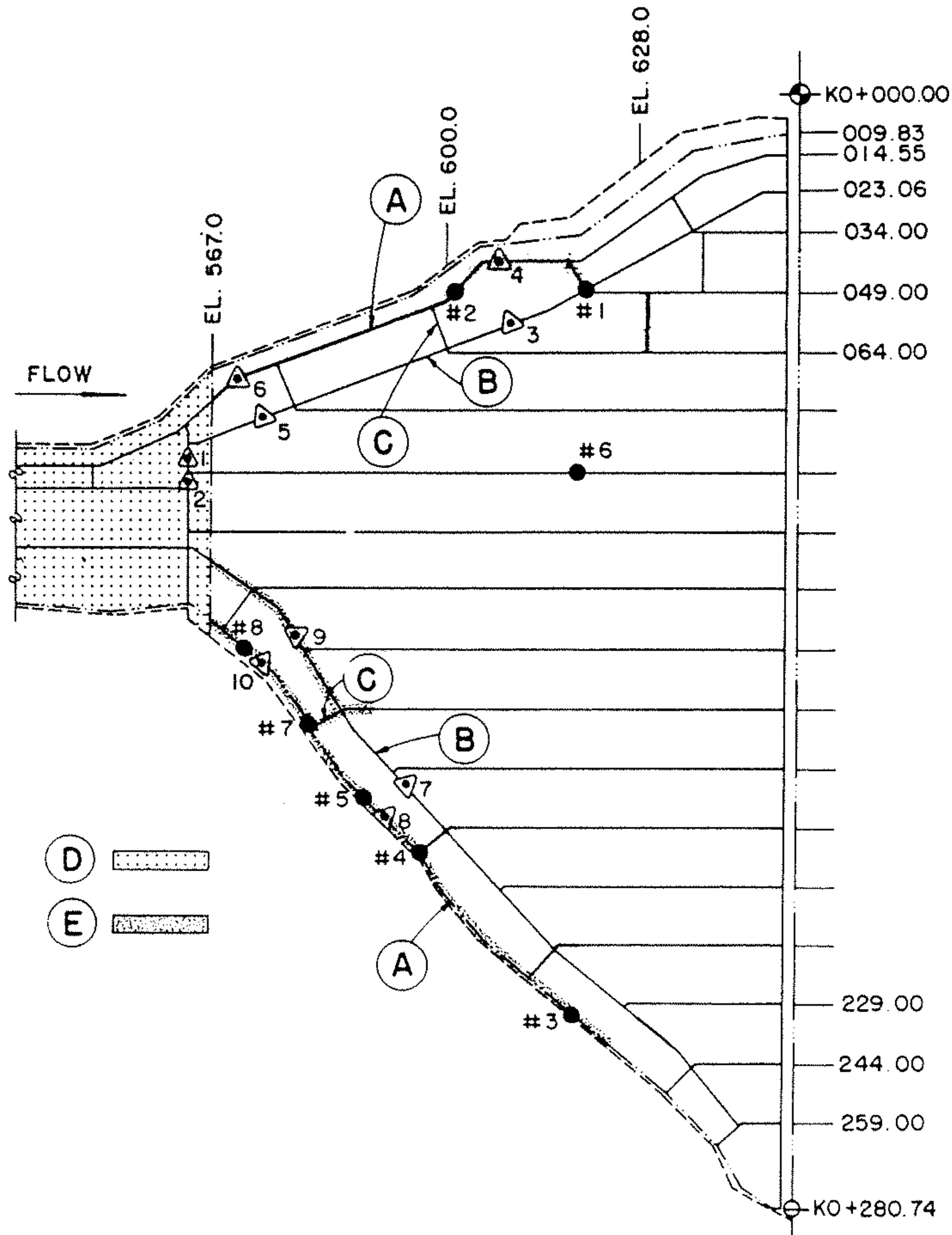
(After Regaldo, et al., 1982.)



(A) Plinth joints.
(B) Perimetral hinge slabs.
(C) Joint type 2.
(D) Vertical joints type 1.
(E) Perimetral joint type 3.
(F) Plinth.
(G) Horizontal joint.
(H) Rubber water stop.
(I) Wood filler.
(J) Paint surface with asphalt.

Figure 8. Alto Anchicaya, Layout of the face and joint details.

(After Regaldo, et al., 1982.)



(A) Perimetral joint.
(B) Inner hinge joint.
(C) Perpendicular joint (hinge slabs).

(D) Impervious material.
(E) Treated zone.
● # 1 Leakage zone.
△ 7 group of joint meters.

Figure 9. Alto Anchicaya, Leakage zones – joint meter locations.

(After Regaldo, et al., 1982.)

the waterstop. Repairs were carried out by filling the joints with mastic and covering with clay. After the second filling the leakage dropped to 180 l/s.

The crest settled 60 mm after the second filling and after one year the total was 110 mm and after seven years 140 mm. The maximum normal deflection of the membrane was 130 mm and after two and a half years 160 mm. Compressive strain was a maximum of 337×10^{-6} in the lower part of the central slab. Joint meters 3, 5, and 7 indicated closed joints and this was confirmed after the first drawdown. Joint meter 4 measured a separation of 6 mm comparing well with the observed opening of 5 mm. Joint meter 8 located close to one of the major leaks, #5, registered a separation of 125 mm, a perpendicular settlement of 106 mm and a 15 mm downslope movement. These movements correlated well with the observations after dewatering, indicating that no significant rebound occurred after removal of the water load. Ref. 30.

5.1.7 Foz do Areia, Brazil.

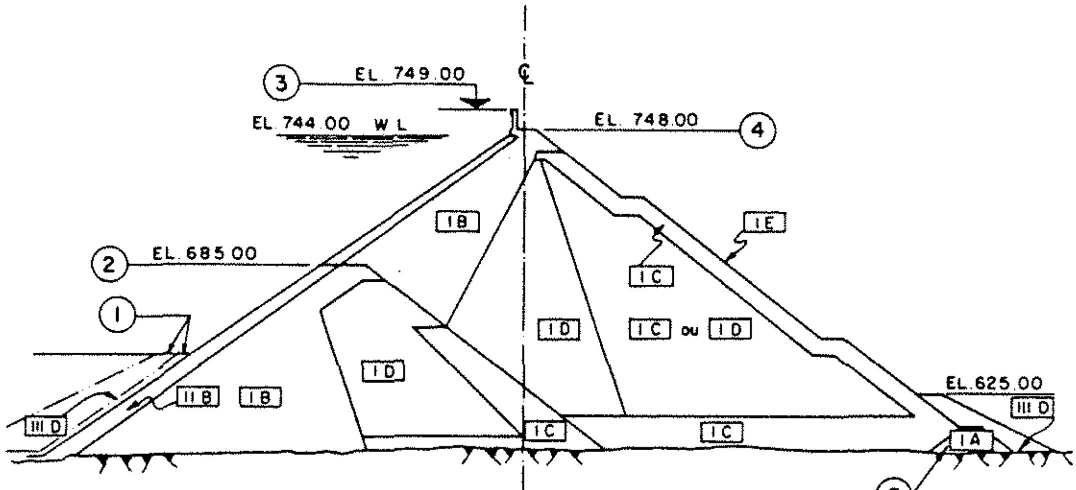
The rockfill for the dam came mainly from the excavations for the powerhouse and the spillway and consisted of basalt and basaltic breccia. The zoning of the dam and the construction details are given on Figure 10. The bedding reduces in width from 10 m at the base to 4 m at the top. Water was used for sluicing the rock at the rate of 25% of the fill volume. A clay protection layer was added to the bottom 30 m of the face.

The thickness of the face slab is 0.8 m at the base tapering to 0.3 m at the crest according to the formula, thickness, T,

$$T = 0.3 + 0.00357\,H$$

where H is the vertical distance from the crest. The width of the plinth is designed to be at least 1/20th of the hydraulic head and convenient sizes of 7.5, 5.5 and 4.0 m were used. Steel reinforcement at 0.4% of the area of the slab was provided in each direction. Extra steel in the form of cages was provided at the edges of all the slabs and the plinth to prevent crushing of the corners of the slabs if movement occurred.

Compared to Cethana or Alto Anchicaya dams, Foz do Areia was expected to have a low modulus of compressibility. 50 MPa was expected compared to 150 MPa of the other dams. The joints were designed with this in mind and the details are shown on Figure 11. A double waterstop protection was provided, copper at the base and rubber at



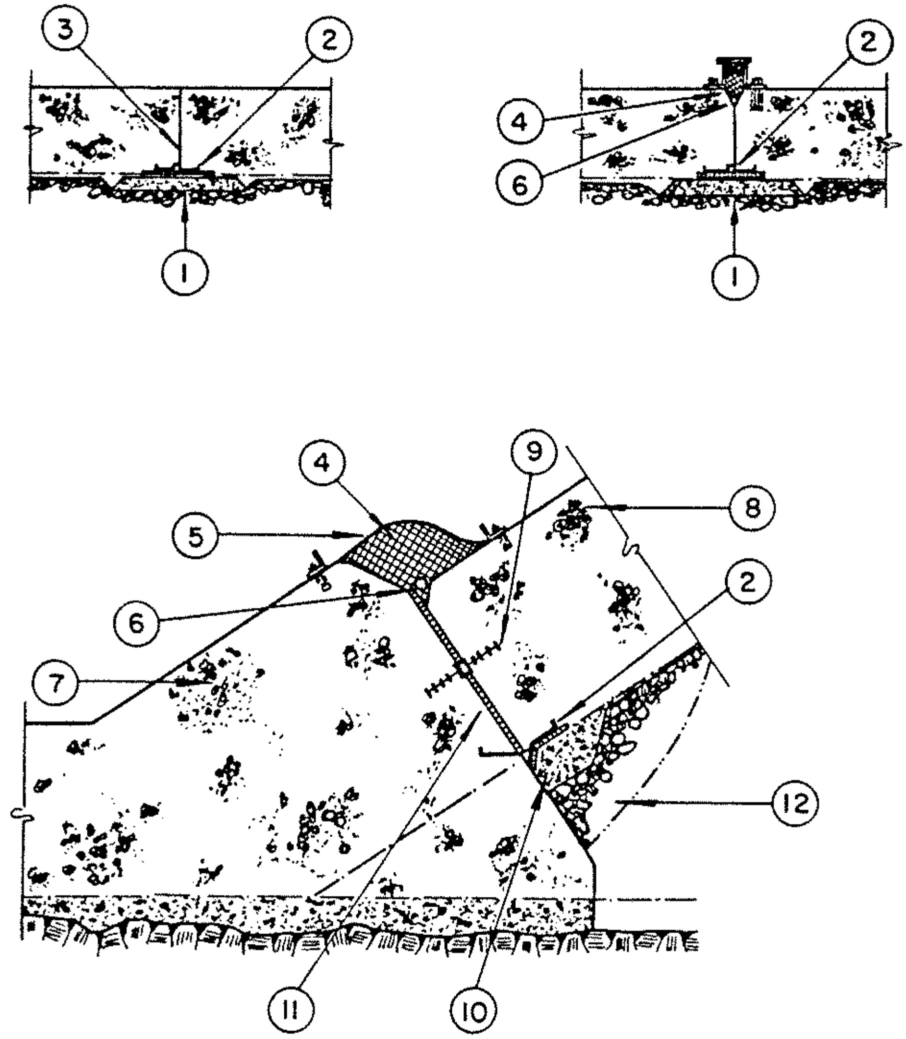
TYPICAL CROSS-SECTION OF EMBANKMENT
(Coupe type du barrage)

TABLE OF MATERIALS				
MATERIAL	CLASSIFICATION	ZONE	METHOD OF PLACEMENT	COMPACTION
ROCKFILL I	MASSIVE BASALT (up to 25% Basaltic breccia)	I A	DUMPED	— — — — —
		I B	COMPACTED IN 0.80m LAYERS.	FOUR PASSES OF VIBRATORY ROLLER (10 ton) 25% of water.
		I C	COMPACTED IN 1.60m LAYERS.	" "
	INTERCALATION OF MASSIVE BASALT AND BASALTIC BRECCIA	I D	COMPACTED IN 0.80m LAYERS.	" "
	MASSIVE BASALT (Selected rock – 0.80m min.)	I E	PLACED ROCK (Downstream face).	— — — — —
TRANSITION II	CRUSHED SOUND BASALT	II B	WELL GRADED - MAX-SIZE 6" COMPACTED IN LAYERS 0.40 m	LAYERS - 4 PASSES OF VIB. ROLLER FACE: - 6 PASSES VIB. ROLLER (upslope)
EARTHFILL III	IMPERVIOUS SOIL	III D	MAXIMUM SIZE 3/4" COMPACTED IN 0.30 m LAYERS	PNEUMATIC ROLLER OR CONSTRUCTION EQUIPMENT.

(1) Clay protection.
(2) El. of First stage.
(3) Parapet.
(4) Crest.
(5) Dike.

Figure 10. Foz do Areia, Zoning of the rockfill dam.

(After Pinto, et al., 1982.)



(1) Concrete pad.
(2) Copper waterstop.
(3) Painted with bitumen.
(4) Mastic.
(5) PVC - cover.
(6) Neoprene tube.
(7) Plinth.
(8) Face slab.
(9) PVC - waterstop.
(10) Sand asphalt mixture.
(11) Timber filler.
(12) Special grading zone IIB.

Figure 11. Foz do Areia, Joint details.

(After Pinto, et al., 1982.)

about mid-depth of the slab. A mastic seal was used, similar to the repairs at Alto Anchicaya. A new feature was the use of a sand bitumen pad instead of the mortar pad more commonly used. It was hoped that this would assist in maintaining watertightness. The vertical joints were only provided with a copper waterstop at the base as it was expected that these joints would be in compression.

The rockfill settlements were monitored by hydrostatic cells for the purpose of determining the moduli of deformation. Settlements of various points on the crest and downstream slope were obtained by surveying. Some strain meters were installed in the slab together with reinforcement stress meters and joint movement meters on the perimetric joint.

The moduli of deformation were calculated and are shown on Figure 12. Low values were obtained with a maximum of 55 MPa and a minimum of 26 MPa. The vertical settlements within the fill reached a maximum of 358 cm at approximately mid-height of the dam. See Figure 13 and Figure 14 for vertical settlements and vertical settlements at the centre line of the dam.

Since filling, the face has settled a maximum of 57 cm near the centre. An interpretation of the results from the settlement cells in the form of equal settlement contours is shown on Figure 15. Back calculations indicate that the horizontal moduli of deformation are approximately three times those obtained from the deformations during construction.

The movement of the perimetric joint is shown on Figure 16 for the three stages of reservoir filling. At stage A, with the reservoir level to within 29 m of F.S.L., movements were small. At stage B, when the water level had risen another 18 m, settlements, separation and shear movements had occurred at all the joints. Some malfunctions of meters occurred during this period. Stage C was when the readings had stabilised with the water at F.S.L. A maximum movement of 55 mm, normal to the slab was recorded on the left abutment.

Strains in the centre part of the slab were compressive reaching a maximum of 665×10^{-6} during filling. Near the abutments a maximum tension of 332×10^{-6} was recorded.

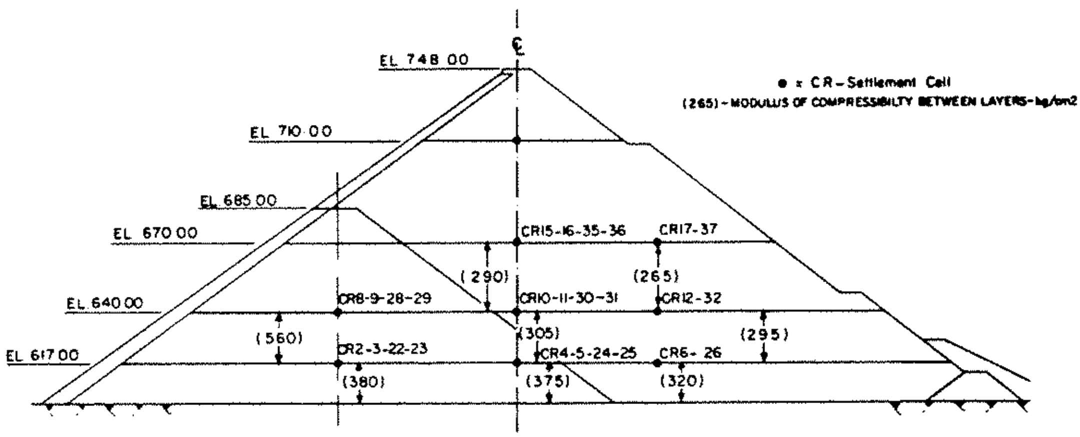
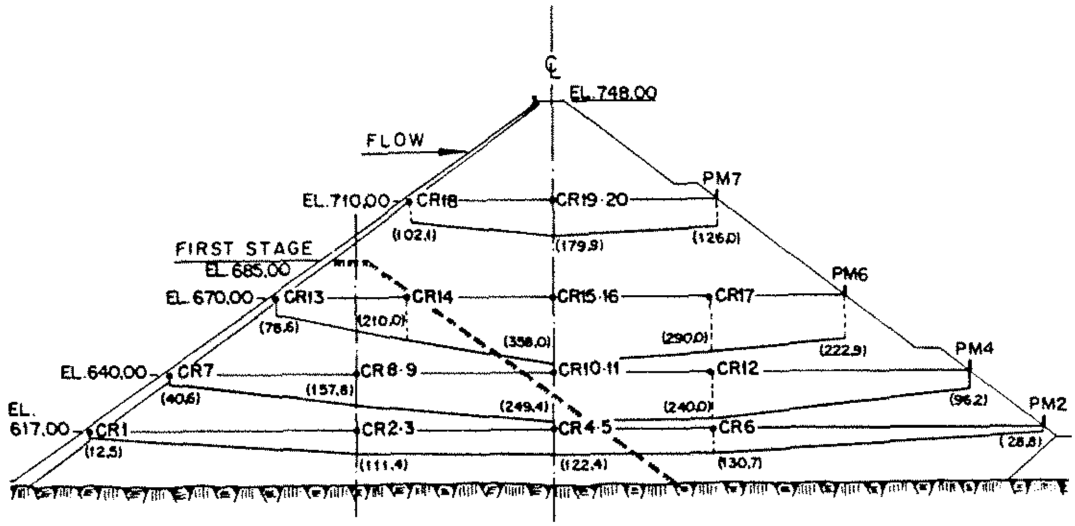


Figure 12. Foz do Areia, Compressibility moduli before reservoir filling.

