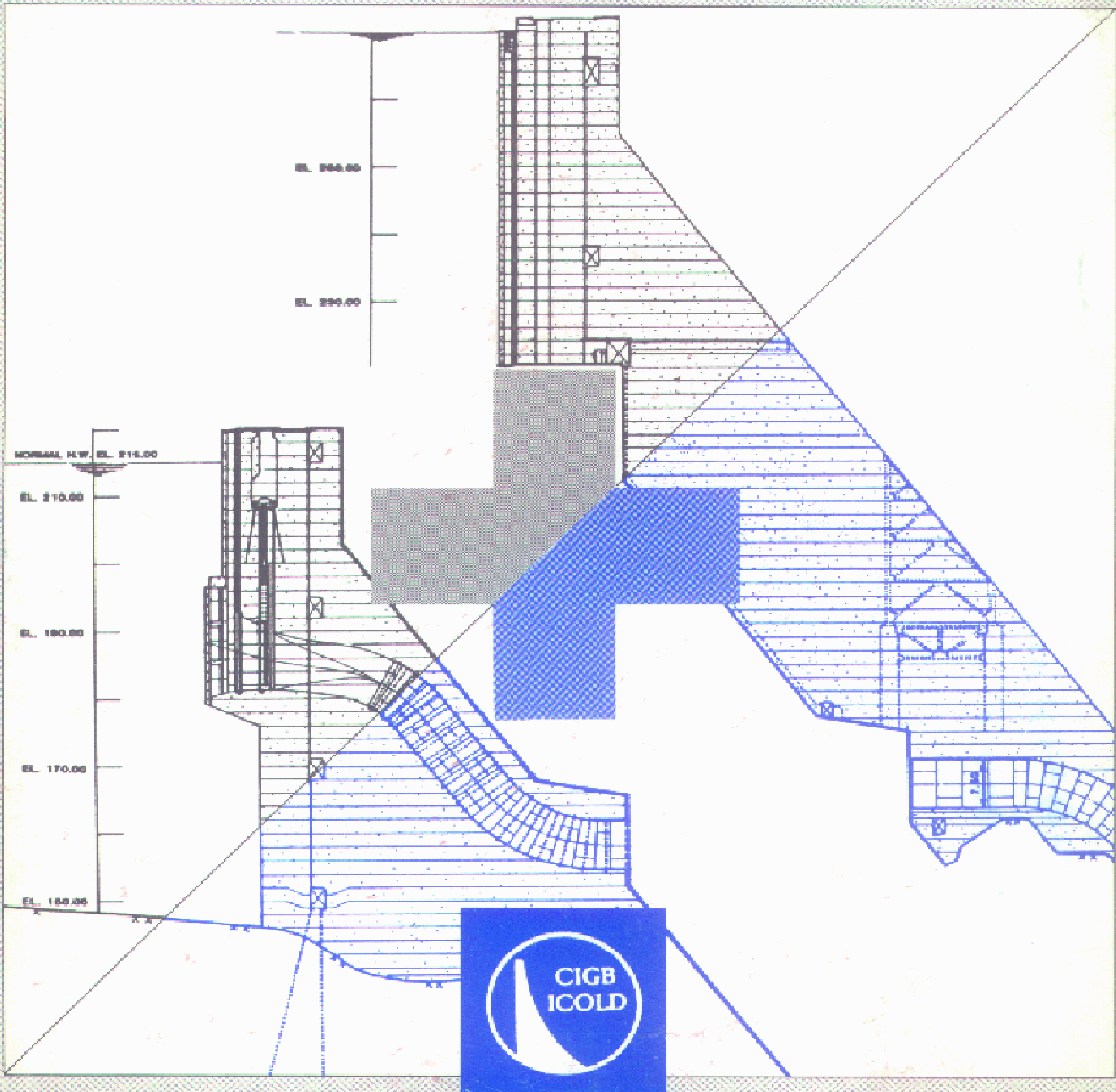
DAM HEIGHTENINGS

Register.

SURELEVATIONS DE BARRAGES

Registre.

Bulletin 64



CIGB ICOLD

1988

DAM HEIGHTENINGS

Register.

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CIGB ICOLD

BULLETIN 64
1988

Commission Internationale des Grands Barrages
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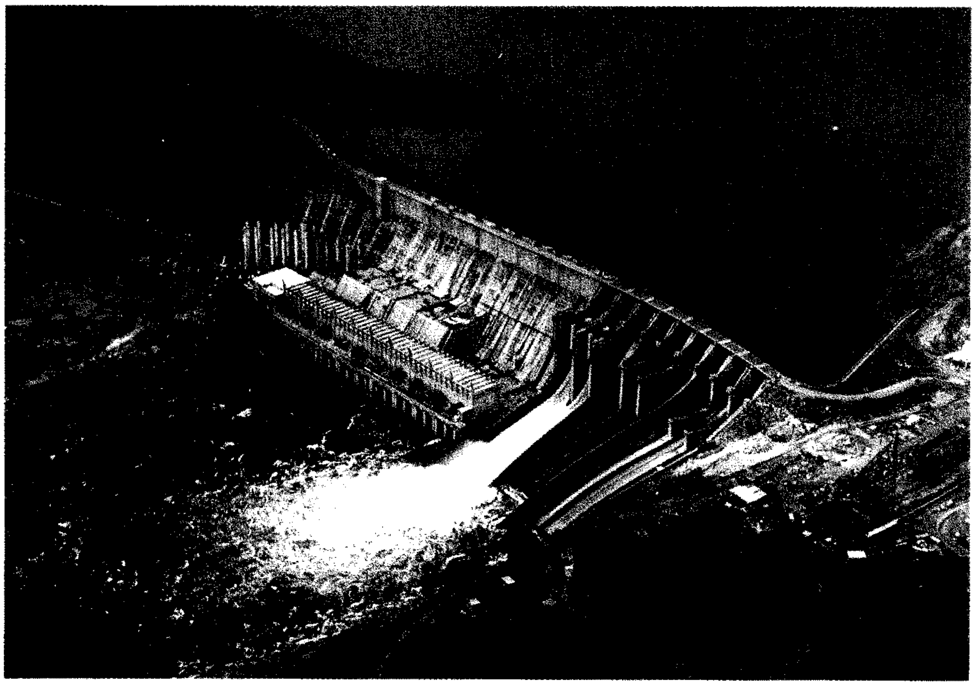

Barrage de Guri en 1978 avant surélévation
(après surélévation, voir photos et principales caractéristiques p. 108 à 111)
Guri dam in 1978 before heightening
(after heightening, see photographs and main features p. 108 to 111)

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INTRODUCTION

A.- CONGRES DE LA CIGB TRAITANT DES SURELEVATIONS DE BARRAGES

Les surélévations constituent un chapitre important de l'ingénierie des barrages. Plusieurs raisons peuvent les motiver : augmentation de la revanche et de la capacité d'évacuation des crues, accroissement du volume de la retenue, etc... Dans chaque cas, l'ingénieur doit considérer plusieurs types de problèmes; ils ont été examinés à fond dans le Rapport Général relatif à la Question 20 (6ème Congrès de la CIGB, New York, 1958). Les Comptes Rendus de ce Congrès n'étant plus disponibles, le texte anglais du Rapport Général, accompagné d'un résumé en français, est reproduit en annexe.

Les rapports présentés lors de ce Congrès fournissent une vue générale du sujet; mais depuis cette date, de nombreux barrages ont été surélevés. La Question 48 (13ème Congrès de la CIGB, New Delhi, 1979)* traite des problèmes étroitement liés à la surélévation; les rapports présentés permettent de connaître les cas les plus récents de surélévations.

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* Les Comptes Rendus de ce Congrès sont encore disponibles au Bureau Central au prix de FF 850 les 5 volumes, frais de port par voie de surface compris. Pour le seul Vol.1, relatif à la Question 48, le prix est de FF 250 seulement.

INTRODUCTION

A.-ICOLD CONGRESSES DEALING WITH DAM HEIGHTENINGS

Dam heightening is a major task in dam engineering. Several reasons may make it necessary : increased freeboard, spillway capacity, reservoir volume, etc. In every case the engineer must consider different kinds of problems which are fully discussed in Question No.20, General Report, 6th ICOLD Congress, New York, 1958. These Congress Proceedings being no longer available, the English version of the General Report is reproduced as an Annex along with a summary in French.

The papers submitted at this Congress provide a general overview on the matter but in the period from that date, many more dams have been heightened. Question 48 (13th ICOLD Congress, New Delhi, 1979)* dealt with problems closely related to heightening; in the papers submitted at this Congress more recent cases of dam heightenings can be found.

Proceedings of the Delhi Congress are still available at Central Office at cost of FF 850 (5 Vol.), postage by surface mail included. For Vol.I, dealing with Question 48, price is FF 250 only.

B.- NOUVELLE ENQUETE

En 1985, la Réunion Exécutive de la CIGB a donné mission au Comité de la Bibliographie et de l'Information d'établir la liste des barrages ayant fait l'objet d'une surélévation.

A cette fin, tous les Comités Nationaux ont été priés de fournir les renseignements nécessaires sous forme de réponses à un questionnaire.

La collecte des informations a été limitée aux cas suivants :

- surélévation supérieure à 5m;
- surélévation de plus de 10% de la hauteur initiale;
- accroissement de la capacité du réservoir de plus de 10%.

Pour figurer dans nos listes, la surélévation devait satisfaire à l'un de ces trois critères.

C.- REPONSES OBTENUES

i) Pays concernés

A fin 1986, 37 Comités Nationaux sur les 77 Comités de la CIGB avaient répondu.

Neuf Comités Nationaux (Argentine, Bulgarie, Chypre, Danemark, Finlande, Rép.Dém. d'Allemagne, Guatemala, Irlande, Pays-Bas) sur les 37 pays membres n'ont mentionné aucune surélévation de barrages.

Dans le Registre Mondial des Barrages, il y a 6 autres pays membres et 3 pays non-membres qui signalent des barrages surélevés.

Les listes sont donc établies avec les renseignements concernant 46 pays.

B.-PRESENT INVENTORY

In 1985 the Executive Meeting of ICOLD entrusted the Committee on Bibliography and Information with the task of drawing up a list of heightenings of dams.

For this purpose, all National Committees were requested to provide the suitable data, on the pertinent forms.

Data collection was restricted to the following cases :

- heightenings greater than 5m;
- heightenings greater than 10% of initial height;
- increased storage capacity greater than 10%.

The dam is considered heightened for the purpose of our list, if one or more of these three criteria are met.

C.-RESPONSES RECEIVED

i) Countries involved

By the end of 1986, only 37 National Committees had answered, out of 77 National Committees in ICOLD.

Nine National Committees (Argentina, Bulgaria, Cyprus, Denmark, Finland, German Dem.Rep., Guatemala, Ireland, Netherlands) out of these 37 reported that no dam has been heightened in their country.

The World Register of Dams reported dam heightenings in another 6 member countries and 3 non-member countries.

Therefore the lists are elaborated on the data related to 46 countries.

ii) **Statistiques**

Pour ces 46 pays, le nombre total de barrages et celui des surélévations est donné ci-après :

Pays	Nbre de barrages		Pays	Nbre de barrages	
	Total	Surélev.		Total	Surélev.
Angola	10	1	Irlande	15	-
Algérie	21	5	Japon	2142	5
Argentine	86	-	Kenya	9	1
Australie	374	25	Liban	5	1
Autriche	112	5	Mexique	487	14
Belgique	15	1	Maroc	29	2
Bolivie	5	1	Nlle-Zélande	72	3
Brésil	489	1	Norvège	219	3
Bulgarie	108	-	Pakistan	38	1
Canada	580	1	Papouasie	3	1
Chine	595	1	Paraguay	3	1
Chypre	39	-	Pays-Bas	9	-
Danemark	6	-	Portugal	75	2
Egypte	5	1	Roumanie	106	1
Finlande	50	-	Afrique du Sud	342	27
France	439	6	Espagne	690	23
RDA	184	-	Suède	134	2
RFA	154	2	Suisse	130	8
Gde-Bretagne	529	12	Thaïlande	41	1
Guatemala	2	-	Turquie	74	2
Guyane	1	-	Etats-Unis	5338	84
Italie	408	8	Venezuela	64	1
Iran	21	1	Zimbabwe	95	3

Le nombre total de barrages existants dans ces pays est de 32 375 ainsi qu'il résulte du Registre Mondial des Barrages (édition 1984). Parmi eux, 26 625 sont des ouvrages en terre et enrochement (82,6%).

ii) Statistics

For these 46 countries, total number of dams and number of heightenings are given below :

Country	Nbr of dams		Country	Nbr of dams	
	Total	Height.		Total	Height.
Angola	10	1	Ireland	15	-
Algeria	21	5	Japan	2142	5
Argentina	86	-	Kenya	9	1
Australia	374	25	Lebanon	5	1
Austria	112	5	Mexico	487	14
Belgium	15	1	Morocco	29	2
Bolivia	5	1	New-Zealand	72	3
Brazil	489	1	Norway	219	3
Bulgaria	108	-	Pakistan	38	1
Canada	580	1	Papua	3	1
China	595	1	Paraguay	3	1
Cyprus	39	-	Netherlands	9	-
Denmark	6	-	Portugal	75	2
Egypt	5	1	Rumania	106	1
Finland	50	-	South Africa	342	27
France	439	6	Spain	690	23
GDR	184	-	Sweden	134	2
FRG	154	2	Switzerland	130	8
Great Britain	529	12	Thailand	41	1
Guatemala	2	-	Turkey	74	2
Guyana	1	-	United States	5338	84
Italy	408	8	Venezuela	64	1
Iran	21	1	Zimbabwe	95	3

The total number of dams in these countries is 32 375 as reported in the World Register of Dams (1984 Edition). Out of this total, 26 625 are earth and rockfill dams (82.6%).

Le nombre de barrages surélevés dans ces pays est de 258 (dont 124 pour les barrages en terre et enrochement).

Soixante-six pour cent des barrages surélevés sont situés dans 5 pays (Etats-Unis, Afrique du Sud, Australie, Espagne, Grande-Bretagne); ils représentent 2,35% des barrages existants.

Globalement, le nombre des barrages surélevés représente seulement 0,8% du nombre total des barrages (0,4% des barrages en terre et enrochement). Mais sans la Chine et ses 18 595 barrages dont un seulement a été surélevé, le pourcentage passe à 257/13 780 = 1,86%; il atteint 7% pour l'Australie.

Le tableau suivant montre que pour 46 pays, c'est essentiellement les barrages en béton, et parmi eux les barrages-poids, qui font l'objet d'une surélévation.

	Barrages en béton		Barrages en remblai	Total
	poids (74%)	99		
	voûte	18		
	contreforts	3		
	voûtes multiples	5		
	deux types	9		

Nbre de barrages surélevés		134	124	258
Nbre total de barrages		5 750	26 325	32 375
% de surélévations		2,3%	0,0046	0,8%
Nbre total de barrages dans le monde		6 085	29 061	35 146

The number of heightened dams in these countries is 258 (124 earth and rockfill).

Sixty-six per cent of the heightened dams are situated in 5 countries (United States, South Africa, Australia, Spain, Great-Britain); they account for 2.35% of the existing dams.

As a whole, the number of heightened dams is only 0.8% of the existing dam stock (0.4% of all earth and rockfill dams). But without China and its 18 595 dams, out of them one only had been heightened, percentage is 257/13 780 = 1.86%; it amounts to 7% for Australia.

From the Table below, we can see that in 46 countries, heightening is mainly for concrete dams and, among them, especially for gravity dams.

	Concrete dams		Fill dams	Total
	Gravity (74%)	99		
	Arch	18		
	Buttress	3		
	Multi-arch	5		
	Two-Type	9		

Nbr of dams heightened		134	124	258
Total number of dams		5 750	26 325	32 375
% of heightenings		2,3%	0,0046	0,8%
Total number of dams in the world		6 085	29 061	35 146

iii) Détail de la statistique pour 28 pays

Pour les 28 pays sur les 37 qui ont répondu et qui ont signalé des surélévations, on a présenté des listes séparées pour les barrages en remblai et ceux en béton (les barrages composites ont été classés avec les barrages en béton), et chaque liste est divisée en deux parties, la première correspondant aux surélévations prévues avant la construction du barrage initial, la deuxième à celles projetées après la construction.

La répartition est la suivante :

	Béton	**Remblai**	**Total**
Prévues avant	25	16	41
Projetées après	93	91	184
Total	118	107	225

On s'aperçoit que seulement 18% des surélévations signalées ont été prévues avant la construction du barrage initial. Ce pourcentage est légèrement plus élevé pour les barrages en béton (21%) que pour ceux en remblai (15%).

José Luis GUITART
Président, Comité
Bibliographie & Information

iii) Detailed statistics for 28 countries

For the 28 countries with heightenings, out of the 37 which have sent answers, separated lists for fill dams and concrete dams respectively are given (composite dams are included in the concrete dam list and each list is divided into two parts : the first one for heightenings planned before the original construction and the second for heightening decided after construction.

The dam numbers in the lists are as follows :

	Concrete	Fill dams	Total
Planned before	25	16	41
Decided after	93	91	184
Total	118	107	225

These average figures mean that only 18% of the reported heightenings were planned before construction. The percentage is slightly higher for concrete dams (21%) than for fill dams (15%).

José Luis GUITART
Chairman, Committee
Bibliography & Information

ANNEXE

ANNEX

(*) Président de la CIGB de 1970 à 1973 (décédé en 1981).

(*) ICOLD President from 1970 to 1973 (deceased in 1981).

COMMISSION INTERNATIONALE
DES GRANDS BARRAGES
de la Conférence Mondiale de l'Énergie

G. R. — E

QUESTION N° 20

SIXIÈME CONGRÈS
DES GRANDS BARRAGES
NEW YORK, 1958

GENERAL REPORT

HEIGHTENING OF EXISTING DAMS INCLUDING METHODS OF CONSTRUCTING NEW DAMS IN SUCCESSIVE STAGES

RAPPORTEUR GÉNÉRAL :
JOSÉ TORAN
I.C.C.P.
Espagne

I PART

I. — INTRODUCTION

> « *Therefore is the name of it called Babel because the Lord did there confound the language of all the earth and from there did the Lord scatter them abroad upon the face of all the earth* ».
> Genesis 11-9

Nature is hostile to man, and so man has to wrest his "habitat". Man wants, besides living, security for it. So he tries to overcome the hazards of his circumstance. The technician is in charge of this task. He has his ideas as tools, and his ingenuity has to direct the common effort to recreate a nature in its surroundings favorable to society.

To warm oneself at an eventual fire, to drink water found while passing by, are natural acts accessible to an animal. The technician succeeds in making fire and storing water to provide for future needs.

Technical action differentiates man from animal. Whenever natural resources are less, the bigger must be man's effort to satisfy his needs. Need is the challenge to technical action.

In Spain, one finds a striking example of the harshness of nature. arising from the scarcity of water and the turbulence of the rivers providing what little water there is. This condition has led Spain to pioneer in dam building. Many examples justify this claim (1). Once again, now when we start a new chapter in the Technique of dam Builders, we find Spain in the forefront and with surprising advance. Such is the case of the Almansa dam, that was built in 1384 with a height of 14 m and was heightened in 1586 by 7 m. This dam is an archgravity type, still in service today.

Several centuries have elapsed since the heightening of the Almansa dam, yet the same principle used there has developed in the construction of huge projects that are the pride of our present-day civilization.

Ross dam (U.S.A.) — and surpassing them all — Grand Dixence dam (Switzerland), the highest structure ever built by man, show the objectives that might be reached through the heightening technique.

Fifteen (15) countries have acknowledged the question, with a total of twenty-nine (29) reports, all of them significant and some very important.

Algeria	(1)	G. Safont, J. Salva.
Egypt	(1)	H. Zaky.
France	(3)	J. Bellier;
		M. Terrasa, H. Vieu;
		H. Chamayou.
Germany	(1)	H. Press.
Great Britain	(2)	J.M. Linton Bogle;
		P.I. Parker.
India	(1)	K.L. Rao, S.K. Dhawan;
Italy	(3)	G. Oberti;
		M. Scalabrini;
		C. Semenza.

(1) Other cases of Spanish pioneering in the construction of dams :
a) Gravity dams Proserpina, Cornalbo (Roman period under Trajan (2) : Alicante (Tibi) 41 m ; 1579-1594 (1), (2), (26), (27).
b) Arch dams Almansa, 14 m. 1384 (1), (2), (27) ; Relleu, 31 m, 1500 (2) : Elche, 23 m, 1632 (1), (2), (27) ; Arguis, S XVII.
c) Buttress dams Albuera Feria, 22 m, 1747. Head buttress dams. Burgobillodo, 26 m, 1928.
d) Dams over 100 m : Camarasa 103 m, (3), (27), 1920 (2) ; Tremp 104 m. 1920 (2).
e) Dams with power station included under the spillway. Gaitanejo (1920).
f) And also... unfortunately the first great catastrophe caused by the breakage of a dam. On April 30th, 1802 the Puent dam, the wonder of its age, suffered a perforation. It was 50 m high ; it caused 608 victims (1), (2).

Japan	(1)	M. Kondo, M. Kakitani.
Norway	(1)	F. Groner.
Portugal	(1)	A.C. Xerez, H.C. Pinto.
Romania	(3)	C. Mateescu;
		R. Priscu, A. Vasiliu, S. Caciulescu;
		R. Priscu, M. Constantinescu.
Spain	(3)	E. Becerril;
		C. Conradi;
		A. Presmanes.
U.S.S.R.	(2)	A.Z. Bassevitih;
		L. Bossovsky.
U.S.A.	(5)	C.J. Hoffmann;
		C.R. Scott;
		E.R. Dexter, F.A. Houck;
		S.W. Stewart;
		J.B. Cooke, J.E. Schumann.
Yugoslavia	(1)	D. Lazarevic.

The first surprise in reading the reports is the number of heightenings already performed. For various reasons, and utilizing different techniques, heightenings have multiplied, especially during the last thirty (30) years. Today, the most ingenious and varied procedures have already developed in use. Lack of data, especially on its detail, has not permitted the complete register of heightened dams. A tentative one, nevertheless, is included in the following chapter. It includes more than 100 cases.

To reach general conclusions of a rigorously scientific order, as to the best method, procedure, technique, or result, in connection with this question which has already received the contribution of engineers of highest qualifications in so many different countries and with so many different approaches, is a very difficult task, and in any case, too much for me.

The responsibility of a general report over a question of such transcendental importance comes to me only because of the supporting acknowledgement by I.C.O.L.D. of the historic doyenship on the matter that would correspond to Spanish engineering. Besides, new weight to this responsibility is added, because of the fact of representing my Spanish collegues — so many of whom are better qualified than I.

With these preliminary qualifications, let me say that my main goal has been to establish an orderly line. I have attempted even an inductive classification of these hybrid — though admirable — monstrosities that are the heightened dams. For it, I have followed the easiest way to review them all, which is to examine both the functional and morphological aspects. On several occasions, I felt the necessity of new words. I did not hesitate even coining them as requested when I thought it could serve to clarify the way. Let us avoid, at least, getting involved in confusion in this new intent of man to conquer heights. My dreams took me many nights to the Tower of Babel.

I-2 — HISTORICAL SURVEY

2-1 — HEIGHTENING

Up to 1900

Besides the Almansa dam, already mentioned, honours must be shared in historical precedence by the curious arch dam of Pontalto (Italy) (1). It was built in 1662 with a height of 5 m. and it was heightened for the first time in 1752 with the addition of 13 m. Proportionally this heightening, 2,5 H probably marks the record. Later a new heightening up to its present height of 38 m took place. We shall come back to this later on.

The examples prior to this century will be complete with the Parramata dam (Australia) (2). It was built in 1858 of masonry to a height of 12.5 m. Forty years later concrete was used to add a further 3.3 m. 0.265 H.

1900-1930

It is this century when important heightenings were undertaken. The first to be mentioned by reasons of chronological priority and the importance of the work, which involved great technical difficulties at the time, is the heightening of the Asswan dam (Egypt) [H. Zaky R-20]. The purpose of this work was to obtain another 7 m of storage height by means of a reduction in the freeboard and a heightening of 5 m in the structure. The original height was 30 m. Consequently 0.17 H.

Almost simultaneously, in 1913, the Ringedal dam (Norway) (3) was heightened for the first time. The addition in height was accompanied by the construction of a "Levy" facing in the upstream face. 11 + 4 m; 36 %.

The first heightening of the Tansa dam in Bombay (India) (1914) [K.L. Rao, R-52] also corresponds to this period.

In 1916 an important dam heightening appears; it is the first one carried out at Lake Spaulding dam (U.S.A.) (41). This is a constant angle type arch dam, which was planned with a view to successive heightenings. This initial height of 68.6 m had an addition of 10.7 m; 0.16 H.

Between 1920 and 1930 the following heightenings are recorded : Arguis (Spain) [Becerril R-13] 22.45 + 4.85 m.; 12,5 %. Total modification of the gravity section, and Cienfuens (Spain) (3), 10,5 + 10 m; 100 %. This dam was later heightened another 10 m always maintaining the same gravity section.

In 1923 Oklahoma dam (U.S.A.) was heightened [S.W. Stewart R-126] 16 + 4.6 m: 29 %.

1930 onwards

In this year we can see the start of systematic heightenings. The second heightening of Asswan (Egypt) [H. Zaky R-20] was completed in 1933; 35 + 5 m; 43 %.

Prestressed cables come to the picture

Around 1930 A. Coyne, completing an intellectual process which grew out of experiences on sea walls accomplished by Considère, introduced the use of poststressed cables in the field of dam construction, and with brilliant simplicity solved the reinforcement and a small heightening of the famous dam of Cheurfas (Algeria) (6), (7).

This occured some time before Freyssinet definitely proved the success of his general prestressed concrete technique in his work at the Gare maritime of Brest. It should be good sportmanship to pick up and clear up this detail which is told by Freyssinet himself (5) in honour to the merit of our great Coyne (we : all those dealing with large dams). In any case "Hommage au génie français".

In 1938 also Freyssinet developed the great possibilities of prestressing in the heightening of the Beni-Bahdel dam (Algeria) (5), (8).

