INTERNATIONAL COMMISSION ON LARGE DAMS
COMMISSION INTERNATIONALE DES GRANDS BARRAGES
151, boulevard Haussmann - 75008 Paris - France
Téléphone : (33) 01 53 75 16 52 - Fax : (33) 01 40 42 60 71
http://www.icold-cigb.org./

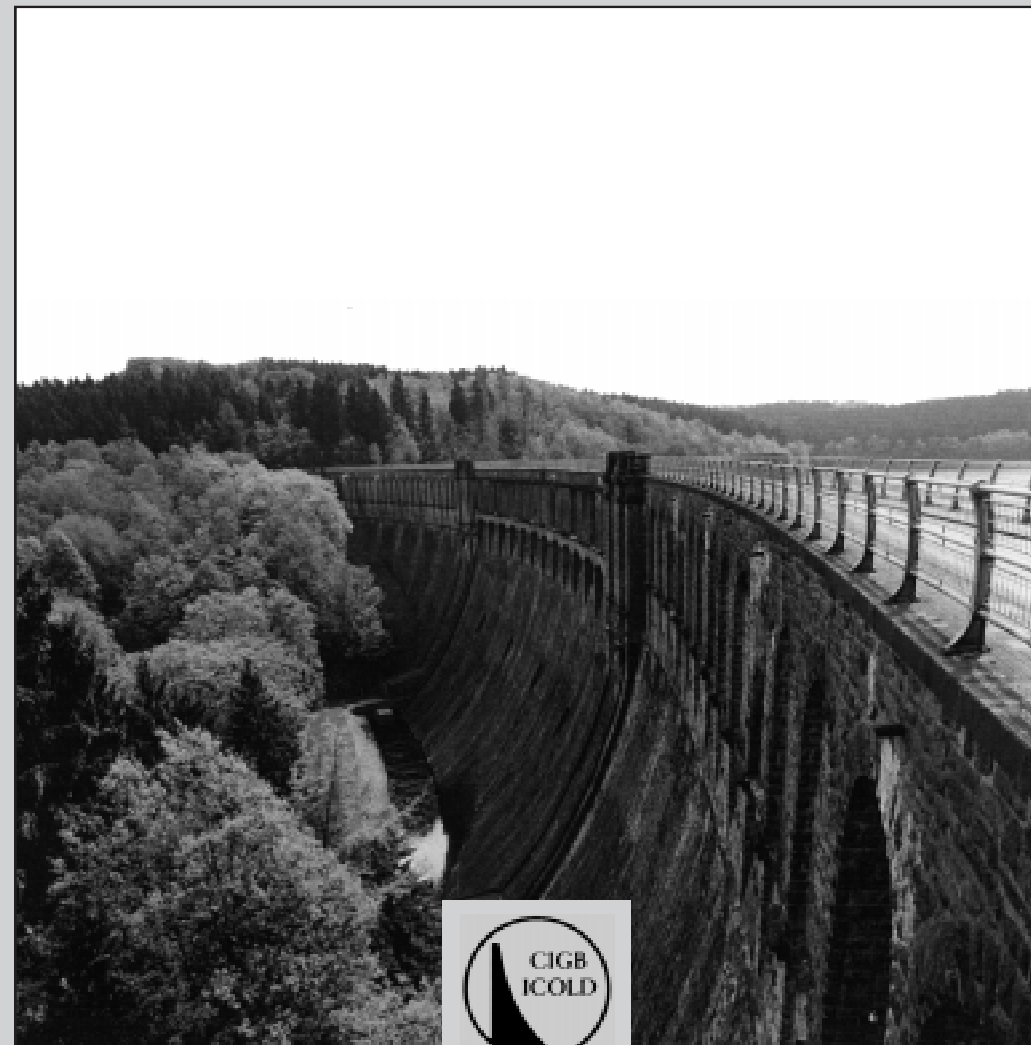
REHABILITATION OF DAMS
AND APPURTENANT WORKS

119

REHABILITATION OF DAMS AND APPURTENANT WORKS

State of the art and case histories

Bulletin 119



CIGB
ICOLD

2000

REHABILITATION OF DAMS AND APPURTENANT WORKS

State of the art and case histories

Commission Internationale des Grands Barrages - 151, bd Haussmann, 75008 Paris
Tél. : (33-1) 53 75 16 52 - Fax : (33-1) 40 42 60 71
E-mail : secretaire.general@icold-cigb.org - Site : www.icold-cigb.org.

COMMITTEE ON REHABILITATION OF DAMS
COMITÉ DE LA RÉHABILITATION DES BARRAGES
(1994-2000)

Chairman/Président	
United Kingdom/Royaume-Uni	G.P. SIMS
Vice-Chairmen/Vice-Présidents	
Germany/Allemagne	P. RISSLER
Portugal/Portugal	J. OLIVEIRA PEDRO
Sub-Committee on management of rehabilitation/ Sous-Comité de la gestion de la réhabilitation	
Austria/Autriche	W. FLOEGL (coordinator/coordinateur)
Sub-Committee on concrete and masonry dams/ Sous-Comité des barrages en béton et en maçonnerie	
Germany/Allemagne	P. RISSLER (coordinator/coordinateur)
Portugal/Portugal	J. O. PEDRO
France/France	D. BISTER
Italy/Italie	L. TOMASI
Sub-Committee on embankment dams/ Sous-Comité des barrages en remblai	
Switzerland/Suisse	M. BALISSAT (coordinator/coordinateur)
USA/États-Unis	R. W. KRAMER
Finland/Finlande	J. PYYNY
Sweden/Suède	K. CEDERWALL
Sub-Committee on appurtenant works/ Sous-Comité des ouvrages annexes	
Australia/Australie	P. J. CUMMINS (coordinator/coordinateur)
Japan/Japon	Y. YAMAMOTO (1)
	A. NIWA (2)
Czech Republic/République Tchèque	L. SATRAPA
Spain/Espagne	M. GOMEZ LAA (3)
	J. L. UCEDA (4)
Sub-Committee on research/ Sous-Comité de la recherche	
Sweden/Suède	K. CEDERWALL (coordinator/coordinateur)
Portugal/Portugal	J. O. PEDRO
Other Members of the Committee/ Autres Membres du Comité	
Cameroon/Cameroun	F. G. ELOUGA (5)
Canada/Canada	M. DESROCHERS (3)
	H. JOBIN (4)
China/Chine	Jinsheng JIA (4)
Iran/Iran	R. BANIHASHMI (6)
	N. KHEYRKHAH (7)
	M. NOROUZI (8)
Syria/Syrie	B. AL-DAGHER

(1) Member until 1999/Membre jusqu'en 1999
(2) Member since 1999/Membre depuis 1999
(3) Member until 1996 /Membre jusqu'en1996
(4) Member since 1996/Membre depuis 1996
(5) Member since 1998/Membre depuis 1998
(6) Member until 1995/Membre jusqu'en 1995
(7) Member since 1995/Membre depuis 1995
(8) Member since 1997/Membre depuis 1997

SOMMAIRE

CONTENTS

TABLE DES MATIÈRES

TABLE OF CONTENTS

LIST OF FIGURES

INDEX OF DAMS

AVANT-PROPOS

Il y a plus de 40 000 grands barrages dans le monde. En 1950, il y en avait 20 000. Actuellement, au moins 20 000 barrages sont donc âgés de plus de 50 ans. Cette situation nous incite à traiter de la réhabilitation, le problème s'amplifiant dans le temps.

Certains de ces barrages, en fait beaucoup d'entre eux peut-être, furent construits dans des circonstances difficiles, les ressources disponibles étant insuffisantes, et la population ayant un immense besoin d'eau pour l'irrigation, l'alimentation en eau potable, l'hydroélectricité, ou la maîtrise des crues.

Une conception et une construction défectueuses sont fertiles en travaux de réhabilitation. On peut être frappé par la comparaison entre deux barrages en maçonnerie situés dans la même région, disons à 100 km environ de distance. L'un est manifestement en mauvais état et n'est âgé que de 20 ans. Par contre, l'autre est un excellent spécimen âgé de plus de 50 ans. Il est évident que les maîtres d'ouvrage ne peuvent apprécier la nécessité d'engager une équipe avec des connaissances profondes sur les principes de conception et de construction impliqués. Je peux y ajouter une sous-catégorie relative au mauvais entretien, un entretien étant souvent défectueux non seulement parce que la gestion manque de fonds ou de compétences techniques, mais du fait que le projet n'a pas prévu comment l'entretien devait être effectué.

Des travaux de réhabilitation peuvent résulter également de progrès dans les connaissances. Nous évaluons mieux aujourd'hui que dans le passé les magnitudes des séismes et les débits des crues, ainsi que leurs effets. Mais il s'agit là de domaines spéciaux qui ne sont pas traités spécifiquement dans le présent Bulletin.

Nous avons sélectionné des exemples pour illustrer les moyens modernes et innovateurs de réhabilitation des barrages. Nous avons choisi des exemples où les données sont facilement accessibles aux ingénieurs souhaitant plus d'informations.

Le Bulletin s'adresse à tous ceux intervenant dans le projet, la construction ou l'entretien de barrages et de leurs ouvrages annexes, et en particulier aux responsables du maintien de leur sécurité et de leurs fonctions de fourniture d'eau ou d'énergie, ou de maîtrise des crues.

Le Bulletin a été préparé par le Comité de la Réhabilitation des Barrages. D'intéressantes contributions ont été reçues d'un grand nombre d'ingénieurs et d'organismes incluant Brown & Root Services, WSP International, Balfour Beatty Projects & Engineering et le Comité National du Royaume Uni qui a apporté son appui au travail du Président. Qu'ils en soient tous vivement remerciés.

Geoffrey P. Sims
Président du Comité de la Réhabilitation
des Barrages

FOREWORD

There are more than 40 000 large dams in the world. In 1950 there were 20 000. That means there are now at least 20 000 dams in the world more than 50 years old. This is the challenge we face in dealing with rehabilitation. The problem is growing.

Some of these, indeed perhaps many of them, were built under difficult circumstances, with inadequate resources available, the population in desperate need of the product be it water for irrigation or drinking, hydro electric power, or flood relief.

Poor design and construction are fertile sources of the need for rehabilitation. We may be struck by the comparison between two masonry dams in the same country, say about 100 km apart. One is obviously in bad condition and is only 20 years old. On the other hand the other is an excellent specimen over 50 years old. It is clear that owners may not appreciate the need for engaging a team with a deep understanding of the design and construction principles involved. I could add to this a sub category of poor maintenance, but so often maintenance is poor not only because the activity is not supported by management starved of funds or organisational skills, but because the design did not envisage how the maintenance was to be done.

Advancing knowledge is another reason for rehabilitation. We have better estimates today of the effects and magnitudes of earthquakes and floods than we did last year. But these are specialist areas and our Bulletin does not deal with them specifically.

We have selected case histories to illustrate modern, innovative means of rehabilitating dams. We have chosen examples where the data are readily available to engineers needing further information.

The Bulletin is addressed to those who are responsible for the design, construction or maintenance of dams and their related structures, particularly those responsible for maintaining them safe and productive of water or energy, or available for flood relief.

The Bulletin has been prepared by the ICOLD Committee on Rehabilitation of Dams. There has been a significant contribution from a large number of individual engineers and organisations including Brown & Root Services, WSP International, Balfour Beatty Projects & Engineering, and the British National Committee supporting the work of the Chairman. To these and to the individuals who have given their time and expertise so generously, we owe a significant debt.

Geoffrey P. Sims
Chairman, Committee on Rehabilitation of Dams

1. INTRODUCTION

1.1 CONSIDÉRATIONS GÉNÉRALES

Le présent Bulletin a pour objet de rassembler l'expérience actuelle à travers le monde en matière de réhabilitation des barrages et de leurs ouvrages annexes. Il vise à être un guide pour les ingénieurs concernés par le projet et la mise en œuvre des mesures de réhabilitation de barrages.

La détérioration des barrages et de leurs ouvrages annexes peut constituer une menace pour la vie humaine et beaucoup d'attention a été portée par la CIGB et d'autres organismes sur la question de la sécurité des barrages et des retenues. Il faut, en particulier, signaler la riche source de données constituée par les rapports de la Question 65 « Vieillissement des barrages et méthodes de réparation » ($17^{ème}$ Congrès, Vienne, 1991), de la Question 68 « Évaluation et renforcement de la sécurité des barrages en service » et de la Question 71 « Détérioration des ouvrages d'évacuation des barrages » ($18^{ème}$ Congrès, Durban, 1994). Même lorsque la sécurité n'est pas la principale préoccupation, les pertes économiques sont parfois sévères. La récolte provenant des terres irriguées et la production hydroélectrique peuvent être réduites au point de mettre en péril l'économie d'un pays en voie de développement. La maîtrise des crues peut être moins efficace que prévu. Les ingénieurs de génie civil savent que les ouvrages rentrant dans le domaine de leurs activités ne sont durables que s'ils sont régulièrement entretenus.

On doit également prendre en considération les besoins urgents, dans certains pays en voie de développement, de renforcement institutionnel en vue de surmonter le manque d'intérêt de politiciens et de décideurs pour soutenir un entretien régulier. Une telle démarche est certainement plus efficace que la fourniture d'une aide coûteuse destinée à la réalisation de travaux de réhabilitation sur un ouvrage particulier qui commence à se détériorer. Ceci est un aspect de la réhabilitation dont l'examen a été très peu développé dans le présent Bulletin. Néanmoins, des procédés d'entretien de bonne qualité, correctement mis en œuvre, sont importants afin d'éviter d'engager des travaux de réhabilitation inconsidérés et coûteux.

La réhabilitation est nécessaire pour faire face à deux facteurs principaux. Budweg (1998) a défini ces facteurs comme suit : les facteurs matériels, d'une part, et les facteurs associés à l'évolution de la technologie, d'autre part. Les facteurs matériels contribuant au vieillissement comprennent :

- Dégradation causée par le temps et altération identique
- Usure du matériel résultant d'un mauvais usage ou du vieillissement
- Altération du comportement après de nombreuses années d'exploitation
- Dégâts dus à des phénomènes naturels, tels que crues, séismes ou glissements de terrain
- Dégâts résultant d'actes de vandalisme ou de guerre.

1. INTRODUCTION

1.1 GENERAL CONSIDERATIONS

This Bulletin seeks to encapsulate the current experience world wide in the rehabilitation of dams and their appurtenant structures. It is intended to be a reference for practising engineers who wish to design and manage rehabilitation projects for dams.

Deteriorating dams and their related structures can threaten life and much attention has been given by ICOLD and others to the safety of dams and reservoirs. Particular mention is made of the rich source of data found in the replies to Question 65 *Ageing of Dams and Remedial Measures*, 1991, Question 68 *Safety Assessment and Improvement of Existing Dams*, 1994, and Question 71 *Deterioration of Spillways and Outlet Works*, 1994. Even where safety is not the major issue, economic loss can be severe. Irrigated crop production or hydropower may be reduced to the extent that a developing country's economy can be at risk. Flood control might be less effective than it should be. As civil engineers we are aware that civil engineering works are only permanent if they are regularly maintained.

We must take regard too of the urgent need in some developing countries for institutional strengthening to overcome the lack of interest by politicians and policy makers to support regular maintenance. To educate the organisation with the responsibility for the infrastructure is certainly more effective than to provide expensive aid to undertake a single rehabilitation project on an individual structure that immediately starts to deteriorate. This is an aspect of rehabilitation that is covered only to a limited extent in this Bulletin. Nonetheless good quality maintenance procedures, faithfully executed, are important in combating the need for disruptive and expensive rehabilitation.

Rehabilitation is needed to counter two major factors. Budweg (1998) has defined these in terms of material factors on the one hand, and those associated with the evolution of technology. Material factors contributing to ageing include :

- Decay through weathering and similar degradation
- Wear of equipment through misuse and age
- Loss of serviceability after prolonged operation
- Damage from natural events including floods, earthquake or landslides
- Damage from vandalism and war.

L'évolution de la technologie comprend les aspects suivants :

- Maîtrise des défauts de construction résultant d'anciennes techniques de conception et de construction. Un exemple est notre perception plus claire du phénomène de sous-pression et de ses effets sur la stabilité des ouvrages. Cela ne constitue pas une critique à l'égard de nos prédécesseurs qui ont appliqué les techniques en vigueur à leur époque.
- Maîtrise des effets résultant de changements dans les conditions d'exploitation, pour une retenue par exemple.
- Meilleure compréhension du mécanisme des changements météorologiques, conduisant à une meilleure prévision des débits de crues et à un meilleur dimensionnement des évacuateurs de crue, par rapport à ce qui était réalisable il y a peu de décennies ; de même, meilleure compréhension des séismes et de leurs actions.
- Prise en compte des changements survenus à l'aval d'un barrage, où le développement et la population sont plus importants que dans le passé.
- Nous connaissons mieux le comportement physique des barrages, et des mesures de réhabilitation sont parfois nécessaires pour satisfaire aux prescriptions réglementaires définissant des critères de fiabilité ou de sécurité acceptables, à la lumière des progrès techniques réalisés.

Le présent Bulletin traite principalement de la maîtrise des effets matériels du vieillissement, mais ne passe pas sous silence les autres aspects.

Les projets actuels de barrages, qu'ils soient de grandes ou petites dimensions, font l'objet, à juste titre, d'une analyse rigoureuse technique et environnementale. La plupart de ces barrages sont bien conçus et bien exploités, et fournissent à la société d'immenses bienfaits. Il importe donc aux maîtres d'ouvrage de maintenir la valeur investie dans le barrage pour les générations futures. Cela est particulièrement un défi lorsque les ressources sont rares et la compétition sévère dans ces circonstances. Le vieillissement des barrages conduit les maîtres d'ouvrage à considérer diverses solutions : réparer, améliorer ou abandonner le barrage. L'éventualité d'un abandon ou d'une mise hors service est fortement contestée par ceux dont les vies sont affectées. Parmi les raisons d'une telle contestation, il y a les importants motifs financiers pour un maintien des bénéfices provenant de l'ouvrage : emploi de main-d'œuvre locale, eau potable, protection contre les crues, production des terres irriguées, ou hydroélectricité. La réhabilitation constitue la solution préférée et le présent Bulletin examine les questions relatives à la conception et à l'exécution de ces travaux.

L'important problème de la sécurité est traité spécifiquement dans d'autres Bulletins de la CIGB. Le présent Bulletin est incontestablement différent. Les Bulletins relatifs à la sécurité des barrages traitent de la nécessité de mesures de réhabilitation, alors que le présent Bulletin se concentre sur la méthodologie des mesures de réhabilitation proprement dites. Un motif important de développer notre capacité et notre compétence dans la technique de réhabilitation des barrages est le maintien ou l'amélioration de la sécurité de ces ouvrages.

Dans un souci de simplification, nous avons établi une distinction entre des termes tels que réparation, travaux de correction, réhabilitation et amélioration, certains d'entre eux étant utilisés de façon confuse dans la littérature.

The evolution of technology includes the following :

- Overcoming construction deficiencies resulting from designs and construction of the former state of the art. An example is our clearer appreciation of uplift pressure and how it affects the stability of structures. This represents no criticism of our ancestors who were working at their state of the art.

- Overcoming the effects of changes in operating conditions, for a reservoir for example.
- Accounting for our improved understanding of the mechanics of weather changes, hence our better ability to predict the magnitude of floods and spillway capacity than was possible even a few decades ago. In a similar vein, to account for our better understanding of earthquakes and their behaviour.

- Accounting for changes downstream of a dam where perhaps there is more development and people now than formerly.
- Our present understanding of the physics of dam behaviour is now improved and rehabilitation is sometimes necessary to meet legislative requirements defining acceptable reliability or safety criteria in the light of technical advances in design.

This Bulletin is more concerned with countering the material effects of ageing but does not ignore the other aspects.

Modern dam projects whether of large or small scale are rightly subjected to rigorous technical and environmental analysis. The great majority are well designed and operated and provide society with powerful benefits. They provide large environmental benefits. An important aim for owners is therefore to maintain the value of the investment in the dam for succeeding generations. This is a particular challenge when resources are scarce and competition for them is severe. As dams age, their owners will consider their options : to repair, upgrade or abandon ? The possibility of abandonment or decommissioning is often strongly challenged by those whose lives are affected. Among the reasons for this are the powerful financial incentives in maintaining the output from the infrastructure whether it is local employment, potable water, flood protection, irrigated crop production or hydroelectric power. Rehabilitation becomes the favoured option and this Bulletin addresses the issues encountered in planning and carrying out these works.

The important issue of safety is dealt with specifically in other ICOLD Bulletins. This Bulletin is however, different in a decisive sense. Bulletins on dam safety deal with the need for rehabilitation while this Bulletin focuses on the methodology of rehabilitation itself. A major incentive however, for developing our capacity and competence in the technology of dam rehabilitation is to maintain or improve dam safety.

We have sought to simplify communication by distinguishing between terms such as repair, remedial works, rehabilitation and upgrading, some of which are used confusingly in the literature.

Entretien (**Maintenance**, en anglais) : travaux requis pour le maintien de l'installation en bon état de fonctionnement. Il comprend les réparations.

Réhabilitation (**Rehabilitation**, en anglais) : est synonyme de mesures de correction. Il s'agit de travaux limités, nécessaires pour que l'installation retrouve l'espérance de longévité de fonctionnement qu'elle avait lors de sa mise en service. Les travaux de réhabilitation ne sont pas conçus pour améliorer les performances mais pour atteindre l'objectif spécifique des mesures de correction – une telle amélioration pouvant cependant résulter de ces mesures – ; la réalisation d'ouvrages nouveaux ou l'installation de matériel nouveau sont limitées à cet objectif.

Amélioration (**Upgrading** ou **Uprating**, en anglais) : Ce sont les travaux considérés comme nécessaires pour maximiser les bénéfices de l'installation existante. Des travaux de construction ou équipement nouveaux sont réalisés lorsqu'ils peuvent être justifiés économiquement.

1.2 STRUCTURE DU BULLETIN

Le Bulletin contient cinq chapitres à la suite du chapitre « Introduction ».

1.2.1 Gestion de la réhabilitation

Le premier de ces chapitres traite des aspects de la gestion des travaux de réhabilitation. Une caractéristique de tels travaux est la difficulté de définir avec précision l'étendue des travaux, et donc de maîtriser leur coût et leur durée. On examine l'optimisation économique de la réhabilitation en tenant compte de la valeur économique de l'installation existante et du niveau de perturbation des fonctions qui lui sont assignées. Il peut être préférable, sur le plan économique, d'améliorer les ouvrages plutôt que de rétablir simplement leurs performances d'origine. La réhabilitation est identique à n'importe quelle autre activité de construction, en ce sens que la solution économique optimale est recherchée à l'intérieur de limites acceptables de sécurité et de risque. Cela est obtenu en comparant les coûts et les bénéfices des diverses solutions, y compris celle consistant à ne rien faire. Une connaissance de la précision des estimations de coût des travaux de réhabilitation revêt ici une importance capitale.

Le but d'une gestion satisfaisante des mesures de réhabilitation est essentiellement de parvenir à une solution technique acceptable, en évitant de se trouver par surprise devant un coût de réhabilitation supérieur à celui budgétisé, ou devant une durée de travaux plus longue que prévu. Une réhabilitation comprend trois phases principales : les reconnaissances initiales, l'étude de faisabilité et l'exécution. La gestion de l'opération cherchera à vérifier qu'une approche pragmatique est suivie, dans laquelle les décisions sont prises à partir d'informations solides et à un coût acceptable.

Il importe d'associer étroitement le niveau des fonds affectés aux travaux de réhabilitation avec la qualité de l'estimation de leur coût final. Le Tableau ci-dessous illustre l'approche, les données utilisées étant très arbitraires. La troisième colonne donne le niveau de confiance. Ce concept représente la confiance avec laquelle on

Maintenance : the work required to keep the installation in working order. It includes repair.

Rehabilitation : is synonymous with remedial measures. It is the limited work needed to restore to the installation the reliable life expectancy it had when it was new. Rehabilitation work is not planned or designed to enhance performance except as a consequence of meeting the specific goal of the remedial measures and new construction or equipment is provided only to the extent necessary to meet this goal.

Upgrading : is synonymous with uprating. It is the work considered necessary to maximise the benefit of the existing installation. New construction or equipment is installed where it can be justified economically.

1.2 STRUCTURE OF THE BULLETIN

The Bulletin contains five chapters following this introduction.

1.2.1 Management of Rehabilitation

The first of these chapters introduces aspects of the management of rehabilitation work. A characteristic of such works is the difficulty in defining precisely the scope of the work, hence controlling its cost and time scale. We discuss the economic optimisation of the rehabilitation through consideration of the economic value of the existing installation and the extent to which it is not performing its designed function. It may be better value for money to upgrade the works rather than simply to restore its originally designed performance. Rehabilitation is the same as any other construction activity in the sense that we are seeking the economic optimum solution within acceptable boundaries of safety and risk. This is achieved by comparing the costs and benefits of alternative solutions, including the one of doing nothing. Of fundamental importance here is knowledge of the accuracy of the estimates of cost of the rehabilitation work.

The aim of the successful management of rehabilitation is essentially to achieve a safe and acceptable technical solution while avoiding being taken by surprise either by the rehabilitation costing more than was budgeted or by taking longer. There are three major phases of a rehabilitation project namely the initial reconnaissance, the feasibility study and the implementation. The management of the process will seek to ensure that a pragmatic approach is followed in which decisions are made on sound information and at an acceptable cost.

It is important to link the level of funds committed to a rehabilitation project strictly to the quality of the estimate of its final cost. The Table below illustrates the approach using largely arbitrary data. The third column is the confidence level, defined as 100 % minus the claimed accuracy of the estimate. This is a concept that alloue de l'argent à l'opération. L'expérience montre que les indications suivantes peuvent être utiles.