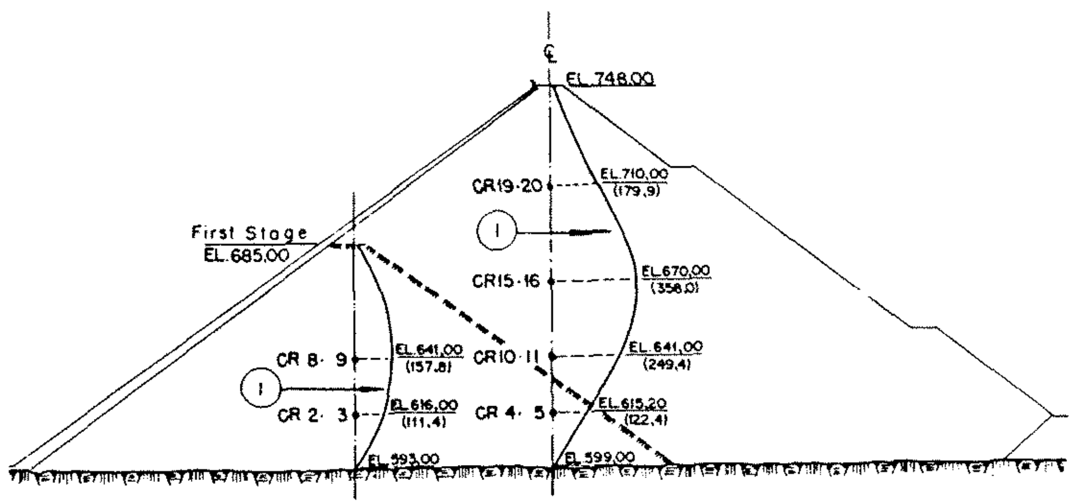
(After Pinto, et al., 1982.)



(Obs.) Largest settlement measured during the construction period - CR 15.16 = 358 cm.
(CR) Settlement cell.
(PM) Instrument house.

Figure 13. Foz do Areia, Vertical settlements before reservoir filling.

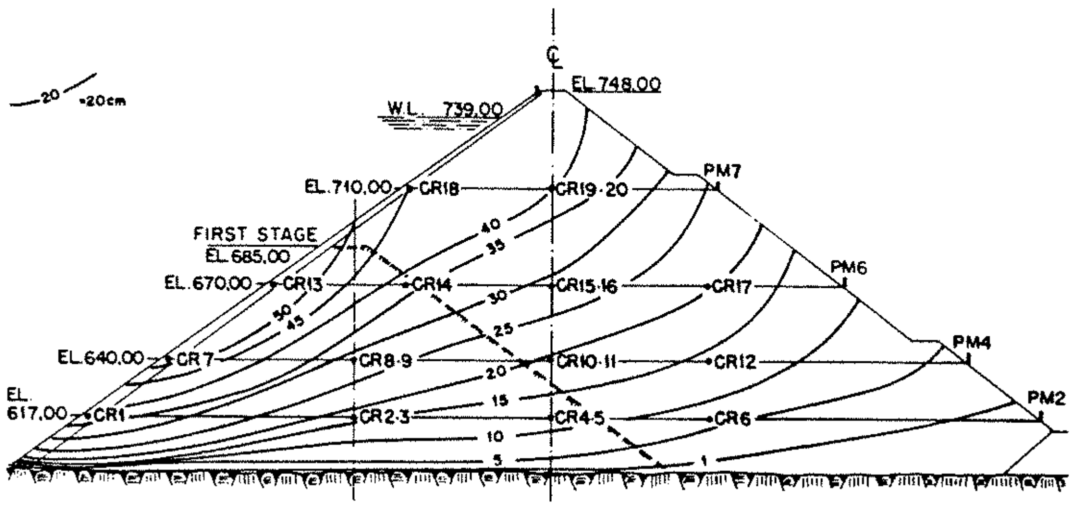
(After Pinto, et al., 1982.)



(1) Vertical settlements.
(CR) Settlement cells.

Figure 14. Foz do Areia, Settlement along the axis, first stage and at completion, before reservoir filling.

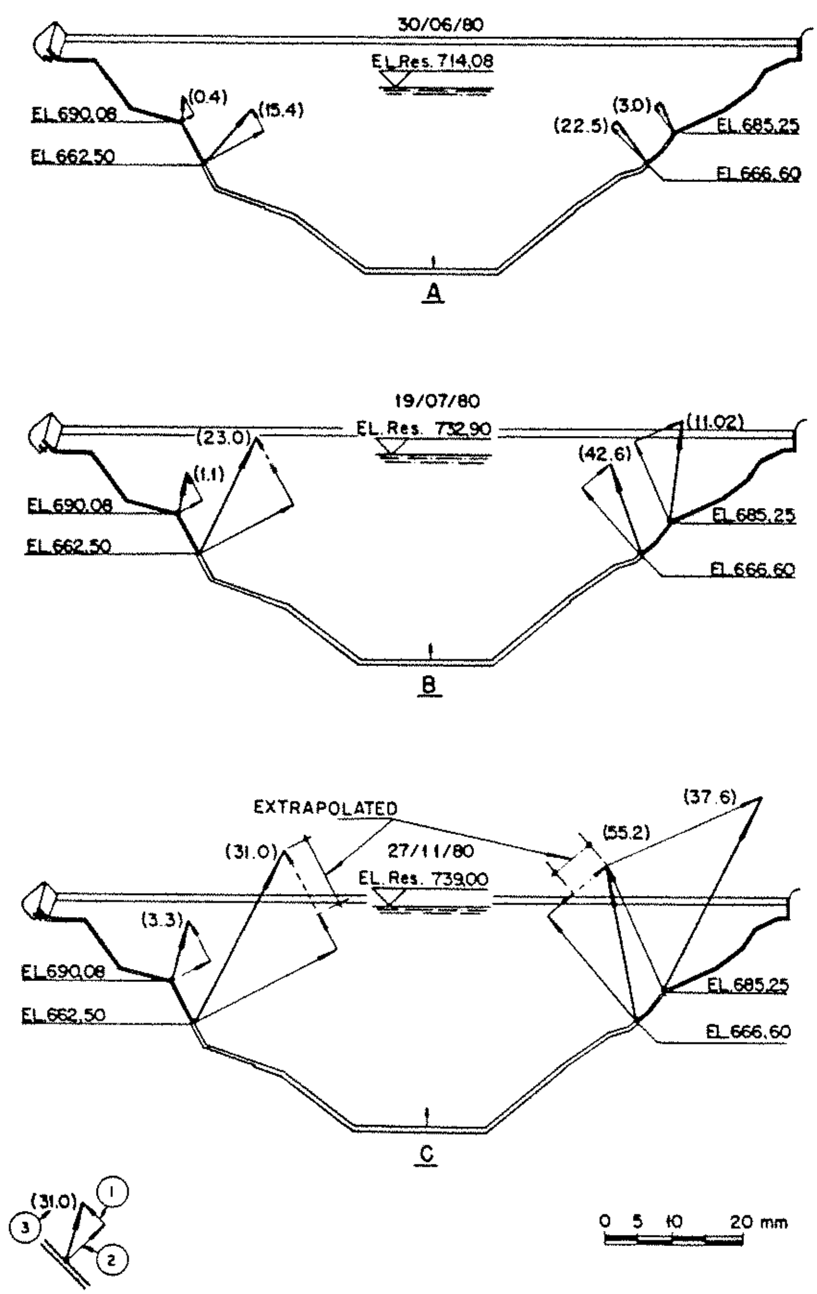
(After Pinto, et al., 1982.)



Equal settlement (in cm) curves after reservoir filling - September, 1980

Figure 15. Foz do Areia, Equal settlement curves after reservoir filling.

(After Pinto, et al., 1982.)



(1) Tangential up-slope movement.
(2) Opening movement.
(3) (31) mm - offset normal to the face.

Figure 16. Foz do Areia, Perimetric joint movements.

(After Pinto, et al., 1982.)

The dam's performance has been judged to be excellent although deformations due to the compressible nature of the fill and wide valley were higher than other concrete faced dams. Maximum recorded leakage was 236 l/s reducing to 165 l/s. Ref. 31.

5.1.8 Outardes 2, Canada.

The rockfill for raising an existing concrete gravity dam was placed in 0.9 m lifts and compacted with four passes of a 10 t vibrating roller. Maximum size of rock used was 750 mm with 5% fines passing the No. 200 sieve.

The slab, a uniform 30 cm thick, was reinforced in each direction with steel at 0.5% of the area of the slab. The reinforcement was increased to 1% at the perimetric joints. At other joints that were considered to be in compression, the steel was also increased to prevent spalling of the corners of the slabs.

Perimetric joints had two waterstops built in. The lower one was of copper and was deeply indented and prevented from collapsing under water pressure by a neoprene rubber insert retained by plastic foam. The P.V.C. waterstop at mid-thickness of the slab was 200 mm wide with a centre bulb. The plinth had a painted coat of bitumen on the surface that was in contact with the slab in an attempt to reduce shear forces. The vertical joints, separating the 12.5 m wide slabs, only had the P.V.C. waterstops at mid-thickness. An unavoidable horizontal construction joint at mid-height had an epoxy coating before the new concrete was placed. Reinforcement passes through this joint, Ref. 32.

5.1.9 Pozo de los Ramos, Spain.

The dam is 97 m high and it is proposed to raise it another 37 m to a total of 134 m at some time. The designers considered an asphaltic concrete face for its ease of jointing when raising. The flatter slopes required for the asphaltic concrete increased the cost and a gunite concrete membrane was chosen because of its low modulus of deformation and the ease of being able to thicken the facing when raising the height. The cracking pattern was expected to be of fine distributed cracks from the distributed nature of the thin reinforcing mesh. Reinforcing mesh was designed to have an area of 0.5% each way of the area of each of the layers of gunite. The percentage of reinforcement was to be increased, to 0.6% of the area, near the abutments.

The composite membrane consisted of a bedding and levelling layer of mortar for a P.V.C. sheet. On top of the P.V.C. porous concrete slabs protect the plastic and act as a drainage layer. 100 mm diameter drains run parallel to the slope and connect to the drainage gallery at the toe. The drains are 12 m apart and below the centre of the 12 m wide concrete facing gunite strips. By using a packer device in the drains leaks can be pinpointed. Various thicknesses of gunite were to be applied to complete the membrane. The maximum was ten layers to a minimum of five, each of 70 mm thickness. The steel reinforcing mesh was to be 8 mm in diameter at 150 mm centres each way, closing to 120 mm centres at the abutments. The strips would be anchored at the toe with a joint capable of some rotation.

Test slabs were subjected to bending tests to examine the effect of different amounts of reinforcement and distribution of reinforcement. The results proved that the modulus of deformation is lower for slabs with distributed reinforcement. Ref. 33.

5.1.10 Yacambu, Venezuela.

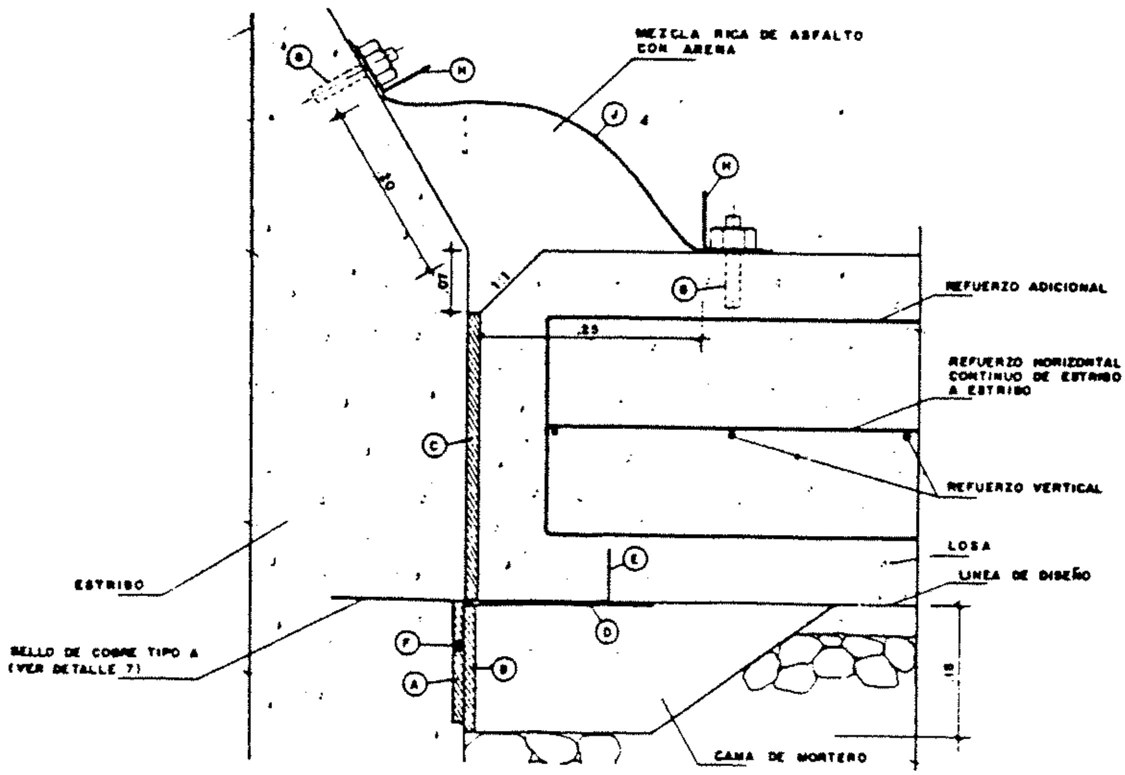
This 162 m high dam of compacted river gravels, features a 50 m high concrete dam at the toe. The remainder of the slope is protected by a concrete membrane. The zone immediately beneath the slab, 25 m at the toe decreasing to 11 m wide at the crest, was of processed 3 in. gravels in 30 cm layers compacted with six passes of a 10 t vibrating roller. The slab varies in thickness from 60 to 30 cm according to the formula, thickness T,

$$T = 1 + 0.0028\,H$$

where H is the head, T and H are in feet.

Reinforcement is of high yield steel and at 0.425 to 0.554% of the area of the slab. Extra reinforcement is provided as cages at the edges of the slab.

The design of the perimetric joint is a development of the idea behind the repairs to the perimetric joint at Alto Anchicaya. The details of the joint are shown on Figure 17. The joint consists of a folded copper seal with sand bitumen mix confined over the joint by a P.V.C. membrane held in place with stainless steel plates and bolts. Its principle is simple. Water pressure should push the sand bitumen into the joint when the joint opens. The premoulded asphalt filler is provided to avoid spalling if the joint rotates without



(A) Premolded asphalt.
(B) Premolded asphalt.
(C) Premolded asphalt.
(D) PVC Band $e = 6$ mm.
(E) Copper seal, type A.

(F) Neoprene cylinder $\varnothing = 1/2''$.
(G) Bolt $\varnothing$ 5/8″ × 3″ at 0.40 m.
(H) Angle 2 1/2″ × 2 1/2″ × 3/16″.
(J) Rubber band $e = 1/8''$.

Figure 17. Yacambu, Perimetric joint detail.

(After Martinez and Carrero, 1982.)

opening. Ref. 34.