In 1939 the heightening of Marshall Ford Dam (U.S.A.) takes place (9). This is a work of important dimensions 60 + 25 m: 42 %. The heightening is effected by means of an addition to the upstream face.

On the experience derived from these antecedents, the cases of heightening occur very frequently in recent years, amongst which there are many making use of the prestressed methods.

Today we can say that the experience obtained embraces the heightening of all classic types or dams. The achievements begin to be sufficient to derive general conclusions. However, the problem presents such a challenge to engineering ingenuity that is obvious that, within the immediate future, we shall see the development of the tendencies today merely suggested, and the introduction of systems which, even though only combining basic ideas already known, will change the approach to the problem, as well as its solution.

2-2 — MULTI-HEIGHTENING

As pointed out in one of the cases mentioned before, in many occasions the heightening of a dam has not been a single operation, but the pressures of demand have forced to repeat the operation, seldom foreseen on the previous instance. Here again the record example corresponds to the little Pontalto dam (Italy) with four increases in height after its construction in 1662. These heightenings, besides that of 1752, took place in 1825, 1850 and 1887: the final formula of Pontalto (Italy) (1) is 5 + 13 + 7 + 9 + 4 m. The original structural type arch gravity was always kept.

Spain registers the case of Irabia dam [Becerril R-13] with three heightenings and one in project according to the formula 15 + 12 + 7 + 6 + (11) m.

Tansa dam [K.L. Rao R-52] in India built in 1890, has undergone besides the heightening of 1914 already mentioned, two others in 1925 and 1948. Finally in 1951 it was reinforced by the system of prestressed cables.

On account of its technical pecularities, which will be described later on, we should mention as a classic case of multi-heightening that of the Asswan dam with the formula 30 + 5 + 15 + (5). The last heightening in project will be abondoned if building of the magnificent project of Sadd-el-Aali takes place.

To end this series we shall refer to the case already mentioned with two heightenings : that of Ringedal dam (Norway) 11 + 4 + 19 m (1913-14-18) and Lake Spaulding 68.6 + 10.7 + 4.6 = 83.90 m.

2-3 — CONSTRUCTION BY SUCCESSIVE STAGES

Building by stages is the logical consequence of the perfectioning of heightening techniques.

We must differentiate between multi-heightening and stage construction. The difference lies in that the former arises when not foreseen, or at least without precautionary measures in the original construction.

It is not strictly stated in the information we have although it may be inferred from a study of its profile that the already mentioned dam, Lake Spaulding, was originally planned to be built by stages. If not this one, we believe that it will belong to O'Shaughnessy dam the title of being the first important dam to which this technique is applied. Built in 1923 with an arch gravity section to a height of 105.1 m, it was increased in 1938 by 26.1 m up to its total height of 131.2 m. A classic case is the Ross arch dam, built in 1937 with a height of 93 m and planned for a second stage of 56.5 m, achieved in 1943. In view of the good results obtained from this heightening, a second one was undertaken in 1948 for 35.1 m up to its present height of 164.6 m. An eventual further heightening of 40.4 m is still under consideration.

At the present time the highest dam of the world is being built by stages. It is foreseen that the Grande Dixence dam (Switzerland) [C-34 Vth I.C.O.L.D. Paris] will come to the formula 182 + 42 + 30 + 30 = 284 m.

Stage construction has now become generally accepted and on this principle the great arch dams at Cancano (Italy) 173 m (two stages) and also Kurobe dam (Japan) 188 m (three stages) are constructed or planned.

3-1 — TABLE A. — REPORTS CONTENT

COUNTRY	REPORT No	AUTHOR	CONTRIBUTION TO THE GENERAL THEORY OF THE HEIGHTENING	MONOGRAPHIES	NOTICES OF HEIGHTENINGS	REFERENCES AND CITATIONS
ALGERIA	101	O.SAPONT J.SALVA		KEPFROUCH: Arch dam built with prefacricated blocks and poststressed cables.		BENI-BAHDEL.
EGYPT	20	H.ZAKY		ASSWAN: Description of its heightenings. Classic case.		
FRANCE	48	J.BELLIER			RASISSE, JOUX, TANSA, STEENBRAS, GAPARSA.	
	49	M.TERRASA H.VIEU		MONT LARRON: Heightening of an arch dam with the construction of two siding arches.		
	81	H.CHAMAYOU		OULE: Heigntened gravity dam.		
GERMANY	3	H.PRESS			SCHWAMMENAUEL: Earth dam, heightened with rock material.	
GR. BRITAIN	2	J.M.LINTON BOGLE		AVON: Arch gravity dam heightened by the COYNE'S method (Use of stressed cables).		
	44	P.I.PARKER	Use of stressed steel cables in the construction and heightening of dams.	Detailed description of COYNE'S method as applied to the dams of TANSA, HENLEY, STEENBRAS, CHEURFAS.		ALLT-NA LAIRIGE.
INDIA	52	K.L.RAO S.K.DHAWAN			BHATGAR, TANSA, JALAPUT, THOLERWADI, SHIRAWTA, WALWHAN, KOYNA, HIRAJUD.	LLOYD, NAGARJUNSSAGAR.
ITALY	14	G.OBERTI	Laboratory tests on the construction by stages of arch dams.	Study on scale model of the dams of PRERA, CANCANO, KUROBE, TOULES.		
	38		Analytical study of concentric arches. Theoretic advantage of independent arches.	PRERA: Arch dam built by stages. Study of monolithism.		LINBERG, ROSS, CANCANO.
	86	C.SEMENZA	Construction by stages of arch dams General study. Devices on the joint face to insure monolithism.			KAE, AMPIESTA, VAIONT, KUROBE, ROSS.
JAPAN	23	M.KONDO M.KAKITANI	Various stresses to be considered and anlysed in the tensional study of a heightening. Determination of the downstream batter under partial hydrostatic load.	Very detailed description of the heightening of ODOMARI dam.		LAGES, MARSHALL FORD, CHEURFAS, BEVER, MULLARDOCH, O'SHAUGHNESSY, ASSOUAN, GRANDE DIXENCE, VERSE.
JUGOSLAVIA	90	D.LAZAREVIC	Different methods of heightening arch dams and hollow dams. Use of tensile steel cables. Reinforcing arches. Type of joint permitting a movement at the contacting surfaces.			
NORWAY	50	F.GRONER		KÖBVAN: Gravity dam heigntened at upstream side.		

COUNTRY	REPORT N.	AUTHOR	CONTRIBUTION TO THE GENERAL THEORY OF THE HEIGHTENING	MONOGRAPHIES	NOTICES OF HEIGHTENINGS	REFERENCES AND CITATIONS
PORTUGAL	40	A.C. XEREZ H.G. PINTO		CHICAMBA: Arch dam constructed by stages.		
RUMANIA	32		General typology. Heightening applicable to the various types of dams. Detailed study of arch and buttress dam cases.			GRANDE DIXENCE, BERNINA, HABRA, CHEURFAS, SABBIONE, PUNTA NEGRA, LA GIROTTE, ROSS.
	33	R. PRISCU A. VASILIU S. CACIULESCU		NEGOVANU: Gravity dam designed to be built by stages. Comparative study of different types		
	34	R. PRISCU M.CONSTANTINESCU	Study of the heightening and construction by stages of gravity dams with prestressed cables. Effect of plastic flow of concrete			
SPAIN	13	E.BECERRIL	The joint in the heightening of a gravity dam. Tangential and effective sliding forces. Suitable location of the joint. Favourable heightening percentage.			PROSERPINA, ALMANSA, ELCHE, ARGUIS, LAS VENCIAS, PUENTES VIEJAS, BURGUILLO, IRABIA, BOLARQUE, BACHIMAÑA, BURGOMILLODO, CAMPOFRIO, SALIME, LOS PEARES, ASSOUAN, HABRA, DIXENCE.
	47	C.CONRADI		TORRE DEL AGUILA: Earth dam, heightened with rock material.		
	137	A.PRESMANES		DOIRAS: Gravity dam heightened. Direct method.		
U.S.A.	91	C.J.HOFFMANN		ALAMO GORDO and PINEVIEW: Earth dam. Spillway problems.		
	92	C.R.SCOTT		MONTGOMERY: Rockfill dam with bituminous water tight deck.		
	121	E.R.DEXTER P.A.HOUCK	Preliminary considerations to all heightening. Numerous bibliographic references.			LAGES, ROSS, STEENBRAS, CHARLOTE LAKE, BURRINJUL, SENNAR, ASSOUAN, O'SHAUGHNESSY, MUNDARING, MARSHALL FORD, LAKE MENTZ, SANTA BARBARA
	126	S.W.STEWART	Heightening of AMBURSEN type dams. Use of mobile gates.			DANVILLE, BOYDS CORNERS, POSSUM KINGDOM, BRISTOL, SHAWVILLE, UTICA, BOYSEN, MARFA, OKLAHOMA, RODRIGUEZ, WESLEY SEALE, OZARK BEACH, ESCABA, CADILLAL, GUAYABAL, MT. UNION, OCAMO, MATHIS, JORDAN
	127	J.B.COOKE J.E.SCHUMANN		BALCH: Heightened arch dam. Thermic study. Use of "Prepakt". Grouting of the joints.		
U.S.S.R.	112	A.Z.BASSEVITH	Analitical study of anchoring with cables. Redistribution of elastic stresses by means of joints, with diferred bonding.			
	123	L.ROSSOVSKY		MIGUETCHAURSK: Earth dam constructed and heightened by hydraulic filling.		

3-2 — TABLE B.

NAME OF THE DAM	COUNTRY	RIVER	STRUCTURAL TYPE	UTILIZATION	APROXIMATE DATE: CONSTRUCTION	APROXIMATE DATE: HEIGHTENING
ALAMO GORDO	U. S. A.	PECOS	E-R	IR		
ALMANSA	SPAIN	BARRANCO GRANDE	G(M)	IR	1384	1586
ARDMORE	U. S. A.		B	WS		
ARGUIS	SPAIN	ISUELA	G			1926
ASSWAN	EGYPT	NILE	G(M)	IR-P	1902	1912 1933 Pr.
AVON	GR. BRITAIN	AVON	G	WS		
AYERS ISLAND	U. S. A.		B		1923	1930
BALCH	U. S. A.	KINGS	A		1927	1957
BENBOW	U. S. A.		B	P		
BENI BAHDEL	ALGERIA	TAFNA	MA	WS-P		
BERNINA	SWITZERLAND		G		1942	
BHATGAR	INDIA		G(M)	IR	1890	Pr.
BOLARQUE	SPAIN	TAJO	G	P		
BOYDS CORNERS	U. S. A.		G(M)	WS		Pr.
BOYSEN	U. S. A.	BIG HORN	B	P	1907	Pr.
BRISTOL	U. S. A.	PEMIGEWASSET	B	P	1924	1932
BURGOMILLODO	SPAIN	DURATON	B	P		
BURGUILLO	SPAIN	ALBERCHE	G	P	1913	
CADILLAL	ARGENTINA		B	IR	1940	
CALA	SPAIN	CALA	G	P	1927	
CAMPOFRIO	SPAIN		G			
CANCANO	ITALY		A	P	1956	Pr.
CHEURFAS	ALGERIA	MEKERRA	G	IR	1891	1930
CHICAMBA	PORT.EAST AFRICA	REVUE	A	P		
CHORRO	SPAIN	TURON	G	P	1921	1948
CIENFUENS	SPAIN	FLUMEN	G(M)	IR		
COAMO	PUERTO RICO		B			
DANVILLE	U. S. A.	DIX	G(M)	WS		1904
DOIRAS	SPAIN	NAVIA	G	P		
ELIZABETH	U. S. A.		B	WS		
ENNEPE	GERMANY		G		1904	
ESCABA	ARGENTINA		B	IR	1940	
FRERA	ITALY		A-G	P		
GAFARSA			G(M)		1910	
GIROTTE (LA)	FRANCE		MA	P	1948	Pr.
GRANDE DIXENCE	SWITZERLAND	VAL DES DIX	G	P		
GUADALMELLATO	SPAIN	GUADALMELLATO	G			1952
GUAYABAL	PUERTO RICO	JACAGUAS	B	IR	1911	

REGISTER OF DAM HEIGHTENING

Reasons for the heightening: Rectification hipot	Strengthening	Silting	Spillway	Heigher demand	Construction by stages	Numerical data of the heightening: h (m)	Δh (m)	W (Hm³)	Δw (Hm³)	C (Dm³)	Δc (Dm³)	Typology: Direct heightening	Prestressed anchorage	Differentiate structure	Syn-monolithism	Postmonolithism	Idiomorphism	Metamorphism	References: VIth I.C.O.L.D. Report No	Others
			◢			43	5	160		1460	260						◢		91	
				◢		14	7					◢							13	
				◢		14	3					◢							126	
				◢		24	3								◢			◢	13	
				◢		30	5	1000	1500							◢	◢		20	
				◢			15		2500					◢				◢	20	
				◢		.	5		5000							◢	◢		20	
				◢																
				◢		29	4	1,4	0,7				◢						2	
					◢	15	9								◢		◢			/9/ DAVIS P.154/E.N.R.Nº7.1933
					◢	29	13								◢		◢		127	
						7	3					◢							126	
				◢		57	7	42	31				◢							L'HOUILLE BLANCHE, 3-IV-1954
				◢			4	15	2,2			◢							32	/23/ CONTESSINI, p. 392.
				◢		27	18	150										◢	52	
				◢		23	13								◢		◢		13	
				◢		18	18								◢			◢	126	
				◢		12	5										◢		126	
					◢	17	9								◢		◢		126	
				◢		26	9								◢			◢	13	
					◢										◢		◢		13	
					◢	60	15										◢		126	
						53		60		99										/2/ GOMEZ NAVARRO, p.1170.
				◢		26	6					◢							13	
					◢	137	36									◢	◢		14-38	
	◢					40		15					◢						13	
					◢	55	20	400	1600						◢		◢		40	
				◢		80	2	80		130		◢								/2/ GOMEZ NAVARRO, p.1173.
				◢		11	10								◢		◢			/3/ KELEN, p.211
					◢		10								◢		◢			
						17	1					◢							126	
				◢		5	1								◢			◢	126	
				◢		94	4	102	16			◢							137	
				◢		4	2					◢							126	
						44	10	2,3	10,3			◢								/3/ KELEN, p.213.
					◢	71	5	126	53						◢		◢		126	
					◢	73	65									◢	◢		14-38	
				◢		10	6	1	6				◢						48	
						48	11	53		80									32	LE GENIE CIVIL, X-1948.
					◢	182	42	100	94	1850	1460					◢	◢			
					◢		30		91		1380					◢	◢			
					◢		30		115		1200					◢	◢			C-34, 5th. I.C.O.L.D.
				◢		56	8	110	52	225	72				◢		◢			
		◢				37	5								◢		◢		126	

3-2 — TABLE B. (continued)

NAME OF THE DAM	COUNTRY	RIVER	STRUCTURAL TYPE	UTILIZATION	APPROXIMATE DATE	
					CONSTRUCTION	HEIGHTENING
HABRA	ALGERIA	FERGOUG	G		1873	
HAMIZ	ALGERIA		G		1885	1935
HENLEY	SOUTHAFRICA		G			
IRABIA	SPAIN	IRATI	G			
						Pr.
JALAPUT	INDIA	MACHKUND	G(M)	P		
JORDAN	CANADA	JORDAN	B	P	1912	
JOUX	FRANCE		G(M)	WS	1905	
KOYNA	INDIA		G	P		
KSOB	ALGERIA	KSOB	MA		1937	Pr.
KUROBE	JAPAN		A-G	P		
LAGES	BRAZIL		G	P	1908	1948
LAKE SPAULDING	U. S. A.	YUBA	A		1913	1916
						1919
LENNEP	GERMANY		B			1905
LIMBERG	AUSTRIA	SALABACH	A	P	1951	
MARFA	U. S. A.	ALAMITO	B	IR	1911	
MARSHALL FORD	U. S. A.	COLORADO	G		1939	
MATHIS	U. S. A.		B	P		
MAUVOISIN	SWITZERLAND		A	P		
MINGUETCHAOURSK	AZERBAIDJAN	KOURA	E	P-IR	1952	1956
MONROE	U. S. A.		B	WS		
MONTGOMERY	U. S. A.	SOUTH PLATTE	R	WS	1957	Pr.
MONT LARRON	FRANCE	MAULDE	A	P		
MÖSVANN	NORWAY		G		1940	
MOUNT UNION	U. S. A.		B	WS		
MULLARDOCH	GR. BRITAIN		G			
MULLHOLLAND	U. S. A.		G-E		1925	
NAGARJUNSSAGAR	INDIA		G(M)			
NEGOVANU		SADU	A	P		
NORDHÄUSER	GERMANY		G			
ODOMARI	JAPAN	OTA	G	P	1935	
OKLAHOMA	U. S. A.	NORTH CANADIAN	B	WS	1918	1925
O'SHAUGNESSY	U. S. A.	HETCH HETCHY	A-G	WS	1925	1938
OULE	FRANCE		G(M)	P	1924	1947
OZARK BEACH	U. S. A.	WHITE	B	P	1911	Pr.
PARRAMATA	AUSTRALIA		A-G	IR	1858	1898
PATILLAS	PUERTO RICO			IR		
PINEVIEW	U. S. A.	OGDEN	E-R	IR	1937	

Register of dam heightening

Reasons for the heightening: Rectification hipot.	Strengthening	Silting	Spillway	Heigher demand	Construction by stages	Numerical data of the heightening: h (m)	Δh (m)	W (Hm³)	Δw (Hm³)	C (Dm³)	Δc (Dm³)	Typology: Direct heightening	Prestressed anchorage	Differentiate structure	Syn-monolithism	Postmolithism	Idiomorphism	Metamorphism	References: VIth I.C.O.L.D. Report No	Others
	◢					35							◢						13-32	/2/G.NAV.p.1176;4th,I.C.O.L.D. R-49.
						41														
							7						◢						44	
				◢		15	12								◢		◢		13	
							7								◢		◢		13	
							6								◢		◢		13	
							9								◢		◢		13	
◢		◢				52	3	724				◢							52	
	◢						1,5					◢							52	
						38	2					◢							126	/2/ GOMEZ NAVARRO, p.1177.
				◢			5						◢						48	
					◢	81	19								◢		◢		52	
						32	15	8	22	30	20									/2/ GOMEZ NAVARRO, p. 1472.
					◢	120	68									◢	◢		14	
				◢		32	28	182	820							◢		◢		E.N.R. VII-1949.
					◢	69	11					◢								E.N.R. I-1936.
							5								◢		◢			
						11	3								◢			◢		/3/ KELEN, p. 213
					◢	120		86		46					◢		◢		38	/2/ GOMEZ NAVARRO, p. 1390.
					◢	21	11	18,5	43,2						◢		◢		126	
				◢		60	23								◢		◢			
						27	2					◢							126	
					◢										◢		◢		38	
						76	4										◢		123	
						8	2					◢							126	
					◢	34	14										◢		92	
					◢	20	12						◢		◢			◢	49	
	◢			◢		16	4	0,8	0,3					◢				◢	50	
				◢		15	4										◢		126	
					◢	36	6									◢	◢			5th. I.C.O.L.D. R-12.
	◢					63	9	5		131								◢		/2/ GOMEZ NAVARRO, p.1180.
					◢												◢		52	
					◢	42	21								◢		◢		33	
							4	0,8	1,2			◢								/3/ KELEN, p.214.
	◢			◢		60	10	13	13						◢		◢		23	
		◢	◢			16	5					◢							126	/2/GOMEZ NAVARRO, P.1442.
					◢	105	24									◢	◢			E.N.R., V-1939.
				◢		38	13	6,7	9,9	42	87						◢		81	
				◢			2					◢							126	
					◢	13	3													/24/ WEGGMANN, p. 538.
				◢			2					◢							126	
				◢		19	9	54,5	81,2	205	115								91	

NAME OF THE DAM	COUNTRY	RIVER	STRUCTURAL TYPE	UTILIZATION	APPROXIMATE DATE: CONSTRUCTION	APPROXIMATE DATE: HEIGHTENING
PONTALTO	ITALY	FERSINA	A		1612	1752 1825 1850 1887
POSSUM KINGGDON	U. S. A.	BRAZOS	B	IR-P	1940	
PUENTES VIEJAS	SPAIN	LOZOYA	G	WS		1935
PUNTA NEGRA			B			
RASSISSE	FRANCE	DADOU	A			
RINGEDAL	NORWAY		G			1913 1918
RODRIGUEZ	MEXICO	TIJUANA	B	WS-IR	1934	
ROSS	U. S. A.	SKAGIT	A		1937	1943 1948 Pr.
SABBIONE	ITALY	SABBIONE	B	P	1950	Pr.
SCHWAMMENAUEL	GERMANY	RUR	E			
SENNAR	SUDAN	NILE	G-E	IR	1918	1925 1952
SHIRAWTA	INDIA		G	P	1920	Pr.
SPARTAMBURG	U. S. A.		B	WS		
STEENBRAS	SOUTH AFRICA		G	WS		
TANSA	INDIA		G-R		1892	1914 1951
TEHUANTEPEC	MEXICO		B	N	Pr.	Pr. Pr.
THOKERWADI	INDIA		G	P	1922	
TORRE DEL AGUILA	SPAIN	SALADO	E	IR	1936	1958
TOULES	SWITZERLAND		A	P		
UTICA	U. S. A.		B	WS	1906	
VADO (EL)	SPAIN	JARAMA	G	WS		
VAIONT	ITALY	VAIONT	A		Pr.	
VENCIAS (LAS)	SPAIN		A			Pr.
VERSE	GERMANY		G			
WALWHAN	INDIA		G(M)	P	1916	
WESLEY E. SEALE	U. S. A.		B-E	WS		Pr.

Register of dam heightening

Reasons for the heightening: Rectification hipot	Strengthening	Silting	Spillway	Heigher demand	Construction by stages	Numerical data of the heightening: h (m)	Δh (m)	W (Hm³)	Δw (Hm³)	C (Dm³)	Δc (Dm³)	Typology: Direct heightening	Prestressed anchorage	Differentiate structure	Syn-monolithism	Postmonolithism	Idiomorphism	Metamorphism	References: VI[th] I.COLD Report No	Others
		◢				5	13					◢								/1/COYNE, LECONS SUR LES
		◢					7					◢								GRANDS BARRAGES, p.59.
		◢					9					◢								
		◢					4					◢								
				◢		58	5	900	655			◢							126	
					◢	28	36								◢		◢		13	
						80	20												32	/26/ CONTESSINI, p. 132.
	◢					32							◢						48	
	◢					11	4											◢		/3/ KELEN, p. 211.
				◢			19													
				◢		73	8					◢							126	
					◢	93	52								◢		◢			
					◢		35		895						◢		◢			
							40		2540						◢		◢		32	E.N.R., IX-1945,IV-1948.
				◢		60	15	26	16	135					◢		◢			L'ENERGIA ELETTRICA,IX-1954
						54	16	100	105									◢	3	
				◢		6,0	0,3	781				◢								L'HOUVILLE BLANCHE,IX-1954
	◢						0,5											◢		
	◢					39	3	186	0,0	478	87	◢							52	
						15	2					◢							126	
				◢		30	2						◢						44-48	
					◢	36	3								◢		◢		48-52	
◢	◢				◢		2					◢	◢				◢		48-52	
					◢	30	9					◢							126	
					◢		21								◢		◢			
◢	◢					60		364		211									52	
				◢		25	4	44	26									◢	47	
					◢	26	59									◢	◢		14	
					◢	9	3								◢		◢		126	
						53	14	38		184										
					◢	210													86	L'ENERGIA ELETTRICA,II-1955
					◢	25	8								◢		◢		13	
				◢		58											◢			/3/ KELEN, p. 214.
◢	◢					27	0,0											◢	52	
					◢	11	3	370	247						◢		◢		126	

II PART

II-1 — REASONS FOR HEIGHTENING

1-1 — Enforced Decision

Very seldom the heightening of a dam is decided by a single reason out of the many which could justify it. Usually heightening results of a multi-reason appraisal. Among them to increase the height might not have been the leading purpose. Heightening occurs as a consequence. Let us consider first the correction of design errors. This is a case which appears with some frequency through the records.

Reinforcement

We refer to cases of underdesign affecting basically the main section of the structure or the loading hypotheses. Actual loads overpassing the latter challenge the stability, and the insufficient section may be corrected with additional weight (this is particularly true for gravity dams which count more numerously in heightening files). If weight has to be added height increase results as a direct sub-product.

Cheurfas dam in Algeria and Tansa dam and Shirawta dam [R-52] in India, (in both uplift pressure being higher than the estimate) could serve as examples.

Repairs

In other instances, it is not the design errors but inadequate construction which enforces the repair of a dam. The most frequent is the bad quality of materials, especially in old masonry dams as Ringedal dam (Norway) and Tansa dam (India). It also happens to relatively modern dams, in which sufficient precautions were not taken against the attack of water with low pH, e.g., Moswan dam (Norway).

Reappraisal of Data

The insufficiency of hydraulic data available during the original design, corrected later by statistical observations also leads to heightening. In dams built for the main purpose of flood control heightening has resulted frequently of two objectives, sometimes simultaneous :

a) Increase of the storage capacity to smooth the overflow peaks and

Spillway

b) to obtain by increasing the water level a higher length on the spillway crest or to reach a new one by a glen in the reservoir contour. Oklahoma dam [R-126], Alamo Gordo dam [R-91].

A strange case which we cannot resist quoting as an example of changing hypotheses and also of reducing height is that of Mulholland dam (U.S.A.), which was reinforced with an earth backfill down stream and reduced in its water level in spite of the opinion of the experts consulted who considered this operation unnecessary, for the simple reason that the population of the valley down stream, under a psychosis of collective fear, pressed the authorities until they obtained the above mentioned changes.