Phase de développement	Erreur possible dans l'estimation du coût (%)	Niveau de confiance
Examen préliminaire du problème	+/– 100	0
Reconnaissances initiales	+/-50	50
Étude de faisabilité	+/-30	70
Estimation du marché de réhabilitation	+/-10	90
Achèvement des travaux	0	100

Le Tableau indique comment les phases d'études permettent d'augmenter le niveau de confiance dans le coût de l'opération. Ainsi, la phase des reconnaissances initiales donne peut-être une confiance de 50 % pour, par exemple, une dépense de 1 % par rapport au coût de l'opération.

Le Chapitre 2 souligne les divers points sur lesquels le chef d'un projet de réhabilitation doit porter une grande attention :

- Préqualification des soumissionnaires. Des compétences techniques et de gestion très développées doivent être requises et la tentation de faire appel à une entreprise locale, à un coût moindre, fera l'objet d'un examen approfondi.
- Choix du type de contrat le plus approprié.
- Mesures pour réduire les perturbations causées par les travaux de réhabilitation. Cela peut résulter d'une moindre utilisation de l'installation ou des impacts environnementaux causés par les travaux de réhabilitation.
- Le contrat doit être clair au sujet de la formation professionnelle et du renforcement institutionnel. Les changements concernant l'organisation doivent être appropriés.

1.2.2 Réhabilitation des barrages en béton et en maçonnerie, des barrages en remblai et des ouvrages annexes

Les trois chapitres suivants traitent des techniques de réhabilitation des barrages en béton et en maçonnerie, des barrages en remblai et des ouvrages annexes. Chaque chapitre a la même forme de présentation. Nous avons adopté comme base 31 scénarios de vieillissement identifiés par la CIGB dans le Bulletin 93 « Vieillissement des barrages et des ouvrages annexes » (CIGB, 1994). Ils représentent un classement safisfaisant des phénomènes de vieillissement enregistrés dans l'importante base de données établie par la CIGB et publiée sous le titre « Détérioration de barrages et réservoirs » (CIGB, 1983). Lorsqu'un élargissement de ces scénarios est possible, cela est indiqué dans le texte.

Chaque chapitre est subdivisé en deux parties principales. La plus étendue traite des travaux de réhabilitation résultant du vieillissement. La plus courte concerne represents the confidence with which project proponents allocate money to it. Experience suggests that the following relationships may be helpful.

Stage of development	Potential error in the cost estimate, %	Confidence level
Initial realisation of the problem	+/– 100	0
Initial reconnaissance	+/-50	50
Feasibility study	+/-30	70
Rehabilitation contract estimate	+/-10	90
Completion of the work	0	100

The Table shows how study phases make it possible to increase the confidence level in the cost of the project. Thus the reconnaissance study gives perhaps 50 % confidence for say an expenditure of 1 % of the project cost.

Chapter 2 outlines those matters to which the manager of a rehabilitation project must give detailed attention. These include :

- Prequalification of tenderers. Highly developed technical and management skills are needed by the rehabilitation contractor and the temptation to use a local firm at a low price is one that should be tested rigorously.
- The selection of the most appropriate Form of Contract.
- Measures to reduce the disturbance caused by the rehabilitation work. This may be as a result of loss of use of the facility or the environmental impact of the rehabilitation works.

The contract must be clear on the requirement for training and institutional strengthening. Organisational changes might be appropriate.

1.2.2 Rehabilitation of Concrete and Masonry Dams, Embankments and Appurtenant Structures

The following three chapters deal with the practice of rehabilitation of concrete and masonry dams, embankments and appurtenant structures. Each follows the same format. Underlying the treatment we have followed is the structure of 31 ageing scenarios developed by ICOLD in Bulletin 93 'Ageing of Dams and Appurtenant Works' (ICOLD, 1994). These represent a reasonable working subdivision of the ageing phenomena recorded in the extensive data base assembled by ICOLD and published under the title 'Deterioration of Dams and Reservoirs' (ICOLD, 1983). Where further development of these scenarios is possible, this has been attempted in the text.

Each chapter is subdivided into two major parts. The larger deals with rehabilitation work required because of ageing. The smaller covers rehabilitation la réhabilitation due à d'autres causes, que le Comité considère comme importante et présentant de l'intérêt pour les ingénieurs. La partie « vieillissement » comprend une section pour chaque scénario. Dans chaque section, le Bulletin traite de trois sujets importants. Le premier concerne la détection de la détérioration nécessitant une réhabilitation et les moyens permettant de contrôler l'évolution de cette détérioration. Le deuxième sujet est un rapport sur les conséquences relatives à la sécurité et à l'exploitation si la détérioration n'est pas arrêtée ou réduite par des travaux de réhabilitation. La troisième section est consacrée au choix des mesures de réhabilitation dont le succès a été démontré par l'expérience.

Le but du Bulletin est de constituer un guide pratique. On a donc recherché des exemples vécus représentant la technique actuelle en matière de réhabilitation, avec utilisation de concepts et matériaux modernes. Les mesures couronnées de succès ont été choisies à partir d'exemples provenant d'une nouvelle base de données établie par le Comité à cet effet. Un questionnaire de sélection a fourni environ 200 exemples de réhabilitation satisfaisant à trois critères principaux :1) ces exemples sont postérieurs à l'année 1975, la précédente base de données (CIGB, 1983) ayant été établie à partir de détériorations observées jusqu'en 1975 ; 2) ils font l'objet de rapports contemporains, de bonne qualité ; 3) ils font preuve d'innovation. La base de données est gérée en Grande-Bretagne par le « Building Research Establishment » (BRE) et il y a lieu d'examiner comment mettre cette base de données à la disposition des membres de la CIGB le plus commodément possible. Le BRE maintient la base de données sous une forme vivante.

1.2.3 Nécessité de recherche

Le chapitre 6 traite de la recherche et du développement continus dans le domaine de la réhabilitation des barrages, ce qui peut être classé en trois grandes catégories :

- Recherche pour améliorer la compréhension du comportement d'un barrage et des mécanismes conduisant à la détérioration.
- Recherche portant sur la grandeur et les effets des phénomènes naturels exceptionnels tels que les crues, les séismes et les températures extrêmes.
- Recherche orientée vers l'évaluation des performances et de la fiabilité des méthodes de réhabilitation actuellement disponibles.

Le but du chapitre 6 du Bulletin est d'identifier les domaines où la recherche est nécessaire au cours des années à venir.

required for other reasons that the Committee considers being important and of interest to engineers. The ageing part is divided into a section for each one of the scenarios. Within each section the Bulletin focuses on three major issues. The first is the detection of the deterioration leading to the need to rehabilitate and how its development is monitored. Second is an account of the consequences on safety and operation if the deterioration is not halted or mitigated through rehabilitation. The final section is a selection of rehabilitation measures that have been shown by experience to be successful.

The Bulletin is intended to be of practical help. Therefore we have sought examples from case histories that represent the current state of the art in rehabilitation, using modern concepts and materials. The successful measures are selected from examples chosen from a new database established by the Committee for this purpose. A selective questionnaire has provided about 200 rehabilitation case histories that have satisfied three major criteria : they have been put in place since 1975, the cut-off date for the earlier data base (ICOLD 1983) ; they are supported by good quality contemporary reports and finally, that they show innovation. The database is managed and held in Britain by the Building Research Establishment (BRE) and it is for debate how this should most conveniently be made available to ICOLD members. BRE are currently maintaining the data base as a live document.

1.2.3 Research Needs

Chapter 6 deals with the continuing research and development relevant to dam rehabilitation, which can be classified into three broad areas :

- Research to improve the understanding of the behaviour of the dam and the mechanisms leading to deterioration.
- Research into the magnitude and effects of extreme natural events including floods, earthquakes and extreme temperatures.
- Research directed to assessing the performance and reliability of the currently available rehabilitation methods.

The objective of Chapter 6 is to identify where research is required in the future.

2. MANAGEMENT OF REHABILITATION

2.1 GENERAL OBSERVATIONS

This chapter discusses the principles to follow in managing the process by which dams, barrages or associated works are rehabilitated. When the funding strategy permits it, a staged approach to the definition of the project is recommended. This has financial advantages for the owner because he spends only on necessary works and he has near-term targets with good technical and financial control. The owner's operational knowledge is important. The rehabilitation is programmed to deal first with critical work needed to make the structure safe or operable. Risk analysis is often helpful in determining the order of priority. The staged approach defines the works fully before a construction contract is awarded. The aim is to avoid surprises during a contract, which delay the work and increase its cost. Rehabilitation work started with insufficient investigation can run into difficulty.

The chapter has been written from the point of view of a consulting engineering firm with responsibility for managing a rehabilitation project. This is the role still carried out by consulting engineers for owners of ageing facilities, particularly where scarce resources have not been allocated to maintenance. However, as is apparent from the case studies reported in this Bulletin, many owners are fully competent to manage even large-scale rehabilitation works themselves. These often are the owners whose system is large enough to allow them to employ full time specialist staff to oversee the rehabilitation of their plants. They are in the fortunate position of collaborating closely with their colleagues responsible for operation, dam safety control and for setting the budget. The problems of managing the interfaces should be less demanding for them.

Probably the majority of rehabilitation is financed by external funding agencies and is undertaken by consultants. The key principles and the logic of the approach set out in this chapter are appropriate in all circumstances. However the comments on interface management will be of particular relevance to consultants and their clients.

The following Table illustrates a logical approach to a rehabilitation project that is based on the procedures in some European countries. Often the owner employs a competent engineer to carry responsibility for the safety of the dam. In some countries this is a mandatory requirement. Where the owner is technically competent there may be a more limited role for a Consultant or a Contractor. The remainder of the chapter describes some of these activities in more detail.

2.2 LEGAL AND ORGANISATIONAL ASPECTS

In some countries there are laws with regard to dam safety that are strictly enforced. In several European countries for example there is a legal requirement for the owner to appoint an individual, the *dam supervisor*, with personal responsibility

Implementer	Action required	Result
Legislature		Appropriate law and guidance
Owner and Consultant	Investigate condition of dam	Report on condition
Owner and Consultant	Preliminary studies, define the aims of rehabilitation, identify priorities	Initial Project Definition Report
Owner and Consultant	Investigate alternative methods of rehabilitation, collect data and assess technical and economic feasibility	Project Plan defining scope, funding and programme
Owner and Consultant	Inform Supervisory Authority	
Supervisory Authority		Approval of Project Plan if appropriate
Owner and Consultant	Design rehabilitation project Complete investigation of finance options Undertake tendering procedure	Project designed Develop a finance plan Appoint a Contractor
Contractor	Carry out rehabilitation work	Successful rehabilitation contract
Owner and Consultant	Supervise and monitor costs, work and time	Successful rehabilitation contract
Owner and Consultant	Review aims of rehabilitation and the extent to which they have been achieved	Production of Final Report
Owner and Consultant	Inform the Supervisory Authority	Completion

for the safe operation of the dam. Substitutes have to be identified in case the nominated individual is not available. In these circumstances there is usually a requirement too for the dam to be provided with monitoring equipment and that this should be read, and the results assessed, on a routine basis. The intervals between inspections may be specified and a full file of documentation and operating manuals be kept up to date. In many countries however there is no such requirement and the engineer who is called upon to investigate the need for rehabilitation has to work with a more limited data base. This chapter is primarily intended for those who find themselves in this position. Nonetheless it is considered helpful to set out the requirements that are frequently made in countries where the arrangements for dam safety are well established.

Thus, good practice and government regulation typically requires that :

- dams and reservoirs be provided with sufficient monitoring equipment to allow a basic assessment of the behaviour of the structure and its foundation ;
- the monitoring equipment must be kept in good condition ;

- measurement data should be routinely evaluated, comparing the readings with the long term behaviour ;
- a regular inspection of the dam, the reservoir and the area downstream should be made, and this should also include geological and hydrological aspects ;
- where unusual behaviour is noted, a call for an independent expert inspection be recommended.

In these circumstances the appreciation of the need for rehabilitation will be based on factual evidence. The following is a typical routine of inspections that will provide a good basis of understanding of the behaviour of the dam. Where there is active involvement of the state dam safety organisation such a routine is often a government requirement :

- annual safety review by the dam supervisor or independent experts ;
- a detailed overall inspection at an interval of five years to ten years carried out by independent experts ;
- a detailed inspection following severe incidents including severe flooding or earthquake ;
- a review of the hydrological and the hydraulic safety every ten years or so ; this might require a recalculation of the probable maximum flood and the design criteria for the spillway in the light of current national or international guidance ;
- risk analysis and risk assessment for management of risks within available resources is increasingly being done as good practice.

The owner should seek to assemble a complete set of documentation of the design and construction of the dam and its foundation. This should include design and material parameters as well as a record of events in the history of the dam. This data provides important information about the long-term behaviour, assisting with the decision as to whether rehabilitation is required, and for the design and construction of the rehabilitation measures.

Operating manuals should be prepared. These should not only contain instructions about operations of the structure and the emergency planning, but also the routine maintenance work and the programme of instrumentation reading.

A regular regime of inspecting and reporting, coupled with a maintenance programme to rectify deterioration, may reduce or even preclude the need for rehabilitation.

2.3 MANAGEMENT OF THE DESIGN OF REHABILITATION

2.3.1 Report on the Condition of the Dam

The need for rehabilitation will either come from a report from the owner of failing performance, or from the outcome of routine inspections referred to above.

In either event it is necessary to prepare a report on the condition of the dam. This will ideally have the following input :

- Reports of comprehensive inspections with the reservoir full and with it empty. Standard texts provide check lists of items to be addressed during the inspection.
- Precise records of the function of the controls of appurtenant works such as current consumption of electric motors, hydraulic pressures in servomotors, opening and closing times
- Remaining tendon load in prestressed structures
- Evaluation of instrumentation data
- Comparison of photographs taken over a range of time
- Study of the dam history
- The design and material parameters in the project documentation
- Review of the maintenance carried out and how this compares with the requirements of the Operating and Maintenance Manual.

The report on the condition of the dam should also refer to the way the scheme is operated and whether this has changed. It should deal with the incidents of floods and earthquakes, the extent of reservoir sedimentation and issues of environmental or legal importance.

An important conclusion of the report on the condition of the dam will be an assessment of where the dam or appurtenant works are in need of rehabilitation. Such a conclusion may be reached with the help of :

- Comparison of the condition of the structure with what it should be
- Comparison of the structure with others
- Physical or mathematical models of static and dynamic behaviour
- Risk analysis
- Further investigations as needed.

The outcome of the report, investigations and an assessment of risk will define which of three paths it may be appropriate to follow :

- Continue monitoring the structure, perhaps increasing the frequency of inspections or instrument readings
- Carry out additional studies and investigations to determine the extent of the problem
- Take decisive action immediately by, for example lowering the level of water in the reservoir.

This chapter deals principally with the second of the categories of response. It assumes that the additional investigations lead to the conclusion that rehabilitation is technically and economically justified. Much rehabilitation work is of a routine nature and is amenable to long term planning ; other rehabilitation is of a more immediate nature and the following section keeps this sort of work in focus.

2.3.2 Rehabilitation Plan

It is good practice to prepare a rehabilitation plan to define the aims of the rehabilitation, to establish priorities and the programme of work. This requires judgement and experience. Important will be the requirement to maintain the structures safe and operational. More detailed requirements may follow. These may include the limitation of seepage for example, or the repair of surfaces damaged by freezing and thawing. Risk analysis and assessment techniques are often appropriate at this stage as discussed below in Section 2.5.

The first step is to develop a practicable programme of investigation work. The full scope and programme for rehabilitation cannot be defined yet as the extent, and often the methods, are still unknown. The owner's experience is established through structured questioning of operating personnel. Specialists may have improved operating rules. Others may have studied the river basin and concluded that the use for the structure be modified. Hydrological studies may suggest an increase in spillway capacity, power studies may recommend more hydroelectric capacity or irrigation and water supply. The river morphology may have changed. Nearby projects built more recently may impact on the works.

Time is always short. A contractor working nearby could negotiate a contract to undertake urgent repairs or use their plant to solve an urgent problem and give time for deeper studies. An example of this is the **Kotri Barrage** Rehabilitation Project in Pakistan where some of the barrage gates were likely to fail before they could be replaced. There were no stoplog gates. An emergency procedure was developed for closing off one or more bays using pre-identified plant sources (Padgett et al 1998). At **Sennar Dam** in Sudan many of the sluice gates were in danger of jamming open due to failure of the original 70-year-old rocking roller tracks. Early rehabilitation of an isolating steel curtain allowed inspection of open sluices. At Kotri Barrage the glacis and downstream apron were damaged by uplift. All the original standpipe piezometers had failed and were replaced with vibrating wire piezometers to quantify the uplift and to formulate solutions. The engineer acted quickly to get data so as not to delay the project.

It is difficult often to take important hydraulic structures out of operation for long periods. Planning must account for operational constraints ; there may be an annual window when measurements can be made, and that failing to use the window can delay the work by a year. The owner and the financing agency may have to agree urgently the most practicable and expeditious way forward. They may agree to single source or price enquiry contracting for specific services for example.

This is a critical period for a rehabilitation project. In view of the complexity of problems, the need for lateral thinking when devising solutions, and the foresight needed to know what data will be required and when, the team must include experienced senior personnel. A brainstorm approach may be appropriate. A site inspection is essential.

A thorough search for data on construction and performance is always worthwhile. Consulting engineers often hold archive records on projects they have engineered. In the developing world paper records are usually stored rather than destroyed. The owner's records may be in remote stores but it is always worth digging deeply for them. Library searches by computer, particularly of the major engineering institutions and research establishments, lead to construction records and statistics in published papers.

Owners usually keep accurate records of information essential to daily operation including irrigation and power flows, water levels, heads and data on sediment movements. However, older instrument installations may not work and the structural behaviour of a dam is often not monitored for lack of equipment and trained staff. Leakage under or through a structure is difficult to quantify and locate. Qualitative records of wear and tear on machinery are often incomplete, although hydropower equipment is usually better documented.

Gathering data frequently involves installing instrumentation such as computer-read piezometers, residual thickness and working stress cells on critical structural elements, and undertaking a detailed survey of condition. For accurate assessment of complex steel hoist structures or bridges inspectors should touch every member and record its condition. Electronic thickness meters ensure that all weaknesses are quantified. On the Kotri Barrage for example engineers inspected each gate member and from this they developed a risk analysis, as a result of which gate replacement took priority over other rehabilitation works (Padgett et al 1998).

It is sometimes possible to assess the scale of rehabilitation work by sampling. This is particularly so on a series of barrages or multiple elements such as gates designed by one firm where important details may have been duplicated. However, it is not always possible to use the sampling technique. If the worst deterioration that could be discovered materially affects the rehabilitation programme or cash flow, then sampling alone is probably not sufficient. The experience of the team will determine whether major surprises could come from opening up each item for detailed inspection during rehabilitation.

The initial phase concludes with the rehabilitation report that identifies the objectives of the rehabilitation, the problems to be solved and the further studies required. It also includes a programme to show key decision dates and the critical path. It identifies the responsibilities for making decisions and the constraints to progress with recommendations for mitigating these.

2.3.3 Preliminary Option Studies and Technical and Economic Feasibility

Having examined the available data and understood the problems, the engineer undertakes general desk studies to identify the possible solutions to each of the problems. These are not engineered in detail at this stage. There may be only one solution, with alternative ways of going about it. At this level of study the engineer identifies the options and examines them from technical and economic points of view. Comparison is usually on the basis of benefit/cost ratios. The bases of costs and benefits are kept consistent from one option to another so that meaningful comparisons between them can be made easily. In this way are identified and excluded options that do not merit costly data collection.

The Feasibility Study follows much the same format as for new projects, except that it is more complex, hence it usually takes longer. The infrastructure generally has to remain in operation throughout the study period. An important decision is the extent of the work to be done. Should it be simply to repair the effects of ageing ? Or should the owner take the opportunity to rehabilitate the structure to the condition it was when new, or even to upgrade it. These decisions are simple in principle, depending on standard economic analysis of the streams of costs and benefits. In practice the decision is influenced by the amount of money and time available, and how the project now fits into the owner's entire system. Thus a study of the whole system is often needed comparing the operation of the system with the proposed upgrading with operation without it. The economic indicators reveal the preferred course.

2.3.4 Finance Plan

Integral with the Feasibility Study is to arrange the financing of the rehabilitation work. The source of funding depends on several factors. Should there be a definable and reliable income from the rehabilitated project, it may be possible to arrange private funding in which an investor takes an equity share in the project for an agreed period. Where the income stream is neither well-defined nor politically reliable, grants or loans from a funding agency become necessary. The financial arrangements depend on the nature of the rehabilitation project, and how it may be structured into fundable contracts without compromising the progress of the works.

Although one agency will usually take the lead, it may be helpful for the engineer to co-ordinate amongst them if more than one agency is funding different parts of the work. One contract must not be allowed to frustrate another by lack of performance.

2.3.5 Detailed Design

The following factors are important when carrying out the detailed design of rehabilitation work.

- Detailed consideration of construction techniques
- The effect on the existing structure of the methods of rehabilitation which might include grouting, prestressing or blasting for example
- The effect on the structure of using materials that have different properties to those of the original materials
- Monitoring the success in meeting the aims of the Rehabilitation Plan, and ensuring that this is documented
- Mitigation of environmental impact.

Often specialist contractors will provide particularly relevant and valuable information.

2.3.6 Project Plan

Discussions with the funders and owner will establish requirements, in particular :

- How the funding agencies relate to each other,

- Who is responsible for paying for extra work uncovered during the rehabilitation,
- The responsibilities of owner, consulting engineer and funding agencies,
- Interrelationships between contracts,
- Reporting and joint meeting requirements,
- A comprehensive suite of work programmes,
- Constraints on owner and funders approvals such as budget year, and financing committee meeting dates.

It is necessary to record and further define these matters in a Project Plan, which is endorsed by the parties.

2.4 MANAGEMENT OF CONSTRUCTION OF REHABILITATION

2.4.1 Contract Packaging

Effective contracts for rehabilitation work require the work to be defined as accurately as possible to avoid surprise as the contract proceeds. Experienced professional advice is essential to choose forms of contract that are equitable for both parties, minimising the points of conflict, and permitting the quick and fair agreement of additional work. Alternative forms of contract for rehabilitation work are discussed below.

The *bill-of-quantities* format published by the Fédération Internationale des Ingénieurs-Conseils (FIDIC, 1992) is popular. Contractors, both local and international, are familiar with the format and a wealth of legal procedure has been established to cover the interpretation of its clauses. The major lending agencies have developed unique forms of contract, or guidelines to modify the standard FIDIC clauses.

Cost-plus contracts seldom gain favour for rehabilitation because it is difficult to control costs, the work process, materials and staff required.

Target cost is a useful form of contract where the owner can benefit significantly from early completion, or lose significantly from delays. Formulae for sharing costs and benefits are built into the contract so that both parties share equally both costs and benefits should there be a deviation from the target cost or target completion date. Decisions must be made quickly for this form of contract to work and both the owner and the contractor must devolve considerable authority to their site personnel.

The *design-construct* form of contract may be appropriate where intimate co-operation between the constructor and the designer will benefit the project in time, cost or quality, or where the contractor has greater knowledge of required specialist designs than the consulting engineer. The designs must always be checked by the consulting engineer for detailed conformance with specified requirements, and this can take as long as the engineer doing the designs himself in the first place. There is usually no saving in engineering costs in these circumstances, but rather the reverse.

The *FIDIC re-measurable contract* can be used when time needs to be saved and a contract started before fully detailed designs are ready. The essential requirement is that the tenderers can prepare realistic tenders. This can usually be accommodated by broadening the bill item coverage and description and providing the contractor with sufficient details on the principal items of cost. Detailed designs are then provided later to match the agreed construction schedules. A site inspection is essential in this context. The unknown element of risk the contractor has to price tends to be less than in a design-construct form of contract. Clearly the Contractor has to price all the uninsured contractual risks and the owner has to weigh up the risks of extra costs arising.

Standard procedures are helpful for the valuation of variations and the analysis of claims. Contracts run into difficulties if variations are not promptly costed and approved. Thus the Engineer's Representative, Project Manager, the Engineer, and the owner in discussion with the funding agency are each authorised to approve additional expenditure to an agreed level. This reduces management time and satisfies the contractor's cash flow requirements without loss of control. Limitations of delegated authority should be incorporated in the contract documents.

Contractor's claims may be a source of friction and if not handled promptly and correctly, can impair the contractor's performance. Strict adherence to the source FIDIC procedures is necessary and the contractor is required to submit fully agreed and priced particulars of his claim at the earliest date. It is in his interests to do so and it enables the Engineer to make prompt decisions on verified facts. A Claims Adjudication Board comprising several independent specialist engineers may have merits in some circumstances.

Owing to seasonal constraints, rehabilitation projects often last several years. Progress is often slower than programmed. Sometimes this is linked to the Contractor's inability fully to appreciate the difficulties in starting a project in a remote developing country. Lack of familiarity with local import procedures, availability of materials, plant and labour law is common. Failure fully to complete mobilisation according to programme often has far-reaching effects on progress. Early delays may increase the volume of work that has to be done later to an extent that is impossible to achieve without major interventions and expenditure. This is disruptive and is to be avoided. Intermediate key completion dates can be established in the contract and justifiable penalties in the form of liquidated damages levied for failure to meet them. The more information about local conditions that is provided to the tenderers, the fewer will be the problems.

Rehabilitation contracts must be of appropriate size to attract appropriately experienced contractors. Too small and you attract an insufficient number of tenders, too large and you discourage the specialised firms whose skills may be needed. It is often recommended that for a critical work item with the potential seriously to interfere with or delay the rehabilitation, it may be advantageous for the owner to designate a specialist subcontractor.

Material and equipment in short supply or on long delivery is ordered ahead of the main contract. Typical examples are ordering steel for multiple gate replacement, ordering new hoist mechanisms, and stockpiling processed aggregates.