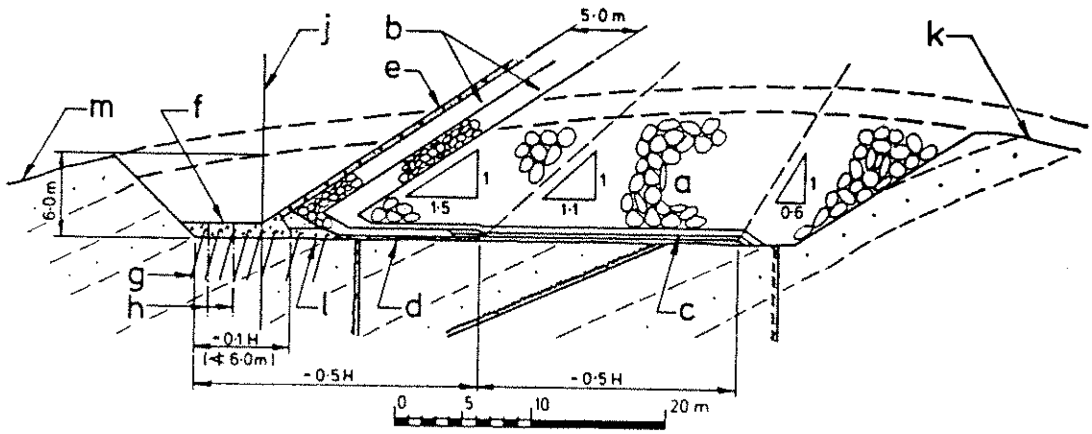
5.1.11 Sugarloaf, Australia.

Sugarloaf is a dam with an upstream membrane constructed where there were less than ideal foundation conditions. The bedrock was highly weathered with seams of clay and it was probably dispersive. Conditions were not considered suitable for the use of a narrow cutoff. The use of steel for facing the dam was rejected on the grounds that the low conductivity of the rock would cast doubts on the effectiveness of cathodic protection, and the facing was in addition vulnerable to construction defects. No Australian equipment or experience was available for the construction of an asphaltic concrete core. An asphaltic concrete faced dam was rejected on the grounds that no advantage was to be gained over an asphaltic concrete core. After careful consideration of these alternatives, a cement concrete face was thought to be most suitable.

High hydraulic gradients are normally acceptable across the plinth but the seams of clay and dispersive bedrock required design modifications from normal practice. The treatment at the plinth area consisted of excavating to at least 6 m below the top of the weathered bedrock. Clay seams were flushed, where possible, and grouted. Downstream of the plinth a blanket of reinforced concrete 150 mm thick, equal to half of the head in length, was placed. See Figure 18. Beyond this there was a zone, again of half of the head in length, of filters and drains. Rock anchors were used to ensure the stability of the plinth with one third of the head acting as uplift, and with the plinth over the most unfavourable clay seam. The clay seam had an effective friction angle of 10^0. Buttresses were installed on the right abutment in areas where there was large overbreak resulting in the plinth to slab joint being a distance above the foundation. Also upstream of these areas additional fill was placed to stabilise the plinth in this direction. Ref. 35.

5.1.12 Wishon and Courtright dams, California.

Both these dams are examples of dumped rockfill dams constructed about 1958. Concrete facing slabs were poured in 18 m long strips parallel to the axis of the dam. The width of the strips varied from 9.5 m to 23.8 m. The thickness of the slab is a function of the head. See Table 13 for thickness to head ratios. The horizontal joints are



(a) Rockfill.
(b) Transition zones.
(c) Filters.
(d) Foundation concrete.
(e) Concrete facing.
(f) Plinth.
(g) Anchor bars.
(h) Grout curtain.
(j) Plinth reference line.
(k) Foundation stripped to highly weathered rock.
(l) Buttress.
(m) Original ground surface.
(H) Hydraulic head at foundation level.

Figure 18. Sugarloaf, Cutoff details.

(After Casinder and Stapleton, 1979.)

three-quarters of an inch wide protected by a copper waterstop and a redwood filler. Vertical joints are 1 to 2 inches wide, protected with a waterstop and sealed with premoulded asphalt joint filler and rubberised asphalt. Ref. 36.

5.1.13 Villagudin, Spain.

The membrane at the cutoff wall and drainage gallery is 35 cm thick but quickly tapers to 30 cm. Slabs are 10 m wide supported by a 10 cm layer of porous concrete. Beneath this is a 50 cm layer of rockfill with maximum rock size of 40 cm. The main rockfill was placed in 1 m thick layers with a maximum rock size of 1 m and compacted by at least four passes of a 10 t vibrating roller (vibrating at 25 Hz.). The rock was sluiced with a volume of water equal to 50% of the rock. The bedding layer was compacted in preparation for the concrete with 15 passes of a 5.5 t (27 Hz) vibrating roller.

The joint details are shown on Figure 19. Both perimetric and vertical slab joints are protected by two waterstops. Both joints have timber fillers and a mastic plug to the top of the joint. Additional reinforcement is provided at the edges of the slabs to prevent spalling. Ref. 37.

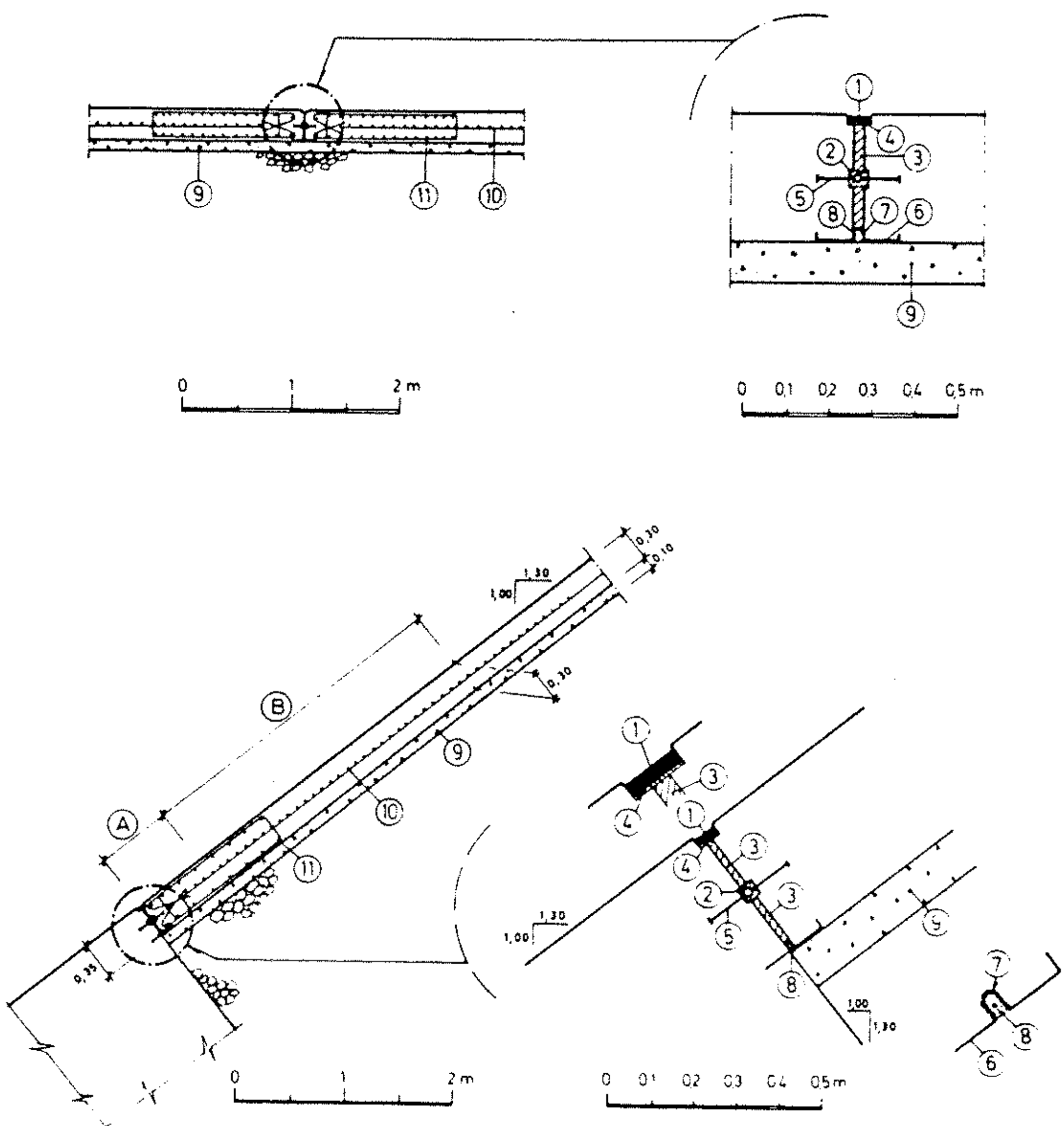
The rockfill volume of 223,700 m^3 was placed in six months. The major part of the slab, 10,070 m^2 was placed in seven weeks. Both of these rates were fast.

5.2 Asphaltic Concrete Membranes.

The details of the membranes of a large number of asphaltic concrete dams are published by ICOLD in Bulletin No. 32. Some case histories are described here. The details of the composition of the membranes, placing temperatures, specified densities, permeabilities and void content can by obtained from the above publication. These details are not discussed here except where they differ from the norm. Some construction details are summarised for the dams discussed on Table 9.

5.2.1 Dungonnel, Ireland.

This dam was completed in 1969 and is 16.7 m high with an upstream slope of 1:1.7. The rockfill was of basalt with less than 15% passing 150 mm and less than 2% passing 38 mm. Lifts were of 1 m, sluiced and compacted with 8 t vibrating rollers. The



(A) Membrane thickness 35 cm.
(B) 35 cm to 30 cm transition.
(1) Poluyrethane adhesive resilient filler.
(2) Deformable inert material.
(3) Creosoted 2 cm-thick beech plank.
(4) Building paper.
(5) 23 cm PCV seal strip.
(6) 1.5 mm copper strip.
(7) Asphalt painted on.
(8) Deformable inert filler (Neoprene).
(9) No-fines concrete.
(10) 14 mm bar, spacing 10 cm both ways.
(11) 10 mm bar on 10 cm centres.

Figure 19. Villagudin dam, Upstream membrane and joint details.

(After Hoyo, 1982.)

drainage layer was sealed at 30 m centres and connected to individual collector drains. The drains were led through the embankment to the downstream toe and flow measuring manholes. No leakage has been reported. The surface of 4,200 m^2 took two months to complete.

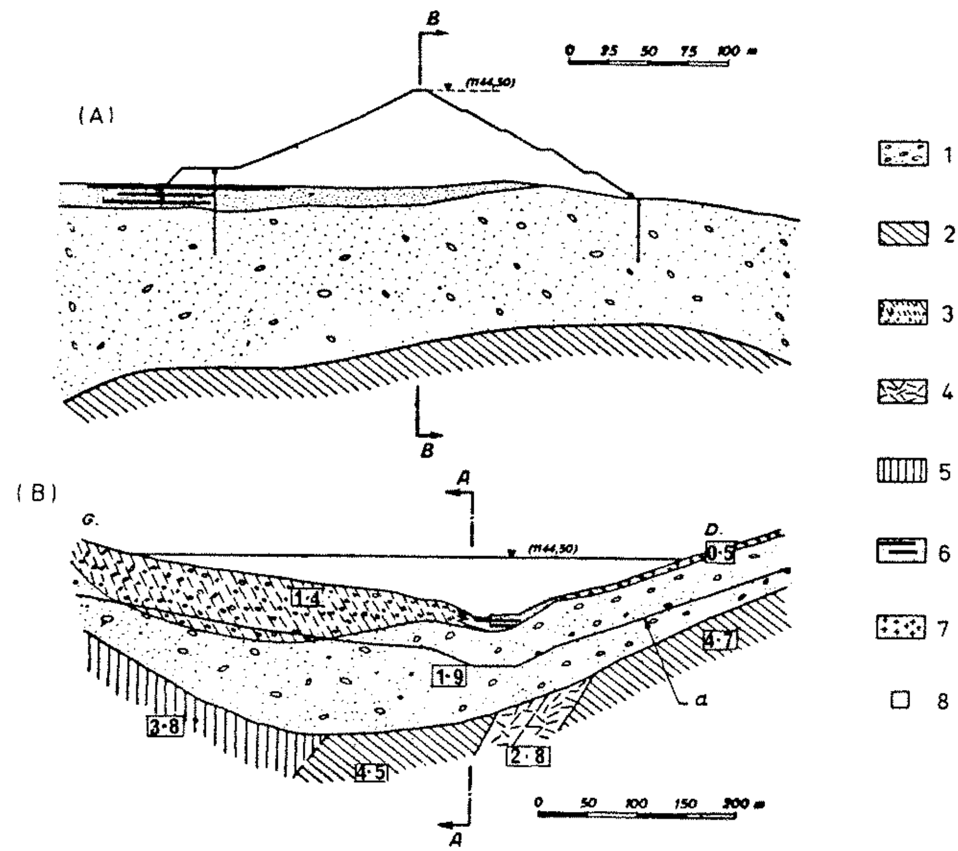
Considerable attention was paid to anti-seepage measures at the upstream toe. The dense asphaltic concrete was thickened to connect with a pocket cut in the top of the concrete cutoff wall. Contact surfaces of the cutoff wall were painted with a thixotropic bitumen paint. The impervious layers were protected further by a copper backed bituminous felt extending a short way up the slope. The whole connection was then protected by a layer of 2 ft wide precast concrete slabs bolted to the cutoff wall. Backfill of maximum size of 100 mm was placed in 230 mm lifts and compacted to a height of 1.22 m above the cover slabs. Ref. 38.

5.2.2 Zoccolo, Italy.

Although this is an earthfill dam, it is an interesting case having been founded on a considerable depth of moraine, outwash deposits and alluvial material. The geological succession is shown on Figure 20. The seismic velocities are shown on the figure to give an idea of relative densities. The moraine material had a density of 2.45 t/m^3 and a permeability of 10^{-4} to 10^{-5} cm/sec. The outwash had a more variable composition and a permeability of 10^{-1} to 10^{-3} cm/sec. The alluvial material was very variable in composition and included lake sediments and had permeabilities as low as 10^{-8} cm/sec.

Prior to the construction of Zoccolo the choice of foundation treatment on thick deposits in river or glacial valleys had been:

1. A rigid concrete cutoff wall built by direct excavation of trenches and shafts down to a depth of 20 – 30 m. Dams using this technique included, San Valentino, the first stage of Vernago, both in Italy and Castiletto in Switzerland.
2. A pile cutoff wall constructed from the original ground level. Dams using this technique included, Maria al Lago and the second stage of Vernago in Italy.
3. A grout curtain of cement and bentonite mixtures. In some cases fine aggregates or chemicals had been added to the mix. Some foundations had been treated only with chemical grout. Dams using grout had included, Silverstein, Germany, Mittmark,



(A) Section.
(B) Longitudinal section.

(1) Moraine.
(2) Mica-schists and paragneiss.
(3) Fan outwash.
(4) Mylonite.
(5) Phyllite.
(6) Alluvial.
(7) Talus material.
(8) Seismic speed.
(a) Cutoff limit.

Figure 20. Zoccolo, Geological sections.

(After Dolcetta and Chiari, 1976)

Switzerland, Serre Poncon, France, and Durlassboden in Austria where the bottom of the reservoir had been partly covered by an impervious blanket.

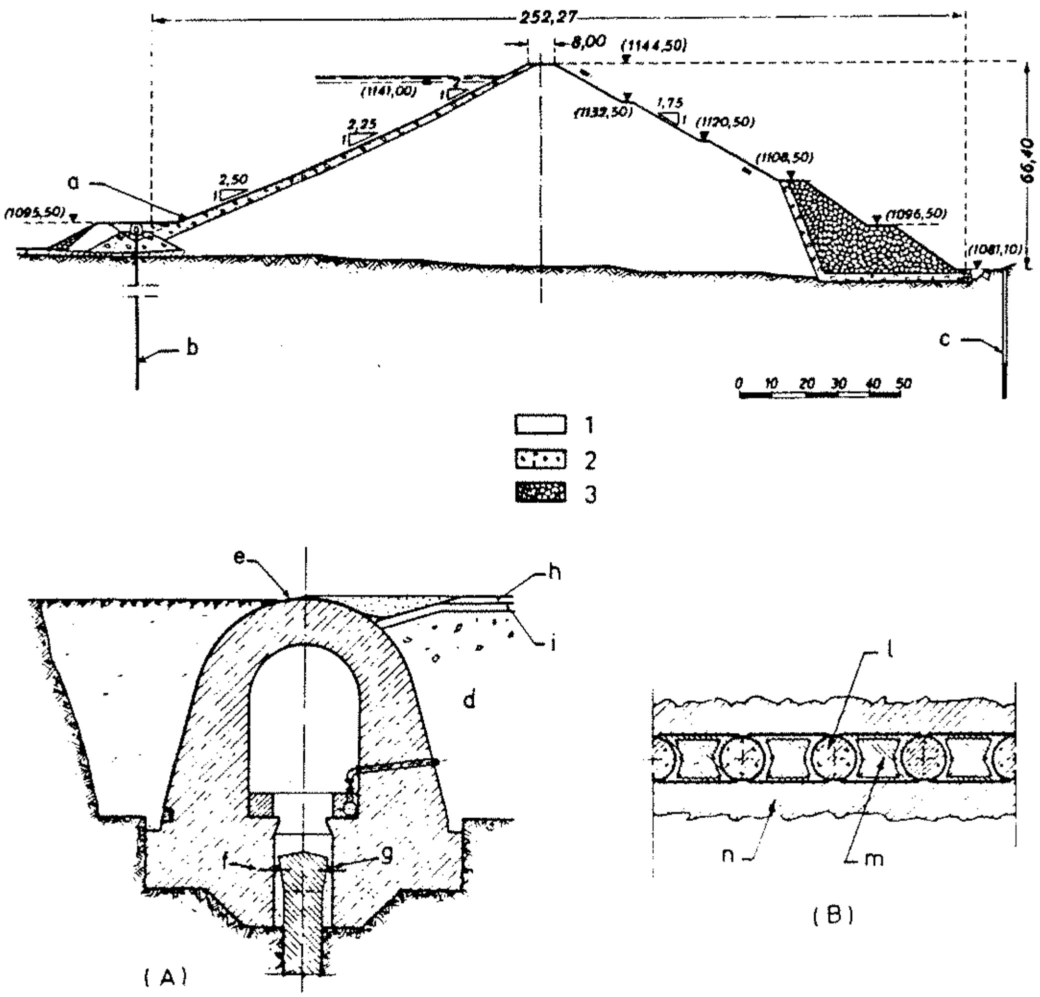
4. A horizontal impervious upstream blanket.