Silting

Dams may also lose their usefulness without any failure in their structure. This happens when silting fills up completely or reduces substantially the storage capacity. We have again as an example our old friend the Pontalto dam which through the centuries had lost successively its utility as a protection of the city of Trento against floods, due to the amount of materials transported by the Fersina river. It was finally relieved of its function by a new dam. The Hamiz dam (Algeria) [R-49, IV° ICOLD], was also rendered useless due to silting in the reservoir and had to be heightened. The same thing happened to Guayabal dam in Puerto Rico, built in 1911, which in 1912 when the heightening was decided, suffered of deposits amounting to 12 % of the reservoir capacity.

Silting, in some way a physiographical disease of reservoirs, is still the great enemy of their utility to store water.. Its solution by means of large bottom sluice gates (Mera dam, (Italy) and Vah dam, (Czechoslovaquia)), would probably alter the concept we have of the structure of dams. Dredging [R-49, IV° ICOLD] and other systems have been tried with varied success. The problem of silting and its solution has been the object of Report N° 14 discussed during the IV° ICOLD. Nothing important has modified the conclusions that were then established [Drouhin — RG-14 — IV° ICOLD]. Many reservoirs will practically die because of silting and will have to be heightened.

Increase of demand

Between the reasons enforcing heightening, we cannot overlook the one corresponding to a bigger demand coinciding with unavailability of sites for a new construction. The fact that this reason is seldom imperative leads to deal with it in following articles.

I-2 – OPTIONAL DECISION

The two elements which, adjective to the existence of water are required for its use are volume W to store it, and head H for power. Both being geographical functions should have inevitable limitations.

Whatever the number of steps in which a water flow is divided to harness it, the sum of them cannot exceed the difference of levels between the source and the sea.

Modern techniques (injections, prestressing, etc.), permit the hydraulic fitting of soils and succeed in correcting their conditions and ability for dam foundation. The number of valleys suitable for the construction of reservoirs is consequently increased and so happens with the total W available for mankind. The limit of W becomes more inaccurate than the linfit of H, although it unquestionably exists for practical uses.

The demand of W is and shall be permanent. But W is not only a strict function of the inflow regulation, but also of the demand and for this the cycle of irregularity (based on man's life and his water needs under all forms) is daily or at most seasonal. A total regulation does not consist in obtaining a constant flow from a natural variable inflow but in reaching a discharge that could follow the demand requirements. Consequently the required W has to be appraised from two terms.

a) A constant term, the asymptotic value to which, through an over-years period, the regulation of the available inflow tends, which is a function of natural variables basically meteorological.

b) Another term, function of the demand, which requires a discharge — even with a constant average value — under a changeable regime.

A yearly regulation is, at the most, sufficient for the consumption's cycle of variations, but nevertheless the discharge to suit the demand may concentrate in very short peaks. In the extreme case in which it could not be possible to compensate the inflow with the required outflow, it would be : $W = W_1 + W_2$.

We have seen before that W_1 has, and W_2 has not, a limit. W_2 increases with mankind their prime needs of drinking and eating (irrigation), and shall still increase in spite of trends to nuclear power. The intermittent utilization of nuclear power plants shall always be expensive. (Hydro-power apparently will have to be in charge of peak supply) (19).

The need of W will be bigger than the topographical possibilities. H and W, the two conditions which are required with water are limited and so as an obvious consequence results :

1) The necessity of exhaustive profit of remaining sites.

2) The revision of previous developments.

Technique has to move indefectively towards the aforesaid points. Nowadays, without the control of an absolute economy planning which, as far as we know, does not exist in any country, it is very unusual the case of heightenings which could not be compared, beforehand, to a new dam construction. An option between both solutions is normally possible. Decision coming from various reasons basically economical shall be reviewed in the next article.

Better Efficiency

The height of a dam and the reservoir volume are absolute geometrical dimensions. However, the useful head and actual storage are relative and depend on water. A proper design could be reflected in the final efficiency of the work.

A conventional dam for power purposes, loses the difference of head between the maximum level and the actual level of the storage, e.g. the difference between the head of the level step occupied by the development and the storage water head at any moment. In a series development this loss can be avoided in the second and following steps by heightening the dam and preparing the corresponding upstream plant for work under head at both sites.

The use of concentration reservoirs to regulate different flows derivated from parallel valleys permits placing the supply channel to the reservoir close to the dam and with a direct access to the power house. In this way the powerplant works in « countercharge » profiting by the difference of heads.

The level oscillation in regulation of reservoirs enforces a great elasticity in the generators. Its unattainability leads to a loss of efficiency. The heightening of the dam could correct this condition by placing the volume affected by the regulation on a higher position which, because of its bigger surface will give a smaller difference in head under the same regulation. The improvement of efficiency is obtained in such a case from two terms : a bigger useful head and a reduction in the head oscillation due to the regulation. This system means a heightening of the whole reservoir, its lower part becoming a support of the useful storage.

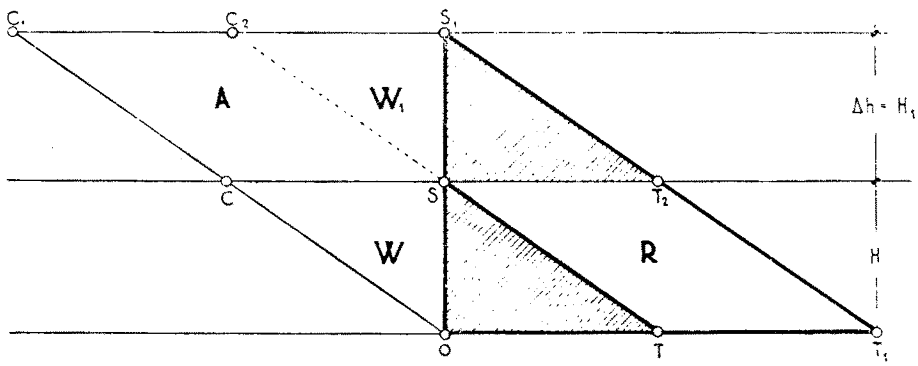
This is an adequate procedure for the hydro-electric utilization of reservoirs which have been built basically for regulation with irrigation purposes.

II-2 — ECONOMIC TOPICS

2-1 — HEIGHTENING COMPARED WITH NEW CONSTRUCTION

We have reviewed the reasons wich may cause the heightening of a dam. Let us disregard the cases in which the lack of site suitable to build a new dam to meet the new requirements enforces the heightening of an existing one, and also those in which the basic object is to reinforce or improve the existing dam eventually damaged.

We shall restrict the field of our economic scheme to the cases in which heightening arises as an optional solution to the building of a new dam.



A dam is represented in our sketch by the area OST with a height H and a reservoir represented by the triangle OSC with W being its capacity. What will result more economically advantageous, the construction of a separate dam, S S_1 T_2, height H_1 in a different location, or the addition to the existing dam of a new section so that Δ h = H_1 ? It is obvious from the sketch that this addition will result in a new dam O S_1 T_1, and the reservoir will be the one represented by the area O S_1 C_1. If a comparison is made between the dam and the reservoir resulting from the heightening, and a new dam of the same height, we will see that the heightening implies an additional construction corresponding to the reinforcement of the original structure represented by the area R limited by the points T, S, T_2, T_1. R represents the additional work necessary so that the existing dam may supply the foundation to the new one. On the reservoir side, however, we get additional storage capacity when comparing the one obtained from the heightened dam to that of a new dam. This volume is represented by the area A, limited by the points C, C_1, C_2, S. On first appraisal a conclusion may be anticipated : If the basic objective is to obtain an increase of head, the solution is not economic. When comparing it to the building of a new dam, the additional cost of reinforcing the existing dam to support the new section will make it inadvisable. However, if the main object is the increase of the storage capacity heightening will be preferable to a new construction in the case that the increment obtained in the reservoir, A has a value higher than the cost of the reinforcement R. In the general case of a multi-purpose for head and regulation, heightening offers with respect to the construction of a new system, an advantage, which is an increased reservoir A, and a disadvantage, with regards to cost in view of the necessity to undertake the construction of R.

Although the drawing offers only a general solution which should be adapted to the circumstances prevailing in each particular case, we believe it to be sufficiently clear to outline an economic study and direct the heightening design with special object of a reduction in R.

In practice the curves giving the areas covered by the reservoir with respect to height are not a straight line nor is the cost of a dam proportional to height. Let us point out, however, the topographical quality of the sketch under comment, that is its remaining invariable under any change in the limiting curves S_1, T_1 and OC_1. Always an area R will represent the disadvantage in cost, and an area A the advantage by reason of the increased reservoir.

The characteristic graphics of reservoirs have already been studied by various authors. R.A. Sutherland formula (18) is currently accepted and has been proved to adjust to reality. It is a parabolic formula $S = az^m$ for the curves $O C_1$. The parameter a and the exponent m are functions typical of the basin. Since the geometric shape depends upon erosion, and this is a consequence of the geological nature of the soil (a climatological influence may also be considered), it follows that m is a fundamentally geological function. More detailed studies about this geometric influence on reservoirs from geologic nature are missed, although some authors have already insinuated them (16). They would open a track for the standing problem of reservoir prospection, especially in research about parallel valleys.

Variation of m ranges between 2 and 4 and the value 2.5 is normally accepted. The parameter a can only be determined by direct measuring of areas affected in each particular case.

The curve S_1 T_1 which gives the variation of the dam areas in connection with the heights measured from the crest, is also a parabolic function dependent on the parameters of the gorge and of the structural type of the dam. By way of suggestion we include a table with the approximate feature of these curves, adjusting them to the classic types of gorge and of the shape factor commonly accepted L/H.

To facilitate the comparison between the two drawings, natural scales have been used in the first one. In practice, however, it is preferable the use of logarithmic scales. Besides the curves characteristic for the dam and for the reservoir should not be arranged on the same drawing as the magnitudes to be represented are of a different order.

2-2. — CONSTRUCTION BY STAGES

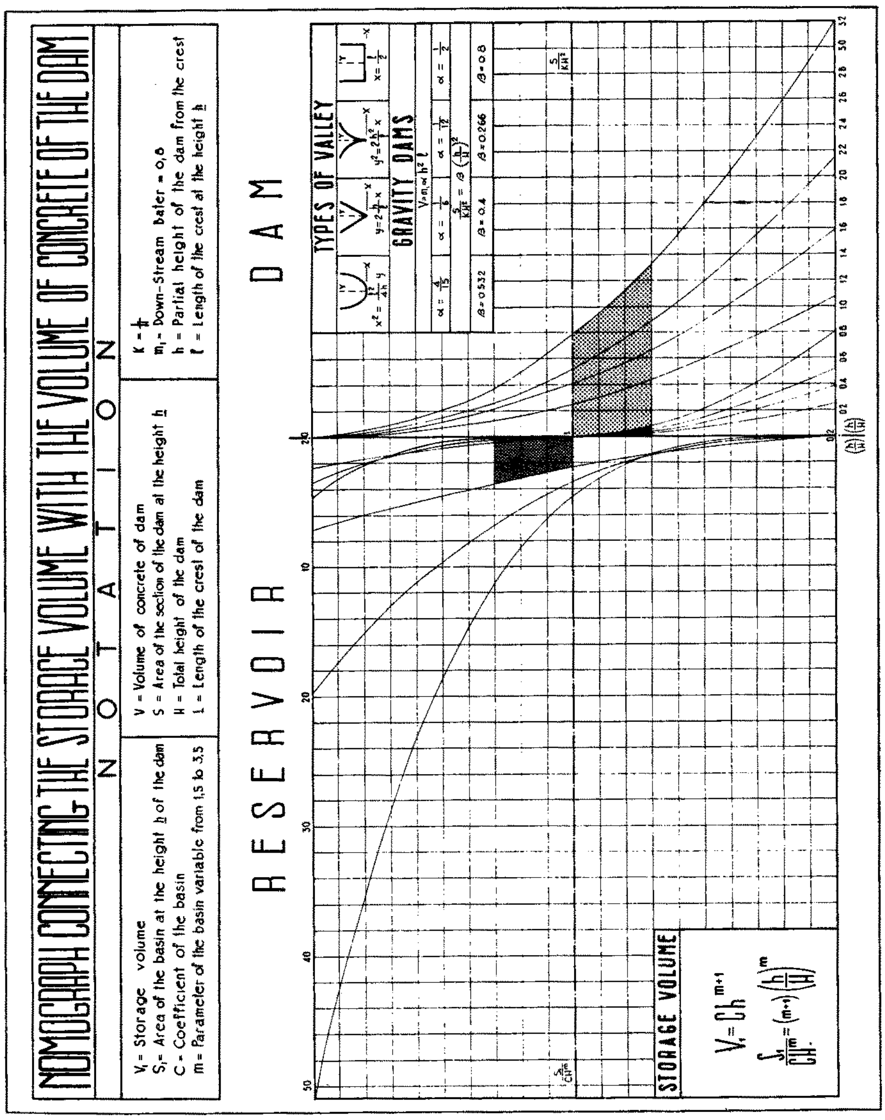
As we saw in the previous chapter it is an unavoidable law that ever increasing demand will use up the possibilities of adequate sites (from both points of view, gorge and basin) for the construction of storage dams. In spite of the development of soil conditioning techniques, with an increase of possibilities over spontaneous geological resources it is obvious that the cases in which an optional solution is allowed will decrease. The heightening of existing dams will have to be contemplated " a fortiori " and it will be imperative the exhaustive utilization of new sites, for the building of dams which should be designed providing for later heightening. If this could be established as general statistical principle, the consequence will be that as the scope of application decreases the moment in which the optional solution will no longer be available is brought forward.

This applied not only to fully developed countries, which for the same reason can see a near end of their topographical resources, but also to underdeveloped countries which should not afford, faced with a rapidly increasing demand, to solve today's problem at the expense of higher costs for future developments.

In the order of absolute economy going beyond the actual financial costs, to consider the real counter value of the work which in the last instance is men's time engaged in work, it is obvious that the most advantageous solution will be to build the total structure as soon as possible.

Man distributes his spare time after life requirements between the terms of the binominal idleness and work. By natural law, the pressure of demand of the first factor is in continuous increase. Technique multiplying work output can reduce the working time and re-establish the equilibrium to a certain extent.

However, the relative possibilities of engineering technique with regards to this compensation are less as higher is the civilization level. The benefit and happiness from marginal idleness, which may be obtained by the application of technique to work, are more obvious when applied to a more primitive stage. The wheel caused more happiness and idleness to man than the jet plane.


NOMOGRAPH CONNECTING THE STORAGE VOLUME WITH THE VOLUME OF CONCRETE OF THE DAM
N O T A T I O N
V1 = Storage volume
S1 = Area of the basin at the height h of the dam
C = Coefficient of the basin
m = Parameter of the basin variable from 1,5 to 3,5
V = Volume of concrete of dam
S = Area of the section of the dam at the height h
H = Total height of the dam
l = Length of the crest of the dam
m1 = Down-Stream bater = 0,8
h = Partial height of the dam from the crest
ℓ = Length of the crest at the height h
R E S E R V O I R
D A M
TYPES OF VALLEY
GRAVITY DAMS
STORAGE VOLUME

The more adjusted is the equilibrium the less effective the the resources of technique will be; man will demand a higher compensation for the forsaking of his idleness. Labour becomes more expensive. This increase in the cost of labour is easier to forecast when the social environment to be considered is at a higher degree of development

Countries technically young can still afford doing their work slowly, and not to use exhaustively the resources provided by nature. It is in highly developed countries where the increase in labour costs can challenge economic balance within a short period. A healthy estimate and future anticipation resulting from the clear government vision or even from financial survey according to the cases described above tend to total construction; that is, to heightening and to immediate and exhaustive construction of the remaining reserves.

However, to do this in the scale of the technical means today available surpasses in most cases financial possibilities. Technique has exceeded the human modulus in the handling of magnitudes, even of a financial order. There are no possible financial means in the world today to cover the prevision of future needs, even to short term if we deal with generations timing.

Once the principle of shortage of means is established, construction by stages becomes an inavoidable solution. It is interesting to quote as a confirmation of this theory that previously in richer countries is where the system has been started in full scale. It is in Switzerland, one the most prosperous countries, where the highest dam of the world, the Grande Dixence, is being built by stages. Further to this, and with great realistic vision, the work has been undertaken by private enterprise desirous naturally to obtain a rentability from their investments within a short delay.

Construction by stages is more expensive; all authors meet in this opinion. This cost increase must have a compensation, otherwise the system would not be applied except in the cases of short financial resources. What factors add to this compensation ? In the first place exploitation can start at an earlier period, and so satisfy a pressing and remunerative demand. If a higher remuneration is caused by necessity, this will not be frequently the case in rich countries in which normally the demand is almost satisfied. It will be only sufficient to study the statistics of consumption per capita and coefficient of the increase of the demand, to observe that the latter is lower as the former is higher.

The balancing item in those countries is the inevitable forecast of higher costs in coming years. The decision then will result in comparing the cost of the total work at the moment in which it would become avoidable with the cost of construction by stages starting at the moment. This solution offers two advantages : a) the profit of an anticipated exploitation and b) lower absolute costs.

From all this, two different cases should be distinguished in the economic problem of construction by stages : the case in which full demand exists (we shall not consider its quality) for the total production and the case in which only a demand is ready only for a part of the foreseen production. Anticipation for future demand should then be considered and it will point to exhaustive utilization. There is no doubt in the first case, since the balance arises from the difference between the rentability of exploitation and the interest of money which we assume will be favourable. The extra cost of the work will have to be paid with this difference.

In the second case the problem is more complex. To absorb a higher cost today, with the only counterpart of an anticipation for the future, goes beyond the financial field to enter that of absolute economy.

II-3 — THE THEORY

3-1. — The problem

Approach to a definition

The first contact with the structural problem of heightening a dam (when it is not necessary to repair damage or correct silting conditions), brings us automatically to the subconscious idea of rectifying, overcoming or correcting. From this complex, we get the rough definition of the design objective, "To increase the existing structure in order to get a higher one able to resist the loads to which it shall be submitted (monolithically), the same way it would have done had it been built originally to its final dimension ".

If the loads to which the final structure is to submitted differ only on the scale from hypotheses of the original design, it seems consequent not to change the type of the structure. The problem becomes one of construction difficulties. The target becomes that of getting a bonding at the joint between the existing dam and the superposed section, sufficient to permit the transfer of stresses with continuity present, and avoiding or reducing local stresses at the contact surfaces (thermal effects, different elastic coefficients, etc.), in order to avoid fissuring detrimental to the resistance of load and impermeability.

Basically, were are facing a renewal of construction with all the difficulties inherent in an old cold joint. The solution depends on the position of the joint within the cross section, as local stresses in the joint coming from the physical differences of the joined materials might be additive at the most critical point to the general pattern of stresses.

The first objection to the above definition is to anticipate in some way the solution of the problem, directing it to such a structure that shall behave as having been built originally to its final height. Such a limitation is perfectly unnecessary, as if the function is accomplished, there is no interest in disguising its heightening origin.

The definition is neither valid, as with respect to its ignorance of the change of hypotheses, if compared with the loads corresponding to a new dam at its total height. The topographic conditions of the new section need not correspond to that of the old dam. (We have to suppose that the initial height did not result from abstract speculation and that local conditions decided it). In any case, the load scheme for the whole dam is never spontaneously equal to the one corresponding to a single equivalent dam. If the hypotheses differ, the solutions will hardly coincide.

Revision of load hypotheses

The artificial modification of the hypotheses is possible, in a certain measure, by means of devices available nowadays to technique, such as prestressing methods, but obviously, it cannot prejudice the definition. Therefore, we are bound to review the definition to properly orient our discussion.

Let us proceed to the analysis of the load scheme characteristic of the heightening of a dam. It has not been spontaneous. Not even today is the proper appraisal of the structural problem of the heightening reflected in all reports. Its specific characteristic is based upon the fact that the superposed section of the heightened dam is not built upon a structure on the zero state of stress of external loads. If the dam is kept in service during the heightening operation — and this is the only interesting case — its structure is under load and correspondingly deformed. The load is a function of the reservoir level and is, for this reason, hazardous. The same applies to the condition of deformation (1).

The stress distribution resulting from this load condition might not be the most critical which will normally become from the full reservoir, but at least, is always a matter of inconvenience or of uncertainty.

Besides, during the heightening operations, the added section acts as an extra load. Its behaviour, as such, varies during the construction processes, during which concrete passes from plastic

(1) See the report of the Swiss Federal Experts on the construction in successive stages of the Grande Dixence Dam. [C-34 V° I.C.O.L.D.].

to elastic condition. Its physical, mechanical properties change. This variation is a complicated function of time, construction procedure, and even of the pouring of the new concrete over the existing structure. Shrinkage, besides the thermal and hydrostatic variations and the delayed deformations of the old concrete, merged with the different elastic behaviour between the new and the old concrete, and all added to the difference of age between both, and the immature reaction of the new concrete, affect the stress distribution created by the hydrostatic head.

Analysis of stresses

Therefore, the stressed condition, while the heightening proceeds is the result of a series of causes, all of them of difficult control and intrinsically variable with time and local conditions. It is not convenient to incorporate and lock into the structure this perturbing status. It will happen thus if the bonding were accomplished simultaneously with the heightening.

Resuming, the stress condition within the original dam when being heightened is the result of superposition over the zero distribution of the elastic effect from the following causes :

a) the original hydrostatic load, depending upon the reservoir level,

b) the temporary loading produced by weight of the progressively incorporated section; (it depends on the physical condition of concrete towards setting),

c) the stresses in the joint because of the contact of materials with a different physical-mechanical behaviour.

Bonding, blocking this uncontrolled strained condition and thus adding it to the stress distribution of the final structure under load, could get the working stresses beyond the acceptable limits. Some kind of local, residual stresses coming from the uncontrolled heightening could be traced in the final dam, invalidating the original intention of obtaining a behaviour equivalent to a new structure.

Even in the trivial case in which the reservoir could be emptied during the heightening, so permitting the disregard of hydrostatic load, the assumption of the persistence of the zero stress condition during the heightening is unacceptable. The stress distribution affected by the weight of the heightening and the bonding phenomena might basically change both in its location and magnitude, from that of a dam built monolithically to the ultimate height.

Unfavourable prestressing

It is an actual prestressing that the added section brings as a dowry for its difficult structural mating with the original dam. (Once again " ces problèmes de peau ", will have a role.) The dowry could become a mortgage, jeopardizing the elastic compatibility of the pair.

It is obvious that the quantitative values of the residual stress will differ, depending on the particular circumstances of the problem, and might perfectly become negligible, or even favourable, in effect. Such is the case of a gravity dam in which the weight of the added section increases the vertical stresses in the upstream face beyond those obtained through elastic computation for a monolithic dam (13).

This favourable effect does not occur from secondary stresses in the joint produced by shrinkage, which, if uncontrolled, could add tensile stresses in the upstream face. In the case quoted by Masao Kondo [R-23 — Japan] of the Odomari dam, these tensil stresses reach 8 K/cm2. (See also [C.M. Roberts, R-2, V° I.C.O.L.D.]).

The difference between the elastic modulus of the old concrete and the new, while green, may also produce tensile stresses in the upstream face, because of the unbalanced distribution between the old dam and the added section in resisting external loads. These tensiles stresses are of minor importance, and they have to be considered only to provide those simple measures for their accommodation.

Tentative Definition

The above considerations suggest the following definition that, for the sake of clarity and an orderly exposition, is presented in detail :

" The design of the heightening of a dam must contemplate a structural combination of increased height, taking advantage of the existing dam reinforced as required. The addition of the new section, which might be of a different type, may employ the monolithic merging — or the simple joining — of the two structures, which must collaborate to resist both mechanical and hydrostatic loads, corresponding to the ultimate scheme and residual constraints arising from construction conditions. The simple joining may produce a structure with differentiated units acting separately and even serving separate functions."

3-2 — Resisting functionalism

Once the definition is established, the following alternatives of mechanical action are offered to the designer. The choice among them depends on the particular conditioning of the problem (geologic, construction, economic, etc.). In a particular situation, each of the possible alternatives provide a solution :

Designer's option

(A) — Elusion of the problem of accommodating the residual stresses; the procedure is to permanently maintain the independence of the old and new sections. Both shall work together, without actual integration. (Differentiated structure).

(B) — Action on the causes of the perturbing stresses; altering, when possible, the storage level and planning all block sectional pours in the heightening in order to eliminate, reduce, or even reverse residual stresses. Steps must be taken in order to control secondary stresses in the bond. Localization of the joint must be considered in order to place it in the least conflicting zone, anticipating the failure to eliminate secondary stresses. Resume : Structural integration (Monolithism, simultaneous with heightening).

(C) — Choice of optimum moment for bonding; maintaining of structural independence between old and new sections until causes responsible for the prestressed condition may be disregarded. Choice also of the most favourable level of the hydrostatic head and scheduling in time and location of joint bonding operation (Post-Monolithism).

(D) — Nullification or reduction of the causes of the residual stressed problem; use of the prestressing techniques combined with action on the controllable external loads, to counteract or redistribute residual stresses — or eventually reverse them — to the most appropriate sense and value. Employment of the prestressing techniques to counteract secondary stresses at the joint should not be considered because of its intricacies. This purpose is better suited by schemes (A) or (B).

We are now facing the true picture of the heightening problem. If an aside may be permitted as an instance for a rough but expressive example, we might say that the problem is not only the one of getting more passengers on the already full bus, but in addition, we want the new passengers to reach their destination, despite the fact that the bus is already in motion. The solutions are not numerous. The first, and most drastic, is to provide another vehicle on the same route; (to provide a new structure for the additional function to work in parallel with the existing dam; Scheme (A). The other solution is to push the passengers into the bus, despite inconvenience.

Options included :

a) To reduce the speed or even to stop the bus, provided we have the authority, in order to allow the passengers an easy jump; (Control of water head or counteraction by prestressing to permit the bonding of the new section at a near zero stress condition: Schemes (B) and (D).

b) To run parallel to the bus and wait for the moment when the speed of the bus decreases to that of the intended passengers; (Await a possible decrease of the head, after reaching the reduction of secondary stresses of the joint, permitting the bond in approximately the zero stress condition: Scheme (C).

Choice

The designer's choice of alternative schemes, considering the optimum economy of the structure, depends upon — we repeat — local possibilities; considerations of safety; availability and adaptability of all means.