2.4.2 Contractor Identification

General

Careful structuring of pre-qualification documentation ensures that the Engineer, the owner and the Funding Agency receive the right information in sufficient verified detail to be able to make sound judgements as to which contracting companies are to be included in the tendering list. The comparative calibre of local and foreign contractors will decide which contracts to let to local firms to aid development of the country's skill base. Joint ventures between local and off shore contractors are favoured for two principal reasons. The local contractor brings intimate knowledge of the local business environment and government procedures while the international company brings project management and technical skills. The form of joint venture can be fully specified and the form of training for the local contractor defined in the contract.

Prequalification

For large international contracts, the preparation of prequalification documents is a significant task requiring careful structuring of the responses required of the tenderer. To ensure that the process is fair and transparent, the owner and Funding Agency must agree with the Engineer the items upon which prequalification is to be judged and the ranking procedure to be used. Weighting of items is done in advance. On prestigious contracts in particular, there can be considerable pressure brought to bear on the owner and Funding Agency. To further support the approach, the ranking procedure can be published with the prequalification invitation, emphasising that data submitted is to be fully verifiable.

There are three important points to consider when prequalifying tenderers for rehabilitation work :

- Do they have the financial stability and resources to complete the contract in addition to their ongoing work ?
- Do they have a proven record for successfully completing similar works ?
- Do they now have the necessary human skill base for the work ?

The owner and the Funding Agency share their qualitative judgement of the experience of each of the prospective tenderers with the Engineer. Formally structured, the evaluation report of the prequalification documentation is presented to the owner, with the Engineer's recommendation.

Tendering and Award Procedures

Tenderers need time and access to the available information to prepare a sound and competitive bid. Rehabilitation is often complex and tenderers need time properly to research the availability of resources in remote areas. Too short a tender period and the tender prices may be high. Failure on the part of a tenderer to adequately price these difficulties may win the owner a lower tender price, but the contract is in danger of being frustrated or at the least generating high administrative cost to handle disputes. Experienced contractors tend to provide more resources to prepare well argued claims, and achieve greater success. Less experienced contractors often present claims badly and have less success. Badly structured contracts tend to have problems. An adequate time for tendering, together with a compulsory site inspection means less chance of impracticably low tenders.

The consulting engineer generally assembles considerable relevant technical and statistical data during the project identification stage. This information affects the tender pricing. Such data should be made available to each tenderer.

Briefing the tenderers at the project site, and conducting explanatory site sessions reduces enquiries and administration during tendering. The discussion at each meeting and the decisions are recorded and circulated to each tenderer. Clarification by the tenderer of particulars in his tender is often necessary. It is important that this clarification does not give him an unfair advantage. Conditional tenders can trigger disqualification or modification of the tender price. What constitutes a condition and what is simply an item with cost implications on the tender price can be difficult to judge. It is therefore important that the principles behind such a situation are clarified with the owner and Funding Agency before tender documents are finalised.

It is sometimes suggested that for rehabilitation contracts, where the contractor has much to offer, that the questions from each tenderer be dealt with individually. Questions relating to errors in the tender documents should still be circulated to all tenderers.

Final negotiations are a matter for the owner and the potential contractor. However, having detailed knowledge of the contracts, the Engineer should be present. Agreements must ensure that the competitive tendering process is not invalidated, and should be covered by precisely worded and legally binding Supplemental Agreements to the contract.

The final task in the tendering process is to prepare a fully conforming contract, to circulate it and have it endorsed by all parties as the Working Document. The exchanges, clarifications, and agreements leading to contract award are assembled in a format, which enables the trail of events to be unambiguously understood.

2.4.3 Construction Management

General Philosophy

The aim is to ensure that the owner receives the product he expects and has paid for, while the contractors and suppliers receive contractually fair compensation for their endeavours. The term “the engineer” is used in this section in the context of the FIDIC Contract. The “engineer” represents the entity employed by the owner to manage the construction contract the owner enters into with the contractor. Local custom dictates whether this is a consulting engineer, a professional construction manager, or the owner himself.

Control Strategies

The engineer’s site manager is given a detailed design brief prepared by the designer outlining the design approach, design parameters, points requiring particular supervision and data to be returned to the head office to allow final design. He is briefed on the contract and required to comment on the contract during construction and at completion. As-built records are also required of the site manager, copies of which are archived.

The engineer needs regular submissions of supporting data from the contractor and needs to generate significant records himself. Experience is needed to anticipate problems as they arise and to arrange for the necessary data to be collected in good time. The site supervisory team must be organised so that every critical piece of permanent construction is witnessed and monitored. Each position in the team carries well-defined responsibilities. The team must always be led by a person with sound technical skills and contract administration experience.

Principal elements of work are covered by a system which records that the contractor and the engineers or inspectors have signed off the work before it is covered up or incorporated in the works. Site-testing procedures, such as non-destructive weld testing by category of weld, grout and concrete mix quality control, paint performance and commissioning procedures are specified in the contract. Equipment is often fabricated off site in other countries. Inspection of these items is handled either by the consulting engineer or by an accredited inspection agency on his behalf. Shipping should not be undertaken until the items are approved. Further inspections take place on arrival at the site to be sure that damage in transit is rectified as soon as possible.

Close financial control is needed to avoid delays in payment to the contractor. The valuation and certification of interim payments should be the responsibility of the engineer's site manager. The contractor's site manager should be similarly authorised to determine the value of interim payments. The owner receives monthly statements of progress versus expenditure, forecasts of routine expenditure he needs to cover such as interim payments, variations, claims and payment of retention monies. The approval of uncomplicated variations is the responsibility of the engineer's site manager while major variations and claims are the responsibility of the head office. Certification of interim payments to offshore suppliers is usually handled off site. Final payment certificates are the responsibility of head office.

Meetings provide a forum for clarification and for developing a spirit of co-operation. Formal monthly meetings between the owner, contractor and engineer following submission of a monthly report, and weekly meetings between engineer and the contractor, are usual. At least two meetings a year between the owner, Funding Agency and engineer are favoured, to be certain that the funding provisions will be adequate and not cause delay of the project. All meetings should be covered by pre-agreed agendas and minuted. It is better to avoid meetings without conclusions and allocated action plans.

Organisational

Continuity of knowledge and practice is particularly important on rehabilitation work. There are benefits if key staff remain with the project to completion. The make-up of the planning, design and construction management teams varies as the stages of rehabilitation unfold. At the project definition stage the team comprises specialists whose experience ensures that all practicable possibilities are considered.

At the design stage engineers, technicians and CAD operators are brought into the team, working through their discipline supervisors. When construction starts, the team undergoes a major change as the responsibility for day-to-day activities is centred on the site manager. The site manager, whether Engineer's Representative or Construction Manager, is vested with the necessary authority for securing proper performance from the contractor. He is selected accordingly. Many good site managers have worked as site agents for contractors and bring to their task a first hand knowledge of problem solving from the contractor's side.

An engineer's site organisation will vary from site to site, but will contain common features. On smaller jobs, duties may be combined but the structure is maintained. On overseas projects it is good practice to use local engineers and technicians where they have the necessary skills, and to include owner's staff in the site supervisory team as part of institutional building and training principles.

Contractor's Staffing

Contractors are required to outline their staffing structure and calibre of available personnel in their tenders. Shortcomings in their proposals are resolved during tender negotiations. The contractor's project manager and his office engineer, who often cover the daily construction planning function, are considered to be key personnel and are carefully vetted for suitability. Experience in contract management and social and governmental conditions similar to those to be encountered are advantageous. The contractor's superintendents for fabrication, erection and installation and plant maintenance must be well experienced in their field and are important for quality and progress control.

The contractor is encouraged to delegate to his site manager the authority he needs to manage site affairs. When the contractor's head office retains control it can lead to delays, labour problems and inefficient contract execution. The contractor's site manager should have the same level of authority and responsibility given to the engineer's site manager and be qualified for this role.

Client Organisation

It is essential to keep the owner informed on matters affecting timely completion and cost of the job ; quality is usually the engineer's responsibility and only in special circumstances is this discussed in detail with the owner. To achieve good communication the owner should set up a site organisation to monitor the contract and to assist to clear bottlenecks in which he may have special competence, such as customs payments, housing, licensing or access. On dams and barrages this may be achieved by strengthening the existing operation and maintenance organisation. The owner's site representative usually has responsibility for the welfare of staff seconded to the engineer but would not be responsible for their actions in supervising the contractor.

2.4.4 Operation, Maintenance and Handover

Where rehabilitation proceeds concurrently with operation, forward planning is necessary so as not to delay the Contractor and to allow smooth operation of the dam, and associated works. The contract documents should make the contractor aware of any operating conditions that may affect the work. Safety of the structures is paramount and always takes precedence.

Most power plants are connected to the grid. Hence alternative electricity supplies can be arranged when it is necessary to take generating equipment out of service for rehabilitation. Alternative arrangements for water supply or for irrigation are seldom practicable. The demands have to be met and have a major influence on the manner in which rehabilitation can be carried out.

Rehabilitation provides an opportunity to second owner's personnel to the contractor whilst complex plant is being installed so that they develop a deep knowledge of the equipment and its maintenance. Commissioned sections of work should be handed over to the owner for maintenance as soon as practicable. Defects that become apparent can then be rectified while the full complement of contractor's and consultant's staffs are resident on site.

At the end of the period for which the contractor is responsible for defects, the engineer's project manager reviews and tests the project, with the owner and the contractor present. It may often be useful to the owner if the engineer provides continuing services to monitor the performance of the rehabilitated project. Monitoring devices are frequently computerised and specialist maintenance contracts are often put in place to help the owner in the initial years.

2.4.5 Training Opportunities and Strategies

The work entailed in rehabilitation of barrages, dams, hydroelectric plants, irrigation and drainage systems and water supply systems is often complex and calls for a range of skills and experience in the water sector. It can provide a training vehicle for the owner's wider institutional development programme. Training opportunities include :

- post graduate training in specialised water resources disciplines such as hydrogeology, hydraulic and mathematical modelling, hydraulic structures design and water resources planning
- providing mentors, devising study tours and seminar participation for top and middle management, installing computer networks and training staff in its optimum use.

Structured training is needed to meet today's social, business and employee's needs. Information technology demands considerable training if its benefits are to be realised. It is not only greater technical skills that are required but also interpersonal and management skills. Whether in a government department or private corporation the level of skill required in planning, technical, contracts and finance departments is greater than ever as we seek to use the world's resources to satisfy the needs of growing populations. Government departments in developing countries carry major responsibilities and are particularly vulnerable. If they cannot satisfy the development needs of their personnel they lose staff to the private sector, cannot attract replacements, and cannot carry out their duties effectively. It is in this climate that the major financing agencies encourage significant, often stand alone, training programmes.

Development programmes can be designed to meet a significant range of needs, from planners, designers, construction, operation and maintenance to top management. All development programmes have a common structure. This comprises definition of what a person needs to be able to do, a schedule of experience he needs to have in order to meet these performance goals, and a mechanism for monitoring progress. Often a *logical framework* approach helps when devising a training programme. This looks at global objectives, goals which when met will realise the objectives, activities to reach goals, and measures to ensure that the goals are met.

Personnel put forward for development are interviewed together with their supervisors to determine their suitability for training and which training modules they should take. Supervisor and employee should agree on what route the training should take. The content of each training module is developed by experienced professionals.

Outlines of the owner's training strategy should be circulated and employees encouraged to apply. Mentors can provide trainees with counselling, guidance and sometimes encouragement to get the best out of the programme.

Encouragement in any course of study is an important ingredient of success. If job descriptions and the skills needed to do the job are available to participants they can see how their study will enable them to progress on their career path. A requirement to demonstrate their new-found skills upon return from training, by presentation or by training others, provides an inducement to perform. A penalty for consistent failure to perform on the course, particularly when the course is overseas, should be built into the programme.

2.5 RISK MANAGEMENT

Dam operators increasingly carry out risk management to assist in the identification and quantification of potential threats to the works, and to manage the risk effectively. This allows better decisions to be made concerning the rehabilitation of ageing structures, taking into account both the value of the asset to the business and the safety of the public. It gives the benefit of allowing prioritisation of investment where lack of funds or constraints do not allow for them all to be done. There is often inadequate operating data available to assess failure frequency, particularly for remote plants, underlining the value of experience and generalised statistical data. It emphasises too the value of a comprehensive data base of operating incidents. Whether or not they reflect well on the operating staff, they are encouraged to report them faithfully. The priority in assessing risk is to understand the consequences of a failure in terms of the business or undertaking that relies on the works. The study should include assessment of the safety of the general public. Several approaches to risk analysis have been proposed to supplement the direct and effective method of inspection analysis and reporting which suffer from the limitation that they principally identify defects that have already developed. The ICOLD Committee on Dam Safety is preparing a Bulletin on the subject entitled "Risk Assessment as an aid to Dam Safety Management". This will address probability and assess the consequences, which together represent the risk of dam failure. It will also give guidance on tolerance risk criteria.

Those undertaking risk assessments or analyses need to be clear and consistent in their nomenclature. It will be helpful to provide two definitions :

Hazard : a situation with a potential for death or injury to people and/or damage to property.

Risk : the probability for specified unwanted event or hazard occurring within a given period.

A technique described by Beak et al (1997) is known as the Failure Mode, Effect and Criticality Analysis (FMECA). This is based on a British Standard (5760 Part 5 1991). It involves applying simple criteria to an engineering system in order to identify the areas of greatest risk. Potential modes of failure are identified for each component of the works. For each failure mode the severity of the event is assessed taking into account a) the effect of failure on the operations, environment and wider community, b) the probability of the occurrence and c) the likelihood of the failure being detected. Each of these three circumstances is assigned a relatively coarse indicator on a rising scale, typically 1 to 5. The term *criticality* is given to the product of the three indicators. The method produces a qualitative rather than a quantitative result. The actual probability of the event occurring is not calculated but the approach identifies which elements of the works pose the greatest threat and is useful in allowing effort to be concentrated in those elements.

Several approaches to a probabilistic risk analysis have been proposed. This technique has been used for a number of years by the chemical and nuclear industries. It has been argued that in both of these industries the works are largely constructed of standard components with well known reliability, whereas every dam is a prototype with the reliability of its components being less easy to define. It has also been asserted that the results obtained from studies of chemical or nuclear plants are less precise than has been assumed.

Typical of the probabilistic approach applied to dams is the procedure detailed by Bowles et al (1990) and Vick et al (1996). This involves developing all the possible failure scenarios that could develop as the result of a triggering event and developing them into an event tree. A probability of failure is then allocated to each component of each scenario. Thus the probability of each of the components of the tree occurring is assessed and the overall probability of each of the failure modes identified is calculated as the product of all the components in that branch of the tree.

This technique provides a quantitative risk of failure for every failure path identified and for each event occurring. It is therefore possible to use this to justify whether to carry out works based on comparison of the calculated risk with a predetermined acceptable risk of loss of life or cost to the community or operator. It must however be borne in mind that the overall risk has been developed as a product of many components. A small error in each component may have a significant effect on the overall assessment. Calculation of accurate probabilities of failure for the various elements of the tree can be difficult and usually involves significant costs.

This technique has been further developed into the Portfolio Risk Assessment (PRA) approach Bowles et al (1998). This is usually based primarily on available information, without performing extensive additional analyses or investigations, the steps are normally conducted at a reconnaissance level and make use of professional judgement which leads to an engineer certifying aspects of safety of the dam. Subsequent to the initial PRA, additional engineering studies will usually be necessary to verify the need for remedial work and to justify the extent of the work. As with the FMECA technique this produces a qualitative rather than quantitative result.

Hoeg (1996) describes a simplified probabilistic risk analysis in the re-certification of existing rockfill dams that seeks to use the rigour of a logical approach via an event tree while making full use of the professional judgement of the engineers involved. The first step is a site inspection, a desk study is not an acceptable alternative. Next, all possible failure modes are visualised and defined. Those that lack technical credibility are eliminated. The third step is to construct an event tree that allows the interrelationships between events to be displayed. Only those events that lead to an uncontrolled release of water are developed at this stage. Each of these events is then reviewed to find those with the greatest probability of occurrence, perhaps using a simple scale of likelihood from *virtually impossible* through *very unlikely*, *completely unknown* to *complete certainty*. The final step is to review the results from the event tree to determine the reasons for certain failure modes giving larger contributions than others.

Other methods include the production of fault trees and Hazop studies. With a fault tree the initial consideration is the failure of the element being considered and a tree is built up to establish the various means by which the failure could develop. As with the event tree the probability of each component is assessed to calculate an overall probability of failure. Hazop is a well established technique used in the chemical industry and bears many similarities to the event tree process but uses standard probabilities of failure for the components used in the construction of the plant.

None of the techniques remove all risk. The aim is to reduce the risk of failure to one that is *As Low As Reasonably Practicable* (ALARP). The basis for ALARP is that risks are acceptable only if reasonably practical measures have been taken to reduce risks. This is commonly taken to mean that the risks have been reduced to the point where it is no longer cost effective to reduce them further.

The procedure with all the methods of risk assessment should involve a team that includes engineers with experience of design, operations and maintenance and may also require advice from specialists in hydrology, geology and seismology in order to develop a comprehensive assessment. A site visit by the team to inspect all aspects of the works is essential, as are discussions with the local operators and the study of such construction drawings and operational records as can be obtained, to enable them to produce a worthwhile analysis. The outcome of the risk analysis is then incorporated into an overall assessment of the risk which, with other input is used as the basis of the decision for appropriate action.

3. CONCRETE AND MASONRY DAMS

3.1 INTRODUCTION

Rehabilitation of dams and barrages is necessary for two main reasons. First is to counter the effect of the ageing process, which is sometimes made worse by defective maintenance. This in turn may be divided between the foundation of the dam and the dam body itself. Second is the introduction of new standards as a result of better understanding of the natural processes at work. For example modern stability criteria against flood and earthquake have resulted in major rehabilitation at a large number of dams. This Chapter is therefore divided into three main sections : rehabilitation of the foundation, rehabilitation of the dam body, and rehabilitation to improve stability.

3.2 REHABILITATION OF THE FOUNDATION

3.2.1 Loss of Strength under Permanent or Repeated Actions

3.2.1.1 Introduction

ICOLD lists 37 case histories in Bulletin 93 (ICOLD 1994). Loss of strength of the rock mass foundations of concrete and masonry dams has been the cause of major incidents. This scenario may occur at the beginning of the operation phase or its effect may be observed after some years of operation.

The main cause of this scenario is related to alternating stresses in the foundations linked with the variations in hydraulic gradient experienced when the reservoir level changes. Thermal effects have little direct impact on the foundation. The alternating stresses lead to foundation deformation, to movements of rock joints, and to initiation and propagation of cracks. This process may also be accompanied and amplified by changes in water content of joint fillings and water pressure in the joints.

For concrete and masonry dams the rock mass is usually strong enough to adopt a new equilibrium state after several years of operation. However, sometimes the rock behaviour changes for one or more reasons :

Periodic change of hydraulic gradient. Permanent deformation accumulates with time or may occur suddenly after an extended period of stable behaviour. This irreversible evolution is often associated with a high fluctuating hydraulic gradient. Such conditions may cause significant joint movements and the washing out of the joint fillings and may cause permanent deformation of the dam foundation, or an increase in the amplitude of the reversible movements. These structural changes may be accompanied by a substantial increase in seepage and uplift pressure. This may result in piping, large scale instability in faulted rock masses subjected to uplift pressure, and stress redistribution in the foundation.

Long-term raising of the groundwater table adjacent to banks and foundation. In porous limestone or sandstone foundations, filling of the reservoir raises the groundwater level in the reservoir banks and around the dam foundation. The ground water level may take several years or decades to stabilise and the effect of the dam drainage system is small. The new water level may modify the equilibrium of the rock mass, depending on the nature and geometry of faults and joints.

Large scale drainage effects. Differential displacements of the dam-foundation interface may be induced by movements of the rock mass foundation owing to external phenomena, such as large-scale drainage and mining.

Chemical and physical alteration of rocks. Chemical and physical alteration of rocks may occur under the new conditions induced by the reservoir and this may lead to weakening of the rock, piping or subsidence.

3.2.1.2 Detection and Monitoring

Usually the loss of strength of the foundation occurs over a long period. Therefore, reliable monitoring is necessary and three measurements predominate : seepage, uplift pressure, and inelastic foundation displacements.

Seepage measurement must comprise the observation of quantity, origin and quality of seepage, particularly the existence of solid matters in the water. At least the majority of the whole foundation area must be included in the monitoring. It should be possible to separate the leakage water coming from the dam and from the foundation. One should be able to distinguish between changes of the seepage characteristics and deficiencies of the collecting systems as for example the blockage by encrustation of drainpipes. Often it is valuable to inspect them by a television camera.

Uplift pressure is measured at distinct points using piezometers. These must be reliable and stable over the long term. The type of the uplift measuring device depends upon the situation. If simplicity and long service life are decisive, one should select simple stand pipes. On the other hand, if many data have to be collected, especially where access is limited, automatic data collection is appropriate provided it keeps on working. Specialised literature from manufacturers deals with the appropriate selection of piezometers.

A loss of strength of a rock foundation is often accompanied by displacements, both elastic and plastic. This can more easily be observed in the dam than in the foundation, and for this reason more effort is put into its investigation. Useful observations can be made by geodetic survey, by pendula or by inverted pendula anchored deep below the dam.

3.2.1.3 Remedial Measures

A rapid response is needed particularly when unusually large irreversible displacements are observed. As a first step, the reservoir water level should be lowered. The horizontal water load on the structure and its foundation decreases with the square of reservoir depth and the bending moment at the foundation approximately with the third power. Therefore lowering the water level is a powerful first aid, not for rehabilitation, but for the safety of the dam.

The problem must then be brought to the attention of experts. The organisation of such a procedure depends upon the regulatory conditions of the country and must be well known to the staff, perhaps as part of an emergency plan.

The foundation rock may be strengthened by grouting. The uplift situation may be improved by a combination of grouting and drainage measures. Impervious aprons may be added to the upstream toe of the dam.

An example of these techniques is provided by the **Albigna** concrete gravity dam, 760 m long and 115 m high, situated in the southeast of the Swiss Alps at an elevation of 2000 m. It consists of blocks of 20 m breadth. Between the blocks there are joint spaces 5 mm wide, intended to reduce uplift at the dam base. The dam is built on a pronounced barrier of coarse-grained granite of high strength. Measurements made with pendulums in the rock showed that this barrier moves in the same direction as the dam when the water level in the reservoir changes. It has three regular sets of joints. Although during design and construction (1956-1959) the foundation rock was considered to be favourable, during the first reservoir filling in 1960 a crack became clearly visible in block 11 continuing into a filled rock joint. In the course of years there was increasing water loss and a slight increase of permanent deformations. During the subsequent investigations an open joint was detected along the upstream heel of the dam, extended over a length of 300 m with a width locally up to 8 mm. It became clear that the crack completely crossed the grout curtain. As a first measure in 1977 the water level was lowered immediately. Then measures were taken to arrest the further propagation of the existing cracks and to minimise their unfavourable effects on the uplift stress distribution. It was important to prevent water percolating into the rock foundation below the dam. Therefore the existence of other extensive fissures had to be assumed. Answers for these uncertainties were to be found with the aid of systematic high precision rock deformation measurements using sliding micrometers (Kovari, 1985). It was observed that the rock mass behaved as a homogeneous monolithic body. Only a few active joints required special attention. But a second fracture system, approximately perpendicular to the first, drained the seeping water in a downstream direction.

The active joints (A and B, Fig. 3.2.1) belong to a principal joint set. The three boreholes M3, M4 and M5 provide an almost complete picture of the deformational behaviour of the bedrock. The remedial measures to stop the underseepage of the dam were carried out in and after 1980.

To repair the dam it was necessary to reduce the water pressure in the fracture opening. Grouting would not have been effective since, as previously mentioned, the rock valley deforms at every change of the reservoir water level. Even grouted, the fracture would tend to re-open and the water would exert its pressure as before. To test its effectiveness in reducing the water pressure in the opened crack, a plastic foil of *hypalon* 2 mm thick and a few hundred mm wide was applied as a seal on the rock surface. The remainder of the seepage still percolating through the fracture was diverted to the lower gallery of the dam by drainage boreholes installed at the same time. These tests proved to be satisfactory. Therefore a watertight lining of neoprene 1.4 mm to 4.5 mm thick was applied and vulcanised to the lower part of

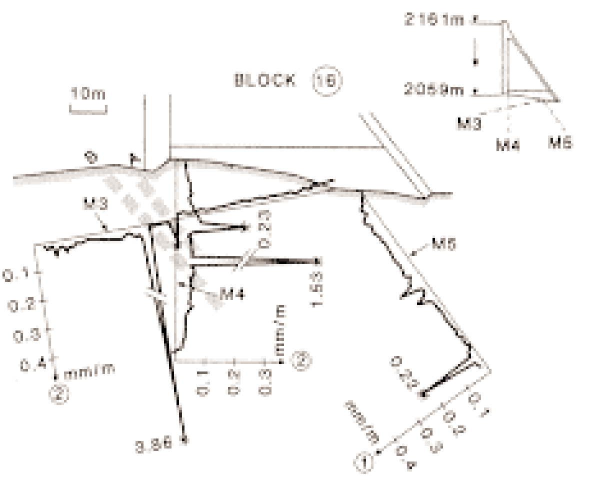


Fig. 3.2.1

Albigna Dam. Strain development due to fall in water level (Kovari, 1985)

1. Compressive strain	*A. Active joint*
2. Tensile strain	*B. Active joint*

the upstream face of the dam and to the bedrock, over a width of 7 m in front of the dam (Swiss National Committee, 1982).

The case history of **Zeuzier** Arch Dam in Switzerland is worth special mention. Although the deficiencies originated in the rock mass of the foundation, they affected the dam body itself. Thus, although the case history is not typical for the scenario of ageing of the foundation, the lessons available from this incident have been extraordinary for the dam profession. Therefore it has been decided to include the case history in this Bulletin.