The choice of ICOS, slurry trench walling was made as grouting was not considered reliable enough and an upstream blanket would not be effective due to the high horizontal permeabilities. Two previous dams had used the ICOS system, Mongulfo and Vodo, Italy. Slurry trench walling is compatible with the use of an upstream impermeable membrane.

The dam section is shown on Figure 21 showing the filter behind the facing and around the drainage gallery. The ICOS panels were 60 cm thick and extended approximately 50 m below ground level. The depth of the cutoff is shown on Figure 20. Extensive studies were made to determine the probable seepage pattern and the flow for a cutoff not reaching all the way to bedrock. Relief wells and a rockfill toe were provided as a result to ensure the stability of the dam.

The horseshoe shaped gallery was expected to move downwards and rotate relative to the cutoff. To allow for the differential movements and still remain watertight the cap to the cutoff was situated 6 cm from each side of the slot in the gallery. P.V.C. waterstops were used to seal the gap and 8 cm of neoprene mastic adhesive provided additional security. See Figure 21 for details of this arrangement. During the first filling the movement was monitored and after 5 cm and 3 cm settlement of the gallery and cutoff respectively, the floor of the gallery was concreted closing the horseshoe. When the movement had reached 6 cm and 5 cm the space between the floor and the cap and between the gallery and cap below the waterstop was grouted. Horizontal movement of the cap and the gallery was about 9 cm. See Ref. 39 for details of the construction and Ref. 40 for details of the dam's early performance.

The dam suffered increasing leakage, 120 l/s to 240 l/s. While the upstream facing was generally effective, in more than ten years of operation the cutoff wall had deteriorated so that repairs were necessary. The remedial program, completed in 1977, involved the injection of cement bentonite grout to form a curtain just upstream of the cutoff and along its full length. On inspection during the drawdown of the reservoir there were found some blisters 10 to 20 cm in diameter, and an open joint in the dense asphaltic



Dam section.

(A) Detail of the joint between upstream facing and cutoff.
(B) Detail of the cutoff.

(1) Embankment.
(2) Filter.
(3) Rock-fill.
(a) Upstream bituminous facing.
(b) Cutoff.
(c) Relief well.
(d) Filter.
(e) Impervious strips.
(f) Polyvinil water-stop.
(g) Neoprene mastic.
(h) Bituminous impervious layers.
(i) Binder.
(l) First type pier.
(m) Second type pier.
(n) Bentonite penetrated zone.

Figure 2.1. Zoccolo, Dam section and details of the cutoff.

(After Dolcetta and Chiari, 1976)

concrete primary layer. Some horizontal cracks were found in the sealing coat. These defects in the membrane were not considered significant. After two years operation, after grouting, piezometric measurements indicated appreciable water pressure differentials upstream and downstream of the cutoff. Seepage was reduced to about half its former value. Ref. 41.

5.2.3 Godey, Switzerland.

The dam was founded on 20 to 30 m of alluvial material. A slurry trench cutoff to rock was used to seal the foundation against seepage. The cutoff would be rigid while the alluvial material could settle under the weight of the dam. The problem of differential settlement was solved by constructing a transition concrete slab. See Figure 2. The slab was designed to distribute settlements between the dam body and the cutoff and to avoid shear failure or cracking of the asphaltic concrete facing. Ref. 42.

Surfacing of this dam took two seasons to complete. During the first season 3,600 m^2 of the membrane was laid. Due to the limited capacity of the hopper it was necessary, on the longer slopes, to form the surface layer in two passes. The layer was rolled only once in order to prevent the formation of two separate impervious sheets with the consequent risk of the formation of blisters.

5.2.4 Luddington, U.S.A.

The embankment was provided, for additional safety, with a vertical chimney drain. A seepage monitoring system is combined with drainage of the 45 cm thick drainage layer. Drains were not allowed to pass through the embankment because of its piping potential. A sandwich type of membrane was used to improve on the watertightness of the membrane. Submersible pumps were installed in the drain holes within the drainage layer behind the outer impermeable asphaltic concrete layer. The capacity of the pumps is such that they can empty the drainage layer at the rate matching the 2.44 m/hr maximum drawdown rate of the pumped storage scheme. Uplift pressures in the drainage layer are thus avoided and the pumps are set to operate only when seepage water reaches predetermined levels. Monitoring of the pumps' operation gives a sophisticated leak detection system. One leak of 19 l/s was detected around the reservoir outlet stucture.

Ref. 43.

5.2.5 Bigge, Germany.

Leakage of 5×10^{-3} l/s occurred for a total area of 46,000 m^2 of asphaltic concrete. The crest settled a maximum of 123 mm at a rate of 1 to 2 mm per year later on in its life. Where two courses were applied blisters occurred in some places. Some of the blisters formed between the two courses during construction. The remedy used was to apply heat to the surface of the lower asphaltic concrete layer to dry it out completely before applying the second layer. Some blisters appeared later and under the pressure of the water travelled down the slope. On examination some of the blisters contained large quantities of water. The water was determined to have come from the drainage layer that was unsealed at the parapet. Ref. 19.

5.2.6 Ogliastro, Italy.

The rockfill embankment was of calcarenite and the surface was sprayed with sodium chloride solution to inhibit plant growth. During construction the joints were preheated by infrared before laying the adjacent strip. At the headwall the asphaltic concrete was reinforced with 'Trivia' net. Any seepages are collected by a drainage gallery on one side and by a 40 cm diameter drain on the other side of the reservoir. The upper section of the completed membrane was sprayed with 1 l per 10 m^2 of lime vinyl solution to counter the effects of high sun temperatures.

During the first filling a leakage of 1.4 l/s occurred at one monitoring station and the total leakage was 2.8 l/s. A 10 m drop in the reservoir level reduced the leakage to 0.5 l/s. One month later again at F.S.L. the leakage increased to 7.3 l/s. Divers spread bentonite over small cracks discovered in flat portions of the membrane. Ref. 44.

5.2.7 Pla de Soulcem and Le Verney, France.

These two dams have been built since the publication of Bulletin No. 32 without a drainage layer directly below the membrane. At Pla de Soulcem the bedding for the membrane is rock with a maximum size of 200 mm in a zone 7.5 to 15 m wide. The permeability of the bedding is estimated to be 10^{-5} cm/s and a transition filter is provided between the bedding and the rockfill with a permeability of 10^{-1} cm/s. The stability of the dam is assured in the case of a major leak with a drain every 15 m vertically with a permeability of 10^{-3} cm/s and a grading in the range 0 – 500 mm. The general rockfill has a grading between 0 – 800 mm. La Verney is similar but the stability is assured with a chimney drain. Ref. 45.

5.2.8 Miyama, Japan.

This dam is one of the largest to have an asphaltic concrete membrane. The reasons for the choice of an asphaltic concrete membrane were, the large fluctuations expected in the reservoir level and the lack of suitable fines for a cored dam nearby. A multilayer membrane with an intermediate drainage layer was chosen. The cutoff wall had a drainage gallery to which the drainage layer was connected. Ref. 46.

An unusual dam was reported in the above reference having been built of volcanic ash. The dam was Ninokwa, and used the ash in 30 cm layers.

5.3 Thin Membranes.

5.3.1 Aguada Blanka, Peru.

This is a recent example of the use of a metal membrane. The dam is 45 m high and uses a 5 mm iron facing. The temperature range is severe from −22°C to +31°C. The geology of the site ruled out a rigid structure because of expected large deflections of the intertuff and lacustrine materials in the foundations. The abutments also contained weak rocks. There were no suitable fines for a zoned dam and the short high nature of the gorge made flat slopes to the dam impossible. Difficulty would also have been experienced in sealing an impervious core to the steep, often overhanging, sides of the gorge.

Iron was chosen with a low percentage of impurities to aid in a slow corrosion rate. The facing was bedded onto a flexible sand bitumen layer which was also expected to protect the downstream side of the membrane against corrosion. The bedding was also intended to provide a smooth working surface, eliminating the danger of rocks penetrating the membrane and to equalise pressure transfer. The facing was painted with a light coloured acrylic paint as corrosion and thermal protection. Two transition zones, 2 m and 3 m wide compacted in 0.25 m and 0.5 m layers respectively, satisfying D_{15}/D_{85} filter criteria, were provided beneath the sand bitumen layer. Compaction was by 4 passes of a 1 t vibrating roller. The main rockfill was placed in 80 cm lifts and compacted with a 5 t vibrating roller.

The thickness of 5 mm was chosen as being the minimum thickness practicable for site welding. Expansion joints were provided with 30 mm of play in a 10 m by 16 m bay. One anchor allowing some movement of the membrane was provided at the centre of each bay.

The dam has performed satisfactorily.

5.3.2 South African Experience.

Experience in the use of butyl rubber used for waterproofing and slope protection of 45,000 and 90,000 m^2 embankments is described in Ref. 47. The water depth was 6 m and the slopes of the embankment were 1:3. The bedding was of a fine selected compacted soil. The membrane of butyl rubber was 0.75 mm thick and was obtainable in 28 m wide strips. Site joints were made with a cold adhesive.

In use field joints deteriorated along water edges on dry freeboard. Hailstones punctured the rubber at some locations causing soil erosion beneath the membrane. Later reservoirs have 1.5 mm thick membranes from 1 m below the F.S.L. to the top of the freeboard. It was noted that ozone attack was reduced if the sheets were not in tension.

5.3.3 Radin Isvor, Bulgaria.

The original design called for a 46 m high dam with a 0.3 m thick membrane of concrete on a levelling layer 15 cm thick. To accelerate completion a synthetic thin membrane was used instead of the concrete. The embankment was over built and then trimmed to the correct profile of the face. The final slope was compacted with a 2 t roller. Longitudinal reinforced concrete beams were fixed to the face and levelled. Sand was compacted between the beams. Precast reinforced concrete slabs were laid over the beams and exposed reinforcement was welded to the beams. A rubber bitumen composition 2 mm thick was stuck to the slabs, a 0.2 mm high density polythene sheet was placed on top and covered by a second layer of rubber bitumen. The whole assembly was then covered with more precast concrete slabs, again welded together and the joints caulked. Ref. 48.

5.3.4 Czechoslovakian Experience.

Repairs to a badly leaking 14 m high earth and sandy clay dam are described using a thin membrane. A 0.9 mm thick P.V.C. membrane protected with an 'asphalted board' on each side formed the new impermeable element. Tests were made to determine the shear resistance of the soil on the boards. The result was used to check the stability of the repair. The membrane was extended at the toe to create an impervious blanket and then anchored in a trench. Site joints were welded with an overlap of 20 cm. A soil protection was placed on top of the membrane and compacted with rubber tyred rollers. Leakage was reduced from 20 l/s to 3 l/s. Ref. 49.

5.3.5 L'Osperdale, Corsica.

This 26 m high granite rockfill dam has an unusual development of membrane. The initial proposal was for a conventional asphaltic concrete design with two layers of asphaltic concrete over a porous asphaltic concrete drainage layer. The new impervious structure begins with 2 m of 25 – 125 mm sized gravel surfaced with crushed 25 – 50 mm gravel. A layer of cold rolled asphaltic concrete provides the smooth surface for a non–woven polyester felt. The membrane, 5 mm thick, consisting of bitumen impregnated polyester felt, glass fibre and terphane film was covered with another layer of the felt. The final protection was provided by interlocking concrete paving slabs. The membrane was anchored by metal plates and bolts at the toe and by burying it in a trench at the crest. Ref. 50.

After filling and three years of operation the dam has settled very little, 7 mm vertically, stabilising after one year. Horizontal movement was 2 to 3 mm. Leakage was 5 l/s probably through the joints in the cutoff wall as after caulking in 1979 the leakage reduced to 2 l/s. Some small movement had occurred in the interlocking facing leaving a gap at the crest which was sealed by mortar. The blocks had also twisted where the dam had bulged slightly. Some resetting of the blocks removed the problem. Weathering of the surface of the blocks has occurred but does not affect the performance of the dam. Ref. 45.

5.3.6 Codole, France.

Sheets of 2 mm thick P.V.C. in 6 m widths were used to form the membrane for the 28 m high Codole dam. A 2 m thickness of gravel in the size range 25 - 125 mm was placed over the upstream face of the rockfill and covered by a 10 cm layer of bitumen bound stones 25 - 50 mm size. The surface was blinded by a 5 cm layer of 3 - 6 mm size rolled to form a smooth surface. Great care was taken to remove all the small stones etc. from this surface before laying a 4 mm thick polyester felt and then the P.V.C. sheets. These were overlapped by 6 cm and welded together by an automatic machine. Further sheets of 4 mm felt were placed over the P.V.C. and held down by 8 cm thick precast concrete blocks. Ref. 45.

5.3.7 Miel, France.

The membrane used for this 15 m high dam was butyl rubber 1 mm thick. A filter 20 cm thick of round gravel was provided directly below the membrane. As surface protection a 20 cm layer of sand was used. The membrane is anchored at its base to the cutoff structure and is buried in a trench at the crest. The upstream slopes are 1:2.5.

5.3.8 Neris, France.

The membrane used was of butyl rubber 1.5 mm thick for the 18 m high, 16.5 m of water, dam. On the embankment, of quarry run stone, a layer 3 to 4 cm thick of cold rolled pervious asphaltic concrete supported the membrane. Protection to the membrane was provided by a layer of geotextile covered with 4 cm thick precast, perforated, concrete slabs, 1 m x 0.8 m. The membrane is anchored at the top and bottom in trenches and sealed to the cutoff wall at the toe by concrete. The upstream slope is 1:1.6.

5.3.9 Dobsina, Czechoslovakia.

The impervious element of this dam is formed by sandwiching a layer of bituminous felt 0.9 or 1.1 mm thick, between two layers of interlocking, precast concrete slabs. The interlock mechanism, shown on the sketch below:

allows some movement of the slope yet provides protection for the membrane from ice and weather. The lower layer of precast concrete slabs is bolted onto the dam at intervals. The dam was completed in 1960, Ref. 7, and reported in 1973, Ref. 8, to be in good condition. The system was used again at the 26 m high Landstejn dam.

6. REPAIRS

The leakage of water through a dam with an upstream impermeable membrane occurs quite often. The acceptability of the quantity of leakage depends on many factors. There are four major reasons for repairing the leaks. These are:–

1. The safety of the dam is threatened by the leakage water. This could be because:
   a. A rise in pore pressure in the downstream slope would cause stability problems there.
   b. Seepage water is raising the pore pressure in the abutments causing stability problems there.
   c. The filters and drains are running full with the leakage water and infiltration. Additional flow could not be carried and the pore pressures in the dam would start to rise.
   d. The water is reducing the shear strength of the rockfill.
   e. Leakage water is causing internal piping or erosion where the water emerges from the dam.
   f. The stability of the dam during an earthquake is reduced with a saturated fill or filters.
2. The rate of settlement is increased by the leakage water. Settlements could become greater than those tolerable by the structures such as the spillway or outlet works. Settlements can be caused either by the reduction in the strength of the rock upon wetting or from internal erosion.
3. Public concern. A leaking dam is regarded by many people as being unsafe. This is especially true if they are living downstream of the dam.
4. The cost of the lost water may not be acceptable. What may be an insignificant loss to a power generating station on a large river may be a significant leak in a dam used for irrigation in a dry area.

The cost of repair has to be weighed against the cost of not doing the repair and accepting the consequences. Evaluation of the risk is left to the engineer responsible. Some case histories where repairs were carried out are presented.

6.1 Concrete faced dams.

6.1.1 Cuga dam, Sardinia.

The dam is 47 m high of vibro-compacted trachite rock with unusually steep upstream slopes of 1:0.7 and 1:0.55. The thickness of the slab varied from 0.5 to 0.3 m. Vertical joints were provided at 12 m centres. One mid-height horizontal construction joint was used. Bedding to the slab was cement mortared stone pitching, 2.8 to 1 m thick.