The first option of our informal example opens a wide panorama of solutions to the structural problem. We are relieved of the necessity of maintaining the original type of structure. It might not be the bus, but the taxi, or the bycicle, (cheap solution, if available, road permitting), the most convenient.

It would be misleading, nevertheless, to extrapolate our example beyond its objective — purely to fix ideas with an easy demonstration — because in fact, the existing structure shall never remain entirely independent and invariable through the heightening process. Heightening implies an increase of function (horizontal components of hydrostatic head) which can be supported by an independent structure, but it implies also an increase of the weight force imposed upon the underlying system (vertical components of hydrostatic pressure acting on the existing storage), which requires reinforcing the existing dam.

The new structure must satisfy two requirements that despite having been reflected on the above definition, should be properly emphasized before proceeding through the morphological consequences of the undisputed solution. It is so, because these two requirements shall have positive influence in the final arrangement of the combined structure. The requirements are :

a) the functional increase;

b) the added load to the existing structure.

Both objectives can be attained with a single structure, or may deserve a distinctly separate structure. If this is the case, a "horizontal" joint must appear. This joint shall separate the already reconditioned old dam from the section superposed for increase in function. A typical example of this distinction between the reinforcing and heightening of structures is the well known heightening of the Aswan dam. The functional increase was satisfied by simple addition of a new gravity section; reinforcing was achieved by building some sort of "buttresses" on the downstream face. The "buttresses" provide the stabilizing weight necessary to counteract the effect of the extra load.

3-3 — Typology

In some instances, the construction of a reinforcing structure can be avoided. It may be unnecessary, either because the existing structure was overdesigned or its heightening was foreseen, or by use of special devices. There is great interest in such cases as we saw in the economical comparison which shows that the cost of the reinforcing structure poses a true disadvantage on the heightening compared to the new construction. The problem of avoiding the reinforcing structure becomes one of balancing the effects of the increased load from the increased head. The stabilizing moment must be obtained by adding weight, or by anchorage to the foundation with stressed cables that incorporate in the structure the weight of a foundation section. In some occasions, it is also possible to act on the external loads, or on the lever arm either of the stabilizing force or of the additional hydrostatic pressure.

The easiest solution arises when it is possible to heighten the structure beyond the necessary rise in water level. By this, the necessary stabilizing weight can be provided. This was the procedure implied in the heightening of the Ennepe dam (3).

The stabilizing action of the additional weight is easily amplified if the new section overhangs the upstream face. This arrangement has been used to advantage in Spain in the Chorro dam, and the Doiras dam. [A. Presmannes, R-137].

The action on the external loads, particularly the uplift, was utilized in Ringedal dam in Norway (3). The possibility of emptying the reservoir permitted the construction of a Levy facing. It permitted the repair and further avoidance of damages occurring in the upstream face caused by acid water attack, plus a reduction in uplift and implied overturning moment.

Gröner (Norway), quotes heightening of dams with an important slope for the upstream face of the added section. In this way the net overturning moment of the hydrostatic load increase is reduced.

In the design of dams with an overflow spillway, the temporary surcharge head produced by the flowing water is considered. The design reserve for this transient load can be employed for a permanent heightening of the storage level by providing mobile gates. S.W. Stewart, (U.S.A.), [R-156], describes numerous cases of heightening, with different purposes, solved by this procedure, including the already quoted example of Bristol dam (12).

Direct heightening by stressed cables

Let us now consider direct heightenings by anchorage to the foundation by use of stressed cables. The procedure first appeared in 1930 in Cheurfas dam. On that occasion, the basic problem was the reinforcement of an underdesigned structure which, as a consequence, resulted in a small heightening. Coyne, who is responsible for the innovation, foresaw, at that time, the extension of his system as a general method for dam heightening (6). Afterwards, there have been many occasions in which Coyne's system has been applied and today it is, most probably, the most standard and popular procedure. Fourteen out of twenty-nine reports received deal with or consider the employment of stressed cables. The number of successful applications is also significant. All of them basically concur with the procedure devised for Cheurfas dam.

Even in the Fifth I.C.O.L.D. Congress, Coyne himself referred to cases of prestressing methods applied to dams as " ... newcomers, the outsiders ... ". We must acknowledge them today with full honours, even if it was only for its importance in heightening problems. Coyne himself stated the sound theoretical legitimacy of his procedure, justifying it as a mere application of the general principle of the static synthesis of structures, as per the definition of Rabut (See French version).

On theoretic grounds, several reports deal with the method, discussing different aspects of the same and its possible extensions. J.M. Linton Bogle, [R-2 Great Britain], analyses the stress distribution created by the stressed cables in horizontal sections depending upon different positions of the cables line of action, all these positions being both in distance and direction near the upstream slope.

A.Z. Bassevitch [R-112, U.S.S.R.] establishes the rigorous equations on elastic theory for the stress distribution due to the force applied to the cable. He extends his study to the case in which the cable, always acting vertically, is displaced parallel to the upstream face. Bassevitch recommends the use of the rigorous elastic methods as the results obtained through them show economic advantage with respect to figures reached through simplified mathematical approaches. The discussion of the same report on the distribution of normal stresses on the foundation resulting from the stressing load is also of interest, as it refers to the increase of such stresses at the heel of the dam, with the accompanying decrease in permeability in the foundation materials, and the improvement of the underseepage condition and the sliding resistance. The aforesaid effects can be amplified by the addition, as per Bassevitch, of a centrally located longitudinal gallery at the base of the dam. This gallery alters the trapezoidal distribution of stresses on the foundation, concentrating them at heel and toe. At the same time, it may usefully serve other purposes, such as a foundation inspection, uplift reduction, drainage and general instrumentation of the dam behaviour.

D. Lazarevic [R-90, Yugoslavia] presents several applications of stressed cables, all of them full of interest for their ingenuity. He deals with hollow gravity dams, and speculates with the use of cables at various locations and directions. He also examines the utilization of prestressed cables for anchoring appurtenant structures in the case of gravity-arch dams. (See also [A. de Montmarin and W. Terminssian, R-67, V° I.C.O.L.D.]).

In gravity-arch dams, cables may be employed to reinforce the vertical cantilevers action, considering them as curved beams fixed at the foundation and supported at the other end by the uppermost arch. It is regrettable that this report is not substantiated by data from field experience.

R. Priscu and M. Constantinescu [R-34, Romania] develop an analytical study of the anchorage effect, based on classical hypotheses about tensile stresses at the upstream face and sliding stability. They extend their study to the general case of a dam with a significant inclination of the upstream and downstream slopes and they state the formula, giving the stress condition at every point of the cross section. They conclude that prestressing is more appropriate when the upstream face is inclined and also for dams of comparatively low height. We must emphasize the interesting discussion of these reporters on the loss of prestressing action, due to plastic flow and creeping of concrete. We shall return, in the following articles, to the technology of the subject, and we shall review it with other reports.

The advantages of the procedure are obvious in connection with stability. It solves not only the static question, but also the addition to compressive stresses on the upstream face restrains fissuring and minimizes the danger of the propagation of uplift pore pressure. The increase of the vertical loads over the foundation implies also improved sliding resistance and reduces underseepage and foundation uplift.

Finally, the creation of artificial compressive stresses on horizontal sections on the portion of the foundation enclosed between the bottom of the cables and the base of the dam, implies an actual mechanical treatment of the material, establishing an elastic condition that may improve its bearing capacity (21).

From the economic point of view of direct heightening by use of prestressed cables is the most advantageous. All the reports coincide in this point. Mateescu (R-32) and Priscu & Constantinescu (R-34), have studied the difference in cost with respect to conventional heightening, and their conclusions have been confirmed by practical experience. Linton Bogle (R-32), Parker (R-44) and Bellier (R-48) also include figures obtained from practical applications. Generally, savings of between 30% and 55% may be estimated. These result confirm the information in our possession from the V° I.C.O.L.D. from which we saw that the cost of a ton of "live weight" resulting from prestressed anchorage is approximately 30 % of the cost of the same weight of concrete. [Montmarin & Terminassian, R-67, V° I.C.O.L.D.].

One disadvantage of the prestressing method is that it does not provide sufficient base at the crest on which to accommodate the superposed structure that is to support the increase of the reservoir. This may be obviated by removing part of the crest to fit the heightening section jointing along the isostatic lines or by arranging this section overhanging the upstream face.

The superposed section may also be affected by the anchorage. Habra dam (R-13), (R-32). It may also be of precast blocks H. Press R-3.

The only serious objection to this system arises from the properties of concrete which vary according to age and which are very much influenced by the permanency of the load. Ageing increases the elastic behaviour under instantaneous loads, but the permanence of the load puts an accent on its plastic properties. This effect added to that of shrinkage is reflected on the total or partial nullification of prestressing. The same kind of phenomena occur in steel, although to a less important degree.

The slow and delayed deformations inherent in the plastic condition of concrete should be taken into account when planning the use of stressed steel cables.

From all this it would follow that prestressing techniques do not suit the dam need for perennity. On the other hand the artificial mechanism and the subsequent "biological" manifestations of prestressing are opposed to the inert and objective nature of the dam. The problem may disappear if the "biology" of concrete in great masses becomes mastered, but at present we cannot as yet depend on prestressing for the same duration we expect from the dam.

The difficulties mentioned do not exist, however, during the phases of construction by stages. Then, durability is not imperative and prestressing can show all its advantages, not only with respect to a reduction of the temporary cross section but also offering the possibility of adjusting the stress distribution in the structure to the most convenient status at the moment of resuming construction. A regulating stress on the cables previously installed for that purpose may abolish the effects of the loading and compensate the bonding stresses.

The specific disadvantages of construction by stages, a) excessive volume of construction with the consequent increment in the initial investment, which may well become prohibitive, and b) difficulties peculiar to load effect when resuming construction, may be easily solved by use of prestressing methods. We wonder why, up to this date, no cases are registered of its utilization for this purpose.

As a last point we cannot help but remember an observation made by Drouhin (8) and emphasized by Coyne (21). Never in the history of humanity have bigger forces been controlled with less mass. Let our mind wander about prestressed cables which may well become the needed reins for control of macrostructures.

Structural Sections

Between the three parts which in the end will make up the final structure (original dam, reinforcing and added sections), there are three boundaries : the horizontal one, basis of the superposed structure, which is compulsory and consubstantial with the heightening. The second boundary separating the main body from the reinforcement may not exist (direct heightening) but it is very important in the mechanical performance of the complete structure. The third is the frontier between the reinforcement and the added section.

The nature of the joints in these boundaries and its influence with respect to monolithism of the complete dam give us the key to the typological analysis of the heightening. As there are three joints each with two possible manners of construction, (there is no point at the moment in differentiating between monolithism simultaneous to the heightening and postmonolithism) the resulting basic types would be eight. The number of possibilities is reduced, however, with some previous considerations related to horizontal joints, which normally coincide in a single one at the separation of the superstructure from the dam and the reinforcement.

The principal joint, that is the one separating the existing dam from its heightening, will normally be monolithic; reasons for it are ; the weight of the superposed body, and the hydraulic requirements which make it difficult to adopt any other arrangement.

A technical solution to the independence of this joint would largely increase the structural possibilities, not only of the heightening but of the basic dam design. Solutions of independent arches (1) would become possible and above all, the analytical problem, especially complex because of the forced monolithic solidarity of the complete structure, would be simplified. Achievement of free sliding at this joint, together with impermeability satisfaction could put a limit to the influence of the vertical cantilevers. They would also provide a solution to many cases of heightening still pending.

The joint between the superposed structure and the reinforcement either does not exist, as in the case when these two bodies are independent, or it will be necessarily monolithic. The reasons for this are the same as in the previous case. Disadvantage for monolithism comes from dimensional discontinuity. The elastic alterations consequent with the latter may be easily discarded since such discontinuity, owing to the different thickness between the reinforcement and the base of the superposed body at the height of the crest, is unimportant as it is precisely at this point where the dam reaches its smallest thickness.

Two of the joints are necessarily monolithic. The typological analysis will therefore be reduced to the morphologic consequences derived from the eventual bonding of the joint between the reinforcement and the original structure.

It should be pointed out here that no differentiation should be made between : a) those cases in which the horizontal joints between the original dam and the added sections become a single one and b) those in which this boundary following an isostatic line is not horizontal. This does not alter our basic classification, since these particular cases may be easily isolated within the typological groups described hereafter.

Subject to the previous reservations, our analysis is reduced to the possibilities of bonding in the joint between the reinforcement

(1) This, howewer, has been solved with good success in some practical applications, which focus the interest on the technical perfectioning of this type of joint. See the C-43 V° I.C.O.L.D. of Prof Peña Bœuf about the dams of Deina (42 m), Benageber (40 m) and Tiemblo. All of them belong to the independent arch type. Also D. Lazarevic R-90 (Yugoslavia) quotes successful achievements of this type of joint in his country, particularly that of the circular dam of Niksic subjet of C-16 at the V° I.C.O.L.D. of A. Bozovic.

and the existing dam, including in these cases those in which the heightening and the reinforcement form a single structural section.

When an independent structure is accepted to meet the increase of function, there should be no objections to a change in its form in order to follow in the best manner the local conditions. Differentiated heightenings may therefore be metamorphic.

The enclosed table should better clarify our track.

		METAMORPHISM	IDIOMORPHISM
STRUCTURAL	DIFERENTIATION	1 MOSSVAN [F. GRONER R-50] ASSOUAN (2th heightening) [H. ZAKY R-20] THE EXEMPLES OF [D. LAZAREVIC R-90]	2 THE ONION DAM [C. SEMENZA R-96] [G. OBERTI R-14] [C. MATEESCU R-32] Kelen /3/ Case of gravity dams
MONOLITHISM	(SYN) MONOLITHISM	3 BURGOMILLODO [E. BECERRIL R-13]	5 ODOMARI [M. KONDO R-23] BALCH [J.B. COOKE et J.E. SCHUMANN R-127] ROSS /15/
MONOLITHISM	(POST) MONOLITHISM	4 LAGES /14/	6 GRANDE DIXENCE /13/ MUNDARING WEIR [C.M. ROBERTS R-12] MULLADOCH [V° C.I.G.B.] ASSOUAN (Ist heightening) [H. ZAKY R-20]

Once the principle of independence of sections is established, there is no need to extend the original dam, which will either become absorbed by the final structure or will remain working in collaboration with the new one in the joint performance of the increased functions. Such is the case in Bever dam where direct heightening could solve the hydraulic function (impermeability), leaving, however, unsolved the stability problem. In order to provide for the latter, for both the original overloaded dam and the superposed section, it was necessary to involve the whole body in an earth fill embankment. This is a clear case of metamorphism with complete dissociation of functions. Here a difference may be observed not only between the sections performing the various functions of resistance but also between these and the structure supplying the hydraulic function.

The classic example of the second heightening of Asswan must also be mentioned to underline its obvious metamorphism as well as the total independence between the added reinforcing section and the one superposed to provide for the increase of function. Total freedom in the reinforcement joint was obtained by means of interposing a stainless wrought iron plate, so permitting the sliding of the "butresses" over the downstream face of the old structure. For sake of routine we keep naming these devices "butresses" in spite of the fact that because its unconstrained possibility of sliding, they do not absorb any longitudinal compression other than the one derived from their own weight.

Nothing objects for extreme cases of resisting performance accomplished by independent sections. Some of these cases are studied by D. Lazarevic (R-90). The most interesting example is the one in which an independent arch is added to an arch-gravity dam. The reinforcement is obtained by means of another arch placed downstream at the height of the old crest and applied to the face, but with complete mechanical independence. Space between the dam and the reinforcing arch may be filled with water. This permits to obtain a horizontal reactive load capable of compensating the hidrostatic overhead from the heightened reservoir level.

We understand that Freyssinet also designed a heightening of this type, substituting, however, the effect of the hydraulic load by jacks. The object in fact is to arrange an independent rib to provide the basis for reactive loads acting on the main structure.

Structural differenciation and idiomorphism

In theory no objections could be raised to maintaining the principle of structural differentiation keeping, however, the original form. This would lead us to final structures that, belonging to the classic types, would have a free movement joint, separating the original body from the added section. From the point of view of elastic behaviour this compound can give a perfect performance, and in some cases with obvious advantage over monolithism. This is the case in the arch type dams, for which Scalabrini (R-38) analytically and Oberti (R-14) experimentally have studied that the normal stresses on the abutments are lower when the ring is formed by two concentric and adjacent arches, than when single arch is used.

C. Mateescu (R-32) and also Semenza (R-86) have reached the same conclusion. The latter has launched the expressive name of " onion type " for this type of structures.

In the case of gravity dams, the heightening over the downstream face by means of an independent slab permits a better distribution of stresses on the foundation, than the one allowed by the monolithic dam (3).

The fact that no realizations of the differentiated and idiomorphic type are recorded, is not due to theoretical nor mechanical reasons. The obvious disadvantages from hydraulic and practical points of view are the ones against this solution. The danger of entrance of water through the joint is always present. Failure of the draining system would cause a very dangerous hydrostatic uplift, which could challenge stability. This also occur if, because construction imperfection, the desired independence was not achieved. A concentration of local stresses could arise by friction.

There is an interesting observation made by Scalabrini on this point, which is also picked up by Mateescu. They point out that the crest zone of the old dam is more influenced by the cantilever action after the heightening than before it, and this results in radial compression which increase the frictions between the adjacent arches.

Monolithism

The bonding of the joints can be done simultaneously to the heightening or deferred, as previously mentioned. (To designate these possibilities we indulge for use of the prefixes "sin" and "post" with the word monolithism).

Generally the metamorphic and monolithic heightenings hold little interest. A change of form occasions sharp dimensional differences, and these may infer elastic disturbances in the points of boundary discontinuity. The problem has further complications when attempting monolithism at the same time as heightening; then the stresses inherent to heightening would have to be dealt with at the worst zone, exactly in that place where the elastic disturbance owing to a change of form, would be enough to justify a critical condition. The smallest section takes worst beating at the point of dimensional discontinuity.

Even though both alternatives may be unsatisfactory, the heightening of a buttress dam by completely filling the space between the buttresses in order to convert it into a gravity dam, is preferable to the contrary case. The formula has been studied (22), in conjunction with monolithism, as an economic solution for construction by stages. There is in Spain the classic example of Burgomillodo dam, in which good results have been obtained. In this case the bonding of the joint took place at the same time as the filling between the butresses.

In Lages dam (Brazil) (14), radial buttresses were built against the gravity curve face. The very well planned precautions taken during construction and above all the scheduling of postmonolithism (four years until final bonding of the joints) have suceeded in spite of the theoretical difficulties explained above.

Sin monolithism

The problem is more important for construction than for design. To determine the slope of the gravity dams heightened by this system, taking into account the effect of the load, does not present serious difficulties and several reports refer to it [Becerril R-13], [M. Kondo R-23]. In the buttressed dam the problem of design is even easier.

Very important works have succeded with this type of heightening; the most popular is the Ross arch dam (15). The heightening of the Balch arch dam (U.S.A.) [J.B. Cooke, J.E. Schumann, R-127] is very interesting for the meticulous technique employed. In gravity dams, the one of Odomari, Japan, gives us the solution to all the problems which can be foreseen today.

We cannot leave this type of heightening without mentioning the O'Shaughnessey dam, especially because of the time in which it was achieved.

Postmonolithism

It comes to the end of our discussion the type of heightening which undoubtedly offers greater possibilities to develop. To keep the joint open until the moment in which the best position of the water head is reached once the secondary stresses at the joint have disappeared seems to be the best solution to avoid residual stresses. This can be obtained without changes in the structure being necessary, and without the necessity of employing artificious procedures which are only fully justified when they were previously foreseen, that is in the case of construction by stages.

Works already accomplished according to this procedure of postmonolithism provide a sufficient basis to derive at definite conclusions. The Grande Dixence dam which we have already mentioned is the outstanding example. The special polilithic procedure followed in its construction was shown to be of more advantage than the superposition of a monolithic slab. It permitted a better coordination of the practical possibilities of concrete pouring, also facilitating the elimination of heat from setting, because of the larger radiating surface of the small blocks.

It is not necessary to resort to polilithic construction in cases of less importance. In the V° Congress we had a report from C.M. Roberts which described in detail the heightening of the dam of Mullardoch (Scotland) similar to that of Mundaring Weir (Australia). In both the monolithic slab was kept almost, freely supported over the downstream face, until reaching the best moment for bonding. In the following chapter we shall discuss the means employed to keep the joint open.

Oberti, and Bassevitch, call especial attention to the possibilities of structural sectioning, not only by simple differentiation of the supporting section, but also extending the procedure to an actually polilithic construction. The object of this would be to create a planned stress distribution, which would afterwards be incorporated and blocked by following the sheme of joint bonding studied to that effect. Bassevitch also includes several examples which we regret are not referred to practical experience data.

The system which may be properly called postmonolithism enjoys all the advantages derived from a stress redistribution affecting the most convenient zones. This can be achieved by simple interaction of the various sections in which the structure may be divided. In many cases the only force, which need to be employed is that resulting from the own weight of each section. The author also discusses ingenious devices to create prestressing in cables, using the weight of the various sections of the dam conveniently arranged to act on its balance.

May not this be the occasion to question the great taboo of monolithism ?

An extreme case is the Meffrouch dam [G. Safont et J. Salva, R-101] now under construction. Complete polylithism becomes stone. masonry. A system of cables and grouting achieves the connection by prestressing. (Geometrical beauty but in pre-fabricated blocks).

Wandering about metamorphism

On the same line of prejudice reappraisal a call for attention cannot be avoided about the perfect functioning of compound structures : the ones resulting from heightening as well as those originally constructed in that manner for other reasons. The experience obtained in this field up to the moment is not sufficient to draw a general conclusion, However, the cases registered appeal to the imagination, because of the possibilities which could be derived if foresaking the prejudice of systematically applying pure or at least classic types in dam design.

H. Press mentioned at the V° Congress the Oker dam which probably constitutes an extreme case. In order to comply with the geometrical conditions of the valley the most economic solution was that of adding a gravity section to a arch-gravity dam. This type should be called arch-gravity + gravity. There is no reason to preclude this solution in many heightening problems. It is perfectly adequate for the heightening of spillways especially when anchoring the superposed section by means of prestressed cables.

If we proceed beyond the subject of heightening we shall find that many arch dams have required a gravity section to adapt them to geological conditions. This was the case in Tignes and Mareges. The famous Italian " pulvinos " may be well qualified as compound structures. Lately, the dam of Peixoto, Brazil (arch between two large gravity section with crest spillway) brings another example of a change in the type of the structure to suit site conditions.

When heightening a dam, but also when building a new one, it has to be faced the problem of adapting it to the shape and geological nature of the gorge. The strict geometry of a dam of a pure structural type seldom matches completely the hazardous geology. There are, of course, more versatile types of dams, but the most (drastic) solution would undoubtedly be the development of a technique combining various types of structure.

Follow nature and eventually remodel nature to meet our need must always direct design.

In Mont-Larron dam [M. Terrassa, R-49] a heigthening was performed transforming the original arch dam into a multi-arch type profiting of two existing rock-ridges for foundation of the intermediate butresses.

Experiences are already on record of almost all possible combinations with existing types. Necessity will, in the last instance, pave the way towards its complete development.

II-4 — TECHNOLOGY

4-1 — The joint

The phenomena to be observed in the joint, stress-and strain problems have two causes :

a) The elastic behaviour of the structure.

b) The physical-mechanical properties typical of the new concrete while soft, and in any case, the differences of behaviour with respect to old concrete.

When a monolithic operation of the structure is desired correcting of deformations caused by elastic behaviour, and consequently the corresponding stresses, is a problem of design. The solution consists in placing the joint in the least harming position. Becerril (R-13) has studied this in detail for gravity dams. The final objective is to obtain that no stresses occur inherent to the elastic behaviour of the monolithic structure, which may jeopardise the constructive perfection of the desired bonding.

Naturally, we are referring to the deformation created under the final load scheme. However, there is another status of elastic deformation. This is the one corresponding to the hydrostatic load if the heightening is carried out while the dam is in service. To compensate these deformations is more complex. It may be attempted, however, and so does M. Kondo (R-23). He studies the increase of downstream slope in a gravity dam able to compensate the reduction of compressive stresses in the upstream face of the dam heightened under load. The best and most practical solution is, postmonolithism, taking advantage of the optimum load status.

If a totally independent operation of the various structural parts of the dam is intended, the construction should be directed to avoid the appearance of unforeseen stresses, basically by friction, which may alter the hypothesis of free sliding. This was the solution given to the second heightening of Asswan dam (R-5). 7 mm stainless plates were used there, which made this a rather expensive solution.

It is possible to achieve a practical structural independence by an adequate designed location of the joint in the zone of minimum shearing stress and by means of a careful treatment of the contact surfaces to eliminate any roughness. This is the believe of Semenza (onion dam), Scalabrini and Mateescu, and although none of them have described any cases in which this method has been applied, all of them coincide in its advantages.

The deformations caused by the load status in the original dam bear some influence even in the case of structural independence. But we do not believe that specifically in the arch dam, even if the difficulties mentioned above were solved, a direct contact could be obtained with the dam in service while heightening unless employing a special " clavage " system to be tightened with the reservoir empty.

With respect to the phenomena of group *b*, the problem is different and so will be the provisions depending on whether monolithism simultaneous to construction is desired, or the design corresponds to postmonolithism.

Cooling

If the bonding is done while construction is in progress precautions should be taken to minimize the phenomena of concrete setting especially that of shrinkage. The most effective method is cooling. This was used for the first time in the O'Shaugnessey dam (11). The water of the reservoir was first tried. This proved to be insufficient and a complete refrigerating plant had to be installed in order to obtain the desired reduction of temperature in the new concrete.

This method of artificial and mechanical refrigeration of concrete was also followed in the Ross dam (15). The same system of horizontal tubing embedded in concrete mass was applied, but using a brine of calcium chloride as refrigerating agent, instead of water.

M. Kondo (R-23) in the Odomari dam, after a careful study decided to use the water of the reservoir, but drawn from an appropriate depth to obtain the desired temperature.

Generally, refrigeration should be applied both to aggregate and water as well as to the poured concrete and should be maintained as long as necessary in order to eliminate shrinkage.