Zeuzier Dam, 156 m high and 256 m long was built between 1954 and 1957. At the end of 1978, after more than 20 years of trouble free operation, abnormal deformations were detected during the regular inspection of the dam. They increased for two years, until May 1980 and then tended to slow down. Finally they almost stopped. In comparison to the geodetic measurements made in 1976, a settlement of 100 mm and an upstream displacement of 90 mm were observed at crest level, as well as a 60 mm shortening of the distance between the abutments of the crest arch. Owing to these deformations, some of the vertical contraction joints in the upper part of the upstream face opened and cracks developed along the downstream toe. The major cracks are located mainly in the lower third of the dam and along the foundation. Other cracks also appeared in the inspection galleries and shafts.

A general ground movement was noticed, attributed to the drainage effect of an exploratory gallery constructed as part of the preliminary investigations for a highway tunnel. Although this gallery is located 400 m below the dam and 1400 m

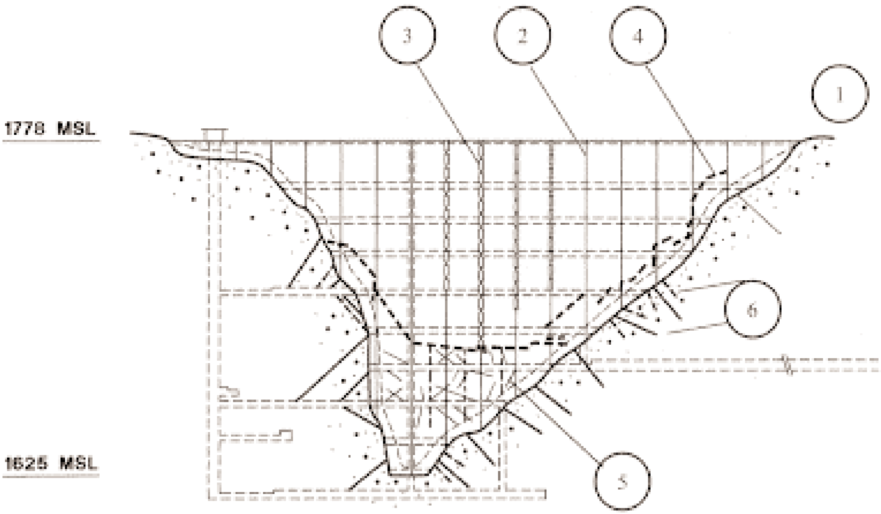


Fig. 3.2.2

Zeuzier Dam. Longitudinal section

1. Left abutment
2. Vertical joints
3. Opening of the construction joints
4. Cracks on downstream face
5. Test and injection drill holes
6. Check holes

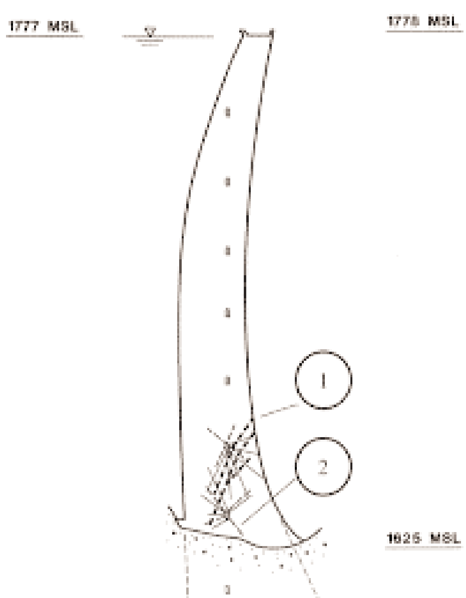


Fig. 3.2.3

Zeuzier Dam. Cross section

1. Main cracks
2. Test and injection drill holes

away from it, its drainage capacity was such that it reduced the pore pressure in the rock mass and consequently it was responsible for the general movement.

A programme was established for examination of the condition of the dam and its foundations and for evaluation of methods suitable for repairing the damage to it. Three zones were defined : grout curtain, abutment and dam body. The grout curtain and abutment were found to be in a satisfactory condition. To facilitate the proper grouting of the smaller cracks crossing the vertical joints in the dam body and subsequently the grouting of the cracks block by block, it was first decided to grout the vertical joints. The location and direction of the drillholes were adapted to the crack pattern and the whole grouting procedure was performed in three phases. As the grouting work proceeded, cores were extracted giving detailed information regarding the location, orientation and extent of the crack pattern, as well as the width of the cracks and the quality of the grouting achieved (Mueller et al, 1985).

Venda Nova arch-gravity dam, 100 m high, was built in 1951 on the Rabago River in Portugal. The foundation is mainly granite, with schistose rocks occurring on the right bank at higher elevations. The rock mass is crossed by several sets of joints and local faults, generally clay and mylonite-filled. The initial treatment of this dam consisted of consolidation grouting at the dam-foundation interface, and waterproofing by a vertical grout curtain, down to a depth of 50 m. Seepage of water was noticed after the first filling of the reservoir, and remained after the initial operation phase. Therefore further foundation treatment was carried out, similar to the initial treatment, and a drainage system installed in 1964.

A monitoring system installed in the dam showed the seepage remaining and the uplift pressure increasing. This was due to the opening of faults and subhorizontal joints at the left bank and valley bottom. It was accompanied by washing out and the dissolution of filling materials. The dissolved salts in the reservoir were up to 22 mg/l, while the seepage water contained much more than this.

Repair work was carried out in two phases in 1984-85 to improve the hydraulic behaviour of the foundation. This is shown in Fig. 3.2.4. This aimed at improving the strength and watertightness at the lower zone of the foundation rock mass. In the first phase an upstream grout curtain was constructed with blast furnace cement up to a depth of 25 m. This was intended to create a barrier to allow the more efficient injection of a resin grout curtain in the second phase. At the same time the rock mass was consolidated in the downstream zone of the foundation down to a depth of about 15 m.

The second phase grout curtain was injected with an acrylic resin. This resin is easy to inject owing to its low viscosity before the polymerisation reaction begins, and proved an appropriate material to fill joints and cracks, owing to its high ductility and swelling in presence of water after polymerisation. About 64 000 litres of acrylic resin were used in 2755 m of drill holes.

The rehabilitation work was effective in reducing the seepage and the uplift in the foundation significantly. The maximum seepage was 1046 l/min before the repairs and only 12 l/min after. The maximum uplift pressure was reduced by factor 4 to 5.

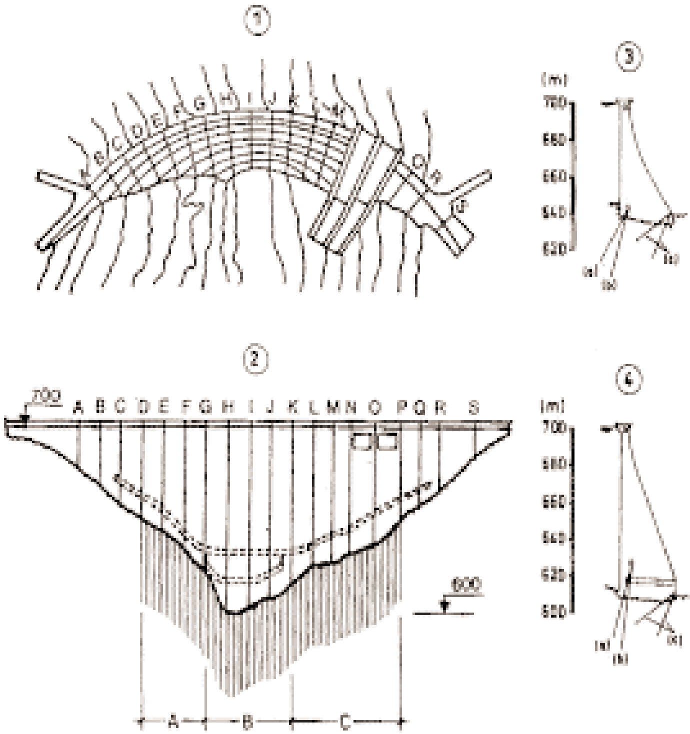


Fig. 3.2.4

Venda Nova Dam. Treatment of the foundation

1. Plan of dam
2. Downstream elevation and foundations A, B and C treated
3. Treatment of zones A and C : cement grout curtain (a), resin grout curtain (b), consolidation grouting (c)
4. Treatment of zone B : cement grout curtain (a), resin grout curtain (b), consolidated grouting

The following Table summarises remedial measures for problems with repeated loadings within the foundation.

Nature of Rehabilitation	Location	Name of the Dam	Reference
Plastic foil applied (hypalon 2 mm and neoprene 1.4 mm – 4.5 mm thick)	Foundation	Albigna, Switzerland	Kovari, 1985
Grouting the dam body	Dam body	Zeuzier, Switzerland	Müller et al, 1985 Pougatsch, 1982 Berchten, 1985
Grouting	Foundation	Venda Nova, Portugal	ICOLD, 1994 Pedro et al, 1998
Grouting	Foundation	Magaribuchi, Japan	Matsuhuij et al, 1993
Drains reconstructed	Foundation	Laouzas, France	CFGB, 1994
Post-tensioned anchors	Foundation, dam body	Pacoima, USA	USCOLD, 1996

3.2.2 Erosion and Solution

3.2.2.1 Introduction

Foundation erosion arises mainly from dissolution and piping in fault and rock joint fillings, soluble rocks, soil foundations, and consolidation grouting. This scenario is often associated with the foundation weakness referred to in Section 3.2.1 above. ICOLD reports eleven case histories in Bulletin 93.

The flow of water in erodible or fractured rock can lead to increasing leakage from the reservoir. Even when the foundation rock mass itself is not soluble, but contains faults or joints filled with fine-grained loam or clay, erosion must be expected when it is under a high hydraulic gradient. More dangerous for the dam are solution processes within the dam foundation, because the erosion process there may not stop at the rock walls of a fault or a joint, but it will continue by attacking the rock mass itself. Large caverns may be the result. In extreme situations, as described by Heitfeld, (1961), the dam may have to be abandoned. But even when the effects can be controlled, extensive rehabilitation work often cannot be avoided.

3.2.2.2 Detection and Monitoring

Rock mapping before the construction of the dam will have helped to design seepage control devices to be installed in a timely manner. Seepage measurements and the measurement of uplift pressure during operation, as described in section 3.2.1, will help to detect such developments.

The following Table shows a useful correlation between the observed trends in seepage and in piezometric pressure. When both are rising it is advisable to review the situation and where both are falling, stability is improving.

	Seepage increasing	Seepage decreasing
Piezometric head increasing	**Unfavourable situation :** rehabilitation may be necessary urgently.	**Unfavourable situation :** it may be necessary to clean or enlarge the drains.
Piezometric head decreasing	**Surveillance required :** risk of internal erosion, hence advisable to check whether there are fine materials in the seepage water.	**Safe situation :** no action necessary.

3.2.2.3 Remedial Measures

The choice of the most suitable remedial measures depends upon the nature of the problem. Grouting of the developed cavities, as far as they are accessible, may be an appropriate rehabilitation measure. The width of the gaps and the volume of the region to be rehabilitated will determine whether mortar or cement suspensions or chemical grouting material should be used (ISRM 1995).

More problematic is increasing seepage caused by solution processes. One must anticipate that the rehabilitation work, as with grouting, will have a limited lifetime. In addition to closing the seepage paths as perfectly as practicable it will be necessary to study whether it is possible to reduce high hydraulic gradients. Such measures are also valuable when erosion of fractures is expected or observed.

Drainage works have caused a high hydraulic gradient to the rock mass which has proved detrimental to the foundation. **Lister Dam** in Germany is a masonry dam 40 m high and 264 m long designed by Prof. Intze. In 1965 the dam was submerged up to 2/3 of its height h_1 from the downstream side as part of the works for the Bigge Reservoir. To improve the stability of the dam, a drainage gallery was excavated at the bottom of the dam, to reduce the uplift pressure. With the gallery pumped empty the uplift pressure distribution was triangular on both sides, with a peak $g_w h_1$ at the upstream side, and $2/3 g_w h_1$ below the downstream face, and zero at the gallery. The hydraulic gradient at the upstream side exceeded 10. After some years it was observed that joint filling material was transported to the gallery, increasing the permeability of the rock below the dam. Additional calculations proved the dam to be stable with the gallery flooded from the downstream side (Bigge Reservoir) and since the end of the 1960s the gallery has been allowed to fill, preventing further erosion. Fig. 3.2.5 shows the uplift pressure distribution under different operating conditions (Rißler, 1998).

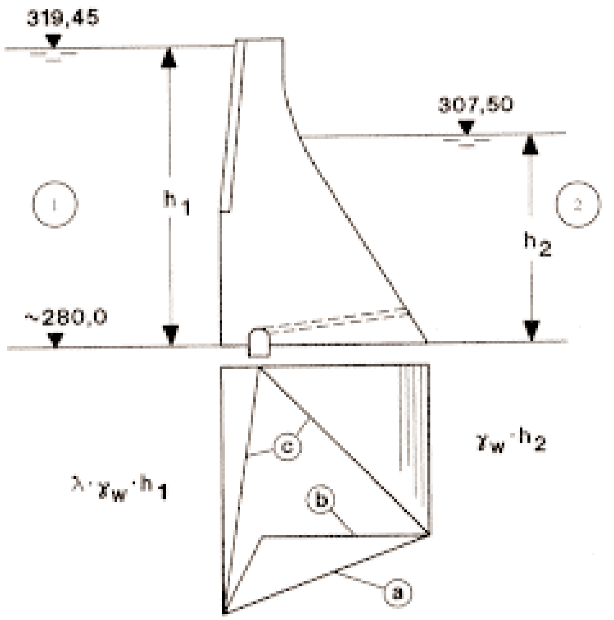


Fig.3.2.5

Lister Dam. Uplift at different operation conditions

1. Lister Reservoir, *a : without drainage gallery*
2. Bigge Reservoir *b : with drainage gallery flooded from downstream*
c : gallery empty

Henne Dam is an example of a dam that has had to be abandoned (Heitfeld, 1961). This Intze masonry dam was constructed in Germany 1905 on soluble limestone formations to form a reservoir of 11 million m^3. After 40 years of operation the dam foundation had become so pervious that remedial work was not economic. A new embankment dam with an internal asphaltic concrete core was built between 1949 and 1955 a few hundred meters upstream, on less soluble foundation rock. An extended gallery system, shown on Fig. 3.2.6, combined with intensive monitoring of seepage ensures the safety of the structure and would enable the owner to start remedial measures rapidly.

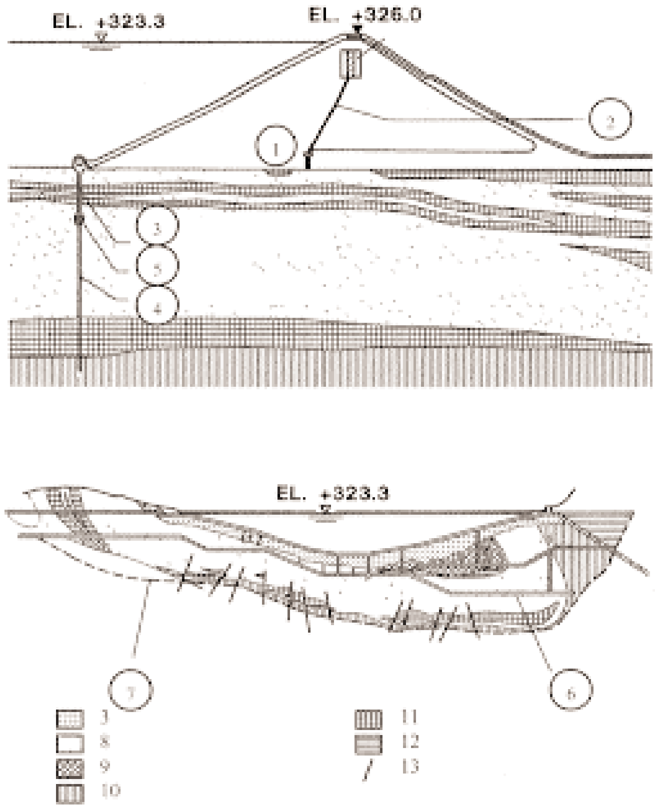


Fig. 3.2.6

New Henne Dam. Cross and longitudinal section ; sealing and drainage measures

1. Rockfill
2. Asphaltic erosion protection
3. Concrete cut-off wall
4. Grout curtain
5. Gallery
6. Galleries
7. Lower limit of grout curtain

8. Lower Flinz series
9. Middle Flinz series
10. Wallner schist
11. Keratophyr
12. Newberrien series
13. Fault

The following Table lists recent case histories of remedial measures against erosion and solution.

Nature of Rehabilitation	Name of the Dam	Reference
Dam abandoned and new dam built upstream	Henne, Germany	Heitfeld, 1961
Gradient diminished	Lister, Germany	Rißler, 1998
Grouting	Kuromata, Japan	Tada et al, 1991
Grouting	Ogakura, Japan	Yanase et al, 1989
Grouting	Magaribuchi, Japan	Matsuhuij et al, 1993
Grouting	Hohnen-Ike, Japan	Fujisawa, 1994
Apron extending from the dam to upstream	Upriver, USA	USCOLD, 1996
Diaphragm wall	Schlegeis, 131 m high, 725 m long, VA, Austria, built in 1971	Floegl et al, 1991 AUCOLD, 1991
Existing pressure relief holes cleaned, new ones bored	Baitings, U.K.	YWS, 1995
Existing pressure relief holes cleaned, new bored	Booth Wood, U.K.	YWS, 1995

3.2.3 Grout Curtains and Drains

3.2.3.1 Introduction

Grouting is the most commonly used seepage control method for concrete dams. Older masonry dams have usually been designed without a grout curtain. Grout curtains are intended to reduce the seepage through discontinuities in the foundation rock. Cement is the most common grouting material, its properties often being improved by mineral additives including bentonite, fly ash, pozzolan, and granulated ground blast furnace slag. Grouts of silicates and acrylic resins have also been used. Drainage systems include boreholes or galleries in the dam body and tunnels in the rock mass. In smaller structures, the drainage is carried out through horizontal channels at the foundation interface.

Degradation of grout curtains and drainage systems may be accelerated under repeated actions or when erosion or solution of the foundation occurs. Grout curtains may deteriorate with time if they are inadequately designed and constructed. Particular problems have been reported due to : inadequate grouting techniques ; stress and deformation in the dam foundation, inducing movements of rock joints and rock fracture ; erosion or solution of grout by seepage water ; and chemical aggression of the water.

3.2.3.2 Detection and Monitoring

Deteriorating seepage control measures can be detected by monitoring the amount and turbidity of seepage water. There have been few reported cases of uplift pressure distribution being changed by the deterioration of grout curtains, although the deterioration of drains does have this effect.

Deterioration of grout curtains may occur slowly or rather suddenly. It has been reported that at arch dams a crack sometimes appears at the base during the first filling, extending over a distance of more than half of the dam thickness. In these circumstances a grout curtain may lose effect with respect to pressure distribution on the base of the dam, but it may still reduce the volume of water seeping through the foundation. Slow deterioration may result from micro-ruptures, erosion or solution processes. Such events cannot be detected by a short inspection of the dam. Many years of seepage observations must be reviewed, together with observations of the water level in the reservoir, precipitation, and temperature, especially during freezing periods. Because the process develops slowly, the owner often has time to fit the rehabilitation work into his other remedial work.

When the grout curtain deteriorates suddenly, and when the seepage volume grows quickly, more severe cracks may be expected demanding a quick response. The stability of the dam must be assessed.

The deterioration of drainage systems in dams may become apparent when for no other reason, the seepage volume decreases, usually slowly. Sometimes this is accompanied by an increase in the uplift pressure near the drains. Sometimes, a blocked drainpipe can cause a change in the flow regime, causing gallery walls that have been dry for years or decades, to become wet. Other drains in the vicinity may show increased discharges. Experience shows that the piezometric pressure underneath a gravity dam is sometimes smaller than theoretically expected. This may be the result of fine material being washed from the reservoir into the joints in the rock below the dam, or another source of increased impermeability in the foundation.

Both for the deterioration of grout curtains and clogging of drains, only detailed and long-term monitoring results of seepage and uplift pressure, starting at the first impoundment of the dam, are a sufficiently reliable tool for the detection of deficiencies.

3.2.3.3 Effect on Safety and Performance

As a consequence of the above mentioned role of grout curtains below gravity and arch dams it should seriously be examined from case to case whether rehabilitation work is necessary. ICOLD reports 24 examples of deterioration of grout curtain or drainage systems in Bulletin 93. The damage has more often affected the performance of the structure, but one of the cases reported led to failure. Deterioration usually leads to increased seepage, higher uplift or erosion. If the seepage is particularly large, the economic value of the lost water may be significant. Frozen seepage water can impede access to galleries. Failure of the upper part of a grout curtain under an arch dam has occurred following deformation of the surrounding rock associated with high stress near the interface of the dam and the foundation. As a result the water pressure distribution is changed and water can seep through newly developed fractures into the drainage gallery.

The three main causes of deterioration of drainage systems of dams and appurtenant works include 1) inadequate design, use of unsuitable pipe and filter materials, and poor quality of construction ; 2) climatic conditions ; 3) clogging of drain holes, pipes and wells (see also the table in section 3.2.2).

3.2.3.4 Rehabilitation Measures

When the deterioration of a grout curtain only increases the water seepage flow and not the stability of the dam, even in the long-term, there is a certain freedom in deciding whether the grout curtain has to be rehabilitated. But such favourable conditions seem to be the exception. Usually a deterioration of the impermeable element in the foundation rock ultimately affects the safety of the dam or the economic return of the reservoir. Therefore remedial measures cannot often be avoided.

It may be appropriate to replace damaged parts of a grout curtain using modern design and construction techniques. This is more likely to be recommended when the seepage has increased slowly, indicating that erosion of joint fillings, solution processes or the development of microcracks are responsible for the deterioration.

However, when the water losses have increased suddenly indicating severe ruptures within the grout curtain, there is doubt that a renewed curtain at the same place would survive, even for a short period. The solution may be to find a new position for a new grout curtain, outside the region affected by adverse stresses.

Instead of a repair of the grout curtain an elastic cut-off element can be applied, as for example at **Schlegeis Dam** in Austria (Floegl et al, 1991, AUCOLD, 1991). Schlegeis Dam is an arch dam, 131 m high, completed in 1973, modified in 1982/83. The volume of seepage water emerging in the inspection gallery at the dam base increased markedly during the final stage of reservoir filling, attaining a maximum rate of about 251 l/s with a full reservoir. Ninety percent of the seepage was concentrated over a 150 m length of the foundation. Investigations showed that the grout curtain, situated between the upstream toe and the inspection gallery, was cracked due to tension. A 5 m deep flexible diaphragm inserted into a slot below the inspection gallery solved the problem, reducing the seepage to 25 l/s and 5 l/s in the treated area. The elastic diaphragm wall consists of a 4 mm thick plastic foil, arranged in 1.5 m broad strips and linked by S-shaped pipes (see Fig. 3.2.7).

Baitings Dam, UK : is a concrete gravity dam, 53 m high, completed in 1958. It was modified in the late 1980s and early 1990s. In 1986, it was required that choked pressure relief holes should be cleared and if this could not be achieved, they should be replaced. A shrinkage crack should stay under regular observation. Nine new vertical pressure relief holes were drilled from the lowest gallery into the foundation rock. Similar work has been done at **Booth Wood Dam**, a 58 m high concrete gravity dam in the UK.

Pacoima Dam in California, USA, is a concrete arch dam 34 m high, completed in 1929. Classified as high hazard, it was modified between 1971 and 1976. The dam was severely shaken by an earthquake of Richter magnitude 6.5. The left abutment proved to have inadequate seismic stability, and rock movement, cracking, and uplift pressures were observed. Interim rehabilitation measures included repair of sprayed concrete cover, stripping of loosened rock, installation of abutment relief

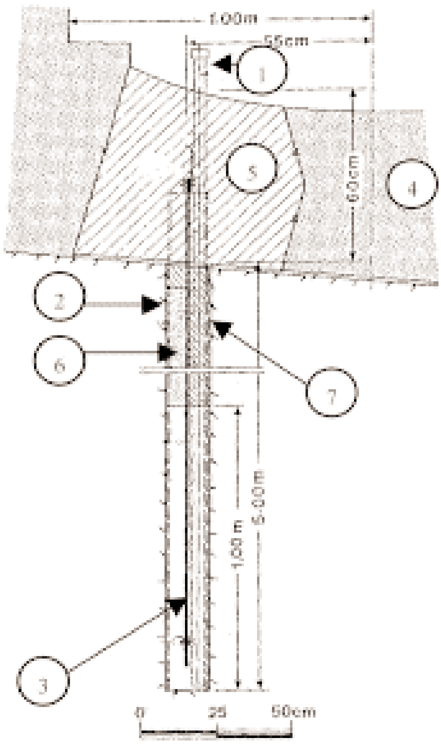


Fig. 3.2.7

Schlegeis Dam. Elastic diaphragm wall

1. Vertical pipe
2. S-shaped pipe
3. 4 mm thick plastic foil
4. New concrete
5. Infill concrete 'B 300'
6. Aggregate plus polyurethane foam
7. Sikaflex T685 and sand

drains, repair of left abutment grout curtain, consolidation grouting of upper left abutment, repair of spillway lining, and construction of an emergency outlet. The permanent rehabilitation work included stabilisation of the rock abutment with post-tensioned anchors. In addition, cracks and the open joint between the dam and thrust block were repaired by cement grouting. The existing grout curtain in the foundation and abutment was repaired and extended. Further consolidation grouting of the foundation rock was carried out (USCOLD 1996).