During construction cracking of the face was so severe that the reservoir was never filled. The reasons for the cracking were determined as being:

1. The dam was founded on a complex volcanic formation with three main faults in the area.
2. Differential settlements occurred in the bedrock and in the overlying alluvial materials.
3. Shear of the bond between the concrete and the stone pitching caused the whole of the weight of the facing to bear on the cutoff wall at the toe. The concrete of the facing crushed and buckled. The cutoff, weakened by the internal drainage gallery, cracked and moved out of line.
4. Grouting of the foundations was thought to have contributed to the movements.

Repair was effected by constructing an impervious clay facing to the concrete and building an upstream rockfill shoulder. Semi-open asphaltic concrete, 0.6 m thick, was placed on the face of the concrete to act as a filter. The original concrete facing was bolted back onto the rockfill and grouted to fill any remaining voids. An impervious membrane was added to the contact surface between the new core and the old cutoff. Gunite was used to smooth off the profile before applying the membrane.

After filling the maximum downstream movement of the crest was 40 mm. Leakage was only 13 l/min. Ref. 51.

6.1.2 Courtright, U.S.A.

The dam is a concrete faced rockfill embankment 97 m high. In concreting the upstream face the lifts were only 9.1 m high and filled many irregularities in the rockfill. The face was thicker than the design maximum of 91 cm, reaching 213 cm in places. The joints between the slab and the cutoff wall were made rigid. In 1961, the joints at the junction between the cutoff and face slabs were modified to allow rotation. Settlement of 178 cm had occurred on the crest. Leakage had increased to 1.26 cumec. In 1968, a complete inspection revealed crushing and buckling of some slabs especially those close to the perimetric joint. The reinforcement had buckled and the concrete had sheared. Frequently a plane of failure coincided with the plane of the reinforcement.

Repairs consisted of removing crushed concrete and adding reinforcement where the new slab was thicker than the old. Reinforcing stirrups were added near the joints. A thrust block was added to counteract the eccentricity of some sections of the cutoff. 1,202 m^3 of concrete was removed and 1,606 m^3 replaced.

As an additional precaution an earthfill blanket was added to the lower 30 m of the slope. Minimum reservoir level is restricted to 1 m above the top of the blanket to avoid drying out of the blanket. Ref. 52.

6.1.3 Alto Anchicaya.

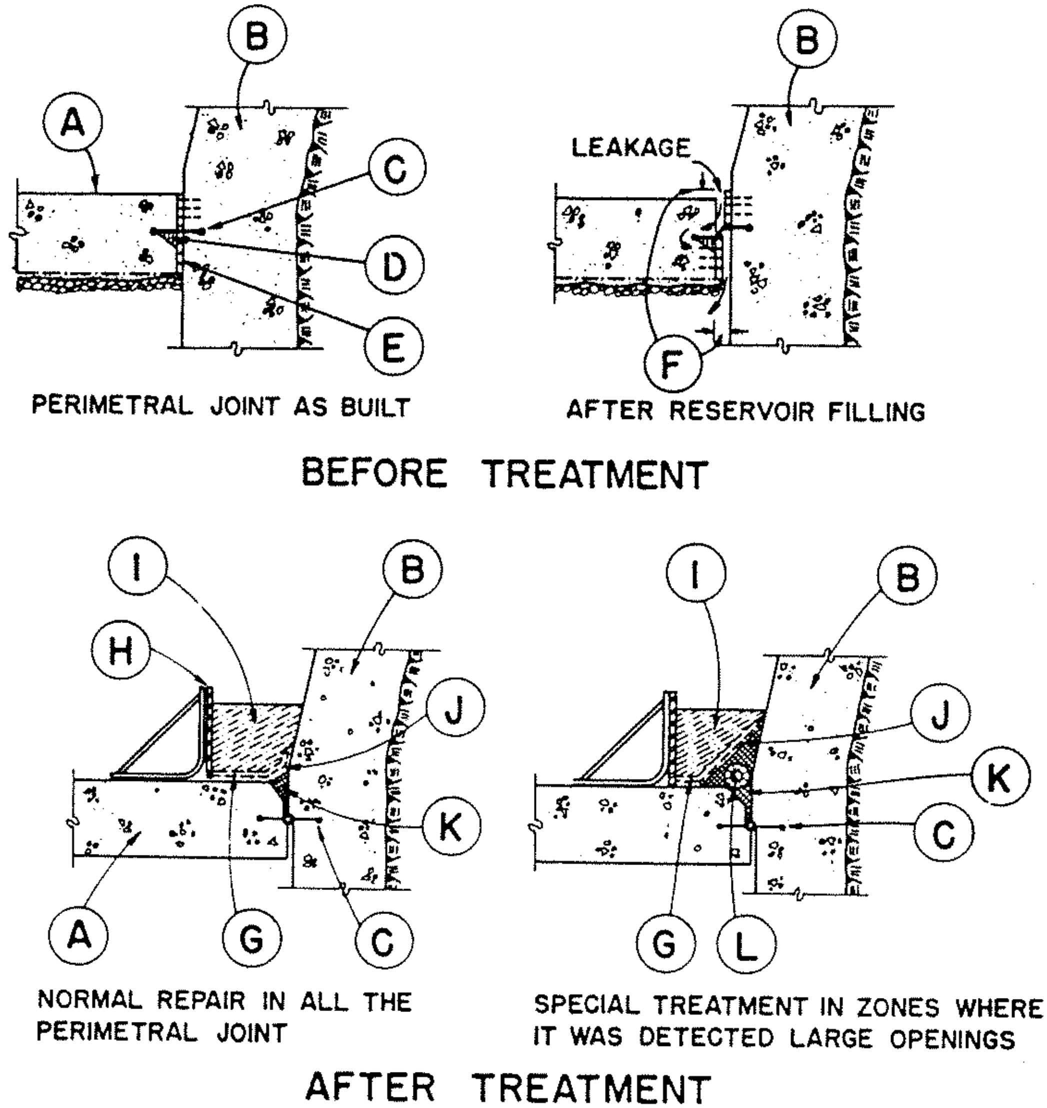
Alto Anchicaya has been described in the previous section. A little more detail of the repairs to the joints is shown on Figure 22. A trough containing compacted clay was constructed over the joint. The joint itself was filled with a bituminous mastic and separated from the clay by a sand bitumen mixture. Where joint openings were large a neoprene tube was embedded in the mastic to provide additional flexibility.

6.2 Asphaltic concrete faced dams.

6.2.1 Scotts Peak, Tasmania.

Extensive cracking occurred of the 43 m high rolled rockfill dam in the asphaltic concrete facing. Cracks were attributed to excessive settlement of the weak argillite rockfill. Differential movements occurred between the rockfill and a zone of compacted gravel, see Figure 23. Leakage of 5 l/s increased to 100 l/s. The water was needed for irrigation in a dry area and losses of this magnitude could not be tolerated.

Butyl rubber sheets were placed over the cracks by divers and temporarily reduced the flow. See Figure 24 for the location of the patches. The permanant repair



(A) Hinge slab.
(B) Plinth.
(C) Rubber water stop.
(D) Zone where concrete did not penetrate well.
(E) Wood filler.
(F) Movement of joint (after reservoir filling).
(G) Chicken mesh.
(H) Retaining plate.
(I) Compacted clay.
(J) Sand-asphalt mixture.
(K) Mastic.
(L) Rubber hose.

Figure 22. Alto Anchicaya, Remedial treatment of the perimetric joint.

(After Cole and Fone, 1979.)

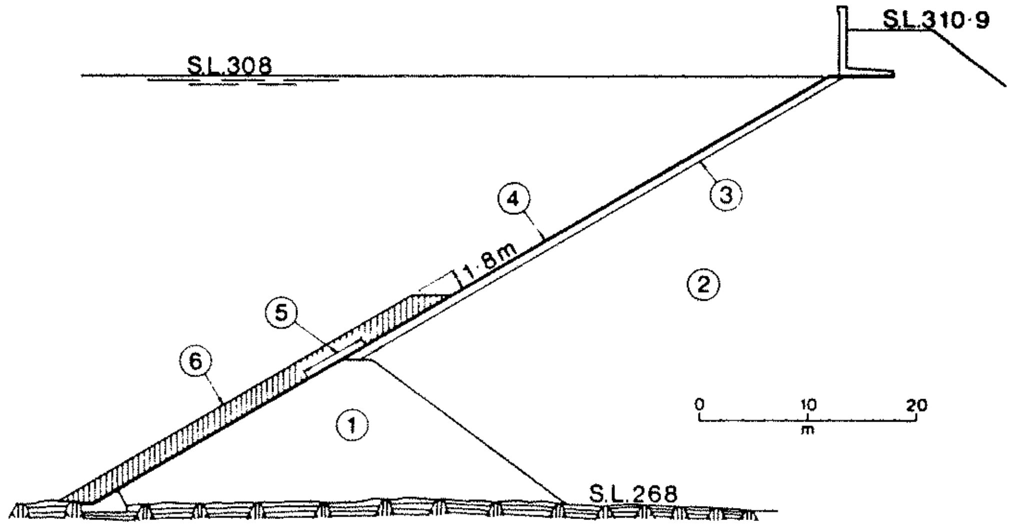


Fig. 1

Scotts Peak Dam – Maximum Section.

(1) Compacted gravel.
(2) Compacted argillite rockfill.
(3) Crushed dolomite (— 100 mm).
(4) Bituminous concrete face.
(5) Location of face cracks.
(6) Gravel blanket as designed.

Figure 23. Scotts Peak, Maximum section.

(After Cole and Frone, 1979.)

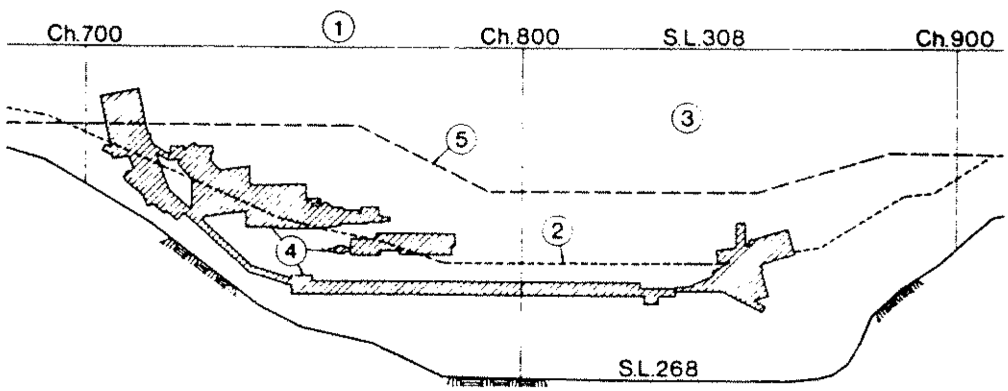


Fig. 2

Scotts Peak Dam – Upstream Face.

(1) Top of upstream face.
(2) Top of gravel zone inside dam.
(3) Bituminous concrete face.
(4) Butyl rubber patches.
(5) Top of gravel blanket.

Figure 24. Scotts Peak, Upstream face location of patches.

(After Cole and Frone, 1979.)

had to be flexible and impervious and dewatering could not be allowed due to the requirement for the water.

The solution adopted was to use an impervious gravel mixture placed under water from a large hopper with opening doors. A barge was used to transport the hopper and its load of 23 m^3 of gravel. 550 separate mooring locations were needed in the four and a half month operation. Surveys of the gravel using echo sounders and plumb lines indicated that the underwater slope of the gravel was shallower than that designed. To achieve the required 1.8 m thickness three separate overlapping lifts were required.

A rapid reduction in the quantity of leakage to 3 l/s occurred. Water in the leakage pond became clear soon after the completion of repairs. The settlement, that was averaging 65 mm a year before the repair, dropped to 8 mm per year.

A 3-D finite element study was performed by successively reducing the modulus of the fill until the observed deformations were achieved. However, at best fit, the computed downstream movements of the crest were only one third of those measured. The study indicated that a zone of tensile strain extended from the foundation up to about 10 m above the gravel zone and covered a length of 300 m. Tensile strains to the left of the deepest section were higher than to the right. This difference was consistent with the lower level of the downstream toe on the left. The tensile zone modelled was fully compatible with the actual cracking of the dam. The presence of tensile zones at the crest was corroborated by the opening of the joints in the crest wall. Inspection of the asphaltic concrete face above water level in this area revealed only surface cracks and no evidence of leakage.

It was noted that the usual filter requirements were not met between the crushed dolomite layer under the face and the segregated boundary of the rockfill. Thus it was postulated that when leakage developed some of the dolomite was washed into the rockfill removing local support from under the face. This allowed the face to deform into the local cavity and cause the cracking. Ref. 15.

6.2.2 Sarno, Algeria.

Constructed in 1952 this 28 m high dam had the asphaltic concrete spread by hand and compacted with 350 kg vibrating rollers. The membrane was completed to half height so that water could start to be impounded. Leakage of 150 l/s occurred. Upon investigation it was discovered that the asphaltic concrete was permeable although laboratory tests had indicated that the permeability of the mix would be satisfactory. It was determined that the laboratory tests had a much higher tamping efficiency than that occurring in the field. The design was based on that used for roads and it was concluded that the road asphaltic concrete was impermeable because the rolling caused the bitumen to rise to the surface forming the impermeable layer there.

Caulking tests with bentonite were tried but were not entirely satisfactory. No guarantee could be given that the dam would remain watertight after a number of filling and emptying cycles. The surface was therefore removed and replaced with a newly designed asphaltic concrete which had 0.2% more bitumen and double the limestone filler (9 to 17% filler). Several areas were found to be pervious after the new surfacing had been laid. These areas were cut out and replaced again. The replaced areas were where poor compaction was suspected. A simple insitu permeability device was developed to test the asphaltic concrete for leaks. Negligible leakage occurred after the repairs.
Ref. 53.

6.3 Lessons.

Several lessons can be learned from these few failures. Cuga dam is a sequence of disasters primarily resulting from inadequate site investigation or interpretation. The consequence of the poor site investigation was that the wrong choice of the type of dam to be used was made. The joint repairs at Alto Anchicaya are significant in the development of a joint design that can accommodate large movements. Inhomogeneity in the dam should be avoided to avoid the type of cracking that occurred at Scotts Peak dam. Designing the dam to act as a filter to prevent erosion by the leakage water is important especially as complete watertightness is rarely achievable. The mix design should take into account the condition under which it is placed to avoid the extensive repairs that were required on the Sarno dam. Settlements of the rockfill can be allowed for in the design of the joints. Rigid connections can lead to severe concrete crushing and buckling as occurred at Courtright dam.

7. COLD WEATHER PERFORMANCE

The details of very few dams constructed in cold weather or operated during cold weather have been reported. Temperatures do drop significantly at higher altitudes and some details of dams operating at the higher altitudes follow. It would be expected that asphaltic concrete or concrete membranes would be constructed during periods of good weather in the summer. Both types of membrane can be constructed rapidly to suit the local conditions. Asphaltic concrete has the advantage that cold joints can be made more easily and operations could start up for only a few hours if necessary.

The possibility of ice lens formation behind the membrane causing bulging and cracking need only be considered for fine grained fills. The grading of normal filters or bedding behind the membrane is such that water cannot be drawn to the freezing front. The freezing of the water from leaks through the membrane is only likely to occur at or just below the waterline. The freezing of the water is then likely to plug the leak reducing the buildup of ice. Fluctuating waterlevels during freezing weather are likely to cause plugging of the filter by ice. The ice would melt out quickly in the spring and should cause no further problem.

No damage to concrete faced dams by the action of ice has been reported by Sherrard, Ref 4. Substantial thicknesses of ice have been driven against the facings during storms without damage. At Genkel dam during most of the winter the ice is observed to stop short of the face of the dam. There is a space of several inches of water between the ice sheet and the membrane. Sherrard speculates that warm air in the drainage gallery at the toe of the slope keeps the membrane sufficiently warm to prevent the buildup of ice. Similar performance has occurred in winter at all other German dams. In the case of Henne dam which has a similar asphaltic concrete membrane the maximum ice thickness reaches about 17 inches and no strong bond forms between the ice and the dam.

7.1 Performance of some dams in cold weather.

7.1.1 Montgomery dam, U.S.A.

Winter temperatures fall as low as −32°C. The dam has an asphaltic concrete facing. A few minor surface cracks 0.25 to 0.5 inches wide and 4 inches deep have been attributed to the cold. The cracks were filled with sand and bitumen and the performance of the dam has not been affected.

7.1.2 Horchwurten, Austria.

An asphaltic concrete membrane was chosen here because there was no suitable impervious core material to be found at the high altitude of the dam. Stage construction was proposed favouring asphaltic concrete's relative ease of making joints. The elevation of the dam, 2,400 m above sea level, means that there is always ice present on the reservoir. The level of the reservoir fluctuates rapidly with the reservoir being used for pumped storage power generation. The slope of the upstream face is 1:1.65 and it was found that floating ice up to 1.5 m thick would be left on the face when the reservoir dropped.

On the occasion when it was necessary to inspect the face with the reservoir level lowered, the sheet of ice remaining on the face would not slide off even when a slot was cut into the toe of the ice by a bulldozer. Experiments were performed and showed that the ice would slide off the membrane, when not actually frozen to it, when the slope was 1:1.5.