Refrigeration may be completed with other measures such as the use of " fly ash " and puzzolan. It is also interesting to control the quality of cement and especially the use of concrete with a rigorous granulometry.

Binders

In the Balch dam, California [J.-B. Cooke, J.E. Schumann. R-28] " prepakt " concrete is being used (construction seems to be still under way). The advantage of its use has two aspects, both contributing towards a better behaviour in shrinkage. The first one better compacting up to contact of the aggregate. The second less increase in the temperature due to concrete setting because of the influence of puzzolan added in the mixture.

In spite of the advantages of using " prepakt ", in Balch dam it has been found necessary to use ice for cooling the ingredients of concrete, and most especially to maintain a strict control of the inner temperature of old and new concrete. This was done by embedding thermo-couples in the mass to regulate the temperature in the new concrete pouring.

The difference of mechanical behaviour of concrete, caused by its age, is of no importance in postmonolithism, but if bonding is carried out simultaneously with construction, the hydrostatic load must be controlled. There is no problem if the reservoir is emptied, but if it is kept in service, load increase up to its final elevation should be avoided until the green concrete has reached its elastic maturity to functionally cooperate with the original dam.

The above refers to an action over the causes of the phenomena in the joint applicable both to group *a* and group *b* but the effects of provisions is by no means complete. In order to obtain the final monolithism of the structure, precautions should be taken also in the design of the joint and treatment of its surfaces to obtain the best bracing.

Keys

In all cases of monolithism, either simultaneous or not, the concrete should be vigorously worked until an extremely rough surface is obtained. Any keying or toothing device will greatly improve the jointure. There are many types of keys which may be used to this effect. A classic type is the "waffle type" used in the Ross dam. Scalabrini comments on this solution as well as on those of the Limberg (Austria) and Mauvoisin (Switzerland) dam, in which the squared throughs were substituted with traigh shapes but bigger, longer and horizontally placed. The conclusion reached by Scalabrini is that such devices are not necessary to insure the monolithism of the arch dam. These may be avoided and so reduce the extra cost of forms by means of a careful grouting of the joint. All this if possible to design the location of the joint in a zone of minimum resulting deformations.

Semenza proposes to achieve solidarity by means of ribs on the face placed in accordance with the construction joints. This solution when extrapolated if hightening is presumed, advises construction of inter-joint blocks with total thickness. It is the case of the central section of the O'Shaugnessey dam. This system saves cost by the construction of blocks with a T shape horizontal section, but in this case Semenza advises the use of a light steel reinforcement to counteract the stresses caused by dimensional discontinuity, and also the different shrinkage behaviour of the two sections with different irradiating ratio (volume exposed surface).

In Grande Dixence dam, the vertical and longitudinal joints of the blocks in a polilithic construction were placed in a saw-tooth form with faces approximately normal to the direction of the isostatic lines.

Summing up and as general rule, it would seem that keying is more effective in gravity dams than in the longitudinal joint of arch dams.

Grouting

The technique of joint grouting will not be discussed here. We shall mention, however, the report R-127 by J.B. Cooke and J.E. Schumann. It describes the way in which this has been done in the Balch dam and the series of precautions taken.

It is interesting, however, to point out that in order to maintain the surfaces of the joint in place during injection. it would be advisable to use a light reinforcement to absorb the normal stresses created by the grout pressure, and use them to insure a resistance to sliding [Scalabrini, R-38].

Interlocks

Metallic interlocks in the joint is the common solution. They were used in the first heightening of Asswan in which the injection was of neat cement. To avoid fissuring in the internal surface of the added section, which is more difficult to be cooled, it would be advisable to place a light mesh reinforcement with the object of distributing and micrifying shrinkage fissures.

When monolithism is deferred, or in cases where structural independence is desired, a difficulty appears when trying to maintain the separation between the original dam and the added section. This may be obviated by the arrangement of a slot in the joint and supporting the new section through a rib system. When forms are removed slots may be filled with dry aggregate, giving a better support to the added section. Afterwards it is all injected while the bonding is done. The use of " prepact " concrete seems obvious.

The metallic interlocks may also play a part in the support of the superimposed section by acting as pitmans in the relative deformations with respect to the original dam.

Drainage

More dangerous than the appearance of secondary stresses in the joint, is the eventual hydrostatic load resulting from fissuring which might connect it with the reservoir. This is the great disadvantage of the longitudinal joints, especially in thin structures (arch dams). Any carelessness in construction may be the cause of it; To avoid such occurrence an efficient drainage of the joint should be provided, in order to relieve any accidental hydrostatic load. In arch dams the drainage is directed towards the downstream face (Balch dam), while in the gravity dams collecting passages should be included within the dam section.

In the horizontal joint supporting the heightening it is normal to arrange a grooving Balch, Chicamba, Lake Spaulding.

4-2 — FOUNDATION

Not much can be said about the foundation problem. Dynamite excavation in the vicinity of any structure is always unadvisable. The conclusion is that should be avoided except in very clear cases in which the firmness of the ground will permit a good support of the reinforcing section. It is partly for this reason that direct heightenings by use of prestressed cables is so popular. In vault dams the subject is as delicate as in gravity dams.

Apart from safety reasons, always relative, (in fact it is difficult to jeopardize the stability of the work whilst its quality might be seriously affected), it is advisable to avoid enlarging the foundations in view of the increase in cost involved. In case of construction by stages all reports coincide in the advisability of building, during the first stage, the foundation for the total structure, or at least its excavation. It is precisely from this that a higher cost of the first stage is derived. Could not this higher cost be compensated reducing the temporary sections by using prestressed cables ? We know of no cases in which this has been done. However, the advantage is not only of an economical nature, but is also reflected in the mechanical operation of the structure, since with already properly installed cables it would be possible, regulating their tension, to compensate the effect of the load during the heightening, bringing now the structure back to a zero stress distribution.

The possibilities of prestressed concrete in foundation problems should not be omitted. The Beni-Bahdel dam is a characteristic example.

Heightening may, in many cases imply the use for support of a geologic area different to that of the original dam, and also to modify the direction and magnitude of the resultant up to become unbearable by the soil foundation. In Beni-Bahdel dam, this resultant was modified by means of prestressed struts. This system may undoubtedly be extended.

To put an end to the foundation problems, we bring up the observation made by Semenza about the case of construction by stages in which the foundation is entirely built during the first stage. In these structures the use of a light reinforcement may be necessary.

4-3 Spillway

It is interesting to note that there is no mention in the reports received, of what seems to be one of the major technical difficulties of heightening, namely the construction of the spillway if the dam is to be kept in service.

All the problems of a heightening which we have discussed are based on the hydrostatic load acting as a parameter; the existence of a crest spillway implies the possibility that this parameter becomes a hazardous and accidental variable. This will influence the construction technique of heightening, since it will first ask for a seasonal planning, and a fast construction, safe from overflow emergencies.

The mentioned solution, as used by H. Press on the basis of prefabricated blocks anchored over the crest spillway with prestressed cables, though fast and efficient, is only applicable when the increase in height does not affect much the impact of the nappe over the waste channel.

Dexter mentions in his report the necessity to modify all the auxiliary works, by reason of the influence which the increase in height may bear on their dimensions and operation. This is particularly important with respect to the devices for dissipation of energy at the foot of the spillway; the funtioning of these is predetermined by the velocity of the water, obviously altered by a modification in the height.

We cannot consider the subject of the spillway in connection with the heightening, without facing the problem — always present — of the very existence of the spillway. The most common death of a reservoir is caused by silting. Heightening may restore its utility to a dam; heightening, in an ideal extrapolation, might also avoid the necessity of a crest spillway. Would not both reasons lead us to the solution of a low gates spillway which may alleviate silting without the necessity to deal with great overflows, which might well be absorbed in a properly capable reservoir ? I do not intend to answer this question; only to point out once again this unsolved problem of the theory of dams. Great spillways mean great losses of water. How long can we afford this ?

4-4 — Artificial prestressing

We have already commented on the Bassevitch report broadening the subject, already outlined by the same writer during the 4th Congress, about the utilization of means other than jacks for prestressing cables. Bassevitch describes several methods all directed towards the same objective. He explains with more detail the injection between the main and the cable anchorage sections and the use between the main and the cable anchorage sections and the use of the dam own weight by ingeniously arranged sections that tilt under control.

The method most widely used up to this date is hidraulics jacks Freyssinet type.

It does not deserve to go further on prestressed cable technology, of which there is very complete information in various reports. Let us mention, as an example, P.I. Parker (R-44) for the detailed description of various cases and the compared economical study of the different aspects.

The cables are normally composed of 1/5" wires, in a number depending on the load reached in prestressing, which may vary from 70 mT as in Tansa dam (India), up to 1,400 mT in the Howden dam (England). Parker sees no difficulties in obtaining loads of 3.000 or 4.000 mT without variation of the present anchoring and stressing procedures.

The diameter of the cable holes varies from 2 1/2" for cables of 70 mT to 14" for cables of 1,400 mT. Special attention should be paid to the construction of these holes, since it represents one of the most important items of cost. Percussion may be used for diameters up to 4" and depths of 60 m; larger dimensions require rotary equipment. 4" diameter is for 200 mT cables, which offer the best economic advantage of this method.

The stressing heads are normally built of concrete; the wires of the cable supply the reinforcement. Also in some cases special heads are used (Tansa). Load distributing slabs should be arranged between the heads and the dam. A sag tension normally between 15 and 20 % should be taken into consideration when calculating the prestressing.

The grouting of the holes is most important since it should secure the permanent protection of the steel in order to avoid any corrosion. The modern use of cables composed of parallel wires permits the inclusion of mortar between wires. This operation must be watched very carefully, since also a second operation will depend on it, namely an improvement in the quality of the surrounding concrete in old and bad quality dams.

II-5 — EARTH-FILL DAMS

We have seen that the main question in the heightening of concrete dams is based on the following points : a) the joint between the original and the new sections, b) the elastic problem derived from the fact that the building of the added section is effected while the original one is deformed. Consequently the appraisal of the heightening problem referring to earth-fill dams should be considered with respect to the same questions.

When the materials are loose there is no difficulty in the joint. If the balance is merely based on the interparticle friction, there is no reason why the solution of continuity marked by joint could be detected (it is easy to take the necessary precautions to insure the homogeneity of the adjoining materials so the " statistical " contact plane may disappear). The same is applicable when the materials are very cohesive and their totally plastic behaviour places them in a condition similar to that of non-cohesive materials with regards to deformations. This is the case of clays in which the deformations due to loads are not reversible.

Since the problem, in both points *a* and *b* is basically a matter of behaviour in deformations, the difficulties will increase progressively as the materials to be used move apart from the extreme mentioned conditions, which, in the last instance, coincide in the point of their no reversibility. Further, and when comparing the difficulties with those encountered in bonding between concretes, many other variables should be considered, or at least the difference in intensity by which the physical-mechanical reaction of the materials is affected. Such are compacting, age, weathering, hygroscopic status, etc., which are conditions of the material very difficult to reproduce or, at least, of no easy control when artificially duplicated.

Since the two extremes which limit, to physical-mechanical effects, the whole range of soil materials are suitable for bonding, there is no apparent reason why the same should not be the case for medium (lean plastic) soils, least of all if the low tixotropic sensivity of the materials is taken into account. The difficulty is not theoretical but practical.

The conclusion reached is that in earth dams the problem of the joint is more difficult that in hard dams, if physical-mechanical homogeneity and continuity are desired.

The material normally used in the nucleus is pure clay. The heightening should be effected with the same material for sake of the function of impermeability which it should supply. Plasticity will insure the mechanical behaviour of the joint whilst impermeability can be guaranteed by the condition that the width should be at least twice the maximum water nead. [C.R. Scott R-92].

In the main section of the dam the most frequent solution is to avoid the problem of obtaining a homogeneity between the existing material and the one incorporated, taking advantage of the stability through deformations of clearly granular and non-cohesive materials. This type of material is the one used in heightenings, thus avoiding any danger of bearing loss because of an eventual deformation of the supporting section. In the four cases presented, Schwammenauel dam (Germany, F.R.) [H. Press R-3], Torre del Aguila (Spain) [C. Conradi R-47]), and Alama Gordo and Pine View (U.S.A.) [C.J. Hoffman R-91] clean granular materials were used in the heightening.

In general it would be senseless to consider the state of déformation in earth dams. To the extent that cohesion is not taken into consideration, the earth dams do not offer a plastic behaviour: besides the deformations are neither reversible and for this reason the problem of eventual blocking of stresses in the heightening does not arise. All this, of course, without any positivness, since the concepts of elasticity and plasticity are only very relative ones in soils in which the mechanical behaviour corresponds to an intermediate stage between these two conditions, and which is further influenced by other phenomena of more difficult theoretical control, such as pore pressure and, in the whole, characteristic tixotropy. Consequently, the difference we have outlined only exists as sharply as we have established, in the extreme cases in which there is absolutely no cohesion or in the presence of a totally plastic condition.

For the above reasons the stability of the whole structure should be checked in the same way as for a zoned dam, since the of the heightening fill always constitutes a zone, even when the existing dam is a homogeneous one. There is no point in abandoning our line with the considerations which would justify that this checking on stability should continue to be effected by means of the trial line method in successive readjustments. Obviously checking should be also done in the joint plane between the old filling and the addition, which, in some cases, may be a specially weak surface; we have already stated that a special element to insure the necessary friction will be necessary.

The main disadvantage of this method springs from its cost. Homogeneous dams are built in that way because granular materials are scarce in site or very expensive to exploit. A heightening effected in accordance with the outlined scheme will therefore be expensive in those cases. As an example we see that the cost of the Torre del Aguila heightening was 49 %, higher than the estimate in a previous design, in which it was proposed to use the same materials which formed the body of the dam. The decision to follow the new design was taken in view of the difficulty to obtain a bonding with the old soils. These appeared very cemented, which is obviously due to their high calcium carbonate content (60 %).

The report on the Montgomery dam presented by G.W. Scott (U.S.A.), deals with a type of dam built with rock fill and with waterproof screen made of bituminous concrete. This type of dam was selected in view of the greater heightening facility. There is no doubt that a perfect bonding shall be achieved as much at the rock fill joint as at the bituminous concrete screen.

Mingesetchaoursck dam studied by L. Bossovsky (U.S.S.R.) is a special case, since the increase in height was decided during construction, and therefore the problems of bonding we have referred to were not present. This is a very interesting case constituting a reversal to the hydraulic fill method out of use during the last years, perhaps not so much on account of the accidents it caused at a time, as by the rapid development of earth moving and consolidating equipment. It might be possible that if the efforts devoted during the last years by the Soil Mechanics to the study of mechanical compacting should have been directed toward hydraulic-fill, this would now be in a more advantageous position. The careful control during construction, allowed in the case studied by L. Bossovsky, to check step by step the safety factor of the structure. This was found to be higher than what had been estimated, a fact which decided the increasing of the height of the dam. From the data presented we may deduce that the tixotropic sensitivity was high, at least before the final consolidation; this could be one of the objections against hydraulic fill, and also and very especially the possibility of a sediment of the sand below the critical density, which would result in a work very sensitive to dynamic effects. This question has deserved careful attention in the dam of Mingesetchaoursk, located in a seismic region, and by use of little known techniques it was possible to determine the sensitivity to dynamic action. It is possible that amongst the developments which modern Soil Mechanics may contribute to the use of hydraulic fill, the means may be found to use deep vibrators during construction to improve the density and stability of sandy soild.

II-6 — SUMMARY AND CONCLUSIONS

More water

1) Dam construction keeps going at an ever increasing rate the world over. Nothing permits to think the peak has been attained. It is presumable, in high developed countries, that within a short delay some of the objectives leading dam construction can be achieved. Such is the case for flood control, navigation, riverbed, relocation or even irrigation that shall reach its limit because of lack of land worth of reclamation.

Powerwise the matter is presently subject to full discussion in face of the nuclear energy possibilities. It seems, however, that opinions agree that nuclear energy development will depend upon water resources. Nuclear plants require tremendous supplies of water. Besides and in spite of the reported improvements of the nuclear plants towards a better adaptability to satisfy the load factor, it is generally accepted that hydropower is bound to provide for the demand peaks.

Water supply both for human and industrial consumption has not a foreseeable limit. Construction of reservoirs has to be continued and even intensified in view of the rapid demographic and industrial increase. It is estimated that the world population will be doubled within the next forty years.

Limited sites

2) In view of these pressing demands an integral utilization of the available water should be undertaken. As the capacity of sites with topographic and geologic conditions suitable for new reservoirs doesn't meet the storage requirements, it becomes imperative :

a) To undertake a revision of all existing dams up to the total utilization of the site (heightening of dams).

b) To plan construction of new dams for the exhaustive utilization of natural resources (construction by stages).

Improvement of natural conditions

3) When creating a reservoir man limits his action as to plug the basin created by nature. The diminishing availability of almost complete natural reservoirs leads to an increasing man-made complement of nature. Impermeability and bearing requirements implie the correction of soils for abutments and foundations. Scarcity also compels construction on seismic zones and carstic soils which years ago whould have been discarded.

All this must be considered for an exhaustive utilization. This also applies when planning the heightening of existing dams (in many dams the height was limited by geological condition which would not support a larger structure or would not permit the utilization of the reservoir from certain elevation up).

Hidraulic potential

4) The value of water increases with altitude. It improves the control of the field of necessity. The ideal solution would be to count with reservoirs capable to regulate all hydraulic resources in conjunction with demand, at the same altitude of the sources. This is in fact the maximum potential of utilization.

It would be interesting to have statistical data about the rate of hydraulic development towards that goal. A reservoir reduces the hydraulic entropy. It would be easy to establish a rule to determine the maximum entropy. It could refer to total volumes or to not regulated flows. The latter would impart the factor a higher practical quality.

Here we refer to regulating reservoirs and not to those which merely act providing head for power; in these the water is a structural element creating a discontinuity in the water level grade without alteration of the hydraulic potential.

Heightening's efficiency

5) The capacity of the reservoir increases as a potential function with an exponent higher than one with respect to its height. There are many factors, however, intervening against this advantage in volume. Such are :

a) Evaporation, a function of the surface, b) Permeability of the soil influenced by the water load, c) Silting and d) Efficiency of the regulation that is occuppancy factor of the reservoir, which will decrease as the storage volume increases.

6) Integral utilization, most imperative to mankind, is not, therefore, a mere structural problem. Hydrology, geology, climatology and finally a thorough large scope economic planning should all be considered when establishing a policy. Coordination of the various demands of water will be the decisive factor. Everything leads to believe that the freedom of action of the user and the designer of dams will be restricted and subject to the requirements of common welfare.

Elastic emergy storage

7) The problem of a dam cannot be considered as an abstract proposition. A maximum utilization of the natural resources should be aimed at — we repeat — regarding the dam as a complement to nature in the creation of the reservoir. When heightening, furtherwise, the remaining resources of nature should also be utilized, as well as the unexploited possibilities of the original structure.

A margin of stability can frequently be found in existing dams. It is certain in an absolute way and very often true to practical effects, that an available elastic resistance margin is present in the existing structure. A concentrated localization of the maximum stresses and the small working stresses to which the material is subject in the classic types of dams, allow at least in certain zones for further loading without going beyond the acceptable working stresses. In this aspect the structure of the existing dam acts as an storage of elastic energy which must be used to increase the bearing function.

Structural comparison of the heightening

8) The structural problems of heightening offer some advantages when compared to those of a new dam. The existing dam always implies a basis of geometrical regularization of the natural contour of the gorge. At least the homogeinity of its materials is another favourable condition for a better distribution of loads over the foundations. The basic difficulties are to be found in the following : a) Deformation status of the structure over which construction is to be done. b) The joint and especially the bonding. c) Enlargement of the foundation. d) Suitable adaptation of appurtenant works (spillway, stilling devices, valves, drainage, etc.).

Heightening technique

9) Using present techniques, with their possibilities to control the temperature of setting, the shrinkage of concrete, grouting and " prepackt " concrete, the bonding of the joint can be safely undertaken. This can be further improved by such precautions as the use of keys and by designing the joint following isostatics lines. However, postmonolithic construction appears as the

advisable solution, since apart from eliminating the problems of control in the joint present in other cases, it solves the elastic problem of blocking of stresses that occurs if construction and bonding are done simultaneously on a deformed structure.

Stressed cables

10) The use of prestressed cables permits the utilization in the heightening of the resources of stability and elastic energy remaining in the primitive structure and also in the foundation. By this means new structures may be added to the original dam, such as spillway piers and even the heightening body itself. Using prestressed anchoring that incorporates the weight of the whole affected section to the structure, the ground can participate in the structural balance as actual weight.

There are also many ingenious devices which permit an improvement in the ground bearing conditions and those of the structural loads to which it is subject.

Objection to stressed cables

11) However, the use of prestressed cables, although solving many acute problems, would seem an emergency solution rather than a permanent one. In a dam, durability should mean perpetuity.

Dam construction has been bypassing conventional reinforced concrete. In general a certain dislike has been manifest to the inclusion of steel. The dam which complements and is incorporated to nature seems to require a rocky and inert existence. We cannot claim to be categorical in these affirmations, but obviously it is still a long way to a complete knowledge of the "biological" behaviour of great masses of concrete. Also, it would involve some risk to depend on metallic materials and mechanisms exposed in places of difficult access, to natural destructive agents.

Eventual prestressing

12) Creeping of the prestressed cables occurs as a consequence of the yielding of steel, but to a higher degree, as a consequence of the plastic behaviour of concrete subject to permanent loads. However, the elastic conditions of concrete under instantaneous loads improve with age. From all this it would seem advantageous the use of prestressing devices in a general manner, or limited to some zones of the profile, so that they would only act under those load conditions statistically less frequent and shorter.

It is easy to imagine systems which would stress the cables at critical moments such as those corresponding to the maximum load, overloads caused by flowing overhead or even at the emptying of the reservoir. A mechanism such as hydraulic accumulators could be used to bring about the prestressing of cables at the required moment. This would correspond to an "opening of the gates" of elastic energy stored in the dam for use in emergency cases. The incidental overstressing conditioned to the appearance of ephemeral extra loads, suits the specific characteristics of the materials. The objections to mechanicism for sake of everlasting dependability persist and need to be emphasized in this speculative survey of solutions.

13) In spite of the previous objections, any system is valid in heightenings. The problem is preconditioned by the existing structure. The solution must necessarily be casuistic, and all procedures are good if the function is achieved.

Construction by stages

14) In construction by stages we enjoy a larger degree of freedom and the field is wider. Without restriction to ingenuity it should be endeavoured to restrain design to a formalistic theory.

It seems that prestressing is able to become an important aid towards reduction of construction costs during the provisional stages, providing reduced profiles to compensate the excess in foundation.

Monolithism should be accepted with all kinds of reservations. The suitable placing of the joint in zones of minimum shearing effective stress obviates or minimizes the bonding problem. If impermeability of the horizontal joints could be easily attained the number of satisfying types of structures composed by independent sections would increase.

The advantage of postmonolithic construction, with complete development of the possibilities to use interaction between the various structural bodies, in order to create a favourable stress distribution both in the foundation and in the structure, results obvious.

The reduction in the size of the blocks towards an actual polilithism would offer great advantages with regards to the degree of general isostasy of the structure, which has to increase to meet the requirements of adaptability to unreliable or heterogeneous foundations.

Concrete for hydraulic use should be prestressed. It appears that a technique for the creation of these prestressing with or without the use of cables, will develop within the foresseable future. This technique might be extended for application to foundations to improve its hydraulic and bearing conditions.

The title of the question under discussion is but a version, or at least a technologically limited fraction of a wider one on which it will be imperative to focus attention. This is the integral utilization of the water resources and the means to achieve it, that is, large dams.

In order to determine the objectives, it will become enforced to obtain statistical data in order to establish a correlation between human needs, both demographic and industrial, and the availability of water and suitable sites for dam construction. In the absence of accurate figures we can still anticipate the conclusion that fatally imposes :

a) A coordination policy for the use of water. This is easily attainable since only simple geometrical conditions (head discharge) are necessary to satisfy the various demanding calls. The theory of the reservoir and that of its multiple and integral utilization should be developed. Many points have already been studied and a simple compilation would prove useful and effective.

b) A policy of systematic prospection of sites and exhaustive utilization of the vailable ones.

The industrial civilization to which we belong, humanity itself were born at the riverside. Civilization has been transmitted through the seas, and it may be possible that in the future even man's subsistence will come from sea. However, man's " habitat " is the continent and on it civilization has progressed and will continue to progress dependent on the availability of water. (It is curious to observe through History the — fluvial, maritime or continental — reaction of various civilizations, in connection with their challenge to expansion).

Dams to control the water required by mankind will increase in size every day and will become actual macrostructures. In less than forty years the maximum height reached by a dam, which was 100 meters in 1920, has risen to nearly 300 meters. Technology has gone beyond acceptable extrapolations from the starting point. The same has occurred to modulus of all kinds. A general revision appears imperative especially of the theory on which the strategic approach to the new problems should be based.

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II-6 — RESUME ET CONCLUSIONS

L'eau

1. — Le rythme de construction des barrages va croissant dans le monde entier. Il est possible que, dans des pays très développés, certains des objectifs qu'elle poursuit se voient bientôt satisfaits. Il peut en être ainsi pour le contrôle des crues, la correction physiographique des lits des cours d'eau, la navigation fluviale et même l'irrigation faute de terrains cultivables.

Pour ce qui est de l'énergie, le thème est en discussion en face de l'énergie nucléaire. Il semble que les conclusions concordent sur le fait que le développement de cette dernière dépendra en grande partie des disponibilités hydrauliques. Les installations nucléaires exigent des débits énormes d'eau traitée. De plus, malgré les grands progrès que l'on enregistre dans l'amélioration du coefficient de variation de charge des installations nucléaires, il semble qu'il faille tendre vers l'utilisation de l'énergie hydro-électrique pour satisfaire les pointes de consommations.

L'approvisionnement en eau, tant à l'usage de l'homme qu'à celui des industries n'a pas de limitation en vue. L'aménagement de retenues doit se poursuivre et s'intensifier en fonction de la croissance démographique et industrielle accélérée de l'humanité : on s'attend à ce que la population du monde double dans les 40 prochaines années.