The following Table lists some typical remedial measures against damage at grout curtains and drains :

Nature of Rehabilitation	Location	Name of the Dam	Reference
Drain reconstruction		Laouzas, France	CFGB, 1994 Bonazzi et al, 1985
Drainage clearing	Foundation drainage	Baitings,UK	YWS, 1995
Drainage clearing	Foundation drainage	Booth Wood, UK	YWS, 1995
Grouting		Magaribuchi, Japan	Matsuhuij et al, 1993
Grouting and post-tensioned anchors		Pacoima, USA	USCOLD, 1996
Successful use of flexible diaphragm wall		Schlegeis, 131 m high, 725 m long, VA, built in 1971, Austria	Floegl et al, 1991 AUCOLD, 1991

3.3 REHABILITATION WORK CONCERNING THE DAM BODY

3.3.1 Chemical Reaction Resulting in Swelling

3.3.1.1 Introduction

There are many similarities between this scenario and the degradation due to chemical reactions discussed below in section 3.3.3. Water plays an important role in this scenario which has two main origins : alkali-aggregate reaction (ICOLD Bulletin 79) and the action of sulphates on concrete and mortar. This scenario occurs in concrete and masonry structures of dams and appurtenant works and Bulletin 93 lists 45 case histories (ICOLD, 1994).

Alkali-aggregate reaction depends on the nature of the cement and aggregate in the concrete, and an ample supply of water. See for example BRE (1988) and ISE (1992). The cycles of saturation and drying of concrete are also thought to be an important factor. In a non-confined or only partially confined structure aggregates are compressed due to swelling and the cement paste around the swelling region is radially tensioned. As a result cracking will occur, relieving the swelling-induced stresses.

The swelling resulting from the reaction varies throughout the dam body and is influenced by the confinement experienced by the structure. This confinement leads to compressive stresses and can reduce or even stop swelling in the confined directions. The swelling is usually confined along the horizontal axis of the dam. Strains of up to about 10^{-4}/year are reported, raising the dam crest by the order of several mm/year (Sims et al, 1988).

Sulphate action has been reported from magnesium (in seawater), sodium or calcium sulphates (ICOLD, 1994). It may also result from the oxidation of iron sulphide (pyrite) contained in some aggregates. The sulphate action takes one of two forms depending on the concentration : 1) crystallisation of complex salts, along with a great increase of volume and loss of binder cohesion ; 2) decomposition of calcium silicates, with a substantial loss of strength. The degradation of the concrete may be more rapid than the development of the alkali-aggregate reaction, as the swelling is accompanied by a significant loss of material strength. The concrete may be destroyed in a few years or even months (Gil Garcia et al, 1991, Cavalcanti et al, 1991).

3.3.1.2 Detection and Monitoring

Displacement and strain measurements are appropriate to detect swelling in dams. Visual inspections are usually less sensitive. Malfunction of electrical and mechanical equipment attached to the dam is not uncommon, as clearances reduce and foundations move. With swelling confirmed, the nature of the process, its stage and rate of development in the different zones of the dam, and the confining stresses, may be assessed by appropriate methods. Model analysis taking into account the anisotropy of the swelling, may then be useful to compare the observed behaviour with that predicted.

3.3.1.3 Effect on Safety and Performance

Alkali Silica Reaction reduces the strength and integrity of concrete. Sims (1991) reports the findings of the Committee of the Institution of Structural Engineers and this is summarised below.

Property	Percentage of unaffected concrete for expansions (mm/m) of		
	0.5	2.5	10
Uniaxial compression	95	80	70
Flexural tension	75	70	–
Elastic modulus	100	50	30

Apart from the effect on embedded equipment, and the loss in structural integrity, the effects and consequences of swelling on the behaviour of a dam depend on its type and characteristics.

Gravity dams : The upstream face of the dam will be the most affected. A general deformation upward and downstream will develop. As the swelling process is not uniform over the whole cross-section, a stress field will result in which compressive stresses occur in the swelling regions and tensile stresses in the adjacent parts. This could lead to cracking at the upstream face, the entrance of water and increasing uplift.

Arch dams : Swelling will cause a radial, upstream displacement together with a raising of the crest. Because the water availability at the upper upstream face part is different from that of the crest, the latter will tend to smaller swelling and therefore the differential strain may cause cracks near the crest.

Buttress dams : Swelling will occur faster in the heads than in the webs, as the latter are not in contact with the reservoir. The differential development will cause a general deformation upwards and downstream, as well as tensile stresses.

3.3.1.4 Rehabilitation Measures

No effective remedial measure against swelling caused by alkali-aggregate reaction is known. Attempts have been made to protect the vulnerable concrete from the effect of water by the use of coatings and by installing on the upstream face a drained PVC geomembrane system (De Beauchamp, 1994), but it is too early to say whether such attempts have been successful. Mitigation of the effects of swelling has been tried by several methods :

- cutting slots to relieve the stress caused by the expansive reaction
- reinforcement or post-stressing in an attempt to reduce the deformation
- additional weight has been used to reduce the stress in the concrete so as to allow it to operate at a lower stress level, although no evidence has been found to show that this is a successful technique
- waterproofing the upstream face of the dam to deprive the reaction of water
- confine the concrete in critical zones to isolate the effect
- break out the damaged materials and replacing them by mortar

Typical case histories are described below for these approaches.

3.3.1.5 Cutting Slots

Chambon Dam (France) is a concrete gravity dam, 90 m high, and 300 m long, completed in 1934, modified in 1992. The dam is affected by ASR swelling which has lead to progressive cracking and irreversible strains. The swelling had the effect of producing a 120 mm deformation upstream and an increase in height of 70 mm over 25 years. After a programme of grouting and the application of an upstream impermeable membrane to reduce the volume of water feeding the reaction, the owner has sawed 11mm wide slots through the dam to relieve the compression within the upper 20 m of the dam and to relieve the left abutment of the heavy thrust of the swelling concrete, see Fig. 3.3.1 and 3.3.2 (Taddei et al, 1996). The work was done in three years : in 1995 cuts S3 and S6 were made. These are 21 m in height. Three further cuts have been made in 1996, and three more in 1997. After sawing and before refilling the reservoir, the upstream watertight membrane was restored.

Before the work started finite element studies were carried out in order to examine the structure's response during and after the work. In addition pendulums, jointmeters, and long base extensometers were used to assess the effect of the sawing on the behaviour of the dam. Three campaigns have been done to monitor sawing effects during one year and the influence of seasons.

Slots were made with sawing machine placed on the downstream face. A sawing cable 11 mm diameter and 68 m long with 40 diamonds per metre was used. High, upstream and downstream pulley blocks, mobile on rack rails were used to make sure that slots were plane and vertical. The average sawing speed was about 2 m^2/hour. After sawing and before refilling the reservoir it was necessary to restore the upstream watertight membrane in front of each slot.

In 1995 the influence of the saw cuts was detectable at about 60 m or 70 m on each side of the slot. There was a displacement of about 10 mm downstream and the slot, initially 11 mm wide, rapidly closed to about 8 mm and closed completely in a few weeks. During the campaign of 1996 the influence of the slot was detected at about 40 m or 50 m and the downstream displacement was 6 mm to 8 mm. In 1997 the influence of the slot was detected at about 30 m and downstream displacements were 2 mm or 3 mm. Several of the joints or slots are still open showing that decompression of the top of the dam has been achieved.

The works were effective in relieving the dam of the effects of the expansion caused by ASR. The owner anticipates that in 5 or 10 years it will be necessary to re-saw the structure. He considers that it will be possible to re-saw the original slots.

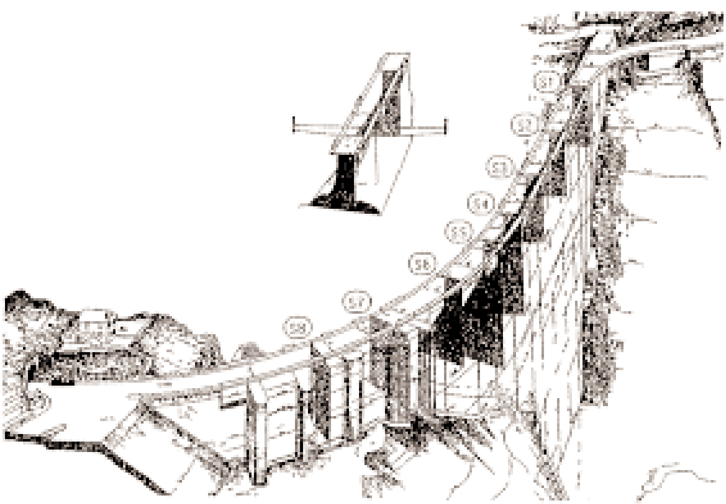


Fig. 3.3.1

Chambon Dam. Slotcuts

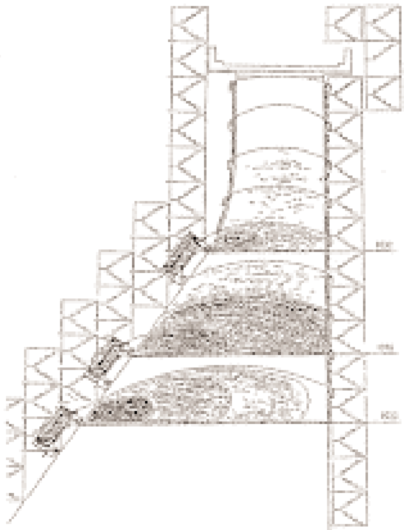


Fig. 3.3.2

Chambon Dam. Slotcut system

3.3.1.6 Waterproofing the Upstream Face

This technique for rehabilitation is used to counter many deterioration scenarios, particularly those whose intensity depends on the seepage of water through the dam. Hence, while typical case studies are listed at the end of this section, the topic is covered more comprehensively in section 3.3.3, dealing with the rehabilitation of the faces of deteriorating dams. Important developments have recently been made in the installation of geomembranes underwater, with the reservoir water level retained at or near its operating level.

3.3.1.7 Post Stressing or Additional Weight

Post stressing and additional weight have been used as components in the treatment of the problems caused by alkali silica reaction. There is considerable doubt however that this is a practicable long term solution. The force required to restrain the expansion caused by the reaction becomes impracticably large. However, the following case histories reveal that prestressing has been used for specific purposes.

Hiwassee Dam (USA) is a concrete gravity structure 94 m high, completed in 1940. To limit the hazard posed by the dam it was modified between 1992 and 1995. Expansion of the concrete was caused by alkali-aggregate reaction resulting in high stresses and deflections in the dam. The nonoverflow sections were post-tensioned in the autumn of 1992 using high-strength multistrand tendons. Two 12 mm slots 15 m deep were cut at blocks 7/8 and 16/17 in the winter of 1992. Other modifications that are required to correct the AAR growth problems over the next 25 years include recutting of closing slots, instrumentation for monitoring the slot behaviour, and permanent monitoring. (USCOLD 1996).

Kamburu Dam (Kenya) is an asphaltic concrete-faced rockfill dam with a separate concrete spillway 56 m high, completed in 1974. Although not dealing with the dam proper, the problems are the same and therefore shown here. In 1982, after 8 years of operation, relative movements were observed between pier 1 and the adjacent stoplog storage. The movement was monitored for 3 years and showed an acceleration of movements during periods with high storage water level. Examination of the spillway revealed alkali-aggregate reaction, cracks, water seepage, and concrete pop outs. The remedial measures placed emphasis on directing water away from the affected concrete, by means of reinforcing the grout curtains, installing water inhibiting devices on construction joints, installing drainage holes into the spillway abutments and by improving surface water drainage. In addition 24 x 1500 kN rock anchors were installed into the abutment rock and stressed to only 600 kN, to allow for further adjustment (Sims et al, 1988).

Gmünd Dam, initially an arch dam in Austria. It is 38 m high, and was completed in 1945, modified in 1993. Continuous measurements revealed that the crest moved at an annual rate of 1.1 mm upstream and 1.9 mm vertically upwards. A great number of cracks have occurred at the upstream side, some cracks were found at the downstream face. In 1989 the reason was found in the alkali-silica swelling of the concrete. The dam was converted to a gravity dam. For this, a 5 m high supporting concrete buttress was placed at the downstream face of the dam. Special care was taken in the selection of the aggregate, which was transported over a distance of 50 km. Also influenced by the swelling was the design of the crest. The crest of the old arch dam was removed down to the level of a horizontal crack that leaked. The deeper crest was covered with a plate with a large waterstop to allow the swelling movements without penetration of silt or any suspended material into the joint. In June 1993 the joint between arch dam and gravity dam was filled for the first time and with it the gravity dam was loaded for the first time. (Wallisch, 1993, and Heigerth et al, 1994)

3.3.1.8 Break out the Damaged Materials

Wimbleball Dam (UK) is a mass concrete buttress dam, some. 60 m high, completed in 1977, modified in 1990. The concrete surface of the dam has suffered from spalling. Even towards the end of the construction of the dam, small pop-outs were observed on the inside concrete faces of the dam parapet walls. These were 10 mm in diameter with soft dark aggregate (pyrites-rich shale) exposed and a rust coloured stain below. The reaction with the calcium in the cement matrix to produce ferric hydroxide and gypsum, which have about, double the original volume. It was considered that the mass concrete dam is safe. Thus the reinforced concrete valve tower and access bridge only need to be protected. The work involved breaking out all patches of soft or reactive aggregate near the surface and patching with a cementitious mortar, then coating the whole surface to reduce the quantity of oxygenated water penetrating the surface. (Phillips et al, 1994).

The following Table lists case histories of remedial measures against chemical reaction resulting in swelling

Nature of Rehabilitation	Location	Name of the Dam	Reference
Cutting slots, water tightening by sealing with a PVC membrane treatment, drain construction, strengthening by grouting, reinforcement	Dam body	Chambon, France	Bister et al, 1991 Taddei et al, 1996
Post stressing	Dam body	Hiwassee, USA	USCOLD, 1996
Post stressing or reinforcement	Dam body	Kamburu, Kenya	Sims et al, 1988
Additional weight	Dam body	Gmünd, 38 m high VA/PG, built in 1945, Austria	Wallisch, 1993
Break out the damaged material	Downstream face	Wimbleball, UK	Phillips et al, 1994
No measures necessary	Downstream face	Val de la Mare, UK	Haws et al, 1994
Upstream PVC barrier	Upstream face	Pracana, Portugal	Silva Matos et al, 1993
Sealing with a PVC membrane	Upstream face	Illsee, Switzerland	Regamey et al, 1995

3.3.2 Shrinkage and Creep Leading to Contraction

3.3.2.1 Introduction

These phenomena are more important early in the life of concrete. But this scenario also includes cases in which the concrete contraction continues for decades, to the extent that it may significantly affect the dam. The effects of the contraction are usually less important in massive gravity dams than in thin arch, buttress and multiple arch dams. More highly stressed structures appear to be more affected by the more highly stressed layer near the surface that is characteristic of shrinkage and creep. (Mehmel, 1964, Hoffmann, 1985, Baecker et al, 1985). Large masonry dams without construction joints, such as for example the **Möhne Dam** in Germany (Idel et al, 1977, Idel et al, 1982) tend not to show shrinkage or creep problems. ICOLD Bulletin 93 lists altogether 23 case histories in which shrinkage and/or creep was relevant. (ICOLD, 1994)

Shrinkage is the deformation of concrete associated with the chemical processes of setting. Drought promotes shrinkage as do high temperature and high cement content in the concrete. Creep is time dependent, inelastic deformation behaviour of concrete under load. The creep rate tends to be high if the concrete is loaded soon after setting and smaller if the chemical processes have come to an end before loading.

Shrinkage and creep effects should be anticipated at the design stage and provisions such as construction joints should be arranged. But it is difficult to predict in advance the resulting displacements, especially at structurally complex locations, such as the foundations of thin arch dams.

3.3.2.2 Detection and Monitoring

Long term monitoring of displacements, especially at critical regions within the dam, by means of geodetic survey, plumb lines, or extensometers is an appropriate means for detecting effects under this scenario. Measurements of foundation displacements, using long-base strain gauges and rockmeters, uplift pressure and seepage give valuable information. Visual inspection is also valuable in describing cracks. The development of cracks may be monitored by means of joint indicators. Important are laboratory tests showing the elastic, creep and shrinkage parameters (Sinniger et al, 1994, and Sinniger, 1994a)

3.3.2.3 Effect on Safety and Performance

The effects of shrinkage and creep can be extensive cracking with the risk of heavy seepage and potential loss of strength.

3.3.2.4 Rehabilitation Measures

The principal method of rehabilitation is to seal cracks with mortar, with cement grout or with epoxy resin. Structural strengthening may be appropriate. Beyond this monitoring must be intensified and it may be found that the dam achieves a new stable equilibrium as the deformation and creep reduce in intensity as the concrete gains strength with age.

3.3.2.5 Monitoring

Vouglans Dam (France) provides an example where investigation and re-analysis confirmed that no physical work was necessary. This is a concrete arch dam, 130 m high, completed in 1968. The instruments installed in the dam revealed deformation and creep in its early years. Careful measurements and subsequent finite element analysis confirmed that the dam had achieved a new position of equilibrium. No rehabilitation work was considered necessary (Bister et al, 1991).

Garichte Dam (Switzerland) is a second example where it was decided that physical rehabilitation was not appropriate. It is a concrete gravity dam, 42 m high, completed in 1931, investigated and grouted in 1932, 1961, 1966, and 1988. During the operation a permanent downstream inelastic deformation of the crest was observed, reaching 23 mm over a period of 58 years with a substantial acceleration over the last 10 years. It was unclear whether the deformation was caused by concrete ageing or by foundation movements or by both. Therefore intense investigations at the site and in laboratory were carried out through which it was discovered that the dam was constructed of poor quality concrete of high porosity made of coarse grained Portland cement with too much water. The concrete has improved its mechanical properties over the years. The permanent downstream deformations observed in the central part of the main dam are not due to creep. They may be attributed to the effect of considerable temperature variations together with the combination of concrete and square stones forming the facing of the dam (Sinniger et al, 1994, and Sinniger, 1994a).

3.3.2.6 Structural Strengthening Works

Olef Dam is an example (Mehmel, 1964, Hoffmann, 1985, Baecker et al, 1985) where it was found necessary to strengthen the dam because of the combined influences of creep, shrinkage and thermal cycling. Olef Dam is a buttress dam in Germany, built in 1955 of un-reinforced concrete. During construction shrinkage cracks appeared in the buttresses. The dam and the cracks are shown on Fig. 3.3.3 and 3.3.4. Additional reinforced concrete support was provided on the inner side of each buttress. This is 400 mm thick at higher levels and 600 mm below (Fig. 3.3.5). A concrete thermal protection wall was installed at the downstream face of the dam between the buttresses in order to reduce the magnitude of the temperature variations associated with climate and season.

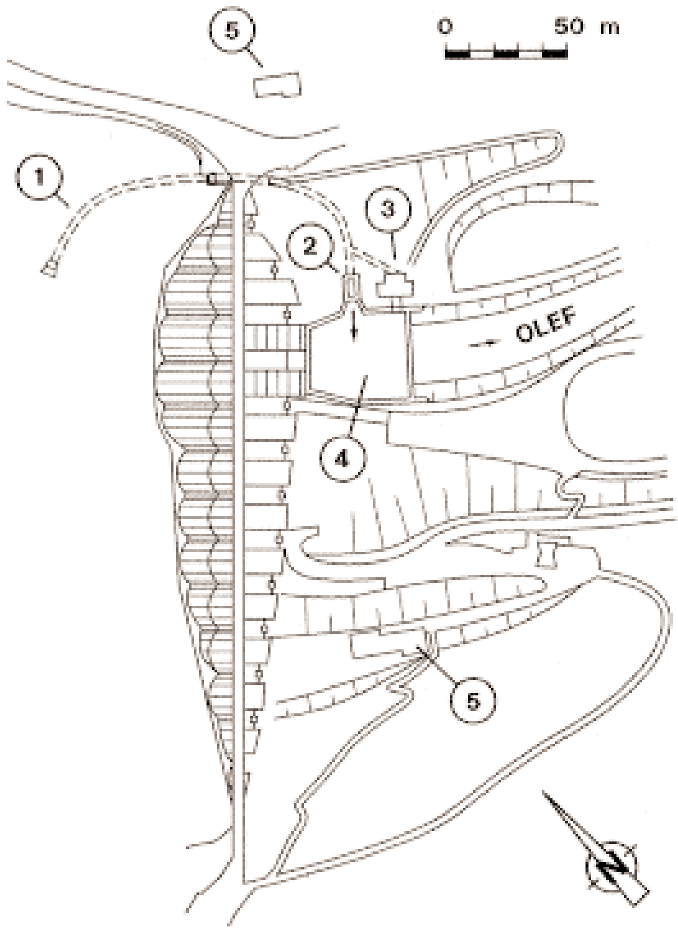


Fig. 3.3.3

Olef Dam. Plan

1. Bottom outlet *3. Power station* *5. Operations building*
2. Valve house *4. Stilling basin*

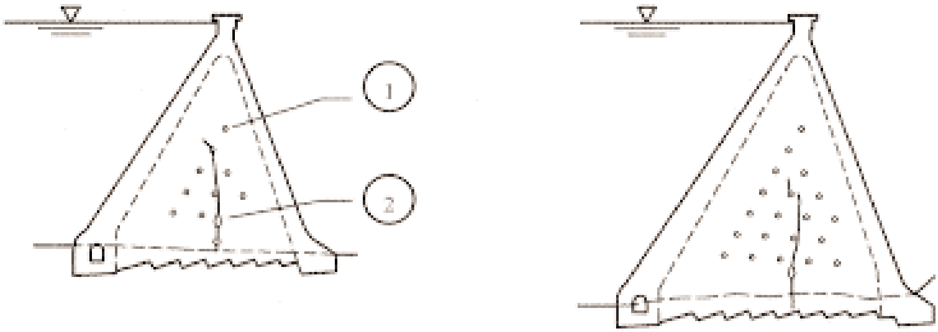


Fig. 3.3.4

Olef Dam. Typical crack pattern in the buttresses

1. Air circulation holes
2. Access hole

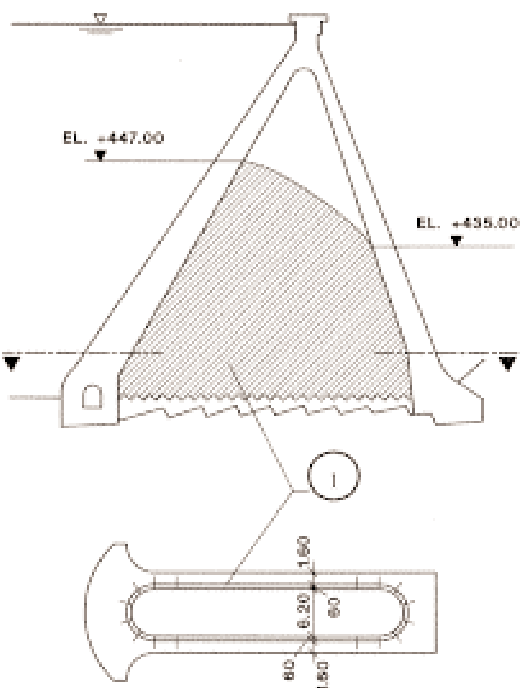


Fig. 3.3.5

Olef Dam. Strengthening of the buttresses

1. Reinforced concrete support

After 10 years of operation of the reservoir, hair cracks were discovered in the upstream face of the dam. Measurement confirmed that the cracks were nearly closed in winter and up to 0.5 mm wide in summer. Typical cracks are shown on Fig. 3.3.6. The cracks were concentrated in the area of the face subject to fluctuating water level. On average the cracks were 200 mm to 250 mm deep, but one was measured to be 800 mm deep.

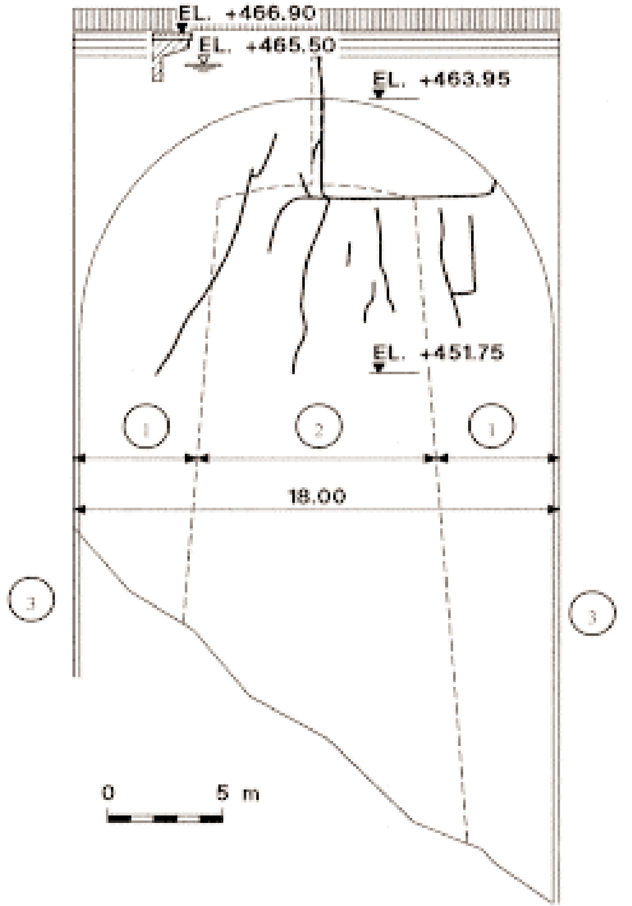


Fig. 3.3.6

Olef Dam. Crack pattern at the upstream face

1. Tension zone
2. Compression zone
3. Joint between buttresses

Finite element simulations showed that within the compression zone of the buttress the stress within the concrete was within allowable limits. But in the areas subject to tensile stress, especially remote from the supporting pillar walls, the tensile strength of the concrete would be exceeded if the crack development were not stopped. Therefore as a first measure the reservoir level was lowered by 10 m.

A 500 mm thick reinforced concrete shell was placed over the upper 17.5 m of the upstream face. Prestressed anchors were used to compress the cantilevers to eliminate the tensile stresses. (Fig. 3.3.7). The cantilevers are supported from the downstream side by a reinforced concrete construction supported on the strengthened buttresses. (Fig. 3.3.8).