When the height of the dam was raised a portable air bubbling system was designed to keep the slope clear of ice by circulating the water in front of the dam. The device worked even though the air temperatures dropped to −35°C. The system used was composed of three-quarter inch hoses laid out in a grid on the face of the dam. There was a hole 1 mm in diameter every 1 m. Air was supplied at the rate of 10 l/min per meter of hose at a pressure of 4 to 5 bar. Provision was made to shut off the air to the hoses above the waterlevel as the reservoir dropped.

Ice related damage to the membrane was not reported, Ref 54. The membrane was of two layers although one layer of 15 cm thickness would have been better because of its longer cooling time. Some rolling out of the underlying filter material occurred when using heavy rollers at the first stage construction joint. Settlement later occurred at this point. The surface was treated with a bituminous emulsion as a pore sealer and then a wear resistant hot asphaltic fine concrete having 10% voids was applied.

7.2 Rockfill operations during winter.

The ceasing of operations during the winter months is common for dam construction. However if the rockfill is dry then there appears to be little technical reason why rockfill placing should not continue during the winter. For example, four dams have been constructed in Sweden with some winter rockfill.

Seitevare	105 m
Letsi	85 m
Ajaure	46 m
Satisjaure	30 m

The winter fill was restricted to 10 m in height and heavily sluiced with water in the spring to thaw any trapped ice and snow. Watering was also used in the autumn to delay frost penetration.

Investigations into the performance of the winterfill placed in these dams concluded that placing rock in the 0.5 m lifts was uneconomic. The repeated travel of the trucks over the lift increased the risk of accumulations of ice and snow in the fill. Test fills were made using 2 m lifts and compaction was by rollers of various weights passing over the fill 10 times. The winter placed fill was sluiced with water the following spring. The results are summarised in the following table:

Roller weight	Winter placed fill Settlement due to Compaction %	Winter placed fill Settlement due to Sluicing %	Summer placed fill Settlement due to Compaction and Sluicing %
8.5 t	2.7	0.1	4.6
10 t	7.5	0.1	4.0

The sluicing of the winterfill gives very little extra settlement although there could be expected further settlement of the winterfill compacted with the 8.5 t roller. The greater settlement of the winterfill compacted with the 10 t vibrating roller is interesting and more data of this type would be useful. The winter modulus, of 200 - 250 kg/cm^2, was obtained from instruments placed in Letsi dam and compares closely with the summer modulus, Ref. 11. Winter fill was also placed for the Gepatch dam, Austria in 2 m lifts and compacted using an 8.5 t vibrating roller. The rock was dry with less than 10% smaller than 2 mm. The specifications were that:

1. Snow cover must be removed and the surface of the previous lift roughened and the snow pockets thawed with salt.
2. The working surfaces must be sloped 10% towards the core.
3. Areas prepared for working must be kept as small as possible, and the next layer placed and compacted quickly.
4. If precipitation was forecast the areas already roughened must be recompacted to seal the surface.
5. At the valley sides the frozen natural rock need not be removed.

The performance of the fill was satisfactory and several conclusions were reached. Ref. 12. These were:

1. A continuous 24 hour, seven day a week operation was required.
2. Output was less than in the summer months.
3. Thawing with salt proved successful.
4. The maintenance of the construction roads was important to keep the traffic moving.
5. When frozen the roughening of the surface was only possible with a heavy ripper.
6. Heavy snowfall stopped operations usually because of difficulty in transporting the fill by truck.

7.3 Conclusions.

One could conclude from the published information that rockfill operations can be continued during winter conditions. The rock needs to be relatively dry and unfrozen before placing. Good compaction can be achieved and sluicing in the spring does not produce much benefit. Areas less sensitive to settlement, such as the downstream half of the dam, should be planned to be constructed during winter.

Membrane construction is subject to the weather but planning the construction so that the membrane is constructed during good weather should overcome most difficulties. The performance of dams operating under cold conditions seems to have been satisfactory.

8. JOINTS

8.1 Concrete joint detailing.

Several examples of successful joints have been given in the section on concrete membrane case histories. The major conclusions for a successful joint can be summarised as follows:

1. Vertical joints between the slabs.
   a. No joint filler is necessary, but painting the surfaces with bitumen is often employed.
   b. Reinforcement need not pass through the joint but extra reinforcement, in the form of stirrups, at the edges of the slabs is recommended to prevent spalling of the concrete at the corners.
   c. Two waterstops are preferable to one, especially on the larger dams.
2. Perimetric joints.
   a. A filler of timber is often used to prevent movement of the slab from cutting the rubber waterstop.
   b. Layout of slabs, hinge joint and plinth should be optimised from a 3-D finite element study to minimise the effect of any shear movements and differential settlements.
   c. Two waterstops are essential.
   d. Additional reinforcement at the edges of the slabs and the edge of the plinth is desirable to prevent spalling of the edges.
   e. Additional waterproofing, similar to that used at the Foz do Areia dam ensures watertightness even with large joint openings.
   f. Instrumentation is useful to determine the performance of the joints without visual inspection.

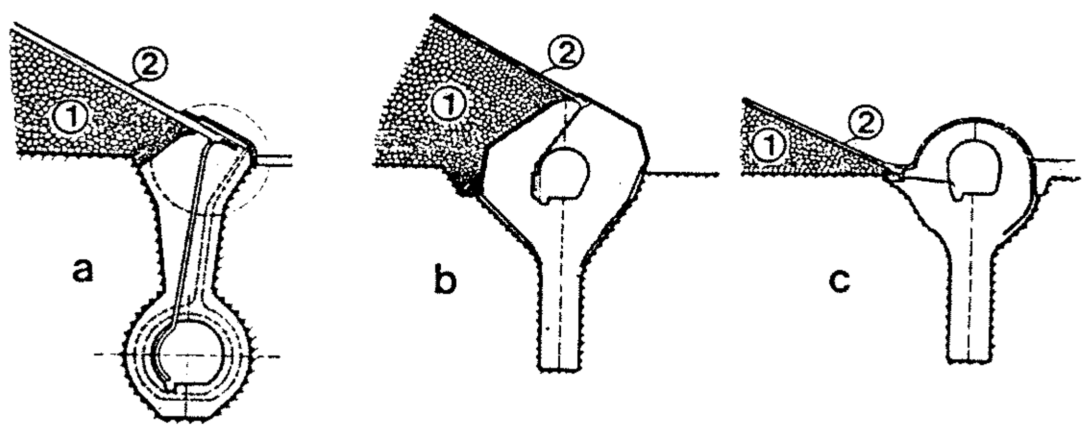
8.2 Asphaltic concrete joint detailing.

The vertical joints between adjacent asphaltic concrete strips are easily made and need not be considered further. The joints between the asphaltic concrete facing and the upstream toe, the crest and concrete structures, require detailing so that the junctions can accommodate some differential movement between the membrane and the structure. Horizontal cracks directly above the cutoff structure have often been reported. Differential movement is always likely to be present especially at the toe of the structure where the relatively stable cutoff and compressible rockfill meet. To these movements can be added the settlement of the foundation. Foundations on alluvial material can give rise to considerable settlements. The joint is required to accommodate this and still remain watertight, Ref. 55.

The first practical requirement of the joint layout is to ensure that the paving machine is able to lay the asphaltic concrete right up to the structure. Consideration of Figure 25 will show the difficulty that could have occurred at Henne dam. Here the cutoff, with the enlargement for the drainage gallery, protrudes above ground level. Many dams have profiles with a large radius at the crest and the toe to make the asphaltic concrete facing tangential to the structure. This ensures that a length of facing is pressed by water pressure against the toe structure. An example of this is Horchwurten dam, Austria, see Figure 26. A length of overlap of 0.5 m is probably a minimum. The contact surface between the asphaltic concrete and cement concrete calls for careful cleaning. An adhesive bituminous tack coat is often used. An arrangement suitable for small movements is shown on Figure 27. Where the facing leaves the concrete structure additional support to the membrane is provided by a wedge shaped thickening.

In alluvial material the plinth is extended to rock or impervious strata by a slurry trench wall or grouting. If the plinth could possibly move, because of foundation spreading, the membrane must be connected via a transition plate. The asphaltic concrete terminates on the plate and the plate is connected to the cutoff by an articulating joint, see Figure 29. An arrangement also using a transition plate is shown on Figure 2.

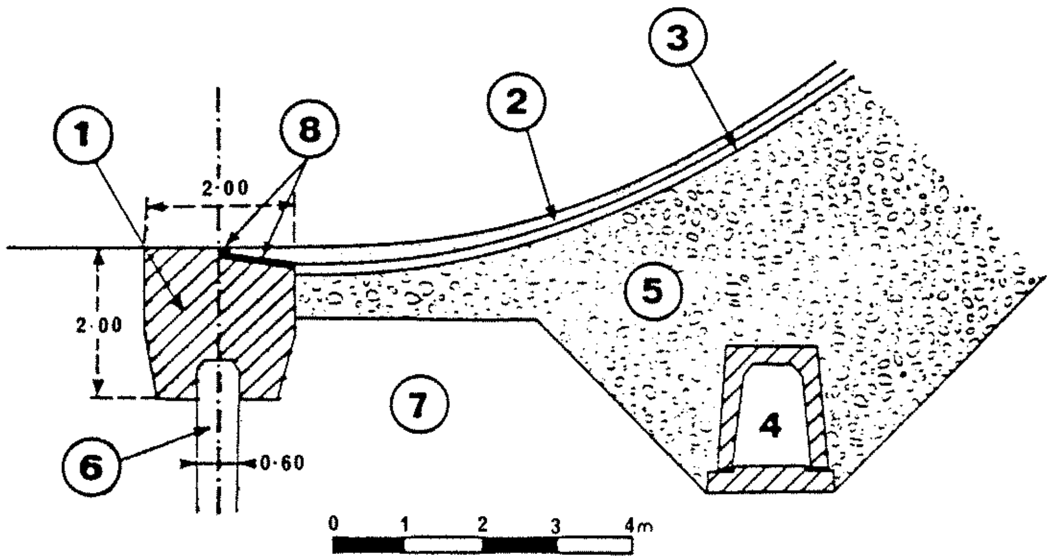
Some provision for sliding of the facing over the cement concrete is often desirable to accommodate deflections. This can simply be achieved by the use of an elastic course between the concrete and the impervious asphaltic concrete. See



(a) Bigge dam 1965.
(b) Genkel dam 1952.
(c) Henne dam 1955.
(1) Rockfill.
(2) Bituminous facing.

Figure 25. Different connections of bituminous facing to cement concrete structures.

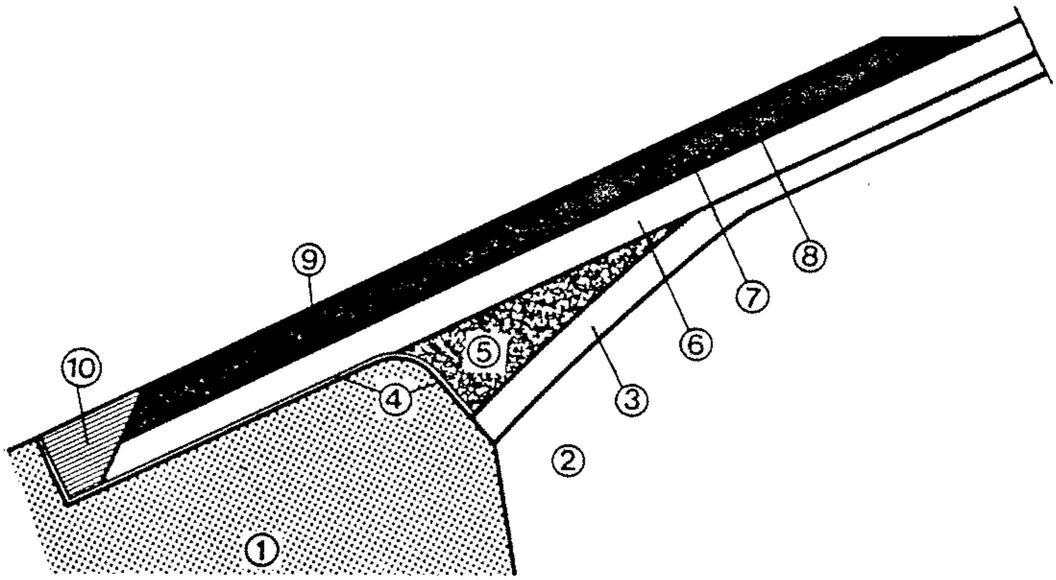
(After Idel, 1979.)



1 Concrete cut off
2 Impervious course
3 Binder course
4 Drainage gallery
5 Filter
6 Concrete diaphragm wall
7 Alluviums
8 Bituminous elastic course between concrete and impervious course

Figure 26. Horchwurten (Austria), Detail at toe.

(After ICOLD Bulletin No. 39, 1981.)



Connection inclined without dilation loop.

(1) Cement concrete.
(2) Rcckfill compacted.
(3) Bituminous levelling course.
(4) Tack coat.
(5) Asphaltic concrete wedge.
(6) Asphaltic concrete impervious.
(7) Reinforcement mat.
(8) Asphaltic concrete protection layer.

(9) Mastic seal coat.
(10) Joint sealing compound.

Figure 27. Simple arrangement of membrane connection.

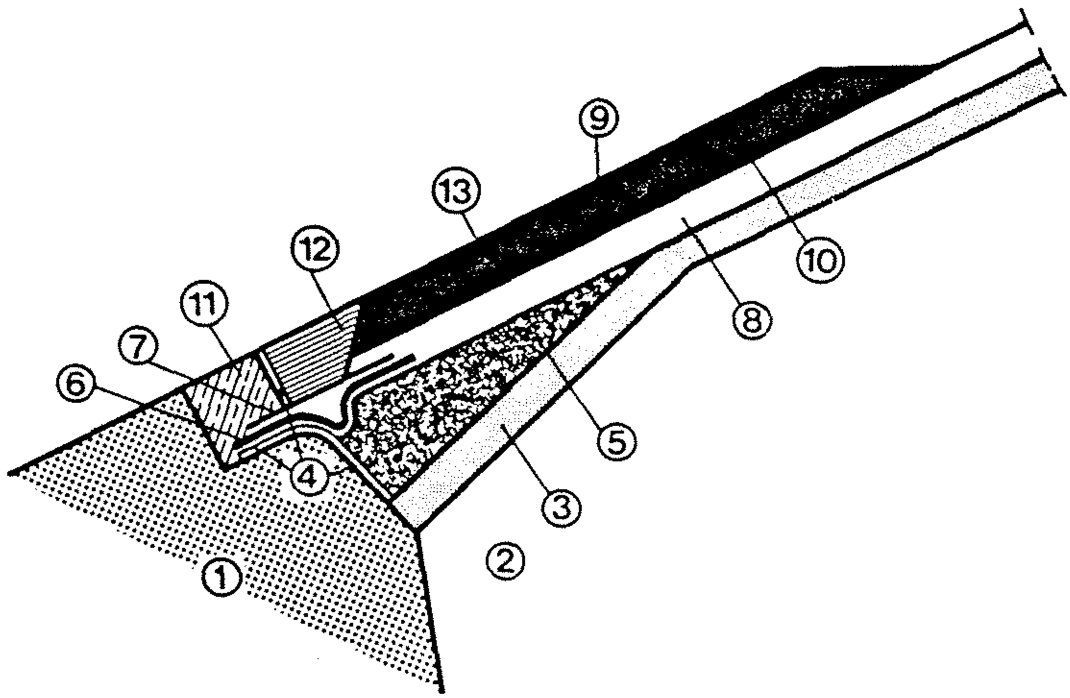
(After Idel, 1979.)

Figure 26. More complex arrangements are shown on Figures 28 and 29. One or two copper waterstops can be provided to allow for considerable movement. A joint with two waterstops is shown on Figure 30 for the Innerste dam, Germany.

The layout of the joint becomes more complex when a sandwich form of construction is used for the facing. Figure 31 shows the connection to the cutoff wall for Bigge dam which is protected by aluminium sheeting.

Many dams have external protection for the joint with concrete blocks, e.g. Dungonnel dam, N. Ireland, or with a clay blanket, e.g. Trapan dam, France. Reinforcement to the asphaltic concrete has been added, e.g. Ogliastro dam, Italy. The reinforcement must however be stable at the high temperatures of placing the asphaltic concrete.

Other examples of connections to structures are shown in Bulletin No. 39 published by ICOLD. Bonding with structures at the crest of the dam is simpler as the joint is not required to be completely watertight.