Nombre limité des emplacements

2. — Devant de telles exigences, il faut arriver à l'utilisation intégrale de l'eau disponible. Comme les ressources topographiques et géologiques qui permettent la construction de réservoirs, sont plus faibles que les exigences de l'emmagasinage, il s'impose :

a) de réformer toutes les retenues existantes jusqu'à leur utilisation intégrale (surélévation de barrages) ;

b) de planifier la construction des nouveaux barrages en vue de tirer parti de manière exhaustive des possibilités naturelles (construction par étapes).

Amélioration des conditions naturelles

3. — Pour créer une retenue, l'homme se limite à construire un barrage qui serve de bouchon au vase créé par la nature elle-même. A la raréfaction des sites utilisables comme retenue, correspond une intervention accrue de l'homme pour suppléer la nature. Les exigences de l'imperméabilité et de la résistance conduisent à la correction des sols et au traitement mécanique des appuis et des fondations. De même la rareté des sites oblige à construire dans des zones sismiques ou dans des terrains karstiques que l'on aurait antérieurement écartés.

Tout cela doit être considéré dans l'utilisation intégrale et compte déjà dans l'étude des surélévations des retenues existantes. Nombreux sont en effet les barrages dont la hauteur a été limitée pour des raisons géologiques qui s'opposaient à la construction d'un ouvrage plus grand ou à l'utilisation du réservoir au-dessus d'une certaine cote.

Potentiel hydraulique

4. — La valeur de l'eau croît avec l'altitude. La hauteur permet de dominer le champ des besoins. L'idéal serait de disposer des réservoirs capables de régulariser toutes les ressources hydrauliques suivant les exigences de la demande, à l'altitude même des sources. C'est en définitive, le potentiel maximum d'utilisation.

Il serait intéressant de disposer de données statistiques sur le rythme du développement hydraulique avec cet objectif en vue. Une retenue diminue l'entropie hydraulique. Il sera facile d'établir un critère pour déterminer l'entropie maxima. On pourrait se référer soit à des volumes totaux, soit à des débits non régularisés, cette dernière éventualité donnant une meilleure qualité pratique au coefficient.

Nous voulons parler des réservoirs régulateurs et non de réservoirs qui interposés sur un cours d'eau, n'ont d'autre objet que de créer une chute en vue de recueillir de l'énergie. L'eau joue ici comme élément structural en créant une discontinuité dans la ligne de pente sans altération du potentiel hydraulique.

Rendement de l'accroissement des retenues

5. — La capacité d'une retenue augmente plus que linéairement en fonction de sa hauteur, mais plusieurs facteurs interviennent à l'encontre de cet accroissement de rendement volumétrique. Ce sont :

a) l'évaporation proportionnelle à la surface;

b) la perméabilité du terrain, fonction de la charge d'eau;

c) l'envasement et
d) le rendement de la régularisation, c'est-à-dire le pourcentage d'occupation du réservoir créé, qui diminue à mesure que volume augmente.

Vision d'ensemble

6. — On ne peut réduire par conséquent l'utilisation intégrale, qui est impérative pour l'humanité à un simple problème de constructeurs. L'hydrologie, la géologie, la physiographie, la climatologie et, en dernière instance, une planification économique d'ordre supérieur, doivent intervenir dans la fixation d'une politique. C'est la coordination des diverses demandes en ressources hydrauliques qui décidera; mais tout laisse à penser que la liberté d'action de l'usager et du projeteur des barrages, dont le champ d'action est malgré tout plus réduit, se trouvera restreinte par l'obligation de s'assujettir aux exigences du bien commun.

7. — Le problème du barrage ne peut pas se poser dans l'abstrait. Il faut arriver à s'ajuster au maximum aux ressources naturelles, le barrage apparaissant comme un complément à la nature pour créer le réservoir. Dans une surélévation, il faut tâcher de tirer parti, non seulement des possibilités naturelles rémanentes, mais aussi des ressources inexploitées de la structure antérieure.

Mise en valeur des réserves structurales

Il est fréquent de trouver, en des barrages déjà construits, une marge de stabilité. On est absolument assuré et souvent pratiquement certain que la structure existante présente une marge élastique utilisable. La localisation des fatigues maxima et les faibles taux de travail auxquels est soumis le matériau dans les barrages de type élastique, permettent, au moins dans certaines zones, de nouvelles contraintes sans que soient dépassés les taux de travail acceptables. La structure du barrage existant se présente à ce point de vue comme un réservoir d'énergie élastique que l'on doit utiliser pour l'accroissement de sa fonction résistante.

Comparaison structurale de la surélévation

8. — Le problème structural de la surélévation présente des avantages sur celui du barrage nouveau. Le barrage existant est toujours un commencement de régularisation géométrique du contour naturel du site. Moins son matériau est homogène, plus les circonstances sont favorables à une meilleure répartition de la charge sur la fondation. Les difficultés résident dans les points suivants :

a) l'état de déformation de la structure sur laquelle il s'agit de construire;
b) le joint et en particulier sa soudure;
c) l'augmentation des fondations;

d) l'adaptation des installations auxiliaires (déversoir, amortisseur d'énergie, vannes de fond, etc.).

Technique de surélévation

9. — La technique actuelle, compte tenu de ses possibilités d'action sur la chaleur de prise, le retrait, les injections, peut s'attaquer avec sécurité au problème de la soudure du joint. Cette sécurité est accrue par des précautions telles que tenons, agrafes et dessin du joint suivant les lignes isostatiques. Toutefois, la construction postmonolithique semble à conseiller, car en sus d'éliminer les problèmes que pose le joint, elle résout le problème élastique de blocage des contraintes quand on construit et soude simultanément sur un structure déformée.

Les câbles tendus

10. — L'emploi de câbles tendus permet à la surélévation d'exploiter les ressources de stabilité et d'énergie élastique rémanente de la structure primitive ainsi que de la fondation elle-même. Ils permettent de fixer sur le barrage primitif des structures nouvelles comme des piles de déversoir ou la surélévation elle-même.

De même ils permettent de faire participer à l'équilibre structural le terrain lui-même et d'incorporer à l'ouvrage le poids de toute la zone influencée par l'ancrage.

Plusieurs dispositifs ingénieux permettent aussi d'agir, en les améliorant, sur les conditions de résistance du terrain ou sur la résultante des charges structurales.

Objection aux câbles tendus

11. — Toutefois, l'emploi de câbles tendus, qui résout beaucoup de problèmes difficiles, semble mieux adapté à une intervention chirurgicale sur la structure qu'à une transformation permanente. Pour barrage, *la permanence doit être à pérennité.*

La construction des barrages a écarté peu à peu le béton armé conventionnel, et manifeste en général une certaine répugnance à toute intromission du métal et des mécanismes. Par un effet de mimétisme, le barrage qui complète la nature et s'y incorpore, semble exiger une existence pétrifiée et inerte. Nous ne pouvons prétendre mettre en avant ces affirmations, mais il n'y a pas de doute que le chemin est encore long pour arriver à dominer le comportement « biologique » du béton en grandes masses, et qu'il est risqué au surplus de dépendre de matériaux métalliques et de mécanismes exposés aux agents destructeurs naturels en des endroits d'accès difficile.

12. — Le relâchement de la tension des câbles est conséquence du fluage de l'acier lui-même, mais dans une plus grande mesure de celui du béton sous les charges permanentes.

Néanmoins, le béton améliore avec l'âge sa condition élastique en présence de charges instantanées. De là paraît découler qu'il convient d'employer des dispositifs de précontrainte, généralisés ou limités à des zones déterminées du profil, entrant en jeu uniquement dans des conditions de charge statistiquement moins fréquentes et plus courtes.

La précontrainte occasionnelle

On peut facilement imaginer des systèmes qui feraient fonctionner les câbles exclusivement dans les moments critiques, par exemple sous la charge maxima, sous la surcharge due à l'élévation de la lame déversante ou même lors de la vidange de la retenue. Pour mettre les câbles en tension au moment voulu on pourrait employer des accumulateurs hydrauliques ou tout autre mécanisme. Cela correspondrait à ouvrir les vannes de l'énergie élastique emmagasinée dans le barrage pour être employée en cas d'urgence. La surtension occasionnelle, conditionnée par l'apparition d'une surcharge extraordinaire et éphémère, s'ajuste parfaitement aux propriétés spécifiques des matériaux. Les objections contre les mécanismes pour des raisons de longévité subsistent et s'accentuent dans cette pure exploration spéculative.

13. — En dépit des objections ci-dessus, tout système est valable pour la surélévation : le problème est préconditionné par l'ouvrage existant. La solution doit être forcément casuistique et tout procédé est bon si la fonction est remplie.

14. — Pour la construction par étapes, le degré de liberté est plus grand et le champ plus étendu. Sans restreindre l'invention, il faut centrer le thème en systèmes plus rigoristes. Il semble que la précontrainte pourra constituer une aide importante pour réduire la construction pendant les étapes de durée limitée. Elle permettra le maintien de profils réduits en compensation de l'excès de fondations.

Construction par étapes

Le monolithisme doit être accepté avec toutes sortes de réserves. La localisation des joints dans les zones où l'effort tangentiel est minimum permet d'éviter ou de minimiser le problème de la soudure. Si l'on réussissait à rendre imperméables les joints horizontaux, les types acceptables de structures composées de sections indépendantes, se multiplieraient. La construction postmonolithique convient évidemment, avec le développement de la possibilité de tirer parti de l'intéraction des divers éléments structuraux pour créer une distribution favorable des contraintes aussi bien dans la fondation que dans l'ouvrage.

La réduction de la dimension des blocs jusqu'à l'authentique polylithisme procurera des avantages considérables en ce qui concerne le degré d'isostatisme d'ensemble de l'ouvrage, qui devra augmenter en fonction de l'adaptabilité à des terrains débiles ou hétérogènes.

Le béton remplissant une fonction hydraulique doit être précontraint. Il semble que la technique permettant avec ou sans câbles, d'obtenir cette précompression ne tardera pas à se développer. Elle pourra s'étendre au terrain et à la fondation pour améliorer leurs caractéristiques hydrauliques et mécaniques.

Le titre de la question que nous étudions n'est qu'un aspect ou, du moins une fraction, strictement technologique d'une autre question plus vaste sur laquelle il est nécessaire et pressant de centrer l'attention. Il s'agit de l'exploitation intégrale des ressources en eau, et des moyens pour y parvenir, c'est-à-dire des Grands Barrages.

Pour rester objectifs, il est impératif d'organiser une information statistique qui permette d'établir une corrélation entre les besoins de l'humanité, tant démographiques qu'industriels, et les disponibilités en eau et en emplacements convenant à la construction de réservoirs. Sans attendre de précision quantitative, on peut anticiper la conclusion qui s'impose fatalement :

a) Une politique de coordination des emplois de l'eau, facile à réaliser du fait que de simples conditions géométriques (hauteurs. débits) sont requises pour satisfaire aux demandes d'origines diverses. Il faut développer la théorie des retenues et de leur exploitation multiple et conjuguée. Beaucoup de points sont déjà étudiés et une simple compilation sera utile et efficace;

b) Une politique de prospection systématique des sites et d'utilisation exhaustive de ceux qui existent.

La civilisation industrielle à laquelle nous appartenons, l'humanité elle-même, ont une origine fluviale. La civilisation s'est transmise à travers les mers, qui assureront peut-être dans le futur la subsistance alimentaire, mais l'habitat de l'homme est le continent sur lequel la civilisation a progressé et progressera en fonction dis disponibiltés en eau. (La constatation dans l'histoire, de la réaction — fluviale. maritime ou continentale — des diverses civilisations, est curieuse, par contraste avec leur projection expansive).

Les barrages, pour contrôler l'eau dont l'humanité a besoin, devront être chaque jour plus grands, macrostructures authentiques. En moins de quarante ans, la hauteur maxima atteinte par un barrage, qui était de 100 mètres dans les années 20, frise les 300 mètres. Les ressources de la technique ont dépassé les extrapolations acceptables, compte tenu des bases de départ. De même, les modules de tous ordres ont été dépassés. Une révision générale paraît obligatoire et, naturellement, celle de la théorie sur laquelle devra s'appuyer la stratégie en face du problème nouveau.

TABLEAUX - NOTES EXPLICATIVES

Les codes sont les suivants :

- colonne "Type" :

TE	Terre
ER	Enrochement
PG	Poids
CB	Contreforts
VA	Voûte
MV	Voûtes multiples

- colonne "Destination" :

I	Irrigation
H	Hydroélectrique
C	Défense contre les crues
N	Navigation
S	Services de distribution d'eau
R	Buts récréatifs

- colonne "Evacuateur de crue" :

L	Déversoir libre
V	Déversoir avec vannes

La colonne "WRD/RMB" comporte les références du Registre Mondial des Barrages, année d'édition et page correspondante.

Si deux codifications sont rassemblées dans une même colonne, la première correspond au barrage d'origine et la seconde au barrage surélevé.

Dans la colonne de droite sont indiquées les publications de référence ou observations disponibles.

TABLES - EXPLANATORY NOTES

The keys are :

- Under the heading "Type" :

TE	Earth
ER	Rockfill
PG	Gravity
CB	Buttress
VA	Arch
MV	Multi-arch

- Under "Purpose" :

I	Irrigation
H	Hydroelectric
C	Flood control
N	Navigation
S	Water supply
R	Recreational

- Under "Spillway type" :

L	Uncontrolled spillway
V	Controlled spillway

The "WRD/RMB" column contains the reference to the World Register of Dams, Edition Year and (page).

If two figures or symbols are in one column, the upper corresponds to the initial dam and the lower to the heightened dam.

In the last column is included the bibliography references and remarks if available.

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EARTH AND ROCKFILL DAMS / BARRAGES EN TERRE ET EN ENROCHEMENTS
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AUSTRALIA — AUSTRALIE	CALLIDE	CALLIDE	TE/ER	I/S	35,5	1 487	46,6	L	5700	1965	1984	—
					45	1 487	127	V	5800	1986	(198)	
	KINCHANT	SANDY CREEK	TE	I	14	670	11	L	570	1977	1984	—
					24	3375	62,8	L	350	1985	(205)	
	MERRIMU	COIMADAI	TE/ER	I/S	37	166	19	L	93	1969	1984	—
					46	347	35	L	306	1984	(200)	
	MIENA	SHANNON	ER	H	22	237	2390	V	52	1967	1984	—
					28	481	3356	V	53	1982	(199)	
	TARAGO	TARAGO	TE/ER	S	34	326	25,5	L	340	1969	1984	—
					34	326	37,5	L	459	1974	(199)	
GERMANY (FR) — ALLEMAGNE (RF)	BREITENBACH	BREITENBACH	ER	S	29	360	26	L	8,2	1956	1984	XIII ICOLD CONGRESS Q 48/R 11
					41,5	740	7.8	L	12,6	1980	(180)	
	RURTAL (SCHWAMME-NAUEL)	RUR	TE	C/S/H/R	61,8	1 700	100,7	V	450	1938	1984	—
			TE/ER		77,2	2 600	202,6	V	450	1959	(176)	
LEBANON — LIBAN	KARAOUN	LITANI	E R	I/H	50	600	52	L	600	1962	1984	VIII ICOLD CONGRESS Q 29/R 21
					68	1900	220	L	800	1965	(608)	
NORWAY — NORVEGE	BLADALSVATN	BLAELV	ER	H	23	37	?	L	230	1963	1984	—
					52	495	?	L	230	1975	(651)	
	LYNGSVATN	LYNGSANA	ER	H	29	180	?	L	135	1964	1984	—
					37	338	?	L	135	1975	(651)	
SOUTH AFRICA — AFRIQUE DU SUD	STERKFONTEIN	NUWE JAAR SPRUIT	TE	S	63	5 700	1203	?	?	1980	1984	—
					93	19800	2656	?	?	1984	(163)	

SPAIN ESPAGNE	GAYA	GAYA	ER	S	50	600	10	L	640	?	1984	XIII ICOLD CONGRESS
					75	1500	60	V	537	1978	(411)	Q 48/R 68
USA ETATS-UNIS	CULMBACK	SULTAN	ER	H/S	61	367	43	L	1620	1965	?	—
					80	565	191	L	1620	1984		
	HUNTINGTON	HUNTINGTON CREEK	TE	R/I	13,7	?	0,43	L	?	1978	?	—
					16,8	?	0,57	L	?	1982		
	LAKE SHERBURNE	SWIFTCURRENT CREEK	TE	I	29	174	84	L	113	1921	?	RAISED WITH REINFORCED EARTH RETAINING WALLS
					33	185	84	L	127	1982		
ZIMBABWE	MWENJE	MWENJE	TE	I	27,1	325	13	L	2230	1969	1984	—
					35,8	775	43	L	2490	1985	(726)	

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AUSTRALIA / AUSTRALIE	GLENBAWN	HUNTER	ER	I/C/S	78	7650	361	L	1700	1958	1984	INSTITUTION OF ENGINEERS AUSTRALIA, APRIL 1983
					101	10735	870	L	5390	1986	(196)	
	MALMSBURY	MALMSBURY	TE	S	22	13	15	V	?	1874	1984	INTERMEDIATE HEIGHTENING IN 1887.
					24	15	18	V	736	1940	(190)	
	MELTON	WERRIBEE	ER	I	35.5	?	21	L	?	1916	1984	INTERMEDIATE HEIGHTENING IN 1937
					35	?	24	L/V	2209	1967	(192)	
	PYKES	PYKES CREEK	TE	I/S	36	?	18	L	?	1911	1984	———
					39	?	24	L	292	1930	(191)	
	SOUTH PARA	SOUTH PARA	TE	S	48	581	45	L	736	1958	1984	———
					48	581	51	V	736	1960	(196)	
	UPPER COLIBAN	COLIBAN	TE	I/S	25	?	19	L	?	1903	1984	INTERMEDIATE HEIGHTENING IN 1917
					28	182	32	L	283	1925	(191)	
AUSTRIA / AUTRICHE	GROSSEE	ZIRKNITZBACH	ER	H	47	380	9.7	L	8	1974	1984	———
					57	740	13.3	L	8	1980	(209)	
	HOCHWURTEN	WURTENBACH	TE ER	H	45	335	7.6	L	21	1974	1984	———
					55	600	12.7	L	21	1980	(209)	
	OSCHENIKSEE	OSCHENIK LAKE	TE/ER	H	65	1000	25	L	6	1976	1984	———
					81	2300	33	L	6	1979	(209)	
BOLIVIA / BOLIVIE	CORANI	CORANI	ER	H	29	300	90	L	1000	1965	1984	———
					34	424	150	L	1000	1983	(213)	
CANADA	UPPER KANANASKIS	KANANASKIS	TE	H	11	5	82	L	?	1933	1984	———
					24.4	235	160	?	30	1943	(257)	

CHINA / CHINE	HAIZI	JU HE	TE	C/I	32	858	54	V	4175	1960	1976	———
					40.5	2267	121	V	3615	1980	(122)	
GREAT BRITAIN / GRANDE - - BRETAGNE	BELMONT	BELMONT BROOK	TE	S	18	?	?	L	?	1827	1984	———
					23	?	1.8	L	?	1849	(501)	
	BUCKIEBURN	BUCKIE BURN	TE	S	21,8	225	0.7	L	?	1905	1984	———
					22,6	226	0.9	L	?	1919	(510)	
	EYE BROOK	THE EYE BROOK	TE	S	15	?	6.9	L	113	1940	1984	JOURNAL OF WATER ENGINEERS MAY. 1959; FEB. 1976
					16	?	8	L	113	1955	(515)	
	LITTLE DENNY	?	TE	S	14.7	130	0.5	L	?	1890	1984	———
					15,3	131	0.6	L	?	1904	(508)	
	NORTH THIRD	BANNOCK BURN	TE	S	14,1	142	2,1	L	?	1911	1984	INTERMEDIATE RAISING, IN 1934
					16.6	144	3,3	L	?	1936	(512)	
	SWINDEN Nº1	CALDER	TE	S	?	?	0.1	?	?	1819	1984	———
					19	?	0.5	L	?	1876	(503)	
	WAYOH	BRADSHAW BROOK	TE	S	23	?	2,3	L	?	1876	1984	———
					31	416	5,1	L	?	1962	(506)	
IRAN	GOLPAYEGAN	GOLPAYEGAN	TE	I	56	?	?	L	2000	1950	1984	———
					64	850	44,5	L	2000	1984	(544)	
JAPAN / JAPON	SHINNARIU-UENNAI	UENNAI	TE	I	21	245	4,3	V	126	1929	1979	———
					26.8	399	6,8	L	150	1977	(243)	
KENYA	SASUMUA	SASUMUA	TE	S	37	512	9.1	L	452	1956	1984	INSTITUTION OF CIVIL ENGINEERS (UK) JAN 1970
					45	700	15,9	L	452	1968	(607)	
PAKISTAN	KHANPUR	HARO	TE	S/I	42	2378	72,5	V	3400	1983	1984	———
					51	4916	132	V	4700	1983	(658)	
PORTUGAL	CAMPILHAS	CAMPILHAS	TE	I/H	35	680	21.7	L	124	1954	1984	———
					35	680	27.2	L	132	1970	(667)	

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SOUTH AFICA / AFRIQUE DU SUD	GRASSRIDGE	GROOT	TE	I	15	370	58	L	741	1924	1984	——
					24	390	74	L	1250	1948	(150)	
	KLEIN MARICO	KLEIN MARICO	TE	I	23	?	4	L	?	1936	1984	——
					27	168	8	L	425	1968	(151)	
	ROOIKRANTZ	BUFFALO	TE	S	27	354	4	L	990	1952	1984	——
					28	413	5	L	990	1969	(153)	
	SUNNYSIDE	RIVIERSON DEREND	TE	I	18	?	?	L	?	1969	1984	——
					25	137	0,5	L	23	1973	(159)	
SPAIN / ESPAGNE	BENAMARIAS	SALGUIRAL	TE	I	14,4	54	0,2	L	10	1972	1984	——
					19,5	85	0,3	L	48	1982	(408)	
	GASSET	BECEAS	TE	I/S	19	360	23	L	14	1909	1984	——
					23	457	41,7	L	14	1984	(387)	
	ODIEL	ODIEL	ER	S	35	94	3,3	L	224	1970	1984	——
					41	146	7,4	L	354	1982	(406)	
	TORRE DEL AGUILA	SALADO DE MORON	TE	I	25	207	44	L	?	1936	1984	VI ICOLD CONGRESS Q 20/R 47
					42	250	70	L	700	1947	(393)	
SWEDEN / SUEDE	STORA STENSJÖN	INDALSÄLVEN	ER/TE	H	20	130	122	V	100	1968	1984	——
					25	350	172	V	100	1978	(687)	
THAILAND / THAÏLANDE	UBOL RATANA	NAM PONG	TE	I/H/C	32	575	2250	V	2500	1965	1984	——
					35,1	690	2550	V	3500	1986	(701)	
TURKEY / TURQUIE	DAMSA	DAMSA	TE	I	26	230	3,3	L	150	1956	1984	——
					33,5	610	7,1	L	150	1971	(705)	

U.S.A. / ETATS-UNIS	ADAMS	SNOW CREEK	TE	I	4,6	?	?	?	?	1890	—	——
					16,2	145	1.3	L	14	1915		
	ARROYO SECO	DRY CREEK	TE	I	17,7	43	1.5	L	64	1957	—	——
					20,4	83	3	L	214	1963		
	BEAVER BROOK n° 3	CLEAR CREEK	TE	S	9,8	?	?	L	?	?	—	——
					17,7	62	0,1	L	13	1910		
	BEAVER CREEK	BOX CREEK	TE	I	11,6	?	?	?	?	1913	—	INITIAL DAM WAS LOG CRIBBING WITH EARTH EMBANKMENT
					17,7	413	1,7	L	48	1953		
	BEAVER PARK	BEAVER CREEK	TE	R	22,9	?	?	L	?	?	1984	——
					29,3	317	5,8	L	1770	1950	(466)	
	BELL CANYON	BELL CANYON	TE	I	15,8	?	0,2	?	?	1948	—	——
					19,8	?	0,5	L	18,7	1958		
	BETHANY FOREBAY	ITALIAN SLOUGH	TE	S/R/I	24.4	230	1,1	L	42	1961	1984	——
					29	879	6,5	L	61	1967	(450)	
	BONITO	RIO BONITO	ER	S	28,7	?	1,4	L	228	1930	1984	——
					34,4	119	1,5	L	1886	1944	(433)	
	BROWNWOOD	BROWNWOOD	TE		37	?	146	V	1133	1933	1984	——
					43.2	?	?	V	?	1983	(434)	
	BYBEE	SHOOFLY CREEK	TE	I	7	?	1,2	L	?	1917	—	——
					16	101	9,9	L	23	1985		
	CABIN	RAPID CREEK	TE	S	15,2	36,3	189	L	54	?	—	——
					27,4	?	986	L	?	1982		
	CAMP FAR WEST	BEAR	TE	I/R	17,4	6	4,7	L	1000	1930	1984	——
					56.4	1493	127	L	3000	1965	(454)	
	CERRO	CIMMARON	TE	S	13.4	?	?	L	?	?		——
					16.1	164	0,9	L	117	1966		