Fig. 3.3.7

Olef Dam. Rehabilitation of the upper part

1. Load distribution	*4. Supporting concrete*
2. Reinforced concrete shell	*5. Prestressing force*
3. Thermal insulation wall	*6. Mass concrete*

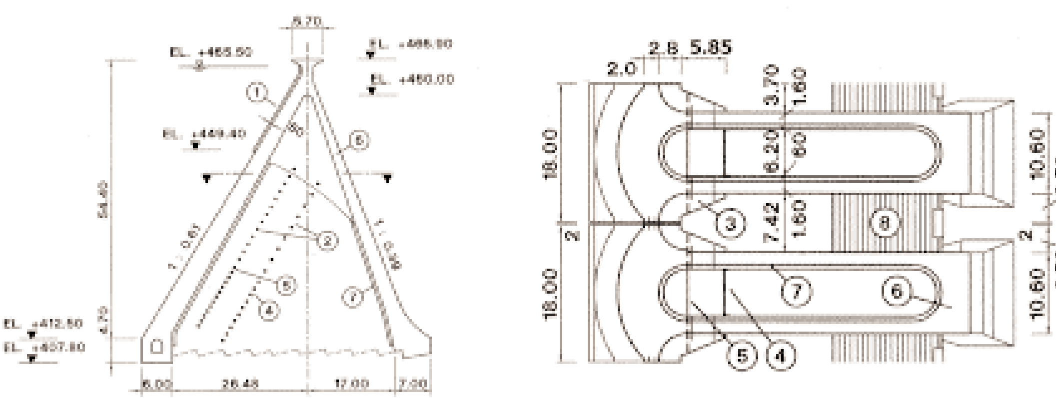


Fig. 3.3.8

Olef Dam. Rehabilitation of the lower part

1. Reinforced concrete shell	*5. Tension element*
2. Precast concrete beams	*6. Mass concrete*
3. Supporting concrete	*7. Reinforced concrete strengthening*
4. Strut	*8. Thermal insulation wall*

There are two separate requirements governing the contact between the new concrete and the old. To resist water loading it was desirable that the two concrete elements should work together, requiring a close connection. However it was considered important that the old concrete should be allowed to respond independently to shrinkage, creep and the effects of thermal cycling. In an attempt to resolve this flat jacks were installed between the old and new concrete. They were adjusted during concreting to allow movement and subsequent grouting. (Fig. 3.3.9).

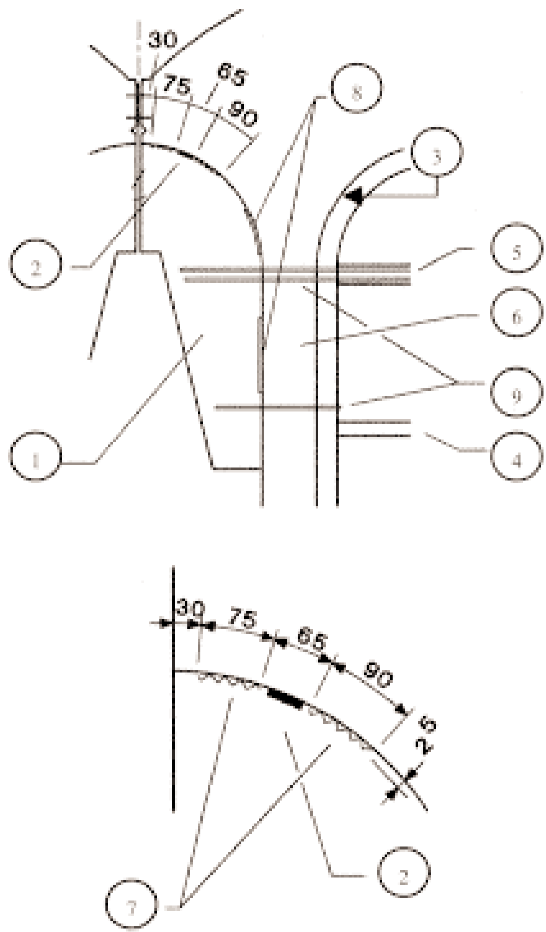


Fig. 3.3.9

Olef Dam. Jacks and injection gaps between cantilever and reinforced concrete support

1. Supporting concrete
2. Flat jacks
3. Reinforced concrete strengthening
4. Strut
5. Tension element
6. Mass concrete
7. Grouting paths of corrugated steel sheet
8. 50 mm thick ‘Poron’
9. Prestressing force

A compilation of recent case histories concerning remedial measures against shrinkage and creep leading to contraction, include the following :

Nature of Rehabilitation	Location	Name of the Dam	Reference
Grouting the dam body	Dam body	Garichte, Switzerland,	Sinniger et al, 1991
Strengthening of pillars	Dam body	Olef, Germany	Mehmel, 1964 Hoffman, 1985 Baecker et al, 1985
Drain reconstruction	Dam body	Vouglans, France	Bister et al, 1991

3.3.3 Degradation at Dam Faces

3.3.3.1 Introduction

Degradation of the mechanical and hydraulic properties of concrete and masonry dams may be caused by chemical reaction between the materials and external agents from the environment, including reservoir and underground water, air and temperature changes. There are four major parameters governing the extent of degradation :

- the permeability of the dam body, which controls the volume of water and gas flowing through it
- these pore fluids may react aggressively with concrete and mortar constituents
- temperature variations cause both cracking and the opening of joints, thereby influencing the overall permeability of the structure
- the growth of plants especially in the joints of masonry.

Degradation is more serious for structures of low quality built in severe climates, structures containing thin elements and those exposed to the action of aggressive waters. The sulphate action mentioned in section 3.3.1 might also be included in this scenario. ICOLD reports 142 case histories concerning this scenario in Bulletin 93 (1994). Of these 51 concern the permeability, 65 the action of aggressive water on concrete and mortar, and 26 cracking resulting from external temperature variation. Many more case histories probably exist.

Concrete and masonry structures exposed to water in cold climates may be damaged owing to their poor resistance to repeated freezing and thawing. The damage is mainly superficial, but nonetheless serious. The properties of materials, quality of construction and the severity of the exposure determines the extent of degradation. Damage of concrete and mortar by frost action depends on the number of freeze-thaw cycles and on the moisture content of concrete as it freezes. The number of freeze-thaw cycles is higher at the beginning and at the end of the cold season. Concrete is particularly vulnerable to frost damage when the moisture reaches a critical saturation point (Vuorinen, 1969). Other factors that may influence the frost action are the rate of cooling, the lowest and highest temperatures attained during the cycle, and the cycle duration.

ICOLD has reported 87 case histories in Bulletin 93 (1994). It also contains an account of the physical processes involved. Frost action has a different severity depending on where it affects the structure :

- Damage on the upstream face is usually limited to the part that is alternately wetted and dried.
- Damage to downstream face is particularly likely where the dam has a high rate of seepage, or where precipitation is heavy.
- The dam crest is sensitive to freezing, as water ponds on the surface.

Materials at the dam faces, being in direct contact with the environment are under the aggressive influence of external agents, particularly water at the upstream face and weathering at the downstream face. Skin effects develop, owing to temperature, moisture, frost, snow, ice, wind and rain, as well as chemical reactions with the elements transported by the water. The major actions of water pressure, temperature variations and earthquake usually lead to stress redistribution within the dam body with highest stresses developing near the dam faces. Therefore, in order to prevent the deterioration of the dam materials near the dam faces, and later into the dam body, it has become current practice to use materials of higher strength near the faces.

Watertight upstream facings were used in old masonry dams and in concrete dams constructed before the development of mass concrete technology. Typical of these is the Lévy type multiple arch concrete structure formed by small arches supported on the upstream face of the dam. Also used were facings of steel, squared stone pitching and earthfill, typically the Intze wedge so often found at older European masonry dams. Downstream faces were protected by squared stone pitching and by means of earthfill. These facings are not used today and modern facings have since replaced many of them.

As concrete technology improved it became possible to produce mass concrete watertight and strong enough to dispense with the use of special facings. Thus the use of facings in concrete dams was abandoned, but the practice persisted of using concrete with a higher cement content near the upstream face, together with steel reinforcement near both upstream and downstream faces. This reinforcement was designed to control the skin effects, preventing the development of large cracks and spalling of the face. Facings of conventional concrete, used also as forms during construction, have been incorporated in RCC dams. Recently some RCC dams have been designed with a PVC membrane on the upstream face as the sealing element (Giovagnoli et al, 1996).

3.3.3.2 Detection and Monitoring

Visual inspections, water flow measurements, and coring to obtain samples for laboratory tests are the most common means of detecting the scenario. In some cases geophysical tests can be used with success. Chemical analysis of the water, water absorption tests and cement grouting tests in holes drilled into the dam body, have also been found useful. Measurement of in-situ density and in-situ sonic velocity has also been useful in identifying poor quality masonry and concrete.

For concrete and masonry of good quality, the boundary between damaged and sound zones is generally clear and can be detected by hammering. For poor quality concrete and masonry, the damage extends deeper into the structure, and sampling is necessary to find the boundary between damaged and sound zones. The extent of seepage or low pressure water pressure tests may give valuable information on the extent and nature of the damage.

3.3.3.3 Effect on Safety and Performance

Loss of structural concrete or masonry may lead to overstressing or to accelerated or aggravated damage in areas subject to flowing water, where the concrete loosened by the effects of freezing and thawing is plucked out by the action of the water. A serious issue arises when particles or pieces of concrete fall from a damaged structure.

3.3.3.4 Remedial Measures

Remedial measures are broadly divided into two types. The first is the prevention of water from entering the dam body, the second reducing the effects of extreme temperature variations. Both of these share techniques such as the replacement and repair of the upstream face. Both approaches are discussed below.

3.3.3.5 Prevention of Water Percolation

Methods currently available to reduce or eliminate percolation through the dam include the following. For all of these it is usually necessary to empty the reservoir prior to the work, although development work is reported in placing PVC membranes underwater (Scuero et al, 1998) as discussed further below.

- barrier coatings to the upstream face of a variety of materials including bitumen, mastic asphalt, resins, chemical impregnation, concrete, metal or PVC membranes
- grouting the upstream part of the dam body
- repair of the facing.

Typical examples of modern rehabilitation under some of these headings are described below.

Barrier Coatings

Agger Dam in Germany is a concrete gravity dam, 45 m high, completed in 1928, and modified in 1967. The upstream face of the dam was attacked severely by the water for a period of 40 years and considerable water losses due to seepage and other phenomena finally required rehabilitation measures. This consisted of a complete renewal of the crest and sealing of the upstream face. The latter comprise a 120 mm asphaltic concrete seal behind a 280 mm reinforced, nearly vertical concrete protection wall fixed by anchors to the dam body. (Feiner et al, 1967). Although this had been done 30 years ago the owner remains satisfied with this solution (Fig. 3.3.10 and 3.3.11).

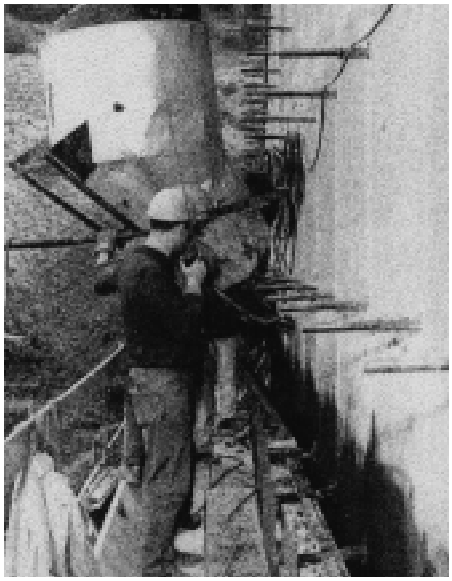


Fig. 3.3.10

Agger Dam. Protection wall and anchors

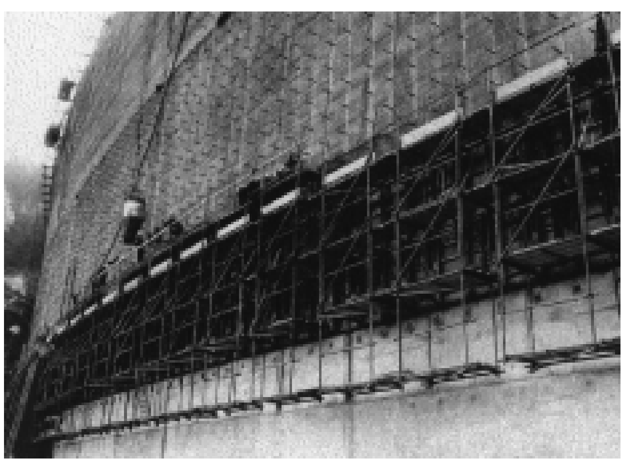


Fig. 3.3.11

Agger Dam. Casting the vertical asphaltic concrete seal

Saints-Peyres Dam (France) : Concrete gravity dam, 61 m high, completed in 1934, modified in 1994. The shotcrete of the upstream face showed cracks and splitting favouring the infiltration of water in the dam body. The upstream sealing has been reconstructed. (Puydebois et al, 1995)

Considerable development is taking place in the application of geomembranes for the rehabilitation of ageing masonry and concrete dams. In 1991, ICOLD Bulletin 78 listed 70 dams waterproofed with geomembranes. By 1999, the number exceeded 150. Scuero et al (1998) refer to the successful repair of **Pracana Dam** in Portugal. This is a 65 m high buttress dam in Portugal built between 1948 and 1951. Comprehensive rehabilitation works followed the confirmed presence of ASR. This included treatment of foundations, injection of a new grouting curtain, construction of a new foundation beam and of two sets of concrete struts between the buttresses on the downstream face. All the cracks were mapped and strengthened by grouting. A new separate spillway and a new water intake were constructed. A drained PVC geomembrane was installed on the upstream face not only to inhibit the flow of water feeding the reaction but also to reduce the uplift pressure in the horizontal cracks in the dam body (Silva Matos et al, 1993). The PVC membrane is connected to the new foundation plinth to achieve continuity between it and the new grout curtain. The efficiency of the system is monitored by separate measurement of water drained from the foundation and from the upstream face. To assist with this the membrane has been separated into ten compartments. An innovative system of wires has been installed between the concrete face and the new membrane to allow the use in future of an electrical leak detection system capable of defining the precise location of any damage to the membrane that might cause leakage.

Fig. 3.3.12 shows work underway on the **Illsee Dam** in Switzerland. This is a 36 m high arch-gravity dam built in 1924 (Regamey et al 1995). Displacements were

Fig. 3.3.12

Illsee Dam. Geomembrane lining

observed that were attributed to Alkali Silica Reaction and in 1997 an impermeable layer was installed on the upstream face in an attempt to reduce the seepage of water into the dam, hence to slow and ultimately to stop the expansion process. The dam is faced with rough masonry and transition layers of geosynthetic materials were used instead of more traditional methods to make good local defects and to prepare the dam surface for the application of the PVC membrane. The three materials shown on the photograph are :

- on the left, the impervious PVC geomembrane,
- on the right, the white material is a relatively thick geotextile, providing protection against puncture thus minimising preparation work on the rough masonry
- the black material above is the drainage mesh against the dam surface.

Drained geomembranes can resist high water heads. **Alpe Gera Dam** (Italy) is a concrete gravity dam, 174 m high, completed in 1964 using a technique similar to modern RCC dams, and with steel facing as impervious upstream membrane. The top 96 metres were galvanised and coated with vinyl acrylic paint. Areas below this were not painted. Most of the corrosion of the steel lining occurred in this lower area. Leakage increased to more than 50 litres per second. In 1994, a drained PVC geomembrane was adopted to protect the corroded steel facing. Steel patches were installed as anti-intrusion supports to the PVC geomembrane over the numerous holes formed in the steel. The amount of water leaking from the area covered with PVC was reduced to practically zero (Fanelli, 1998)

The underwater installation of a PVC geomembrane, accomplished in 1997 at **Lost Creek Dam** in USA (Harlan et al, 1998), represents a significant advance in rehabilitation of the upstream face of a dam. Rehabilitation without dewatering can provide substantial operational, economic and environmental benefits. Lost Creek Dam is a 36 m high arch dam built in 1923-24, whose reservoir cannot be emptied. The permeability of the concrete, and the consequent deterioration as a result of alternate freezing and thawing, resulted in progressive loss of concrete and decreasing strength of concrete at the downstream face. The stability of the dam was not an issue. The rehabilitation options considered included : chemical sealant, shotcrete, micro-cement grouting, RCC buttress on the downstream face and the installation underwater of an upstream PVC membrane system. The membrane was the preferred option. At present, this is the only reported case of complete repair performed underwater with a synthetic membrane.

ICOLD is revising Bulletin 78 on Waterproofing Geomembranes. It intends to include a database of dams where geomembranes have been installed as a rehabilitation measure, or since construction.

The use of geomembranes to rehabilitate embankment dams is discussed further in section 4.6 below.

Grouting the Upstream Part of the Dam Body

Kuromata Dam (Japan) is a concrete gravity dam, 24.5 m high, completed in 1927, and modified in 1981. The dam had served for more than 50 years with a progressive deterioration, resulting in considerable leakage from the downstream surface of the dam. Therefore, grouting was applied to the dam body and its foundation rock to reduce their permeability to water. To reduce leakage of grout, a hardening accelerator was used and the actual grout programme was based on the results of test grouting. This provided a successful improvement which has prevented further deterioration of the dam. (Tada et al, 1991). The injection hole interval was specified as 1.0 m pitch in one row for grouting of the overflow channel on the right-bank where the dam height is low. For grouting of the other parts of the dam, the injection hole interval was specified as 1.5 m pitch in two rows with a row to row interval of 1.0 m. As grouting material, hardening accelerator (DENKA ES) for soil stabilisation was used for injection into the dam. The gel-time of the material changes according to the amount of ES setter added. The usual mix pressures applied were dependent upon the depth (0 – 20 m, 3.0 bar ; 20 m to 30 m, 5.0 bar ; >30 m, 7.0 bar). The results may be seen tabulated below.

Section	Take of Material Before treatment (l/m)	 After injection (l/m)
Right-bank side overflow channel	352 – 2292	80 % of values <37
Centrally-situated river course zone	312 – 4308	80 % of values <45
Left-bank side flushing channel zone	304 – 3147	80 % of values <40

Mesce Dam (France) provides a second example of this approach. This is a masonry gravity dam, 65 m high, completed in 1917 and modified in 1967-1970. This old masonry dam, built without any drainage devices, did not meet the current standards. Among others the seepage at the whole downstream face contributed to this judgement. To minimise seepage, a drainage gallery had been excavated in the foundation of the dam and a system of vertical drains installed from the crest to the gallery. Epoxy resin grouting has been performed over the lower third of the height and near the upstream face, and a grout curtain carried out from the upstream toe into the bedrock. (CFGB, 1976)

Facing Repair

Sasanagare Dam was the first concrete buttress dam in Japan. It is 25.3 m high, was completed in 1923 and modified in 1983/84. Deterioration occurred at all parts of the dam, particularly the buttresses and slabs. A preliminary repair was made in 1948/49 but this only revealed temporary improvement to the extent that in 1965, concrete began to fall off from the surface of the dam. In 1983/4, after a careful ageing survey, the buttresses were overlaid at both sides by 700 mm to 1100 mm thick concrete layers and the upstream slab by 300 mm to 700 mm thick concrete. The whole dam was overlaid by new concrete. (Shibata et al, 1991)

The concrete mix used consisted of the following, expressed in kg/m^3 with 0.25 % of air entraining agent.

cement	234.6
admixture	41.4
water	144.0
sand	759
Aggregate 0.5 mm to 20 mm	562
Aggregate 20 mm to 40 mm	609

Similar work has been performed at the Olef Dam as described in section 3.3.2.

La Girotte Dam is a concrete multiple arch dam 510 m in length completed in 1949 and situated in French Alps at 1753 m above sea level. It comprises 18 vaults each about 2 m thick. Two main problems arose :

- In winter the reservoir water level is usually at the bottom of the arches and the effects on the concrete of freezing and thawing are at their maximum. Many cracks have been observed and these are attributed to thermal effects on the thin concrete elements.
- The quality of concrete is not suitable to combat the acid waters coming from the glaciers.

Maintenance is carried out on a routine basis. Access to the upstream face is possible in the interval between end of winter and the beginning of seasonal filling. There is time each year to rehabilitate only one or two vaults. This has given the opportunity to test a range of products with the following results (Bister et al, 1993). The upstream facing of the rehabilitated concrete has to be tough enough to resist the abrasive effect of the ice cover on the reservoir.

The performance of coatings of shotcrete or epoxy resin protected with a polyethylene coating was not encouraging. The best results were obtained by applying a resin facing on the concrete surface that had been cleaned by sandblasting. One coat of epoxy resin was applied and this was covered with two or three coats of polyurethane-resin, applied at a rate of about 1.5 kg/m^2. This technique has been used for 25 years, since 1970. It was found to last for 12 to 15 years with some local repairs. After this it was necessary to replace the facing. The major disadvantage was the time it took to complete the sandblasting to take off the old facing. This was considered to be too costly and alternatives were sought. Since 1994 the owner has experimented with a new PVC geomembrane facing. Five vaults have been covered and it is anticipated that the results concerning the behaviour against ageing will be better than those with resin. This PVC membrane is shown on Fig. 3.3.13 and 3.3.14.

Fig. 3.3.13

La Girotte Dam. Upstream face

Further examples of facing repairs are described below under the heading of Countering the effects of extreme temperatures.

Fig. 3.3.14

La Girotte Dam. Details of PVC facing

3.3.3.6 Countering the Effects of Extreme Temperatures

The most usual method of repairing frost-damaged concrete has been the construction of a new facing of frost-resistant concrete. The damaged concrete has been removed and a new facing constructed either by spraying or by casting new concrete in situ. Sometimes it has not been necessary to replace the whole face and a partial repair has been possible with a saving of money. Alternative facings have also been used including steel linings and epoxy resin coatings. Since 1970 the use of thick PVC has become popular, especially in Mediterranean countries. The use of geomembranes is discussed above.

Prestressing of concrete damaged by frost has also been used with success.

At the downstream face, it is important to design the remedial work so that water within the structure can escape. The downstream face must be permeable. Drains must be designed to be effective even in extreme weather conditions.

In Middle European countries where the climate is continental, with a wide range of temperature between winter and summer, it is customary to check the quality of stones and of mortar in masonry dams every few years. Not infrequently it is found that the stones have split, particularly along the cleavage planes in sedimentary rocks. Special problems occur with damaged stones exposed at corners of valve towers, for example, or at pillars. Damaged material has to be removed and replaced. Because it is difficult nowadays to find experienced craftsmen, the owner must ensure detailed supervision of the work. It appears that rehabilitation work of this kind to a lesser or greater extent has to be done each generation, that means, every 30 years.

Seepage Reduction and Insulation

Wissota Dam (North and South Section) (Wisconsin, USA) is a flat slab and buttress concrete dam of the Ambursen type associated with an embankment, 21 m high, completed in 1918. The sloping surface of the north and south hollow dams were affected by freeze-thaw actions. Deterioration of concrete was most severe at the buttress-slab, cold-joint junctions, where joint filler materials apparently failed, and near the edge of previous repairs. Rapid deterioration developed due to seepage through cracks and joints on the upstream slab, which saturated the downstream face. The dam was declared to be of high hazard and rehabilitation was carried out between 1988 and 1990. The remedial measures included installing steel cofferdam sheeting along the downstream buttress faces down to bedrock ; removal of soils between sheeting and slab face ; cleaning of rock contact surfaces and concrete, and mass concrete pours for entire height. The structure was monitored for movements by installing numerous strain gauges. (USCOLD, 1996)

The use of a thermal insulation against climatic extremes has been mentioned in connection with the Olef Dam (section 3.3.2).

Prestressing

Cedar Falls Hydro Dam (Wisconsin, USA) is an Ambursen-type buttress dam 15 m high, and was completed in 1912. The concrete buttresses and corbels were significantly deteriorated due to seepage coupled with freeze-thaw cycles to the extent that the structure was judged to be a high hazard. The dam was repaired in 1985-1986. The repair programme consisted of innovative corbel post-tensioning, as well as the more traditional repair using buttressing. (USCOLD, 1996).

Repairs to Facing

Haweswater Dam, UK, is a concrete buttress dam, 27.5 m high, completed in 1942 and rehabilitated in 1987. The buttresses forming the dam are dumbbell-shaped in plan, giving a continuous face downstream as well as upstream. The surface of the concrete showed spalling over 30 % of the downstream face, mainly in the lower two thirds of the face, with penetrations up to 100 mm (Hopkins et al, 1988). The problem appeared to be caused by high levels of condensation on the internal faces of the buttresses and of freezing and thawing of saturated concrete.

The whole of the downstream face was repaired using a three-element system. Following trials of commercially available materials the system used included the following elements :

- The concrete was prepared by mechanical cleaning and grit blasting.
- A surface sealer incorporating saline primer and acrylic top-coat. The purpose of the saline is to penetrate the concrete surface and by chemical reaction to form a hydrophobic block.
- A fairing coat or render which smoothes the surface, fills minor blemishes and helps to prevent weak spots through which the cementitious mortar can be attacked.
- A cementitious mortar applied to the damaged area with an acrylic bonding agent.

It appears that the repairs have been successful in the main. After ten years there is some cracking of the repair mortar. Seventeen 2 m long cores were drilled horizontally into the downstream face. Of these ten were in the original concrete and seven were through areas that had been repaired. In three of the cores the bonding to the original concrete was breaking down.