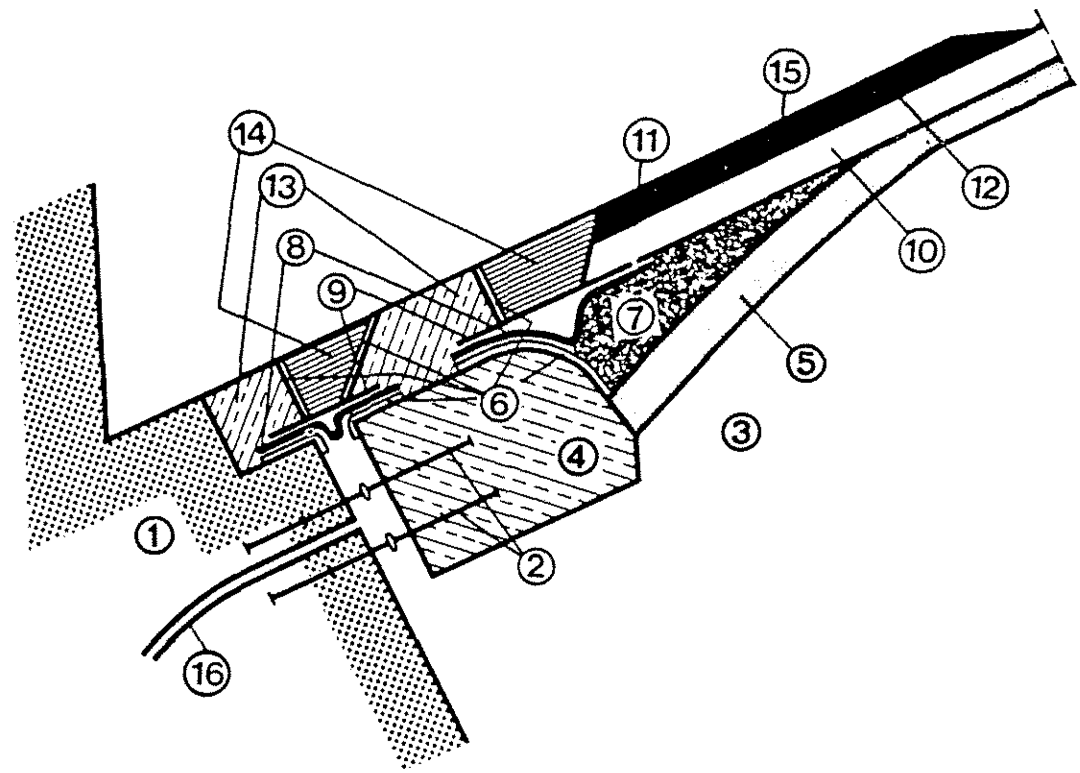


Connection inclined with dilation loop.

(1) Cement concrete.
(2) Rockfill compacted.
(3) Bituminous levelling course.
(4) Tack coat.
(5) Asphaltic concrete wedge.
(6) Dilation loop.
(7) Copper cover sheet.
(8) Asphaltic concrete impervious.
(9) Reinforcement mat.
(10) Asphaltic concrete protection layer.
(11) *In situ* cement concrete.
(12) Joint sealing compound.
(13) Mastix seal coat.

Figure 28. Connection with one waterstop.

(After Idel, 1979.)

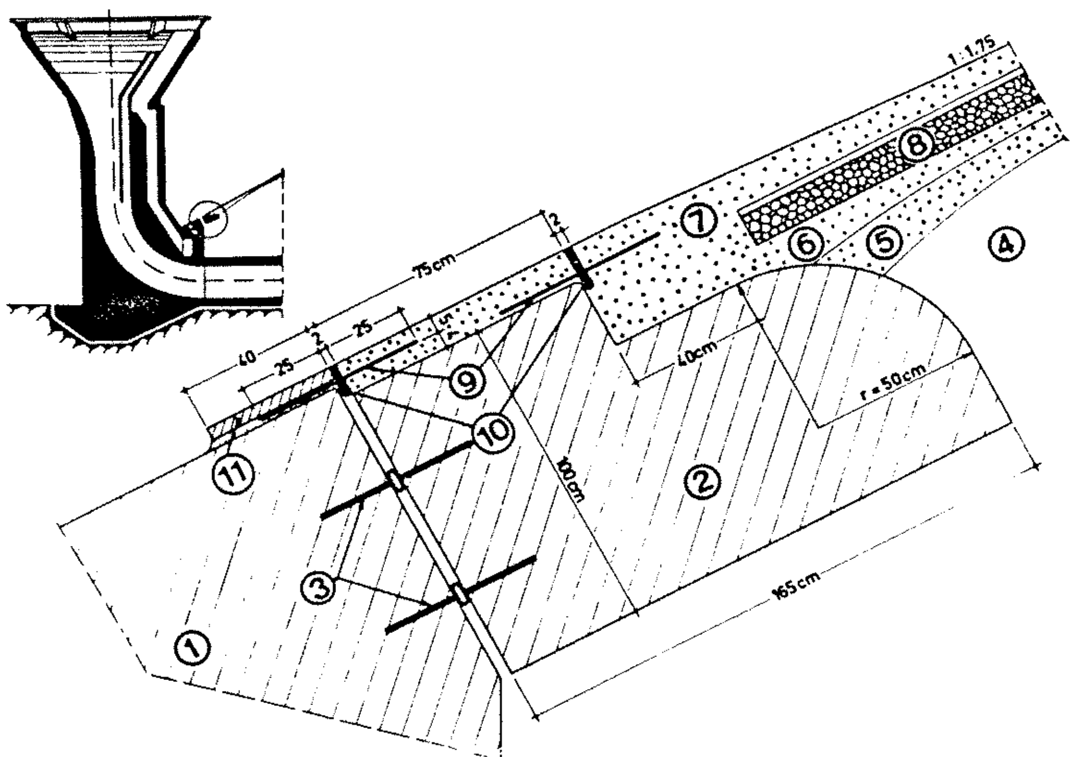


Connection inclined
with concrete slab and dilation loops.

(1) Cement concrete.
(2) Waterstop tape.
(3) Rockfill compacted.
(4) Reinforced concrete slab.
(5) Bituminous levelling course.
(6) Tack coat.
(7) Asphaltic concrete wedge.
(8) Dilation loop.
(9) Copper cover sheet.
(10) Asphaltic concrete impervious.
(11) Reinforcement mat.
(12) Asphaltic concrete protection layer.
(13) *In situ* cement concrete.
(14) Joint sealing compound.
(15) Mastic seal coat.
(16) Drainage pipe.

Figure 29. Connection for expected large movements.

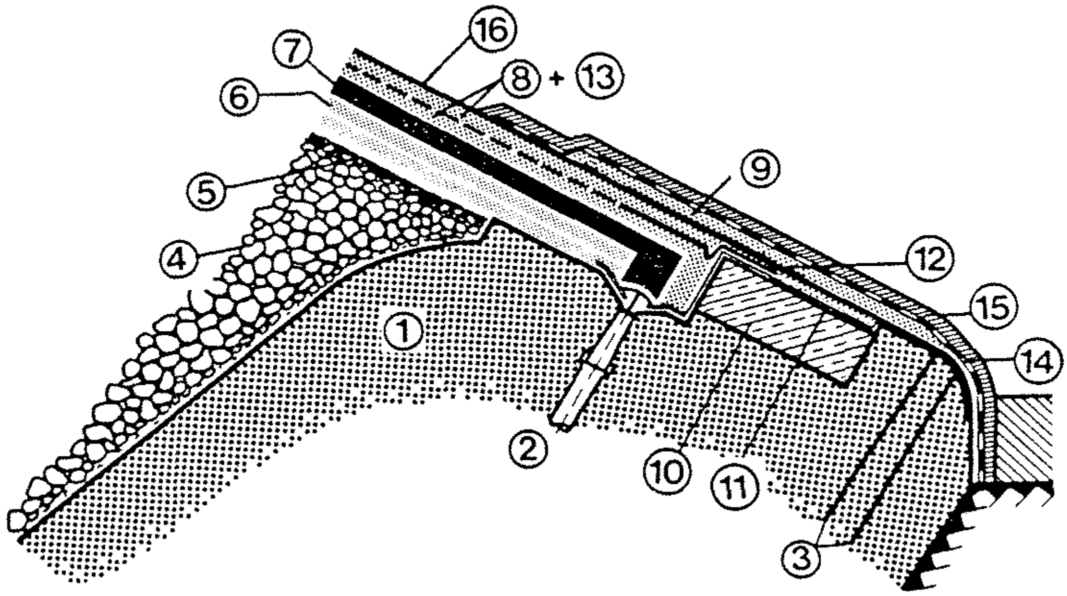
(After Idel, 1979.)



(1) Intake tower.
(2) Reinforced concrete slab.
(3) Waterstop tape.
(4) Rockfill compacted.
(5) Bituminous levelling course.
(6) Asphaltic concrete wedge.
(7) Asphaltic concrete impervious.
(8) Drainage.
(9) Dilatation loop.
(10) Mastic with elastomere additives.
(11) Precast concrete brick in resin sticker.

Figure 30. Innerste dam intake tower, Connection of bituminous facing.

(After Idel, 1979.)



(1) Cement concrete.
(2) Drainage pipe.
(3) Water stop.
(4) Rockfill.
(5) Bituminous levelling course.
(6) Asphaltic concrete impervious.
(7) Drainage course.
(8) Asphaltic concrete impervious in two layers.
(9) Asphaltic concrete.
(10) Cement concrete.
(11) Emulsion tacke coat.
(12) Dilation loop.
(13) Asphaltic concrete impervious.
(14) Laminated aluminium sheets.
(15) Protection cement concrete reinforced.
(16) Mastic seal coat.

Figure 31. Bigge, Bituminous facing connection to the cutoff wall.

(After Idel, 1979.)

9. TRENDS

There appears to be a trend towards the use of concrete membranes where foundation conditions are suitable for upstream membranes. The figures for completed dams in Spain are illustrative:

1970 - 1973	3 asphaltic concrete	1 concrete
1973 - 1976	2 asphaltic concrete	No concrete
1976 - 1979	No asphaltic concrete	4 concrete[1]

[1] Includes one dam reported in the 1973 - 1976 figures as being asphaltic concrete faced. The dam was later redesigned with a concrete face.

The reason may be purely economic due to the rise in the price of oil and oil products. The recent completion and successful performance of several large concrete faced dams may accelerate this trend.

Several trends in construction methods can be identified for both asphaltic concrete and cement concrete membranes.

9.1 Cement concrete.

A clear trend emerges from the study of the case histories in the design of the joints. The trend is towards the use of a sophisticated, but simple in principle, waterproofing measure for the perimetric joint. The development of the joint can be traced in the following stages:

1. Butt joints with a compressible filler were used first with one waterstop.
2. The design of Cethana dam confirmed the use of two waterstops, one at mid-slab thickness, the other at the base.
3. The failure of Alto Anchicaya perimetric joint required a new design of the joint for the repairs.
4. At Foz do Areia the design of the perimetric joint incorporated the ideas used for the repairs to Alto Anchicaya. Larger settlements were expected here than had occurred at any of the previous dams. To summarise this design the elements of the joint from the outside are:
    a. A P.V.C. cover retained by stainless steel plates and bolts.
    b. A mastic joint cover with a neoprene tube embedded for additional flexibility.
    c. A joint filler.

d. A P.V.C. waterstop at mid-slab thickness.

e. A joint filler.

f. A copper waterstop at the base of the slab. To prevent collapse of the centre rib under the action of water pressure neoprene or foam inserts can be used.

g. A sand bitumen pad.

h. A filter zone beneath the joint to prevent movement of fines if leakage develops.

5. The waterstop at mid-slab thickness was omitted at Yacambu.

Future experience may show that the second waterstop is not necessary.

There is some trend for the reduction in thickness of the face slab in relation to the height of the dam, see Table 1. The empirical design formula,

$$1 \text{ ft} + 0.006 \text{ ft} \times \text{Height}.$$

has been replaced by the less conservative,

$$0.3 \text{ m} + 0.002 \text{ m} \times \text{Height}.$$

for dams such as Cethana and Yacambu.

Reinforcement expressed as a percentage of the area of the slab has remained essentially constant at about 0.5% in each direction. Additional reinforcement is now generally provided at the edges of all the slabs to prevent spalling of the corners. At Foz do Areia the area of the reinforcement was only 0.4% of the area of the slab. It remains to be seen if this reduction in area of reinforcement leads to any loss in performance. The case for 0.5% reinforcement is backed by a considerable depth of usage, see Table 13.

The plinth has developed into being a reinforced slab about 0.6 m thick, dowelled into rock. The width of the plinth is a minimum of about 3 m and is usually 1/20th to 1/10th of the water head at that point. It is used as a working platform for the grouting operations on many sites.

The construction of higher concrete faced membranes has resulted in the need for staged construction of the membrane and rockfill. While it is advantageous to limit the deformations of the concrete face to those resulting only from the water loads, it is more practicable to allow impounding to start as soon as possible. Horizontal construction joints can be made while maintaining the vertical continuity of the slab. The additional joint openings, due to the settlememt of the fill during the construction of the subsequent stages, have been protected at Alto Anchicaya and Yacambu by an upstream clay blanket on the lower slopes.

9.2 Asphaltic concrete.

A trend towards the use of one dense asphaltic concrete impervious layer as the main waterproofing element is emerging. One thicker layer has the following advantages:

1. It retains the heat longer allowing compaction to be more easily achieved before the layer cools down.
2. Larger aggregate sizes can be used.
3. Only one pass of the paver is required.
4. The danger of the formation of blisters is eliminated.
5. The greater heat stored in the asphaltic concrete can melt the surface of the underlying layer and therefore increase the bond.

The advantages in reducing the number of joints through which leakage is possible has also been recognised.

The excellent watertightness of many asphaltic concrete faced dams has led some designers to omit the drainage layer directly beneath the membrane. The dams have had their stability ensured with semipermeable material beneath the membrane and by adequate filters built into the body of the dam. Examples of this form of construction are Pla de Soulcem and La Verney in France. Uplift forces on the membrane could develop in the case of rapid drawdown if the body of the dam did not drain quickly.

Reinforcement has been added to the asphaltic concrete to improve its tensile properties, enabling it to span local settlements without damage. The stability of the reinforcement used should be ensured at the high temperatures used for placing the asphaltic concrete. Polypropylene, a common geotextile, has a melting point of below 120°C, a common placing temperature.

The surface coating is useful for sealing small leaks and protecting the dense asphaltic concrete against deterioration due to daylight and atmospheric oxygen. The addition of reflective, light coloured paint to the surface can reduce the heat buildup in the asphaltic concrete in sunlight. Lower temperatures will reduce the speed of the weathering process.

A development of asphaltic concrete has been tried in France. The bitumen has been replaced by vinyl pitch and the material has been used on three dams, Revin, 22 m, l'Etang, 32 m, and Monnes, 57 m. The pitch is composed of 80% coal pitch, 15% of oil from anthracite, and 5% P.V.C. It is made on the spot by melting granules. The advantages of the vinyl pitch are:

1. It melts at 140°C compared to 180°C for bitumen.
2. It compacts satisfactorily at 60 - 70°C compared to 120°C for bitumen.
3. Deformations due to creep are much reduced. The following table illustrates the difference at a constant 70°C.

	7.5% Bitumen	9.3% Vinyl
Slope	1:3	1:3
After 2-7 days	77 - 100 mm creep	3 - 100 mm creep
After 7-8 days	6 - 100 mm creep	0 creep
Slope		1:2
After 2-7 days		10 - 100 mm creep

4. The flexibility is better than with bitumen bound aggregates. Tests performed at 20°C on 200 x 600 x 50 mm specimens supported at 350 mm centres are summarised in the following table.

7.5% Bitumen		9.3% Vinyl	
Time	Deflection mm	Time	Deflection mm
0	0	0	0
10 min	2	1 min	13
16 hr.	12	10 min	23
40 hr.	19	20 min	30
80 hr.	25 fissures	30 min	34
104 hr.	32 fissures	110 min	43
		180 min	50 no fissures

5. Shear resistance tested at a constant 50°C was better than that of bitumen. For a shear load of 2.5 kg. on a 65 x 50 x 35 mm sample, bitumen failed after 2.5 hours, (normal load 5 kg.), vinyl ruptured with an 8 kg. shear load after 15 hours, (normal load 8 kg.). See Ref. 56.

Revin dam was completed in 1972 using an experimental section of 10,000 m^2 of 6 cm thick vinyl. After 10 years, favourable performance of the test section was reported. Permeability averaged 2 x 10^{-8} cm/s and at worst was 4 x 10^{-4} cm/s. Better watertightness was achieved with the vinyl although some cracks appeared above the waterline. The modulus of the vinyl had reduced by 75% while that of the asphaltic concrete had reduced by only 40%. The absolute value of the modulus of the vinyl was still higher than that of the asphaltic concrete.

The density of the vinyl is 1.2 compared to 1.03 of bitumen. This means that 1.17 times more vinyl is required for the same proportions by volume. See Ref. 57.

9.3 Rollcrete.

Rollcrete is roller compacted concrete. The concrete uses the largest aggregate size practical with the smallest proportions, by weight, of cement, (2.5 to 7%). The concrete can be delivered to site, spread and compacted by conventional earth moving equipment at a rate similar to the rate for placing rockfill. The physical state of fresh rollcrete is such that it displays a relationship between compacted dry density and moisture content similar to that found with engineering soils. A dense concrete can be produced with air contents of only 1 to 3% indicating a low permeability. Strengths of 14 MPa are easily achieved and Schrader, 1982, Ref. 58, designer of the Willow Creek dam, comments that "rollcrete has been shown to have strength equal to that used in gravity dams and occurs earlier". Shrinkage characteristics are different and significantly less than conventional concrete.