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	CLEVELAND	SPRING CREEK	TE	I/R	14	?	4	?	?	1930	—	
					18,6	?	6,6	L	?	1978		
	COONEY	RED LODGE	TE	I/C/R	29,6	1041	29,8	L	405	1937	1984	
					31,1	1181	34,8	L	1473	1982	(435)	
	DEERFIELD	CASTLE CREEK	TE/ER	I/S	40,5	508	18,8	L	14000	1947	1984	USCOLD NEWSLETTER MARCH 1984
					52	880	18,8	L	86500	1984	(439)	
	EAST BARRE	WINOOSKI	TE/ER	C	17	304	14,8	L	481	1935	—	
					20	326	14,8	L	1050	1961		
	ENCINO	ENCINO CREEK	TE	S	32,3	383	4	L	10,8	1924	1984	
					51,2	2415	12,1	V	48,7	1963	(429)	
	FISHING CREEK	FISHING CREEK	TE	S	14,9	65	261	L	63,7	1933	—	
					17,7	89	186	L	439	1984		
	GREAT WESTERN	WALNUT CREEK	TE	S	10,9	?	?	L	?	?	—	
					21,3	655	4	L	860	1958		
	GURLEY	GURLEY CANYON	TE	I	6,1	?	?	L	?	?	—	
					20,1	747	12,4	L	257	1961		
	HARRIS	WEST RIFLE CREEK	TE	I	7,9	?	?	L	?	?	—	
					15,2	46	0,2	L	524	1949		
U.S.A. / ETATS-UNIS	HOG PARK	HOG PARK CREEK	TE	S	19	126	0,4	?	?	1966	1968	
					34	669	27,9	L	62	1985	(148)	
	HOLMES	HOLMES CREEK	TE	I	12,2	?	?	–	–	1880	—	
					21,3	?	0,9	–	–	1903		

HULET	SINKER CREEK	TE	I	18	?	?	L	?	1910	1984	ORIGINAL DAM FAILED IN 1943
				29	?	5,3	L	34	1976	(466)	
JEMINA K	NORTH FORK BEAVER CREEK	TE	I	15,9	?	0.01	L	?	?	—	
				20,1	?	0.04	L	7	1977		
JUNIATA	KAHNAH CREEK	TE	S	7,9	?	?	L	?	?	—	
				29,6	245	3,3	L	255	1954		
LAKE GREGORY	HOUSTON CREEK	TE	I	24,4	92	2,3	V	99	1938	1984	
				27.4	118	2,8	V	201	1966	(435)	
LAKE KEMP	WICHITA	TE	I/C	30,2	1150	355	L	1812	1923	1984	
				35,1	2930	373	L	15133	1974	(428)	
LAKE WOHLFORD	ESCONDIDO CREEK	ER/TE	I/S/H	23	?	3,1	L	46	1895	1984	
				30	119	8,6	L	88	1924	(429)	
MATHEWS	CAJALCO CREEK	TE	S	66,5	?	76,5	L	566	1938	—	
				80,5	7309	224,5	L	382	1961		
MILLER FLAT	MILLER FLAT CREEK	TE	I	16,8	?	?	?	?	1949	—	
				22,3	?	6,9	L	56,6	1953		
NEW CASTLE	PINTO CREEK	TE	I/R	22,6	?	4,2	?	?	1956	—	
				25,3	?	6,5	L	538	1974		
NINEMILE	NINE MILE CREEK	TE	I	13.4	?	3	—	0	1926	—	
				16.8	?	4,3	L	5,5	1982		
NOVATO	NOVATO CREEK	TE	S	17,1	138	2,1	L	269	1951	1984	
				21,7	452	5,5	L	244	1959	(442)	
PARADISE	LITTLE BUTTE CREEK	TE	S/I	44,2	368	7,9	L	226	1957	1984	
				53,3	590	14,2	L	361	1977	(447)	
PATERSON CARL SMITH	WEST LEROUX CREEK	TE	I	10,7	?	?	L	?	?	—	
				16,8	279	1.1	L	493	1965		

ICOLD / CIGB **DAM HEIGHTENING / SURELEVATION DE BARRAGES**

EARTH AND ROCKFILL DAMS/BARRAGES EN TERRE EN ENROCHEMENT

HEIGHTENING PLANNED AFTER CONSTRUCTION/ SURELEVATION PREVUE APRES CONSTRUCTION

COUNTRY / PAYS	NAME / NOM	RIVER / COURS D'EAU	TYPE	PURPOSE / DESTINATION	DAM HEIGHT / HAUTEUR (m)	DAM CONTENT / VOL.BARRAGE ($10^3 m^3$)	RESERVOIR (hm^3)	SPILLWAY TYPE / TYPE EVACUAT.	SPILL.CAPACI / CAPACITE EVA (m^3/s)	YEAR / ANNÉE	W.R.D. / R.M.B.	BIBLIOGRAPHY / BIBLIOGRAPHIE / REMARKS / OBSERVATIONS
	PENROSE - ROSEMONT	EAST BEAVER CREEK	TE	S	21,3	?	?	L	?	?	1984	——
					27,4	383	3,1	L	102	1961	(451)	
	PETERS	LAGUNITAS CREEK	TE	S	57,3	?	20,4	L	623	1954	1984	——
				S/I/H/R	70,1	716	41	L	623	1983	(445)	
	ROB ROY	DOUGLAS CREEK	TE	I/S	29	320	11	L	41	1965	1984	——
					43	963	43,7	L	159	1986	(458)	
	SCOTTS FLAT	DEER-CREEK	TE	S/I	42,7	482	32,4	L	425	1948	1984	——
					53,3	852	60,4	L	793	1965	(440)	
	SEVIER BRIDGE	SEVIER	TE	I	19.8	?	0.13	?	?	1908	—	——
					28,1	?	0,29	V	156	1916		
	SILVER LAKE	NORTH BOULDER	TE	S	9,4	?	?	?	?	?	—	——
					21,6	557	4,9	L	1876	1964		
	SKAGWAY	WEST BEAVER CREEK	TE	R	20,6	?	?	L	?	?	—	——
					23,1	197	3,8	L	2213	1980		
	STANDLEY LAKE	WOMAN CREEK	TE	I	27.9	?	?	L	?	?	1984	——
				I/S	34,4	5328	52,3	L	3398	1928	(467)	
	STONE CANYON	STONE CANYON CREEK	TE	S	49.1	558	9.8	L	4,4	1924	1984	——
					57.3	2558	12,8	L	8.1	1956	(429)	
U.S.A. — ETATS-UNIS	THOMPSON	MIDDLE CANYON	TE	S	21.3	50	0,4	L	113	1925	1984	——
					34,8	103	1,2	L	260	1966	(430)	
	VALMONT "A"	BOULDER CREEK	TE	I	7.7	?	?	L	?	?	—	——
					20.4	876	13,8	L	2860	1964		

VOUGA	RAZOR CREEK	TE	I	14,9	?	?	L	?	1958	—	———
				18	51	0.6	L	635	1978		
WATAUGA	WATAUGA	TE/ER	H/C/R	96.9	2674	1063	L	2093	1948	1984	———
				100	2724	1222	L	2207	1983	(440)	
WELLINGTON	BUFFALO CREEK	TE	I	13,1	?	?	L	?	?	—	———
				17,1	156	5,4	L	175	1950		
WENAS	WENAS CREEK	TE/ER	I/R	16,3	32	1,6	L	43	1925	—	———
				27	200	3 9	L	828	1982		
WHITE PINE	WHITE PINE FORK	TE/ER	I	10.6	?	0.04	?	?	1938	—	———
				16.5	?	0.39	L	11.6	1967		
WILKINSON	BOHMAN WASH	TE	I	9.8	?	0.07	L	?	1946	—	———
				16.2	?	0.35	L	?	1957		
WOODRUFF NARROWS	BEAR	TE	I	18	79	34	L	144	1962	—	———
				21	123	70	L	221	1981		
WRIGHTSVILLE	WINOOSKI	TE/ER	C	29	859	?	L	498	1935	1984	———
				35	941	25	L	1740	1959	(434)	

ICOLD / CIGB **DAM HEIGHTENING / SURELEVATION DE BARRAGES**

CONCRETE AND COMPOSITE DAMS / BARRAGES EN BETON ET MIXTES

HEIGHTENING PLANNED BEFORE CONSTRUCTION / SURELEVATION PREVUE AVANT LA CONSTRUCTION

COUNTRY / PAYS	NAME / NOM	RIVER / COURS D'EAU	TYPE	PURPOSE / DESTINATION	DAM HEIGHT / HAUTEUR (m)	DAM CONTENT / VOL. BARRAGE ($10^3 m^3$)	RESERVOIR (hm3)	SPILLWAY TYPE / TYPE EVACUAT.	SPILL. CAPACI. / CAPACITE EVA. (m3/s)	YEAR / ANNÉE	W. R. D. / R. M. B.	BIBLIOGRAPHY / BIBLIOGRAPHIE / REMARKS / OBSERVATIONS
AUSTRALIA / AUSTRALIE	CASCADE	CASCADE CREEK	CB	S	7,5	?	0,03	L	?	1908	1984	
			VA		17	4	0,16	L	4	1915	(191)	
	COTTER	COTTER	PG	S	21	?	?	L	?	1915	1984	
					31	26	4,7	L	850	1951	(192)	
	GLENMAGGIE	MACALISTER	VA/PG	I	33	77	129	L	2775	1927	1984	
					37	77	190	V	2775	1958	(192)	
	HUME	MURRAY	TE/PG	I/H	51	4419	1540	L	5200	1936	1984	POST TENSIONING OF HUME DAM AUST. CIVIL ENG. TRANSAC. (MARCH 1962)
					51	4419	3038	V	7940	1961	(193)	
	LESLIE	SANDY CREEK	PG	I	31	108	47,5	L	2080	1965	1984	
					34	114	108	V	3430	1985	(205)	
	LAURISTON	COLIBAN	CB/ER	S	33	42	15	L	609	1941	1984	
					33	42	20	V	609	1949	(194)	
	WOODFORD CREEK	WOODFORD CREEK	VA	S	12,6	?	0.16	L	?	1928	1984	
					18	4	0.86	L	110	1948	(194)	
NEW ZEALAND / NOUVELLE - ZELANDE	MAHINERANGI	WAIPORI	VA/PG	H	21	95	68	L	560	1931	1984	
					34,4	134	243	V	200	1946	(653)	
NORWAY / NORVEGE	STOREFOSSEN	BERGDALSELV	PG	H	13	?	?	L	?	1926	1984	
					19	9	12	L	240	1948	(644)	
SOUTH AFRICA	BEERVLEI	GROOT	MV	C	29	42	58	L	1133	1957	1984	
					31	42	94	L	1133	1967	(154)	
	CHELMSFORD	NGAGANE	PG/TE	S	20	216	83	L	722	1961	1984	
					25	332	199	L	1230	1982	(155)	

AFRIQUE DU SUD	DOORNDRAAI	STERK	PG	I/S	26	32	21	L	1246	1953	1984	
					31	32	47	V	1246	1971	(153)	
	KOSTER	KOSTER	VA/TE	I	20	?	5	L	660	1964	1984	
					23	50	11	L	660	1972	(156)	
SPAIN ESPAGNE	PUENTES VIEJAS	LOZOYA	PG	S	48	?	?	L	?	1925	1984	VI ICOLD CONGRESS Q 20/R13
					66,2	144	49	V	400	1940	(392)	
SWITZERLAND SUISSE	ILLSEE	ILLBACH	PG	H	18	12	?	L	?	1927	1984	
					25	24	6,5	L	4	1943	(689)	
	SANETSCH	SARINE	PG	H	37	?	1,6	L	?	1962	1984	
					42	37	2,7	L	30	1965	(692)	
USA ETATS UNIS	CLEAR CREEK	TIETON	VA	I/R	19	3	2,3	L	113	1914	?	USBR PROJECT DATA BOOK P. 1337
					25	4	6,5	L	195	1918		
	JACKSON LAKE	SNAKE	TE/PG	I/C/R	14.8	?	469	V	?	1911	?	USBR PROJECT DATA BOOK P. 639
					20	376	1044	V	?	1916		
	LAKE MALOYA	CHICORICA CREEK	TE/PG	S	16.8	?	1.4	L	48	1917	1984	
					30.5	404	4.9	L	336	1948	(440)	
	MARSHALL FORD	COLORADO	TE/PG	C/I	60	1400	678	L	14385	1940	1984	
					85	2726	2410	L	14385	1942	(438)	
	MC CLURE	SANTA FE	TE/PG	S	18.9	?	0.6	L	71	1927	1984	
					33.8	345	3.3	L	287	1947	(430)	
	MOUNTAIN DELL	PARLEYS CREEK	MV	S	19.8	?	1.2	L	?	1917	1984	
					32	14	3.9	V	11	1925	(427)	
	SAN JUAN	—	TE/PG	S	18.3	?	1.6	L	25.5	1972	?	
					24.1	1070	3.2	L	25.5	1981		
	UTE	CANADIAN	TE/PG	R	36.6	1288	135	L	16000	1963	1984	
					39.9	1622	336	L	16000	1984	(454)	

ICOLD / CIGB — DAM HEIGHTENING / SURELEVATION DE BARRAGES

CONCRETE AND COMPOSITE DAMS/ BARRAGES EN BETON ET MIXTES

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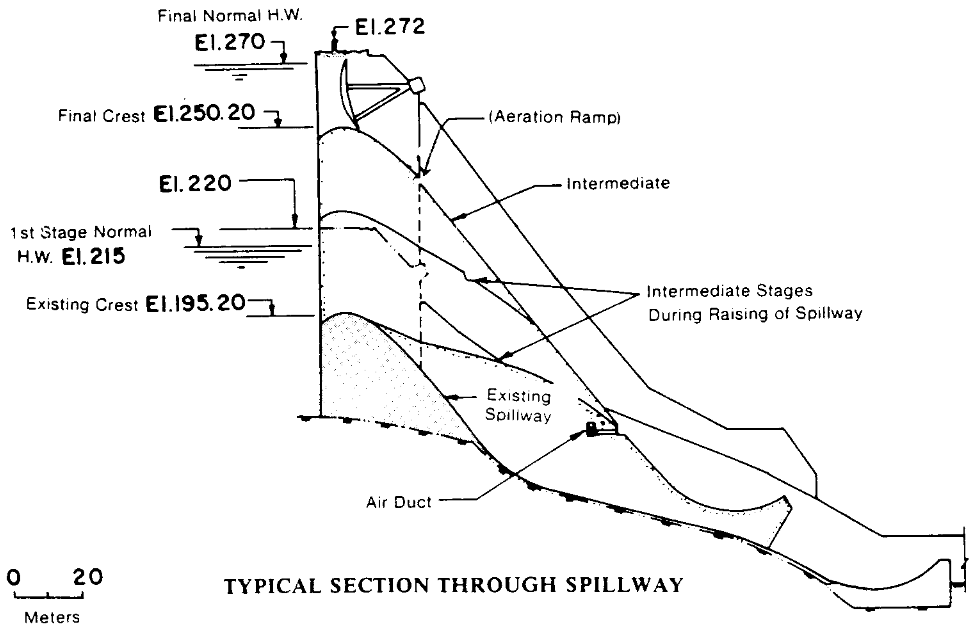
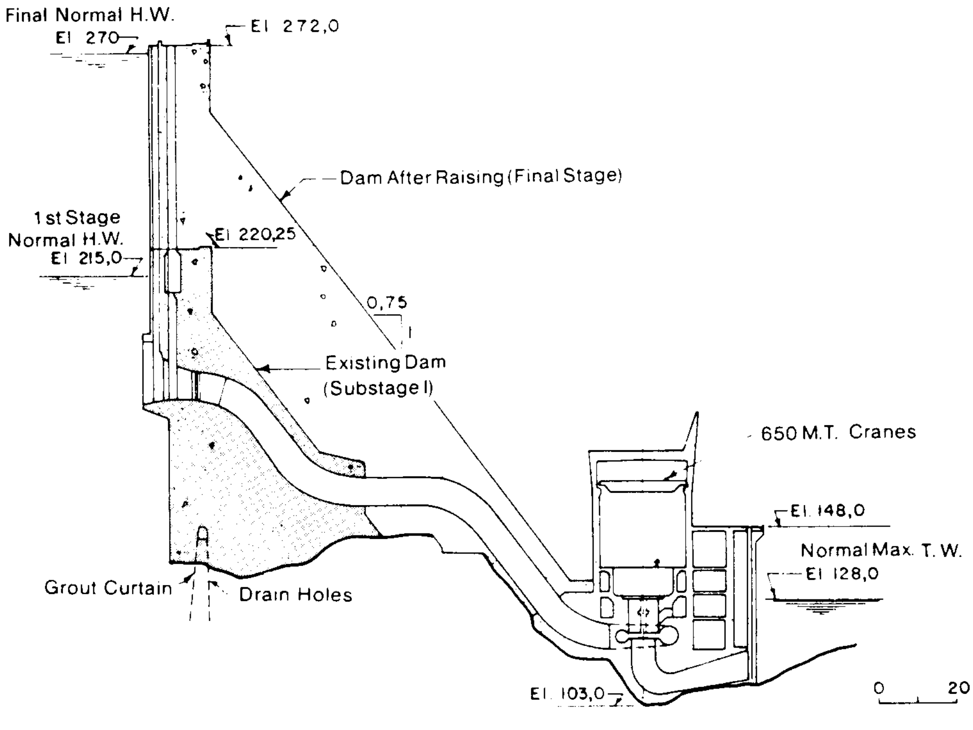
COUNTRY / PAYS	NAME / NOM	RIVER / COURS D'EAU	TYPE	PURPOSE / DESTINATION	DAM HEIGHT / HAUTEUR (m)	DAM CONTENT / VOL.BARRAGE ($10^3 m^3$)	RESERVOIR (hm^3)	SPILLWAY TYPE / TYPE EVACUAT.	SPILL.CAPACI. / CAPACITE EVA. (m^3/s)	YEAR / ANNÉE	W.R.D. / R.M.B.	BIBLIOGRAPHY / BIBLIOGRAPHIE / REMARKS / OBSERVATIONS
VENEZUELA	RAUL LEONI (GURI)	CARONI	PG/ER	H	120	2100	17000	V	30000	1975	1984	
					172	75700	135000	V	30000	1986	(718)	

GURI DAM HEIGHTENING



TYPICAL SECTION THROUGH SPILLWAY



SECTION THROUGH RAISED DAM AND POWERHOUSE NO. 1

ICOLD / CIGB **DAM HEIGHTENING / SURELEVATION DE BARRAGES**

CONCRETE AND COMPOSITE DAMS / BARRAGES EN BETON ET MIXTES

HEIGHTENING PLANNED AFTER CONSTRUCTION / SURELEVATION PREVUE APRES LA CONSTRUCTION

COUNTRY / PAYS	NAME / NOM	RIVER / COURS D'EAU	TYPE	PURPOSE / DESTINATION	DAM HEIGHT / HAUTEUR (m)	DAM CONTENT / VOL. BARRAGE (10^3 m³)	RESERVOIR (hm3)	SPILLWAY TYPE / TYPE EVACUAT.	SPILL. CAPACI. / CAPACITE EVA. (m3/s)	YEAR / ANNEE	W. R. D. / R. M. B.	BIBLIOGRAPHY / BIBLIOGRAPHIE / REMARKS / OBSERVATIONS
AUSTRALIA / AUSTRALIE	CLARK	DERWENT	VA	H	61	153	315	V	566	1949	1984	RAISED BY PRE-STRESSED
					67	159	541	V	651	1966	(194)	WALL
	HARVEY	HARVEY	PG	I/S	16	3	2.4	L	?	1914	1984	———
			PG/TE		24	95	9.1	V	133	1932	(192)	
	MANLY	CURL CURL CREEK	PG	S	11.5	?	0.3	L	?	1892	1984	RAISED 0.5 m IN 1909
				R/C	20	8	2	L	210	1922	(190)	POST TENSIONED IN 1981
	MOUNT BOLD	ONKAPARINGA	VA	S	54	100	30.2	L	1020	1938	1984	———
					58	105	47.3	V	1020	1962	(194)	
	MUNDARING	HELENA	PG	S	59	62	21	L	880	1902	1984	———
					71	124	76	V	1019	1951	(190)	
	WELLINGTON	COLLIE	PG	I/S	20	26	31	L	1432	1933	1984	———
				I/S/H	37	90	185	L	1432	1960	(193)	
	WYANGALA	LACHLAN	PG	I	61	187	375	L	7200	1936	1984	GRAVITY DAM UTILISED AS
			ER		85	3580	1220	V	14700	1971	(201)	TOE WALL IN NEW DAM.
AUSTRIA / AUTRICHE	SPULLERSEE NORD	ALFENZ	PG	H	26	24	13.1	L	0	1925	1984	———
					29	27.4	15.7	L	2.8	1965	(206)	
	SPULLERSEE SUD	ALFENZ	PG	H	36	63	13.1	L	19.8	1925	1984	———
					39	69	15.7	L	36.8	1965	(206)	
BELGIUM / BELGIQUE	LA GILEPPE	LA GILEPPE	PG	S/C	52	260	13.3	L	?	1876	1984	———
			ER/PG		68	1433	26.4	V	185	1971	(212)	
BRAZIL / BRESIL	LAJES	LAJES	PG	H	35	68	199	L	?	1907	1984	———
			CB		63	190	1052	L	71	1958	(219)	

FRANCE	BOUILLOUSE	LA TÊT	PG	H/I	23	46	13	L	?	1910	1984	———
					25	50	17.5	L/V	67	1947	(479)	
	COMBES (CONFOLENT)	LA CREUSE	PG	H	26	35	2.4	L	?	1921	1984	———
			PG/CB		35	41	4.7	L	350	1927	(479)	
	LA CHAZOTTE	LIGNON DU	PG	S	29	?	5	L	?	1914	1984	———
	LA VALETTE	VELAY		S/H/R	60	215	41	V	1400	1948	(479)	
	JOUX	TURDINE	PG	S	25	8	0.8	L	60	1906	1984	VI.- ICOLD CONGRESS Q20/R48 XIII.- ICOLD CONGRESS Q48/R15
					28	10	1.1	L/V	60	1954	(478)	RAISED BY POST-TENSIONING
	OULE (L')	OULE (L')	PG	H	33	41	6.5	L	?	1923	1984	VI.- ICOLD CONGRESS Q20/R81
					51	128	18.6	L	69	1950	(479)	XIII.- ICOLD CONGRESS Q48/R15
	ST. ENGRACE	SAISON	PG/VA	H	32	10	0.09	L	?	1916	1984	XIII.- ICOLD CONGRESS Q48/R15
					42	11.3	0.34	L	110	1953	(479)	
GREAT BRITAIN / GRANDE BRETAGNE	ARGAL	ANTER BROOK	PG	S	14	?	0.65	L	15	1942	1984	RAISED BY POS-TENSIONING
					17	30	1.35	L	22	1961	(515)	
	BURRATOR	MEAVY	PG	S	24	?	3	L	?	1896	1984	———
					27	32	4.7	L	243	1928	(509)	
	MULLAROCH	CANNICH	PG	H	42	?	?	L	?	1951	1984	PROC. I.C.E. SEPT. 1953
					48	219	223	L	?	1951	(516)	
	PRESCELLY	SYFNWY	PG/VA	S	20	?	0.3	L	?	1931	1984	———
					23	129	0.6	L	350	1941	(514)	
	SPELGA	BANN	PG	S	30	?	2.7	L	56	1958	1984	———
					32	54	3.3	V	108	1974	(518)	
JAPAN / JAPON	KAWAKAMI	TONDA	PG	C/S	47	55	6.1	V	540	1962	1984	———
					63	163	13.7	V	600	1980	(598)	
	KURODA	KURODA	PG	H	35	41	4.5	V	26	1933	1984	XIII.- ICOLD CONGRESS
					45	145	11	V	210	1978	(597)	Q 48/R 25

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DAM HEIGHTENING / SURELEVATION DE BARRAGES

CONCRETE AND COMPOSITE DAMS / BARRAGES EN BETON ET MIXTES

HEIGHTENING PLANNED AFTER CONSTRUCTION / SURELEVATION PREVUE APRES LA CONSTRUCTION

COUNTRY / PAYS	NAME / NOM	RIVER / COURS D'EAU	TYPE	PURPOSE / DESTINATION	DAM HEIGHT / HAUTEUR (m)	DAM CONTENT / VOL. BARRAGE ($10^3 m^3$)	RESERVOIR (hm^3)	SPILLWAY TYPE / TYPE EVACUAT.	SPILL. CAPACI. / CAPACITE EVA. (m^3/s)	YEAR / ANNÉE	W. R. D. / R. M. B.	BIBLIOGRAPHY / BIBLIOGRAPHIE / REMARKS / OBSERVATIONS
JAPAN / JAPON	NAKANO	KAMEDA	PG	S	53	75	0.7	L	141	1935	1984	———
					75	276	3.3	L	305	1984	(583)	
	ODOMARI	OTA	PG	H	63	102	18	V	670	1935	1984	VI - ICOLD CONGRESS Q 20 / R 23
					74	178	31	V	804	1959	(580)	
MOROCCO / MAROC	EL KANSERA	BEHT	PG/CB	I/H	63	192	235	L	1800	1935	1984	———
					68	200	297	L	1750	1969	(612)	
	LALLA TAKERKOUST	N'FIS	PG	I/H	62	150	52	V	1500	1935	1984	———
					71	200	78	V	2240	1980	(612)	
NEW ZEALAND / NOUVELLE-ZELANDE	MCLAREN FALLS	MANGAPAPA	VA	H	23	1.2	0.1	L	?	1924	1984	———
					25	2.2	0.17	L	835	1979	(653)	
	WAITAKERE	WAITAKERE	PG	S	20	?	0.77	L	50	1910	1984	———
					25	25	1.85	L	55	1927	(653)	
PORTUGAL	COVA DO VIRIATO	POIOS BRANCOS	PG	S	24	12	0.8	L	?	1962	1984	RAISED BY PRESTRESSING
					28	19	1.5	L	4	1982	(669)	
	ALBASINI	LEVUBU	PG/TE	I	31	?	21	L	1458	1952	1984	———
					34	330	29	V	1458	1971	(153)	
	BOSPOORT	HEX	PG/TE	I	18	?	4	L	620	1933	1984	INTERMEDIATE RAISING OF 2.4 m IN 1953
					27	48	19	L	620	1969	(151)	
	BUFFELSPOORT	STERKSTROOM	VA	I	29	17	5	L	796	1935	1984	INTERMEDIATE RAISING OF 2.4 m IN 1959
					34	19	11	L	796	1967	(151)	