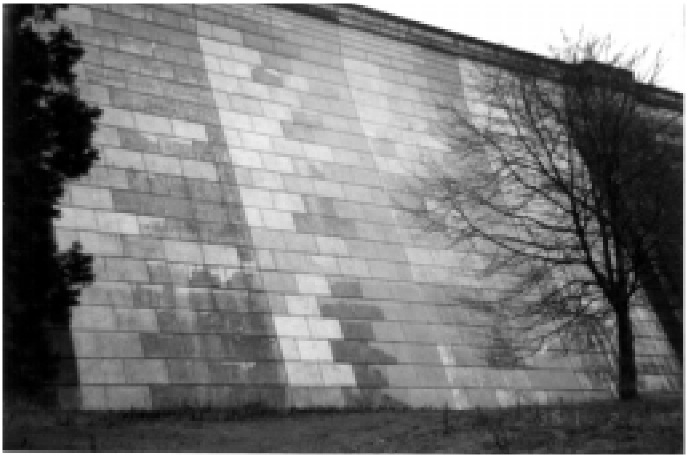


Fig. 3.3.15

Haweswater Dam. Downstream face after repair

Repairs to Damaged Masonry

The **Möhne Dam** in Germany is shown on Fig. 3.3.16 and 3.3.17 and 5.4.1. The masonry gravity dam is 40 m high and 650 m long and was completed in 1913. The dam was damaged by bombing in 1943 and repaired rapidly in the same year. The downstream face of the dam had not been investigated since the reconstruction in 1943. At the beginning of the nineties, a detailed inspection was made of the 25 000 m^2 of the downstream face using specialised access platforms. It revealed that extended rehabilitation work was necessary. The masonry was repaired by replacing stones of the face and by renewing the mortar in joints. Afterwards the faces of the two towers of the dam were rehabilitated in the same way and this was followed by rehabilitation of the 105 pillars between the flood escape openings at the crest. Altogether 1500 m^2 of stones were replaced and 15 000 linear m of joints treated.

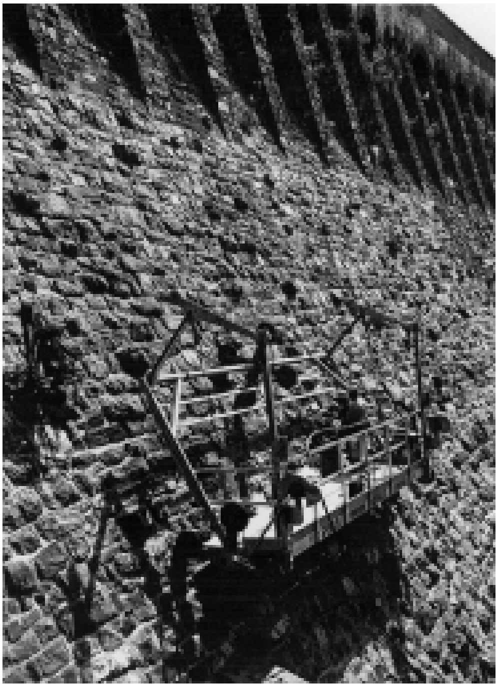


Fig. 3.3.16

Möhne Dam. Inspection and working platform

Fig. 3.3.17

Möhne Dam. Working from the platform

Some recent case histories concerning remedial measures against degradation due to chemical and physical actions are compiled below.

Nature of Rehabilitation	Location	Name of the Dam	Reference
- Sealing of upstream face with elastomeric polyurethane	Upstream face	Saints-Peyres, France	Puydebois et al, 1995
- Sealing of upstream face with epoxy resin	Upstream face	Mesce, France	CFGB, 1976
- Increasing thickness of upstream wall	Upstream wall	Magaribuchi, Japan	Matsuhuij et al, 1993
- Upstream concrete lining	Upstream face	Hohnen-Ike, Japan	Fujisawa, 1994
- Barrier coatings	Upstream face	Agger, Germany	Feiner et al, 1967
- Sealing dam body with a hardening accelerator	Dam body	Kuromata, Japan	Tada et al, 1991
- Reconstruction of deteriorated zones, new dam crest		Rempen, Switzerland	Zurfluh, 1985 b
- Reinforced shotcrete	Upstream face	Schöni, Switzerland	Diethelm, 1983
- Reinforced shotcrete	Upstream face	Schräh (Wägital), Switzerland	Zurfluh, 1985 c Bischof, 1993
- Sealing with PVC-membrane	Upstream face	La Girotte, France	Charvoz, 1995 Bister et al, 1993
- Seepage reduction and insulation		Wissota, USA	USCOLD, 1996
- Post-tensioning of cracked corbels of a buttress dam	Corbels	Cedar Falls Hydro, USA	USCOLD, 1996
- Successful application from PVC membrane	Upstream face	Lost Creek, 36 m high VA, USA	Harlan et al, 1998
- Sealing by acrylic bonded cementitious mortar	Downstream face	Haweswater, U.K	Hopkins et al, 1988
- 800 mm of frost resistant concrete, successful	Downstream face	Pack, Austria, built 1930, PG 33 m high	Kroell, 1991
- Successful repair of masonry	Downstream face, valve towers,	Möhne, Germany	Rißler, 1994, Rißler, 1998

3.3.4 Loss of Strength Due to Permanent or Repeated Actions

3.3.4.1 Introduction

This ageing scenario concerns the inability of some concrete and masonry dams to withstand the actions occurring under normal operating conditions. Permanent, steady state loading is distinguished from oscillating or transient loads. Clearly the water load is the most significant load on the dam but temperature variations are important too. Shortcomings in the ability of the dam to withstand the loading will often be revealed by cracking. The development of cracks sometimes begins during construction and develops over time, its influence on the dam behaviour only becoming significant after some years. ICOLD have reported 75 case histories under this scenario in the ICOLD Bulletin 93 (1994).

Filling and the routine operation of the reservoir lead to permanent and repeated variation in water pressure. The resulting hydraulic gradients cause seepage through pores and cracks in the dam, with associated fields of pressure and force. The operation of spillways, bottom outlets and powerhouses may also contribute to variations in hydrostatic and hydrodynamic pressures on the structure.

Temperature variations of a daily or seasonal nature influence the dam behaviour throughout its life. The magnitude of thermal deformations depend not only on the type and thermal properties of materials used, but also on the design and the construction techniques. The magnitude of the deformations and consequent problems are likely to be in proportion to the range of external temperature variations.

Influences from outside may also threaten a dam, especially when it is sensitive against deformations as it usually is at arch dams.

3.3.4.2 Effect on Safety and Performance

The inability of masonry or concrete dams to withstand permanent or repeated actions is manifested by cracks and by their development with time. To evaluate the safety and performance of cracked structures it is necessary to understand the geometry of the cracks, particularly their extent, their density, their depth, their continuity, and the origin of cracking. The change of the cracks with time is important. It is then necessary to analyse the behaviour of the structure under normal and under exceptional conditions. The consequences of these developments depend on the type and characteristics of each scenario :

For gravity dams, hydrostatic pressure may cause tensile stresses near the upstream face that exceed the tensile strength of the material and lead to horizontal cracking or opening of construction joints. Repeated variation of reservoir water level may increase cracking and thereby make possible the development of leakage paths and associated uplift pressure.

Cracking may appear in arch dams in many forms (ICOLD 1994) :

- horizontal cracking, typical of poor lift joints ;
- cracking near the abutments, approximately normal to the support surface, usually occurring early in the life of the structure without subsequent development ;

• cracking with random pattern ;

• cracking near reinforced zones ;

• cracking at the boundaries of zones of different inertia or stiffness.

Some cracks are not critical to the safety of the dam, others are. Cracks in arch dams, occurring during the first impoundment or even after many years, has threatened the survival of the whole construction and has led to emergency rehabilitation work (Zurfluh, 1985, Mueller et al, 1985, Pougatsch, 1982, Berchten, 1985, and Schöberl et al, 1993).

Cracks occur frequently in massive and hollow buttress dams. Vertical cracks in the buttresses and in the webs of the hollow blocks usually occur at an early age when they may be explained by the effect of the dissipation of the heat of hydration and to shrinkage. They are not ageing phenomena. During operation they may develop as a consequence of hydraulic and thermal cycling, large bending stresses and cracking may develop on the faces of buttresses and between the inner and outer faces of the hollow chambers. See also the discussion on Olef Dam in section 3.3.2 and Mehmel, 1964, Hoffmann, 1985, and Baecker et al, 1985.

Cracking in multiple arch dams usually develops in the buttresses, and often follows the geometry of the principal compressive stresses, taking the form of arches from the upstream face towards the foundation. Cracks are also found in the arches, particularly near the support of the arches and near the foundation.

3.3.4.3 Methods of Detection

The cracking resulting from the effect of repeated loadings is generally detected visually and through seepage measurements. Quantitative data are required to assess the extension and rate of growth of the cracks. This will be obtained by regular measurement of the movements of selected cracks. When the cracking of a dam progresses, there will come a time when a review will be required of its safety and performance. This may include complete mapping of all existing cracks, not only those readily visible from accessible areas, including for example the crest, the upstream and downstream faces and galleries. Divers and closed circuit television cameras can inspect superficial cracks in the concrete underwater. Cracks and voids within the dam body may be investigated using core drilling and water pressure tests or ground-penetrating radar. Owing to the large dimensions of the structures, special care will be required in designing an appropriate approach to these studies. Further understanding of the processes at work and the condition of the dam can be obtained by determination of the physical properties of the concrete, joints and cracks by means of laboratory tests. Alternatively or additionally, measurement of the in-situ stresses in the dam body by means of flat jacks and overcoring methods, among others may be helpful.

3.3.4.4 Remedial Measures

The remedial measures for this scenario aim at reducing or removing the causes of cracking by strengthening the structure or by the physical protection of exposed faces. The aim is to reduce the importance of the fluctuating loading on the structure or to reduce the range of temperature variations it experiences. These rehabilitation measures include therefore :

Strengthening the structure to reduce the level of stress in critical locations by :

- grouting in joints and cracks ;
- construction of additional buttresses and backfilling, and filling of bays between buttresses and hollow chambers in buttress and multiple arch dams ;
- providing additional vertical loading through prestressed anchors or additional mass.

Reducing the range of temperature variations by :

- physical insulation and protection of the faces of gravity, arch and multiple arch dams ;
- closing the bays between buttresses to reduce the circulation of air ;
- grouting works, to reduce leakage through open joints and cracks ;
- improvement of drainage.

Strengthening Using Prestressed Anchors

La Bourne Dam (France) is a masonry gravity structure, 18 m high, completed in 1878. This is an example of a dam rehabilitated through the addition of prestressed anchors. It is discussed in a little more detail in section 3.3.6. (Lino et al, 1991).

Strengthening by Remodelling

Ternay Dam (France) is a concrete gravity dam, 41 m high, completed in 1867, modified in 1990. Stability problems and cracks were repaired by strengthening by shape correction. An earth and rockfill embankment was placed downstream (Lino et al, 1991).

Strengthening by Grouting

Zillergründl Dam (Austria) is an arch dam, 186 m high, completed in 1985, and modified in 1987 and 1888. In September 1987, during first impoundment, at a water depth of 175 m, suddenly a 1 mm wide flat crack developed in block 10 near the dam bottom, extending to an area of about 350 m^2, and allowing the release of some 160 l/s of seepage water.

Rehabilitation measures consisted of force-locking injection of the crack with synthetic resin carried out in two stages. The crack area was opened with grout holes. The primary injection was carried out with the reservoir water level of 120 m, whereas the secondary injection took place with the reservoir empty. As a result of the level of the injection pressure of the grout secondary joints appeared and also had to be sealed. Among the lessons learnt was the need to design the level of grout pressure taking account of the details of the actual conditions. Specifically, to ensure that local tensile stresses do not exceed the tensile strength of the rock or concrete. (Schoberl et al 1993)

Some recent case histories concerning remedial measures against loss of strength due to permanent or repeated actions are compiled below.

Nature of Rehabilitation	Location	Name of the Dam	Reference
Grouting the dam body	Dam body	Roselend, France	CFBG, 1976
Strengthening by anchors	Dam body	La Bourne, France	Lino et al, 1991
Downstream earth and rock embankment with	Dam body	Ternay, France	Lino et al, 1991
Reinforcement	Dam body	Hohnen-Ike, Japan	Fujisawa, 1994
Strengthening by resin grouting, successful	Dam body	Zillergründl, 186 m high, 506 m long, VA, built in 1986, Austria	Schoeberl et al, 1993 Schoeberl 1996
Reinforcement, thermal insulation	Dam body faces	Olef, Germany	Mehmel, 1964 Hoffmann, 1985 Baecker et al, 1985

3.3.5 Structural Joints

3.3.5.1 Introduction

Joints are man-made discontinuities in the dam structure that result from the construction process, or are introduced to improve the structural behaviour. Construction joints, are interfaces between successive construction lifts or between different materials, such as those between dam and foundation, or old and new materials when repairing or rising the works. Contraction joints are called transverse or longitudinal, depending on their orientation in relation to the dam axis. They prevent cracking of mass concrete due to the shrinkage and temperature variations developed in the construction phase. Peripheral construction joints in arch dams allow relative movement between the springing of the arches and the pulvino. Construction joints at the crown of arch and multiple arch dams decrease the crown stiffness. They also separate the dam body from adjacent structures, for example plinths at the dam heel or spilling basins ; they allow differential movements in the foundation (ICOLD, 1994, Gilg et al, 1987, Demmer et al, 1985).

Construction joints must ensure adequate bond, shear strength, and watertightness, between the two surfaces (Pacelli et al, 1993). This is true too for joints designed to improve structural behaviour. Contraction joints are usually grouted in the cold season, after dissipation of the shrinkage effects and cooling of the concrete. Old concrete and masonry dams, and some recent RCC concrete dams, have been built with few or no contraction joints. Although joints generally increase the cost the need for them, even in RCC dams, is usually accepted.

To ensure the watertightness of a joint, waterstops are placed near the upstream face. Waterstops of copper, stainless steel, natural and synthetic rubber and most commonly of polyvinyl chloride are in use.

3.3.5.2 Detection and Monitoring of Deterioration

Deterioration of joints leads to loss of their mechanical and hydraulic properties, particularly the bond and shear strength, and the watertightness. In general, joints are the weakest parts of a dam. Tensile, compressive or shear stresses, as well as opening, closing and shearing movements between the sides of the joints accelerate their deterioration. This scenario is worsened with the leakage through the joint and development of uplift forces. The stability of large blocks of the dam may then be endangered, and leaching of the concrete or of the mortar may occur in the course of time (ICOLD, 1994, Léger et al, 1997).

Deterioration of waterstops is frequently related to damage of the two corners of the joint or to the upstream facing. Waterstops are more vulnerable under high water pressure, particularly where there are significant and frequent fluctuations of the reservoir water level (ICOLD, 1994, Pacelli et al, 1993)

Visual inspection for detection of cracking and wet zones, as well as measurement of seepage and leakage, are the methods mostly used to monitor the behaviour of joints in concrete and masonry dams. Leakage is observed in galleries and at the downstream face of the dam. This is sometimes revealed by lime efflorescence on the concrete face. The relative movements across the joint, measured at surfaces of the dam and galleries or inside the dam body are also appropriate monitoring variables.

Submerged zones may be inspected by divers or by a remote controlled vehicle equipped with a TV camera. Core sampling and geophysical methods have also been used (ICOLD, 1994). Sonic tests have also been used to measure the development of cracking and to assess the efficiency of repairs (Portuguese Work Group, 1991, Silva et al, 1993)

At the dam-foundation interface measurements of seepage, uplift and movements of the dam base in relation to fixed points into the dam foundation are currently done. Measurements of the dam base slope are also occasionally carried out.

3.3.5.3 Rehabilitation of Joints

The main purpose of the rehabilitation of joints is to restore their watertightness and also for many joints, their bond and shear strength. These rehabilitation works are usually carried out at the upstream face, or inside the dam body, either near the upstream face or at the surface of galleries. Typical techniques are indicated below for a range of materials and methods.

Breaking out and reconstruction of the joint, with a new waterstop is the first type of repair work. A particular difficulty is the bond between the new concrete and the original. Cement and epoxy are two materials reported to have been used with success. Another technique is to seal across the joint on the upstream face of the dam, with the seal adhering to both sides of the joint. It has been found helpful to cut a shallow rebate in the upstream face. The slot should be wide and deep enough to span the joint and to remove the deteriorated material near the surface. The sealant should not sag in to the slot, should bond to the concrete, remain flexible with time, and should not extrude. The sealant should be selected to be stable under the operating conditions, considering temperature variations ultraviolet light, submersion in water, wetting and drying, and freeze-thaw cycles (Schrader E, 1980).

A successful technique consists of drilling a hole through the joint, and filling it with an appropriate filler. This filler should displace water and have properties similar to those indicated above for the sealant. Sealing the joint by means of sheets of rubber, neoprene or other materials, glued or fixed by stainless steel plates bolted to the face, is also referred to (Schrader, 1980, CFGB, 1976, Pendse et al, 1998).

Grouting of joints and cracks is a current technique to restore the joint watertightness. From the dam face or from the face of a gallery, an array of holes is drilled crossing the joint or crack through which an appropriate sealant is grouted. Normal and fine grade cement grouts have been used, as have chemical grouts with mineral or cement fillers, resins and water reactive expansion polymer gels (Schrader, 1980, CFGB, 1976, Gouvenot et al, 1991, Brighton et al, 1991, Diaz, 1994). Sometimes the zone to be treated has to be confined by separate works before grouting starts. Special sealing devices must be used where important movements are to be accommodated by a joint. Typical in this category are those between aprons and the dam heel, or peripheral joints between the arches and the pulvino. (Demmer et al, 1985). Filling with slightly damp cement mortar, rammed into the joint to a depth of twice the width has been reported.

Grouting with cement or epoxy resins is the current technique to restore the bond and the shear strength in joints and cracks. Typical are contraction joints, which are grouted after completion of the construction in a cold season, and sometimes re-grouted during the operation phase (CFGB, 1976, Gouvenot et al, 1991, Portuguese Work Group, 1985).

Transverse cracks may develop due to shrinkage and cooling of the concrete in dams constructed with few or no contraction joints, such as RCC and some conventional concrete dams. Cracking may develop also in concrete and masonry dams owing to other actions, under normal operating conditions, and due to unforeseen or exceptional occurrences. It has been often observed that the pattern of such cracking tends to follow the construction joints. Larger cracks, are in general treated individually with cement grout, and smaller ones are sealed en masse with fine grade cement or epoxy resins (Silva et al, 1993).

Leakage through joints and cracks in dams are often associated with the deterioration of the upstream facing and with the use of pre-stressing. These scenarios are discussed in subsequent sections of this Bulletin.

3.3.5.4 Examples of Rehabilitation

Eleven cases of rehabilitation works concerning joints and cracks at concrete and masonry dams of different types are indicated below. The following examples concern gravity dams :

Eguzon Dam, completed in France in 1926, is a concrete structure. The joints and waterstops were rehabilitated by demolition of the squared stone masonry of the upstream face and the cyclopean concrete of the dam body, near the joint corners, and reconstruction with concrete, after inserting a new waterstop (ICOLD, 1994, ICOLD, 1984). **Saint-Sernin** masonry dam and **Lake Margaret** concrete dam were treated by means of polyurethane resin and by application of pre-stressing forces respectively (ICOLD, 1994, ICOLD, 1984).

Arlanzón concrete dam was constructed without contraction joints. Fifteen large vertical cracks developed, from the upstream to the downstream faces, causing leakage that damaged the dam, particularly at the downstream face, owing to frost. The rehabilitation works included the treatment of the cracks, by grouting with an epoxy resin, as well as the treatment of the dam faces and the consolidation and drainage of the dam body (Diaz, 1994).

Olivettes and **Seven Mile Dams** illustrate rehabilitation works in recent dams. The Olivettes RCC dam, with a crest length of 254 m, was constructed without contraction joints with concrete containing 130 kg of cement per cubic metre. Three vertical cracks developed, the treatment of which included the application of an elastic sealant in strips a few millimetres deep and 50 mm to 100 mm wide at the upstream face ; and grouting the joint near the downstream face (Gouvenot et al, 1991). At Seven Mile Dam the contraction joint of the spillway blocks, constructed without waterstop, was repaired by grouting a water reactive expansion polymer gel (Brighton et al, 1991).

The following examples of rehabilitation works concern arch dams :

Cabril Dam showed cracking at the downstream face, concentrated in a narrow band about 10 m below the crest and following the construction lift joints. The transverse contraction joints opened, particularly at the downstream face, and when the reservoir operating level was low. The rehabilitation works carried out in 1981 included grouting of the cracks and transverse joints with epoxy resin (Portuguese Work Group, 1991, 1985). **Flumendosa** and **Monceaux la Virole Dams** are other examples of similar rehabilitation works. At the first, the cracks developed along construction lift joints, at the higher elevations of the upstream face. These were grouted with epoxy resin (Silvano et al, 1997). At the second the contraction joints, which were constructed without a waterproofing system, were treated first with a bituminous mastic and some years later with silicone (ICOLD, 1984).

Les Toules Dam illustrates a structure constructed in phases, the interface between the old and the new concrete being in general a critical zone. This 86 m high double curvature arch dam was initially constructed to 26 m high only, and the prepack joint between the old and new concrete had to be re-grouted after a few years of operation (ICOLD, 1984).

El Atazar Dam is an example of underwater treatment of a large crack developed on the upstream face, at great depth. Important repair works of that crack were first carried out in 1978 and 1979, by caulking the crack at the face and then grouting resin from the neighbouring galleries. The increase of leakage and the inspections made in 1990 and 1992 with a vehicle equipped with a TV camera, pointed out to the need for a new treatment. These were made by divers and included the detection and sealing of the suction points at the upstream face. Grouting was also carried out with a highly fluid resin (Prieto et al, 1994).

The following cases illustrate the rehabilitation of joints and cracks in buttress dams :

Pracana Dam shows extensive cracking, owing to an expansive process, during the initial 25 years of operation. Important repair works were developed in 1985 and these included the treatment of the cracks by grouting using cement and epoxy resin. The joints between the diamond heads of the buttresses were treated as well and the upstream face was sealed with a PVC geomembrane (ICOLD, 1994, Portuguese Work Group, 1991, Silva et al, 1993, Pedro et al, 1979).

Storfinnforsen Dam, for which leakage and soaking of lime developed, associated with cracking at the front deck and spalling at the upstream face. The repair works included the treatment of the upstream face with shotcrete and grouting of the cracks downstream with cement grout (Eriksson et al, 1994).

Further examples of the rehabilitation of joints include the following :

Nature of Rehabilitation	Location	Dam	Reference
Prestressing	Contraction and construction joints, horizontal and vertical	Lake Margaret, Australia, built 1918, PG, 17 m high, 243 m long	ICOLD, 1984
Grouting with a water reactive expansion polymer gel	Contraction joints on spillway blocks	Seven Mile, Canada, built 1980, PG, 79 m high, 348 m long	Brighton et al, 1991
Reconstruction of joints and new water stops	Upstream face and dam body	Eguzon, France, built in 1926, PG, 61 m high, 300 m long	ICOLD, 1984, ICOLD 1994
Contraction joints filled with bituminous mastic and subsequently silicone	Dam body	Monceaux la Virole, France, built 1946, VA, 34 m high, 123 m long	ICOLD, 1984
Elastic sealant over vertical cracks Grouting	Upstream face Downstream face	Olivettes, France, built 1987, RCC, PG, 36 m high, 254 m long	Gouvenot et al, 1991
Grouting with polyurethane resin	Construction joints and horizontal cracks	Saint-Sernin, France, built 1921, PG, 18 m high, 122 m long	ICOLD, 1994
Grouting with epoxy resin	Lift joints at downstream face	Cabril, Portugal, built in 1953, VA, 135 m high, 290 m long	Portuguese Work Group, 1985, 1991
Grouting cracks with epoxy resin, sealing of upstream face, consolidation and draining of the dam body	Cracks through dam body, upstream face	Arlanzón, Spain, built in 1933, PG, 47 m high, 267 m long	Diaz, 1994
Caulking at the upstream face in 1978 and 1979 ; grouting with resin. Further sealing and grouting by divers in 1992	Upstream face, near base of dam	El Atazar, built in 1972, VA, 134 m high, 484 m long	Prieto et al, 1994
Shotcrete to the upstream face ; cement grouting of cracks	Upstream face	Storfinnforsen, Sweden, built in 1954, CB, 40 m high, 640 m long	Eriksson et al, 1994
Grouting of contact area	Junction between original dam and raised parts	Les Toules, Switzerland, built in 1963, VA, 86 m high, 460 m long	ICOLD, 1984
Sealing of joints using steel plates and a range of filling materials	Upstream face	Koyna, India, 103 m high, built in 1962	Pendse et al, 1998

3.3.6 Pre-Stressed Structures

3.3.6.1 Introduction

Although the use of pre-stressing forces in structures is a current technique, it is not much used in dams. The first applications of pre-stressing to dams date from the thirties (Monfort et al, 1991). However the technique has not been popular, perhaps owing to the hazardous nature of the construction and operation of the pre-stressing devices, and the difficulty of ensuring their durability in an aggressive environment for the long lifetime required. An important benefit of using pre-stressing forces for the rehabilitation of dams is that it is not necessary to empty the reservoir. Pre-stressing systems should be designed conservatively at the lower limit of current practice, and applied with care.

Pre-stressing forces are usually applied by tendons or bars, bonded full length or unbonded, and with corrosion protection. Hydraulic flat jacks have also been used to apply pre-stressing forces in the joints of dams. The purpose of pre-stressing is to improve the stress distribution at specific zones of the dam body, by countering the tensile stresses developed there. They comprise a system of balanced compressive forces applied at the anchor heads and distributed along the full length of the tendons. Because of this, the application of pre-stressing force together with grouting may improve the dam watertightness.

Typical applications of pre-stressing forces in dams are as follows :

- to increase the compressive stress at horizontal sections of gravity dams, particularly at the dam-foundation interface. The need for such an increase may be associated with the raising of the dam, the deterioration of the materials or because of changes in national regulations. This aspect is dealt with in section 3.4.2 (Pedro et al, 1994, Rißler, 1994, Cooper, 1991).
- to increase the thrust between the cantilever blocks or against the abutments in arch dams (Umana et al, 1973) ;
- to compensate for the tensile stresses developed next to the points of application of large concentrated forces, such as those near the trunnion bearings of large sector gates (Heitor et al, 1993).