The use of rollcrete at Tarbella for repairs established its practical large scale use. 500,000 m^3 was placed in six weeks, the smallest dump trucks were 70 t, and the batching plant had a capacity of 10,000 m^3 daily. The speed of construction and the low unit cost have made the material attractive to designers. Willow Creek, Oregon has recently been completed and partially filled. Leakage has been severe, at 95 l/s, mainly through the lift joints. See Ref. 59. Remedial measures are estimated to cost U.S.$3 million and could well put rollcrete dams out of favour for some time. It should be pointed out that the use of rollcrete was saving U.S.$10 million over the conventional use of rockfill. The successful use of rollcrete at Guri, Venezuela, Ref. 60, and in Japan, for

Shimajigawa dam, Ref. 61, indicates that leakage could be kept small.

The use of rollcrete for forming upstream membranes has not yet been explored by the designers. The low modulus, high rate of placing, and the use of conventional construction plant, should make rollcrete attractive for the construction of facings. Two systems for its use are possible.

1. The rollcrete and the rockfill could be placed at the same time.
2. The rollcrete would be placed after the completion of the rockfill.

A zone of rollcrete laid horizontally parallel to the dam axis, about 10 m wide, would form the membrane. The recognised potential for seepage at the lift joints has been solved, in Japan, by applying a mortar layer, or, at Guri, by scarifying the surface. The specification of allowing 1,600°F hours (870°C hrs.) at Willow Creek before the joint required treatment has been recognised by the designer as being too ambitious. The use of 400°F hours may well have reduced the leakage.

An alternative application of rollcrete is to form the bedding layer for slipformed concrete facing. It has already been noted that an increase in the stiffness of the bedding material will reduce the stresses in the membrane and rollcrete is stiffer than the conventional materials used for the bedding. The rollcrete zone would also satisfy many of the requirements for membranes discussed in Section 3. Replacing the rockfill by more than a thin zone of rollcrete must return the design back to the realms of gravity structures.

Some other useful rollcrete references are Refs. 62, 63, 64, and 65.

10. DESIGN

As mentioned in the introductory section, few design methods have been reported in the literature. Both asphaltic concrete and cement concrete are practically impervious, when well constructed, so that the thickness of the membrane has not been related to permeability. Durability and practical limits on the placing of the materials has almost certainly determined the thickness of the membranes. For example, it will be seen in the following sub-section, on concrete design, that a thickness of one foot has been considered a minimum. Similarly 5 cm has been considered a minimum for the construction of asphaltic concrete impervious membranes.

Whether the performance of a membrane is adequate or not, will depend on the amount of leakage through the membrane itself or through the joints. As most membranes are impermeable the joint performance will be critical. The amount of leakage through open joints is proportional to the square of the size of opening. Joint opening is dependent on the deformations of the rockfill, hence the characteristics of the rockfill are most important. A stiff rockfill can be obtained by the methods outlined in Section 4. The estimation of deformations of the rockfill is covered in the next chapter.

10.1 Rockfill.

The deformations of a number of rockfill dams have been collated by Sodemir and Kjaernsli, 1979, Ref. 66. Vertical and horizontal deflections of the crest and deflections of the facing normal to the upstream slope can be estimated from their charts or from empirical formulae for compacted rockfill:

Vertical settlement after initial reservoir filling Sv,

$$Sv = 0.001\ H^{1.5}$$

and long term

$$Sv = 0.003\ H^{1.5}$$

Horizontal deflection after initial reservoir filling Sh,

$$Sh = 0.00005\ H^{1.5}$$

and long term

$$Sh = 0.0015\ H^{1.5}$$

Maximum membrane deflection after initial reservoir filling Sn,

$$Sn = 0.002\ H^2$$

Where H is the height of the dam in meters and S is in meters.
Their data base for compacted rockfill was only nine dams, none of which were more recent than Cethana, 1970, so the deflections should be regarded as an upper bound solution. Heavier construction equipment in use today should result in lesser deformations.

Deformations can be calculated by F.E.M. The actual techniques are not considered in this report. The selection of a modulus of deformation for the rockfill is required whatever method of calculation is used. Wilkins, Ref. 67, gives some values of moduli and their stress dependence, for various rocks in Australia. These values are typical for rockfill, see Table 12, which gives moduli as calculated from the actual deformations of the dams.

These moduli can be used as a guide in the initial stages of design but the following outline method can be used for obtaining reasonably accurate forecasts of deflections:

1. Install settlement gauges within the fill at various levels as the construction proceeds.
2. Back calculate the moduli of the rockfill from the observed settlements.
3. The moduli obtained during construction can be used to calculate the additional deflections due to the water load.

Joint details can be finalised at this stage of the design.

10.2 Cement concrete membrane design.

The thickness of the membrane has varied from 2.0% to 0.47% of the water head. See Table 1 for the range of values used. Cethana is the least conservative design and appears to have suffered no detrimental effects. The empirical design equation is recommended to be:-

$$T = 0.3 + 0.002\ H\ m.$$

This results in a design that has proved itself adequate structurally, adequate for durability and practicability of placing.

The amount of steel reinforcement used for a number of dams is shown on Table 5. The common use of 0.5% and the absence of failures due to inadequate

Table 12

Typical deformations and moduli for some dams with upstream membranes.

Dam	Height (m)	Modulus from Performance (MPa)	Maximum Deflection Normal to Membrane (cm)	Crest Horizontal Movement (cm)	Crest Vertical Movement (cm)	Other Movements (cm)	Remarks
Guadalupe	28.5				210		
Bigge	55				12.3		Rate of 1-2 mm year in later life.
Courtright	90				178		Dumped rockfill. After 12 years.
New Exchequer	148			12.2 d/s	45.7		First filling
Rama	100				69		0.089 mm/day
				4.2			0.038 mm/day after filling
Kangaroo Creek	59				15		During construction
			5.2		4.3		After filling
Cethana	110					425	At midheight before filling
						100	During and after filling
		185				26.6	Base to 1/3rd height
		135				24.9	Base to 1/2 height
		112				27.2	1/3rd to 1/2 height
				4.1 d/s	6.9		
		204	11.7				
Scotts Peak	43				19.5		After 3 years
					0.8		per year after
Alto Anchicaya	140	98 - 167					
					10.6		On first filling
						1.5	Downslope
						6	On 2nd filling
			13			11	After 1 year
			16				After 2.5 years
Foz do Areia	160	28 - 55				358	At 1/2 height before filling
						57	After filling

d/s - downstream
u/s - upstream

Table 13

Reinforced concrete faced dams.

Dam	Country	D.O.C.[1]	Height m	Steel Reinforcement % of area	Upstream Slope	Downsteam Slope	Remarks
Bucks Creek	U.S.A.	1928	36	0.75	1:1.4	1:1.5	
Don Martin	Mexico	1930	30	0.31	1:1.75	1:2	
Kangaroo Creek	Australia	1969	59	0.55	1:1.4		
Huinco	Peru	1970	15	0.5	1:1.5	1:1.5	
Cethana	Australia	1971	110	0.5	1:1.3	1:1.3	
Outardes 2	Canada		55	0.45	1:1.4		
Pozo De Los Ramos	Spain		97	0.5 to 0.6 at abutments	1:1.3	1:1.3	Gunited
R.D.Bailey	U.S.A.	1978	96	0.5			
Yacambu	Venezuela	1980	162	0.425 to 0.554	1:1.5	1:1.6	
Foz Do Areia	Brazil	1980	160	0.4	1:1.4		
Villagudin	Spain	1981	33	0.5	1:1.3	1:1.3	

[1]Date of Construction

reinforcement leads to the recommendation that 0.5% continues to be used in the absence of a more rigorous design method. Extra reinforcement at the edges of the slab to prevent spalling is recommended. In the publication by A.S.C.E., Current Trends in Design and Construction of Embankment Dams, there is a note to the effect that 0.3 to 0.4% reinforcement is being used. This may point the way for a reduction in the amount of reinforcement . Design of the slab and reinforcement to resist the tensile and compressive forces may be possible if a 3-D F.E.M. analysis is performed. Measurements of the strain in the concrete of some dams have indicated that the strains are low compared to the strains to failure of the concrete. The use of this design criteria could result in a very thin membrane that is clearly not practicable for long term durability.

The design of the joints in the membrane is covered in the section, Joints.

10.3 Asphaltic concrete membrane design.

The design of the asphaltic concrete mix is covered in many publications, such as those by Shell, and by national organisations. This part of the design process is not essential to the dam design and is not considered further.

Sawada et al., 1973, Ref. 68, have developed an expression for the required thickness of asphaltic concrete, Tn, that takes into account the permissable leakage through the membrane.

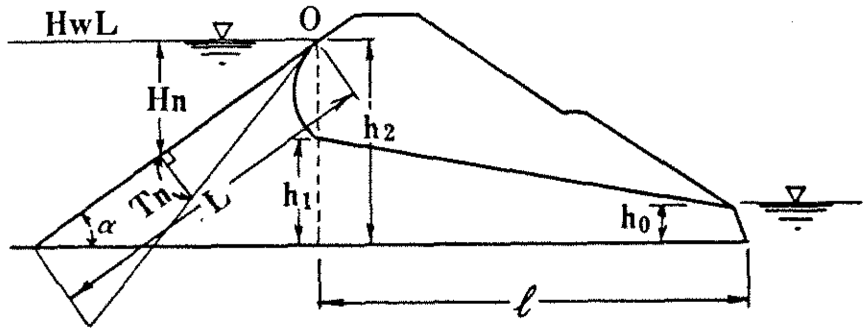
$$Tn = \frac{k_1 \quad Hn \quad h_2}{q} \text{ cosec alpha}$$

or more conveniently:

$$Tn = \frac{k_1 \quad Hn}{q'}$$

See Figure 32 for definition of the symbols.

Several dams are claimed to have been designed in Japan using this formula. An example given is the Muyama rockfill dam, h_2 = 75.2 m, with an assumed freeboard of 1.5 m, a permeability of 2 x 10^{-8} cm/s, membrane area of 44,700 m^2. An allowable seepage of 2 x 10^{-4} m^3/min per m^2 was assumed in the design with 1/50th of this coming through the membrane. At the maximum water depth Tn is calculated to be 22 cm. The average thickness required for the total leakage through the dam of 149 l/s is 11 cm. The actual membrane used two layers of 6 cm each.



$$T_n = k_1 H_n h_2 \operatorname{cosec} \alpha / q = k_1 h_n / q'.$$

where

q = allowable seepage for 1 m bank length m^3/mn (m).
q' = allowable seepage for 1 m^2 bank slope m^3/mn (m^2).
k_2 = permeability coefficient of embankment (m/mn).
k_1 = permeability coefficient of impervious layer (m/mn).
h_0 = height of internal water table or height of at the toe of drain.
h_1 = water head needed for seepage up to ho height (m).
h_2 = High water level (m).
α = slope (degrees).
l = horizontal seepage length in embankment (m).
L = $h_2 \operatorname{cosec} \alpha$ = permeable slope length (m).
H_n = Water head at any point on slope (m).
T_n = The thickness of impermeable layer for any water head H_n.

$$T_h = 60 \sqrt{\frac{P}{\sigma_b}} F_s$$

where :
T_h = Thickness required (cm).
p = impact pressure of wave (kg/cm^2).
σ_b = allowable bending strength.
F_s = Safety factor 1.0 — 1.5.

Figure 32. Definition of symbols used in text.

(After Sawada, et al., 1973.)

The design also involves the checking of the stability of the embankment. For this the position of the phreatic surface is often required in the downstream slope. Sawada et al. also give an expression relating the amount of seepage, permeability of the dam and tailwater level, to a linear portion of the phreatic surface. See Figure 32 for details.

$$h_1 = \sqrt{2\,lq/k_2 + h_0^2} < h_2$$

The thickness of the membrane required to resist wave pressures is given by Sawada et al., by the empirical relation:

$$T_h = 60\sqrt{\frac{P}{\sigma_b}F_s}$$

The above expression is also used to check the stability of the membrane against the water pressure in the case when support is removed from under the asphaltic concrete. A comprehensive series of tests have been done by these authors to determine the properties of the asphaltic concrete for use in design. Some of these properties are summarised on Table 14.

Finally, the thickness of the drainage layer should be designed to carry the full allowable leakage. The thickness required, x, is given by:

$$x = \frac{q_d}{k\ 1/i\ 1/\Psi}$$

where
Ψ – is an experimental factor conservatively taken as 1.
i – the hydraulic gradient in the drainage course (the slope).
k – the permeability of the drainage course.
For the Miyama dam,
k = 2.4 x 10^{-2} m/min,
i is 2.15 as the slope of the facing was 1:1.9,
q = 4 x 10^{-6} x length of the slope.
The thickness required is therefore 5.4 cm.

Table 14

Smmary of mechanical properties of asphaltic concrete as reported by Savada et al., (1973).

	Compression		Tensile	Bedding	
Temperature	Strength kg/cm^2	Stiffness kg/cm^2	Strength kg/cm^2	Strength kg/cm^2	Stiffness kg/cm^2
-10^0C	120	9×10^3		120	3×10^4
0^0C	45	2×10^3	35	80	8×10^3
$+10^0C$	20	8×10^2	10	25	8×10^2

An 8 cm drainage layer was used.

The Swiss National Committee on Large Dams suggest that the thickness of asphaltic concrete, w, should follow the equation:-

$$w = 5 + \frac{z}{25}$$

where
w is the thickness of the membrane in cm.
z is the water depth in m.

They also make the point that in their opinion multilayer sealing courses are inadequate because of the possibility of blister formation. A single, thick layer of good quality asphaltic concrete placed in dry weather is recommended.

The recommended design method is empirical using the Swiss method and checked using the method of Sawada et al. in the absence of any other design method.

10.4 Rigorous design method for concrete membranes.

A rigorous design method would involve predicting the deformations of the fill under the action of the water load, and designing the joints and the slabs accordingly. The use of a 3-D finite element analysis is fundamental to achieve this end. The steps in the design would be:

1. Instrumentation of the rockfill with settlement gauges, extensometers, and surface movement surveying.
2. Monitoring the instrumentation as the rockfill is being placed.
3. Using the F.E.M., attempt to back calculate the moduli of deformation for the fill. It is expected that the moduli will be stress dependent. There have been successful attempts at modelling the deflection of dams, see Ref. 69.
4. Using the moduli obtained the additional deflections due to the waterload can be calculated. An incremental raising of the water may be required to check for the development of tensile strains near the abutments.
5. Different slab layouts could be modelled in an attempt to minimise the opening of the joints. Separation or rotation of the joints along the axis of the slab, can be handled by current joint technology, but any form of shear movement cannot.

The expense of a detailed F.E.M. analysis may only be justified in the case of the higher dams. Sufficient empirical experience is available for the satisfactory performance of smaller dams to enable empirical design of these dams.

The watertightness of membrane faced dams has been determined, in the past, solely by the performance of the joints. Concrete of sound quality has sufficient impermeability, even in thin slabs, not to be of great concern in the design.

10.5 Beyond the limits of current practice.

For progress to be made some element of risk in using new approaches must be accepted. However, with logical reasoning, based on past experience, the risk may be small. Take for example the need to construct a 240 m high dam in an area where there is no fine material. Perhaps the foundations are on waterbearing gravel. An upstream membrane dam is advantageous due to material requirements. The settlements of the foundation while being large would not be excessive.

A construction approach for a concrete upstream membrane based on the best performance to date might be:

1. Construct all of the rockfill. Settlements of the rockfill and foundation would be essentially completed before laying the membrane. Rockfill could be sluiced during compaction by vibrating rollers in reasonably small lifts.
2. The slab would be slipformed in 12 to 15 m wide strips with waterstops in the vertical joints but no joint filler should be used. The plinth to slab joint would have two waterstops and additional mastic protection as on Yacambu, Venezuela. The slab thickness would be adequate according to the Cethana dam formula, giving a 780 mm thick slab at the base. Reinforcement would be provided at 0.5% of the area of the slab in each direction. The reinforcement should not be placed at the centre of the slab where the slab is thicker than about 500 mm as it is required to control surface cracking. Additional steel should be provided at the edges of the slabs and plinth to prevent spalling of the corners. The use of low heat cement, chilled aggregates and iced water to reduce the possibility of thermal cracking should be considered. An air entraining agent should also increase the durability of the concrete.

3. As the slabs are being constructed impounding could start. Horizontal construction joints may be necessary but these joints have not given much trouble in the past. The deflections due to the water loads would be occurring as the membrane was being placed. This should reduce the joint deflections higher up the slope.

The dam should be designed with a semipervious bedding to the membrane. In addition it should be stable if serious leaks develop. The whole dam should act as a filter to prevent internal movement of the dam's material. A stiff bedding to the membrane is advantageous in reducing the stresses in the membrane. Rollcrete could be used here laid in horizontal lifts at the same time as the rockfill.

Other design features that could be studied are:

1. The dam could be gently arched upstream to assist in keeping the facing in compression.
2. Starting the construction of the slabs before the rockfill has been completed. Generally the lower half of the face of a dam bulges out as consolidation of the rockfill occurs. The bulge could be opposed by the water loadings and hence reduce the deformations. A F.E.M. check would be required using an incremental water and fill height approach to check if any benefit could accrue.

An asphaltic concrete membrane would be equally watertight but at a significantly increased cost because of the flatter slopes usually used.

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