SOUTH AFRICA / AFRIQUE DU SUD	BULK RIVER	BULK	PG	S	22.5	6.8	0.56	L	?	1907	1984	———
					25.5	7.9	0.82	L	128	1919	(150)	
	CLANWILLIAM	OLIFANTS	PG	I	37	53	67	L	?	1935	1984	———
					43	70	121	V	1530	1969	(151)	
	HARTEBEESPOORT	CROCODILE	VA	I	59	68	168	L	2322	1925	1984	———
					59	68	212	V	2322	1971	(151)	
	HENLEY	UMSINDUZI	PG	S	18	14	3	L	1274	1943	1984	———
					25	17	7	L	1274	1959	(152)	
	KOPPIES	RENOSTER	PG/TE	I	19	?	8	L	?	1925	1984	———
					25	728	47	L	1420	1971	(151)	
	LINDLEYSPOORT	ELANDS	VA	I	36	25	12	L	850	1938	1984	———
					38	33	16	L	850	1968	(152)	
	LOSKOP	OLIFANTS	PG	I	44	235	208	L	2830	1939	1984	———
					53	415	360	L	2886	1977	(152)	
	NAHOON	NAHHON	PG	S	33	59	7	L	646	1966	1984	———
					44	91	22	L	1275	1980	(157)	
	NJELELE	NJELELE	VA	I	41	?	31	L	640	1948	1984	———
					47	14	57	V	640	1968	(152)	
	POORTJIESKLOOF	KINGNA	VA	I	33	12	6	L	420	1952	1984	———
					38	13	10	L	420	1968	(153)	
	SAND RIVER	SAND/PALMIET	PG	S	18.5	7.7	0.8	L	155	1905	1984	———
					26.1	8.5	2.7	L	165	1929	(150)	
	STEENBRAS	STEENBRAS	PG	S	19	17	3	L	53	1921	1984	INTERMEDIATE RAISING OF 13 m IN 1928 POST TENSIONED IN 1954
					36	56	34	L	481	1954	(162)	
	STETTYNSKLOOF	STETTYNSKLOOF	PG	S	43	52	5	L	963	1954	1984	———
			PG/ER		58	495	15	L	1000	1981	(153)	
	UMZIMAI	UMZIMAI	VA	S	16	?	0.1	L	155	1957	1984	———
					24	4	1.2	L	155	1966	(154)	

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SOUTH AFRICA / AFRIQUE DU SUD	VAALDAM	VAAL	PG/TE	I/S	54	440	880	L	5097	1938	1984	INTERMEDIATE RAISING OF 6 m IN 1956
					63	1380	1380	V	25000	1984	(152)	
SPAIN / ESPAGNE	ALMANSA	BELEN GRANDE	VA/PG	I	19	?	?	L	?	1384	1984	VI ICOLD CONGRESS Q 20 / R 13
					25	6	2	L	48	1586	(386)	
	ALSA	TORINA	PG	H	42	60	12.6	L	124	1921	1984	——
					49	86	22.9	V	50	1981	(389)	
	ARGUIS	ISUELA	PG	I	25	?	1	L	?	1704	1984	VI ICOLD CONGRESS Q 20 / R 13
					29	13	3	L	72	1929	(386)	
	BECERRIL	BECERRIL	PG	S	25	15	0.8	L	9	1930	1984	——
					32	42	2	V	80	1968	(390)	
	BOLARQUE	TAJO	PG	H	36	?	?	L	?	1910	1984	V ICOLD CONGRESS C-33
				H/I	46	160	31	V	1800	1954	(395)	VI ICOLD CONGRESS Q 20/R 13
	CAMPOFRIO	CAMPOFRIO	PG	S	29	?	2.9	L	13	1883	1984	VI ICOLD CONGRESS Q 20/R 13
					35	50	43	L	30	1956	(386)	
	CONDE DE GUADALHORCE	TURON	PG	I/H	70	130	30	V	200	1921	1984	——
					74	135	74	V	628	1947	(389)	
	DOIRAS	NAVIA	PG	H	89	223	98	V	2000	1934	1984	——
					98	231	124	V	2650	1958	(391)	
	EL BURGUILLO	ALBERCHE	PG	I/H	82	292	190	V	800	1913	1984	VI ICOLD CONGRESS Q 20/R 13
					91	350	217	V	1400	1931	(387)	
	EL ESPARRAGAL	ENCARNACION	VA/PG	I	26	9.8	1.7	L	186	1970	1984	——
					27.5	9.8	2	L/V	203	1984	(406)	

	EL SANCHO	MECA	PG	S	45	?	?	V	1380	1962	1984	XIII ICOLD CONGRESS Q 48 / R 68 RAISED BY PRESTRESSING
					50	92	58	V	500	1973	(401)	
	GUADALMELLATO	GUADALMELLATO	PG	I/H/S	53	225	110	V	900	1928	1984	X ICOLD CONGRESS Q38 / R 9 XII ICOLD CONGRESS Q47 / R 28
					61	430	162	V	1120	1965	(390)	
	IRABIA	IRATI	PG	H	14.2	?	?	L	?	1925	1984	RAISING OF 8 m IN 1929 RAISING OF 6 m IN 1934 VI ICOLD CONGRESS Q20/R13
					44	45	13.6	V/L	387	1947	(389)	
	MEDIANO	CINCA	PG	I/H	79	?	312	V	3000	1966	1984	———
					91	437	438	V	3246	1974	(409)	
	PRIAÑES	NORA	PG	H	27	10	1.1	L	275	1953	1984	———
					27	12	1.6	V	320	1967	(395)	
	ULLDECONA	CENIA	PG	I	61	129	?	V	400	1967	1984	XIII ICOLD CONGRESS Q48/R68
					66	129	11.5	V	400	1984	(404)	
	VALDEINFIERNO	LUCHENA	PG	I	34	?	?	L	1072	1806	1984	———
					49	60	14	L	550	1897	(386)	
SWEDEN SUEDE	KRANGFORS	SKELLEFTE	CB/TE	H	27	?	?	?	?	1928	1984	———
					33	50	1200	V	760	1948	(683)	
SWITZERLAND SUISSE	BERNINA NORD	LAGO BIANCO	PG	H	12	5	11	L	55	1912	1973	———
					15	7	18	L	55	1942	(922)	
	BERNINA SÜD	LAGO SCALA	PG	H	23	10	11	L	55	1912	1973	———
					26	13	18	L	55	(922)	(922)	
	KÄPPELISTUTZ	SEKLIS	PG	H	10	?	-	L	40	1945	?	XV- ICOLD CONGRESS GP 9
					18	3.4	0.1	L	130	1983		
	LIST	GSTALDENBACH	PG	H	10.5	0.5	-	L	6	1901	?	XV.- ICOLD CONGRESS Q59/R 23
					16.5	2	0.1	L	78	1982		
	MUSLEN	MUSLENBACH	PG	H	22	3	-	L	34	1908	1973	XV.- ICOLD CONGRESS Q59/R 23
					29	8	0.1	L	65	1982	(922)	

ICOLD / CIGB

DAM HEIGHTENING / SURELEVATION DE BARRAGES

CONCRETE AND COMPOSITE DAMS/ BARRAGES EN BETON ET MIXTES

HEIGHTENING PLANNED AFTER CONSTRUCTION/ SURELEVATION PREVUE APRES LA CONSTRUCTION

COUNTRY / PAYS	NAME / NOM	RIVER / COURS D'EAU	TYPE	PURPOSE / DESTINATION	DAM HEIGHT / HAUTEUR (m)	DAM CONTENT / VOL. BARRAGE ($10^3 m^3$)	RESERVOIR (hm3)	SPILLWAY TYPE / TYPE EVACUAT.	SPILL. CAPACI. / CAPACITE EVA. (m3/s)	YEAR / ANNÉE	W. R. D. / R. M. B.	BIBLIOGRAPHY / BIBLIOGRAPHIE / REMARKS / OBSERVATIONS
SWIZZERLAND / SUISSE	PIORA	FOSS	PG	H	10	?	32,5	V	?	1920	1973	——
					27	36	53,9	V	86	1952	(923)	
TURKEY / TURQUIE	PORSUK	PORSUK	PG	I/C/S	44	77	141	?	–	1948	1984	——
					65	220	431	?	150	1972	(706)	
U.S.A. / ETATS-UNIS	BIG CREEK 1	BIG CREEK	PG	H	40	?	?	L	?	1913	?	——
					52	?	110	V	541	1917		
	BIG CREEK 2	BIG CREEK	PG	H	24	?	?	L	?	1913	?	——
					36	?	110	V	541	1917		
	BIG CREEK 3	BIG CREEK	PG	H	38	?	?	L	?	1913	?	——
					50	?	110	V	541	1917		
	BOONE	HOLSTON	TE/PG	H/C/R	48	580	320	V	5655	1952	1984	——
					51	605	365	V	6317	1984	(442)	
	CATAWBA	CATAWBA	PG	H	22,7	?	?	L	?	1904	1984	——
					28,8	?	?	V	?	1925	(427)	
	ENTERPRISE	LITTLE PINE	VA	I/R	15	?	1,5	—	?	1926	?	——
					17	?	2,1	L	3,8	1940		
	FISHING CREEK	CATAWBA	PG	H	18,4	?	?	L	?	1916	1984	——
					23	?	?	V	15	1927	(426)	
	GIBRALTAR	SANTA YNEZ	VA	S	43	46	10,1	L	850	1920	1984	——
					51,5	57	12,3	V	2550	1974	(428)	
	GREAT FALLS	CANEY FORK	PG	H/R	17,4	20	?	V	?	1926	1973	——
					28	48	105	V	5037	1925	(323)	

	LAY	COOSA	PG	H	26.1	?	179	V	?	1914	1984	—
					39.5	?	325	V	11810	1968	(426)	
	NOLICHUCKY	NOLICHUCKY	PG	H/R	18	?	?	L	?	1913	1973	INTERMEDIATE RAISING IN 1925
					28.7	40	23	V	3446	1973	(318)	
	PARR SHOALS	BROOD	PG	H	12.5	?	2.5	L	5890	1914	?	—
					15.2	?	38.2	V	4700	1978		
	SOLOMON LAKE	SOLOMON CREEK	VA ER	H	15	?	?	?	?	1930	1984	—
					37	161	38.8	V	494	1980	(468)	
	THURLOW	TALLAPOOSA	PG	H	7.6	?	5.9	L	?	1897	?	—
					18.9	115	22.5	V	8840	1930		
	URRACA	URRACA CREEK	TE/PG	S	15.8	?	0.03	L	158	1955	?	—
					18.3	?	0.05	L	336	1983		
	WILLOW FALLS	WILLOW	VA	H/R	16.2	?	0.3	?	?	1914	?	—
					18.3	?	0.9	V	170	1924		
	YATES	TALLAPOOSA	PG	H	12.2	?	21	L	?	1902	?	—
					26.5	?	66.6	L	8770	1928		
ZIMBABWE	MAZOE	MAZOE	VA	I	34	14	22	L	?	1920	1984	ABUTMENTS STRENGTHENED BY PRESTRESSED CABLES
					37	16	35	L	420	1961	(725)	
	SEBAKWE	SEBAKWE	CB	S	40	49	155	L	2600	1957	1984	PRESTRESSED BY CABLES
					47	104	270	L	2450	1985	(725)	

ICOLD / CIGB

DAM HEIGHTENING / SURELEVATION DE BARRAGES

HEIGHTENINGS REPORTED IN THE WORLD REGISTER OF DAMS = MEMBER COUNTRIES
SURELEVATIONS MENTIONEES DANS LE REGISTRE MONDIAL DES BARRAGES = PAYS MEMBRES

COUNTRY / PAYS	NAME / NOM	RIVER / COURS D'EAU	TYPE	PURPOSE / DESTINATION	DAM HEIGHT / HAUTEUR (m)	DAM CONTENT / VOL BARRAGE ($10^3 m^3$)	RESERVOIR (hm^3)	SPILLWAY TYPE / TYPE EVACUAT.	SPILL CAPACI / CAPACITE EVA (m^3/s)	YEAR / ANNÉE	W.R.D. / R.M.B.	BIBLIOGRAPHY / BIBLIOGRAPHIE / REMARKS / OBSERVATIONS
ALGERIA / ALGERIE	HAMIZ	ARBATACHE	PG	I	?	?	?	?	?	1879	1984 (169)	
					53	20	21	V	750	1935		
	CHEURFAS	MEKERRA	PG	I	?	?	?	?	?	1882	1984 (169)	
					37	45	14	L	850	1935		
	BAKHADDA	MINA	ER	I/S	?	?	?	?	?	?	1984 (169)	
					60	320	56	L	2000	1935		
	KSOB	KSOB	MV	I	?	?	?	?	?	1939	1984 (169)	
					46	60	29	V	1900	?		
	ZARDEZAS	SAF SAF	PG		?	?	?	?	?	1949	1984 (169)	
					64	65	31	V	3200	1977		
EGYPT / EGYPTE	ASWAN I	NILE	PG	I	36	300	?	?	?	1902	1984 (378)	
			PG/CB	I/H	53	545	5000	V	14500	1959		
ITALY / ITALIE	LAGO LAVEZE	GORGENTE	PG	S/H	?	?	?	?	?	?	1984 (550)	
					40	55	2.99	V	300	1927		
	LOMELLINA	GAVALUNO	PG	I	?	?	?	?	?	1896	1984 (550)	
					21	?	0.27	?	?	1927		
	CODELAGO	ARBOLA	PG	H	?	?	?	?	?	1912	1984 (550)	
					26	53	16	V	335	1921		
	LAGO D'AVINO	CIAMPERE	ER	H	?	?	?	?	?	1913	1984 (550)	
					20	33	6	L	33	1927		
	LAGO VENINA	VENINA	MV	H	?	?	?	?	?	1926	1984 (552)	
					50	95	11	L/V	89	1942		

	LAGO TRUZZO	TRUZZO	PG	H	?	?	?	?	?	1927	1984 (553)	
					30	62	21	V	143	1942/6		
	OZOLA	OZOLA	MV	H	?	?	?	?	?	1929	1984 (554)	
					25	7	0.06	L	323	1940		
	TAVERNELLE	METAURO	PG	H	?	?	?	V	2200	1959	1984 (561)	
					22	13	1.96	V	2200	1965		
MEXICO — MEXIQUE	EL COTO	GARCIA	PG	I	?	?	?	?	?	1903	1984 (619)	
					21	?	14	L	?	?		
	COINTZIO	GRANDE DE MORELIA	TE	I/C/H	?	?	?	?	?	1939	1984 (621)	
					46	600	84	V	600	1964		
	JOSE TRINIDAD FABELA	LAS LAJAS	TE	I	?	?	?	?	?	1945	1984 (621)	
					22	104	6.5	L	125	1960		
	ALVARO OBREGON (EL GALLINERO)	TENASCO	PG	I/S	?	?	?	?	?	1946	1984 (621)	
					260	3	13	L	550	1980		
	DANXHO	COS OMATE	TE	I/C	?	?	?	?	?	1949	1984 (622)	
					32	395	34	L	240	1976		
	SOLIS	LERMA	TE	I/C	?	?	?	?	?	1949	1984 (623)	
					57	3084	1217	V	?	1982		
	CEJA DE BRAVO	CEJA DE BRAVO	TE	I	?	?	?	?	?	1955	1984 (624)	
					14	?	4.5	L	100	1979		
	MIGUEL HIDALGO (EL MAHONE)	RIO FUERTE	TE	I/C/H	?	?	?	?	?	1956	1984 (624)	
					86	10200	3290	L/V	16450	1964		
	SAN PEDRO PIEGRA GORDA	SAN PEDRO	PG	I	?	?	?	?	?	1957	1984 (624)	
					31	22	2	L	250	1981		
	OJO DE AGUA	TEPEHUAGE	TE	I	?	?	?	?	?	1958	1984 (625)	
					20	340	2.9	L	14	1978		

ICOLD / CIGB   DAM HEIGHTENING / SURELEVATION DE BARRAGES

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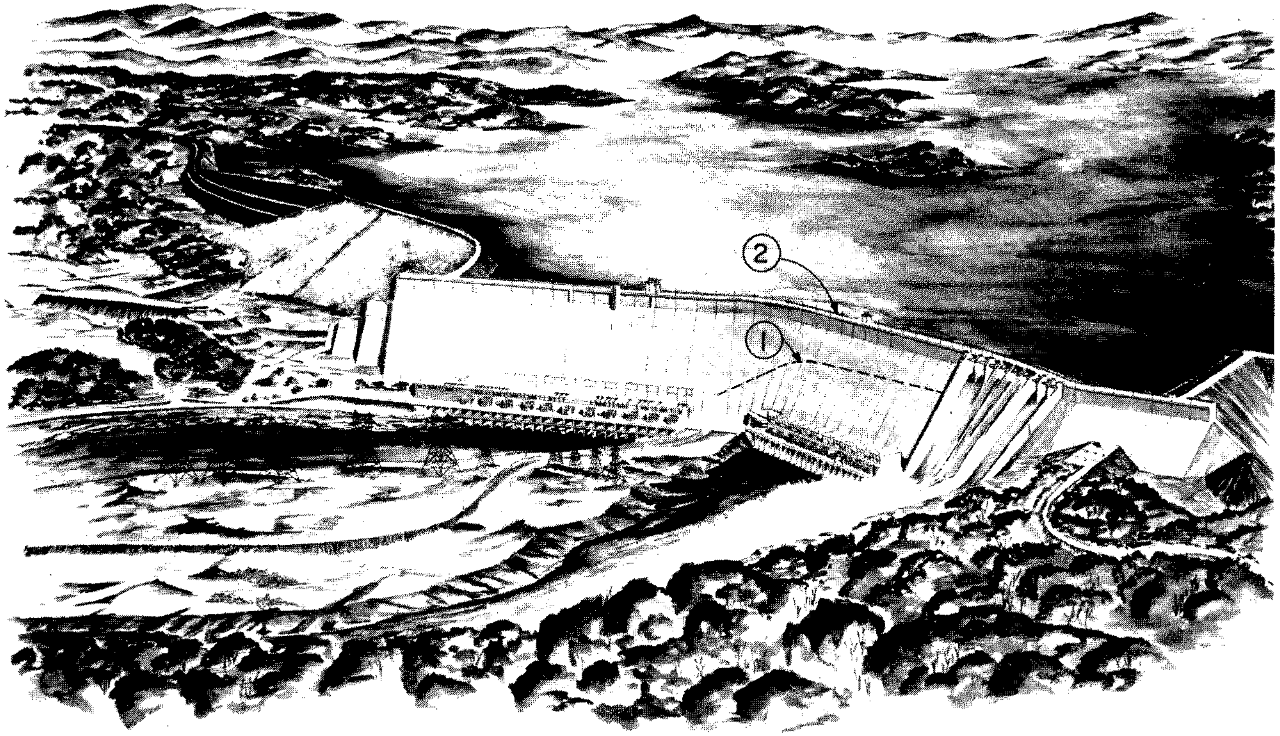
COUNTRY / PAYS	NAME / NOM	RIVER / COURS D'EAU	TYPE	PURPOSE / DESTINATION	DAM HEIGHT / HAUTEUR (m)	DAM CONTENT / VOL BARRAGE ($10^3 m^3$)	RESERVOIR (hm^3)	SPILLWAY TYPE / TYPE EVACUAT	SPILL CAPACI / CAPACITE EVA (m^3/s)	YEAR / ANNÉE	WRD / RMB	BIBLIOGRAPHY / BIBLIOGRAPHIE / REMARKS / OBSERVATIONS
MEXICO / MEXIQUE	PEÑA BLANCA	SANTOS O TEPOZAN	TE	I	?	?	?	?	?	1959	1984	
					32	124	4.5	L	240	1970	(625)	
	LA CALERA	RIO DEL ORO	TE	I	?	?	?	?	?	1962	1984	
					32	606	66	L	5100	1968	(627)	
	EL CINCO	ARROYO LOS BUEYES	TE	I	?	?	?	?	?	1969	1984	
					12	101	2500	L	610	1975	(631)	
	CHIHUILA	CHIHUILA	PG	I	?	?	?	?	?	1970	1984	
					48	7	6	L	60	1975	(631)	
PARAGUAY / PARAGUAY	ACARAY SUPERIOR (YGUAZU)	YGUAZU	ER	H	38.5	931	4080	L	5300	1976	1984	
					42	?	?	?	?	?	(659)	
ROMANIA / ROUMANIE	DRAGOMIRNA	DRAGOMIRNA	TE	S/I	14	?	3.4	?	?	1968	1984	
					30	2400	22	L	140	?	(673)	

ICOLD / CIGB

DAM HEIGHTENING / SURELEVATION DE BARRAGES

HEIGHTENINGS REPORTED IN THE WORLD REGISTER OF DAMS = NON - MEMBER COUNTRIES
SURELEVATIONS MENTIONEES DANS LE REGISTRE MONDIAL DES BARRAGES = PAYS NON - MEMBRES

COUNTRY / PAYS	NAME / NOM	RIVER / COURS D'EAU	TYPE	PURPOSE / DESTINATION	DAM HEIGHT / HAUTEUR (m)	DAM CONTENT / VOL BARRAGE ($10^3 m^3$)	RESERVOIR (hm^3)	SPILLWAY TYPE / TYPE EVACUAT	SPILL CAPAC / CAPACITE EVA (m^3/s)	YEAR / ANNÉE	W R D / R M B	BIBLIOGRAPHY / BIBLIOGRAPHIE / REMARKS / OBSERVATIONS
ANGOLA / ANGOLA	CAMBAMBE	CUANZA	VA	H	68	200	20	L	7000	1963	1984	
					87.5	?	?	?	?	?	(729)	
GUYANA / GUYANE	SAND-LADING	MAZARUNI	ER	H	55	?	4	L	2000	?	1984	
					65	2700	40	V	2000	?	(738)	
PAPUA / PAPOUASIE	SIRINUMU	LALOKI	ER	H	25	?	?	?	?	1963	1984	
					32	?	333	L	200	1971	(745)	



Rendu d'architecte de l'aménagement de Guri (après surélévation).
Architect's rendering of Final Stage of Guri Projet (after heightening).

Une importante surélévation de barrage parmi les plus récentes,
celle du Barrage de Guri sur la rivière Caroni (Venezuela)
— la hauteur au-dessus de la fondation passe de 110 à 162 m
— la retenue devient la 7e plus grande au monde (135 km^3)

One of the major recent heightenings of dams,
Guri Dam on the Caroni River (Venezuela)
— *height above foundation increased from 110 to 162 m*
— *reservoir is the 7th man-made lake in terms of capacity (135 km^3)*

SIGNIFICANT DATA OF GURI PROJECT

	1st Stage (1968-78)*	Final Stage (1983-86)*
RESERVOIR		
Catchment area	85 000 km^2	85 000 km^2
Minimum operating level elev.	195 m	240 m
Normal water level elev.	215 (1)	270 (1)
Maximum water level elev.	219.50 (2)	271 (2)
Area at normal water level	765 km^2	4 250 km^2
Volume at normal water level	17 km^3 (3)	135 km^3 (3)
Volume at minimum water level	5.9 km^3	49.6 km^3
Probable maximum flood	48 100 m^3/s	48 100 m^3/s
CONCRETE DAMS		
Type of dam	Gravity	Gravity
Roadway elev.	220 m	272 m
Parapet elev.	220.91 m	273.30 m
Height above foundation	110 m	162 m
Height above riverbed	100 m	152 m
Length of the right gravity dam	483 m	1 073 m
Length of the left gravity dam	179 m	169 m
Downstream nominal slope	1/0.75	1/0.75
Crest width of the right gravity dam	2.50-16.25 m	2.50-21 m
Crest width of the left gravity dam	3 m	Varies 11.45 m max.
Maximum depth of the grout curtain	75 m	115 m
Concrete volume	1 127 000 m^3	5 280 000 m^3
SPILLWAY		
Type of spillway	Creager Profile	Creager Profile
Type of gates	Radial	Radial
Crest elev.	195.20 m	250.20 m
Gate size (w x h)	15.23 × 20.76 m (1)	15.24 × 20.76 m (1)
Capacity at normal water level	27 000 m^3/s	27 000 m^3/s
Maximum capacity	40 000 m^3/s	30 000 m^3/s
Concrete volume	327 000 m^3	746 000 m^3
Length	183.76 m	183.76 m

État des travaux en août 1984.

Works in process in August 1984.

RIGHT EARTH AND ROCKFILL DAM		
Crest length	220 m	4 000 m
Maximum height above foundation	90 m	97 m
Crest elevation	221.30 m	277 m
Crest width	12 m	11 m
Upstream slope	1/2.5	1/3
Downstream slope	1/1.75	1/2.5
Total volume	2 089 000 m^3	47 430 000 m^3
LEFT EARTH AND ROCKFILL DAM		
Crest length		2 000 m
Maximum height above foundation		102 m
Crest elevation		276 m
Crest width		11 m
Upstream slope		1/3
Downstream slope		1/2.5
Total volume		21 700 000 m^3
POWERHOUSE No. 1		
Number of units	10	10
Rated capacity per unit	180 to 370 MW	218 to 400 MW
Total installed capacity	2 660 m	3 005 MW
POWERHOUSE No. 2		
Number of units		10
Rated capacity per unit		610 MW
Total installed capacity		6 100 MW
POWERHOUSES No. 1 + No. 2		
Number of units		20
Total installed capacity		9 105 MW
Average output		50 TWh (4)
POWERHOUSE No. 3 in 2050		
Planned extension		10 000 MW
POWERHOUSES No. 1 + No. 2 + No. 3		
Total installed capacity (planned)		20 000 MW

(*) 1st date : first units in service
2nd date : all units in service
(1) 1.50 m of gates extension is not included
(2) Operation of fusible dike starts
(3) 1 km^3 = 10^9 m^3
(4) 1 TWh = 10^9 kWh

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Par pure coïncidence, ce Bulletin est édité au moment où se tient pour la 2e fois aux États-Unis un Congrès de la CIGB (San Francisco - juin 1988). Il y a trente ans, la surélévation des barrages était traitée pour la première fois dans un Congrès : c'était à New York en 1958.

Incidentally, this Bulletin is issued at the same time as an ICOLD Congress is held for the second time in USA (San Francisco - June 1988). Thirty years ago, dam heightenings were dealt with, for the first time in an ICOLD Congress : it was in New York in 1958.