3.3.6.2 Deterioration of Pre-Stressed Dams

The typical deterioration scenario of pre-stressed structures is the loss of the pre-stressing force, due to creep of the steel or of the dam materials, as well as the deterioration of the tendons, anchor bars and jacks.

The main cause of that deterioration is corrosion. Corrosion of the steel elements of the pre-stressing devices, particularly those under tension, is likely to occur in the saturated dam environment, particularly where the grouting or corrosion protection is defective. As corrosion reduces the section of the tendons, the stress increases or is partially transferred to the dam material, leading eventually to failure. In order to prevent the corrosion of the anchor heads corrosion inhibiting products are used, as well as covering with concrete (ICOLD, 1994 ; Pedro et al, 1994).

Monitoring the pre-stressing force is important. To facilitate the visual inspection of the anchor heads they are often placed inside pockets, filled with polyurethane foam and covered by a mortar cap that can be easily chipped out. These arrangements include also the installation of load cells and extra sleeves through the anchor blocks, to facilitate the future installation of additional tendons (Brighton et al, 1991). Tests of gamma radiography of the anchor heads have also been done (ICOLD, 1994). The monitoring of the pre-stressing forces is in general done through measurements of load cells. Such measurements however, owing to the possible transfer of stresses from the steel to the dam material, may not allow a reliable assessment on the condition of the tendons.

Loss of the pre-stressing force may also be revealed by an increase in deformation of the dam and cracking in the pre-stressed zone. Deformation measurements are carried out by means of plumb-lines, rockmeters, joint-meters and strain gauges ; cracking may be monitored by visual inspection or by means of periodic sonic tests. However these measurements may indicate only the occurrence of extreme situations, when it is too late to take the necessary preventive measures.

3.3.6.3 Rehabilitation of Pre-Stressed Dams

The principal aim of rehabilitation of pre-stressed dams is to restore the pre-stressing force lost in the course of time. This can be achieved by installation of additional tendons and anchor bars, or by re-stressing, replacing and repairing the existing ones. Such works may be particularly difficult for dams constructed before the sixties. Pre-stressing technology has improved since that decade, partly as a result of development in the nuclear industry. As referred to above, arrangements have been adopted in recent applications of pre-stressing forces that make it easy to monitor the forces, hence the effectiveness of the rehabilitation works (Brighton et al, 1991).

In designing rehabilitation works the total pre-stressing force applied to the dam is divided among several tendons or anchor bars. Smaller forces make it possible to obtain more favourable distributions of stress and easier rehabilitation works.

In general, the concentrated forces applied by the tendons and anchor bars, at the crest or in a gallery, are transferred as a distributed force to the dam body by means of a rigid structure. Frequently this structure is a reinforced concrete beam, which, if applied on the dam crest, increases the dam height and, if appropriately shaped, can form the crest of a spillway (Lino et al, 1991).

Pre-stressing forces are used in the piers at the trunnions of radial gates to counter the large concentrated tensile forces there. It has been considered advantageous to use several tendons for the application of the pre-stressing force to each pier. This can be achieved by placing each pair of tendons in a skew symmetrical position in relation to the plan of the pier, with the anchor heads at the faces of the pier and at the supporting blocks of the gate axis (Heitor et al, 1993).

3.3.6.4 Examples of Rehabilitation

Illustrative examples of pre-stressed dams are given below, including structures that were designed originally to incorporate pre-stressing forces.

Cheurfas Dam, a masonry structure completed at the end of the last century, which was rehabilitated was reinforced in 1936 by the installation of 37 tendons of 10 000 kN (1000 tonnes) capacity. These tendons were periodically inspected at intervals of 4 to 5 years. In 1967, after the detection of the failure of two tendons, a further 30 tendons each of 2000 kN were installed. However, the inspections continued to reveal deficiencies, including failures of some of the new tendons. Therefore, doubts remained concerning the reliability of the pre-stressing system and the dam was decommissioned in 1975 (Monfort et al, 1991).

Rassisse Dam is an arch dam completed in 1954, with an artificial abutment on the right bank, which is reinforced with two tendons each of 12 000 kN. These tendons were designed to provide additional downforce equivalent to 80 % of the weight of the abutment. The safety conditions were under control in 1991, in spite of uncertainty over the condition of the tendons, because a model analysis showed that the required pre-stressing force is smaller than that initially assessed (Monfort et al, 1991).

Neubeur Dam is a multiple arch dam completed in 1954. The blocks of the spillway and of a gravity abutment, on the left bank, are reinforced with 22 tendons of 12 000 kN. Unlike modern pre-stressing systems, these tendons cannot be re-tensioned and doubt remains over their condition. Therefore, their replacement by new ones was envisaged (Monfort et al, 1991).

More recent examples of the application of pre-stressing forces, particularly for the rehabilitation and raising of dams of moderate dimension, are presented below :

Stave Falls and **Cova do Viriato** are both concrete gravity dams, the first completed in 1911 and rehabilitated in 1985, and the second completed in 1962 and raised ten years later. At Stave Falls dam, load cells were installed in some anchor heads and arrangements were made to facilitate the inspection and the re-tensioning of the tendons, should this become necessary (Brighton et al, 1991).

Auberives (La Bourne) masonry gravity dam, completed in 1878 and rehabilitated in 1984, was rehabilitated by adding tendons of 500 kN capacity, at 2 m centres along the dam crest. The anchor heads were incorporated within a reinforced concrete beam, shaped as a spillway crest, which also improved the discharge of the flow over the dam (Lino et al, 1991).

Fratel Dam, completed in 1973, illustrates the application of pre-stressing forces to improve the stress distribution in the piers supporting the thrusts of high sector gates. Fourteen tendons each of 500 kN capacity were used to pre-stress the blocks with the anchor heads against the pier. The monitoring and the periodic inspections of the dam have confirmed that the behaviour is satisfactory (Umana et al, 1973).

The following Table summarises the case histories used in this assessment of the rehabilitation of pre stressed elements of dams.

Nature of Rehabilitation	Location	Name of Dam	Reference
Provision of downforce, dam abandoned after 39 years	Dam body	Cheurfas, Algeria	Montfort et al, 1991
Provision of downforce, load cells to monitor load	Dam body	Stave Falls, Canada	Brighton et al, 1991
Provision of downforce,	Dam body	Auberives, France	Lino et al, 1991
Provision of downforce	Dam abutment	Rassisse,	Monfort et al, 1991
Support of sector gate trunnion bearing	Gate piers	Fratel, Portugal	Umana et al, 1973
Provision of downforce, dam raised	Dam body	Cova do Viriato	Brighton et al, 1991
Provision of downforce, not possible to monitor stress	Spillway and abutment	Neubeur, Tunisia	Monfort et al, 1991

3.4 REHABILITATION TO IMPROVE STATIC STABILITY

3.4.1 Introduction

The rehabilitation of a dam may be designed to improve not only its performance but also its safety. The owner and the public are interested in a dam fulfilling its task safely and reliably hence the optimal design of rehabilitation works will address both aspects. The ICOLD Committee on Dam Safety will shortly publish its Bulletin ‘Safety Improvement of Existing Dams’. They are well advanced with their next Bulletin ‘Risk Assessment as an aid to Dam Safety Management’ They have described rehabilitation work from the point of view of improving the safety of existing dams. In contrast, the emphasis is laid here on the methodology of rehabilitation and on the experience gained in this field. This includes a more detailed handling of the examples and where helpful a reproduction of sketches in order to improve the understanding of the reader. Rehabilitation for reasons other than ageing is often undertaken in response to changes in regulations. In this chapter therefore we describe successful approaches to improving the stability of dams.

Until the 1930s the causes and effects of uplift under a gravity dam were not fully understood. It was not until the 1960s that the foundations of gravity dams were routinely and systematically drained at depth (Sims, 1994). A characteristic of the design of the masonry gravity dams built in Germany for example in the first decade of this century is that uplift was not included in the stability calculations, either for overturning or for sliding. As a result the dams were found to be unsafe in terms of modern design standards. No drainage was provided. Rehabilitation work is then necessary to increase the stabilising moment or sliding resistance of the cross section.

The engineer should judge the stability of the dam on the basis of in-situ investigations. Rehabilitation work is expensive and one of his tasks is to avoid unnecessary cost. He must assess the stability of the dam, taking advantage of the actual strength of the foundations. Particularly for older structures, there may be no contemporary design information to guide him on foundation quality and conditions. Defects in the foundation that may affect the resistance to sliding include faulting and crushed areas of rock. These may be close to the interface between the dam and the foundation or may be deeper. Suitable investigation measures include core borings, investigation adits, galleries, closed-circuit television camera inspections, water pressure tests and geophysical testing. Laboratory testing will be required. Despite this array of possible measures, it remains difficult to get a reliable picture of the sliding stability of the rock mass.

The resistance to overturning or sliding of a dam on its foundation rock may be improved by several methods :

- Providing downforce by enlarging its profile, adding ballast or through prestressed anchors.
- Draining the dam and its foundation to reduce uplift.
- Providing additional friction on the sliding surface by grouting.
- Providing horizontal force by, for example placing a concrete block against the downstream face or within the dam body from a gallery.

Engineers are refining their understanding of the occurrence of earthquakes. Particularly in countries where earthquakes were formerly regarded as occurring with negligible frequency it is becoming routine to review the design of older dams from the point of view of their resistance to seismic loading. The Engineering Guide on this subject recently published in the UK illustrates the point (BRE 1991). Rehabilitation may be necessary to achieve acceptable stability against seismic loads.

3.4.2 Remedial Measures

3.4.2.1 Improvements by Adding Down Force

The three major methods of increasing the downwards force on the dam are by enlarging the profile of the dam, by adding ballast, or by installing pre stressed anchors. **Muslen Dam** (Wulliman et al, 1985) is an example of rehabilitation by increasing the cross section. **Morris Shepherd Dam** (USCOLD, 1996) is an example where ballast has been used.

Eder Dam (WSB, 1994) is one of several examples of the stability being improved by adding prestressed anchors. Important considerations for such a solution include the need to ensure that the anchors maintain their load throughout the life of the dam. Thus corrosion protection and regular monitoring are essential. Eder Dam, a masonry gravity structure, was built between 1909 and 1914 in Hessia, Germany. It is 48 m high and 399 m long. It was damaged in 1943 during World War II, but was rebuilt in the same year. A drainage gallery was installed during the repairs at the base of the dam. Drainage and grouting work was undertaken at the upstream face and around the gap made by the bombs. The dam and the rock abutment were grouted in 1962 and 1963. Fig. 3.4.1 shows the cross section as it was at the end of 1963.

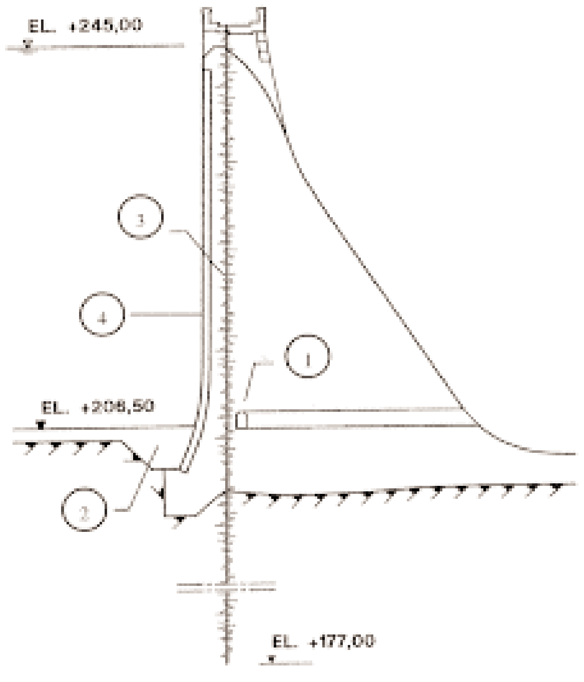


Fig. 3.4.1

Eder Dam. Cross section after grouting in 1962/63

1. Longitudinal gallery
2. Clay infill
3. Grouted zone
4. Masonry facing

In the early eighties the stability of the Eder Dam was considered to be inadequate. The analysis was done amid intense technical discussion in Germany that triggered alterations of the German standards. For example no tensile stress is now permitted at the upstream face. In some states there exist new administration standards with regard to uplift assumptions. This led not only to the rehabilitation of the Eder Dam but also to rehabilitation work at other dams.

The uplift distribution shown in Fig. 3.4.2 was used for the stability calculation. It was found that the dam stability was not sufficient to meet the required standards. As an immediate step the reservoir water level was lowered by 1.5 m while feasibility studies were put in hand permanently to improve the stability of the dam. The six options considered are shown on Fig. 3.4.3.

1. Prestressed anchors between crest and foundation rock. For this it was planned to build a new dam crest including a gallery for the anchor works

2. Ballast loading on the dam crest

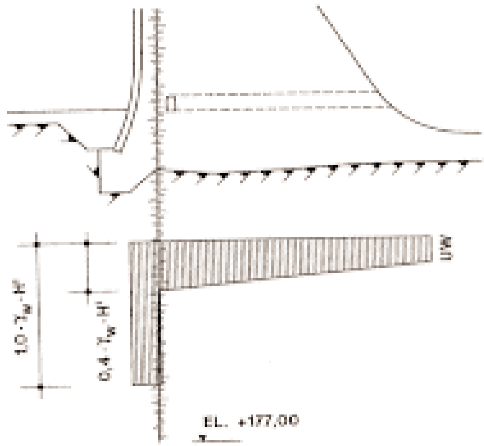


Fig. 3.4.2

Eder Dam. Uplift distribution

3. Add a concrete wall to the upstream face, draining the space between the wall and the original structure. A new control gallery at the upstream toe from which grouting and drainage is carried out in the foundation rock
4. Enlargement of the dam profile at the downstream face by adding mass concrete. This concrete to be connected to the existing dam with a shear connection to make a composite structure
5. Support of the dam from the downstream side with a rockfill embankment or mass concrete
6. Rockfill embankment or mass concrete loading the upstream face. An impermeable layer to be placed between the new fill and the existing dam with a drainage layer behind and a control gallery at the bottom.

The owner rejected alternative 2 because the German standards require the assumption of a water level at the crest level for an extraordinary loading case. He rejected alternatives 4 and 5 because of the complexity of the design problems and the high cost. Alternative 3 was rejected because it required the Eder lake to be emptied for several years.

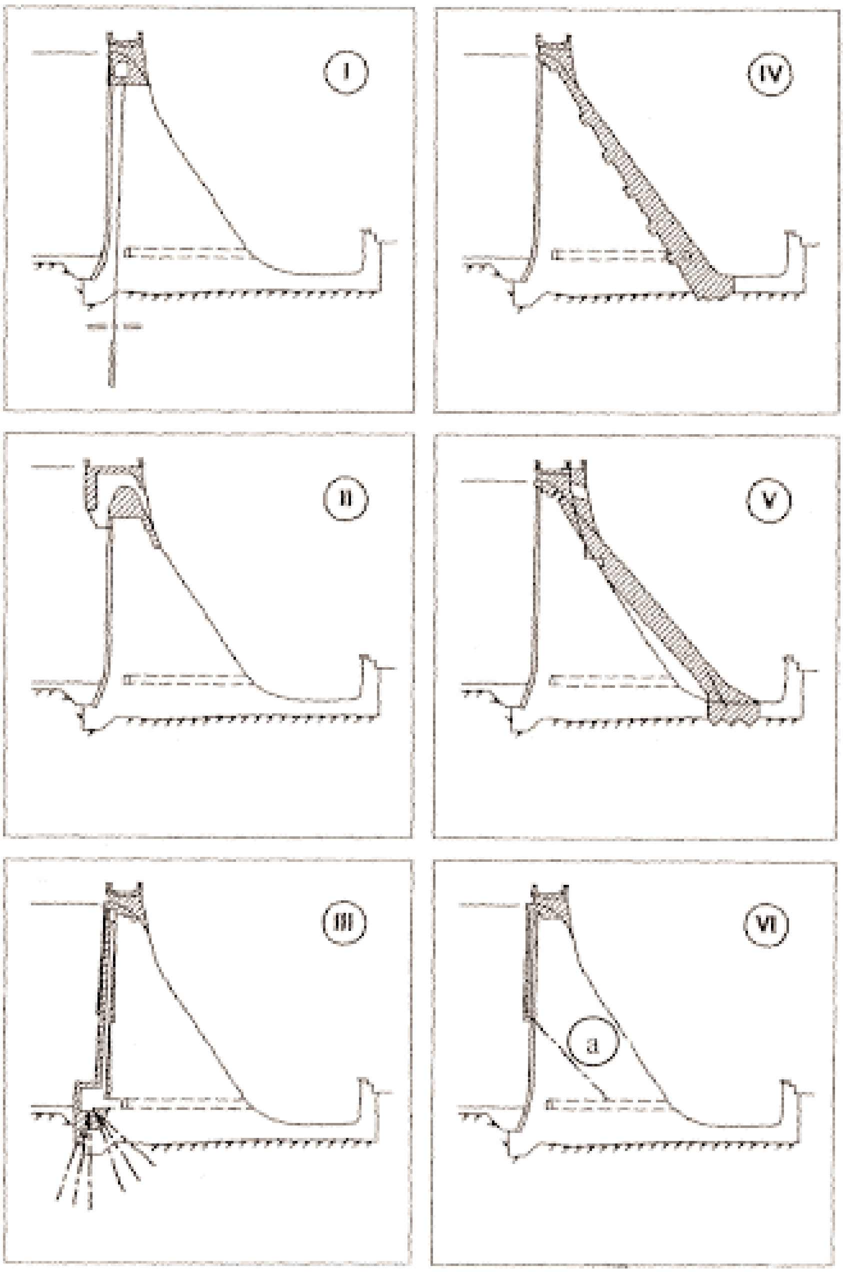


Fig. 3.4.3

Eder Dam. Six alternatives for rehabilitation

a. Drainage holes

Alternatives 1 and 6 were evaluated and alternative 1 was chosen because of its smaller influence on the lake level, because of its smaller alteration of the outer profile of the existing dam and because of lower risks during construction. The configuration finally adopted is shown on Fig. 3.4.4.

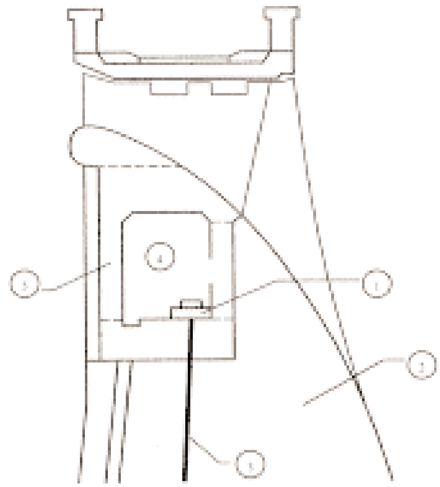


Fig. 3.4.4

Eder Dam. Final crest configuration

1. Anchor head
2. Masonry
3. Post tensioned anchor
4. Gallery
5. Concrete

104 pre-stressed anchors, 70 m long, each with 4500 kN working load, were used. Anchor cables were developed for the project during intensive qualification investigations at site. Each anchor consists of 34 strands of St 1570/1770 steel, (yield point 1570 MN/m^2, strength 1700 MN/m^2), 150 mm^2. The heads were arranged in the gallery at the crest, the anchoring length about 30 m below the base of the dam. The 273 mm diameter holes were inclined 3.2° from the vertical with an accuracy of 1 %. The cables were assembled at a workshop at the site and were transported without bending to the crest where they were taken by a steel frame designed to avoid strong bending.

3.4.2.2 Improvements through Drainage – Drainage Gallery by Blasting

Urft Dam in Germany is a masonry gravity structure built between 1900 and 1904. It is 58 m high and 226 m long. Its crest is curved with a radius of 200 m. It is founded on sandstones, siltstones and slate. No gallery was provided in the original design. In the eighties lime encrustation from the mortar was observed on the downstream face of the dam and this prompted an investigation of its behaviour. Coring showed that the mortar consists of lime, *Rhenish trass*, and cement. The mortar content was between about 35 % and 40 % of the masonry volume. No remarkable damage had occurred through the seepage of water through the dam and the solution of salts from the mortar.

Water pressure tests showed that the permeability of the masonry was 10^{-6} m/s to 10^{-7} m/s with higher values in the foundation. The elastic modulus of the masonry was between 3000 MN/m^2 and 6000 MN/m^2. The siltstone in the foundation was slightly weathered. The jointing was more developed in the siltstones than in the sandstones. The water pressure tests showed that permeability decreased with depth. Dilatometer tests showed Young's modulus of 4000 MN/m^2 to 8000 MN/m^2.

Preliminary three dimensional finite element analyses confirmed adequate stability (Wittke et al, 1995). But further work was needed for the dam to meet current German standards. For this, two control galleries were excavated as shown on Fig. 3.4.5 and 3.4.6. The first at the contact between dam and foundation rock. The second one at about one third of the height of the dam on the line of the vertical drainage holes. By this means it became possible to monitor these devices and if necessary to rehabilitate them. Instruments were installed in the dam as shown on Fig. 3.4.7 to 3.4.9 to measure displacements, piezometric heads and temperatures. Displacements are monitored by extensometers, by inclinometers and by plumb lines and inverse pendulums. The alignment and level of the crest were measured. In both galleries large flat jack tests were done to evaluate more precisely the Young's modulus. The finite element analyses were refined with the measured data.

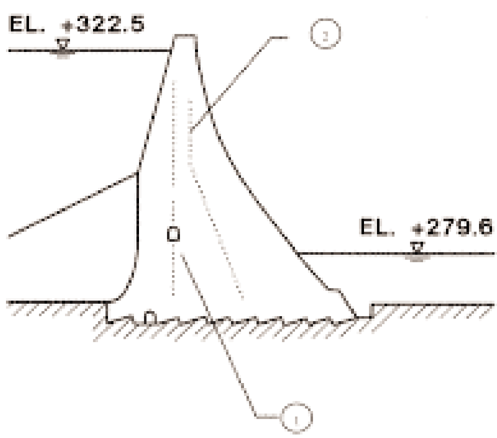


Fig. 3.4.5

Urft Dam. Cross section

1. Inspection gallery
2. Drain holes

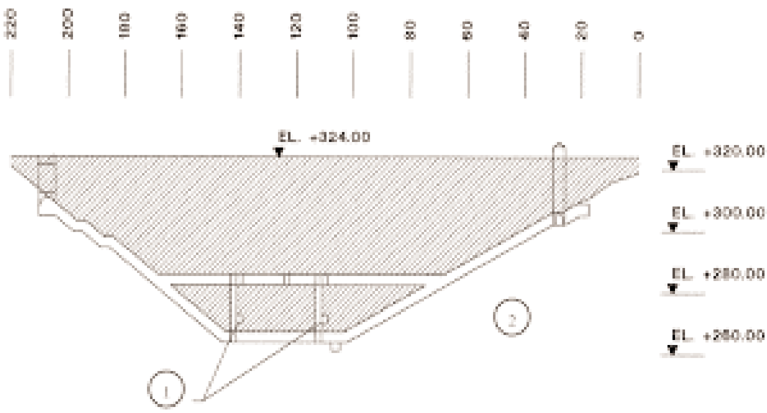


Fig. 3.4.6

Urft Dam. Longitudinal section

1. Bottom outlets
2. Right abutment

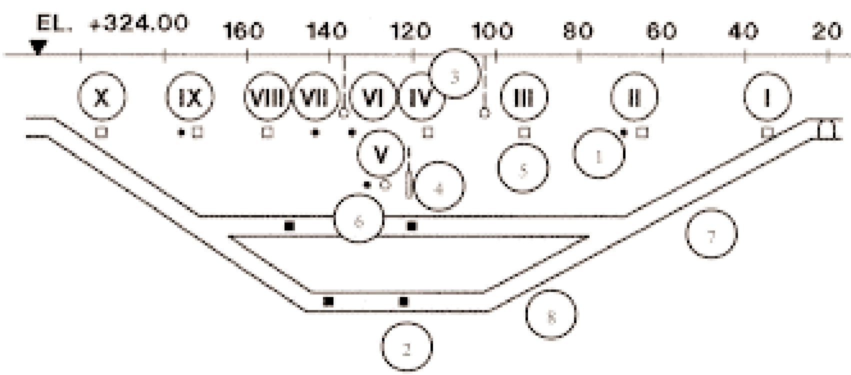


Fig. 3.4.7

Urft Dam. Instrumentation

1. Extensometer
2. Flat jack
3. Pendulum
4. Inverted pendulum
5. Piezometer
6. Thermometer
7. Upper gallery
8. Lower gallery

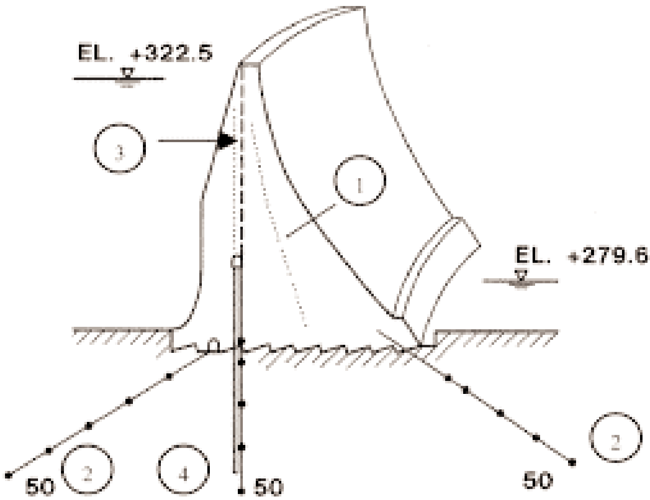


Fig. 3.4.8

Urft Dam. Instruments at cross sections V to VII

1. Drain holes | *3. Pendulum*
2. Extensometer | *4. Inverted pendulum*

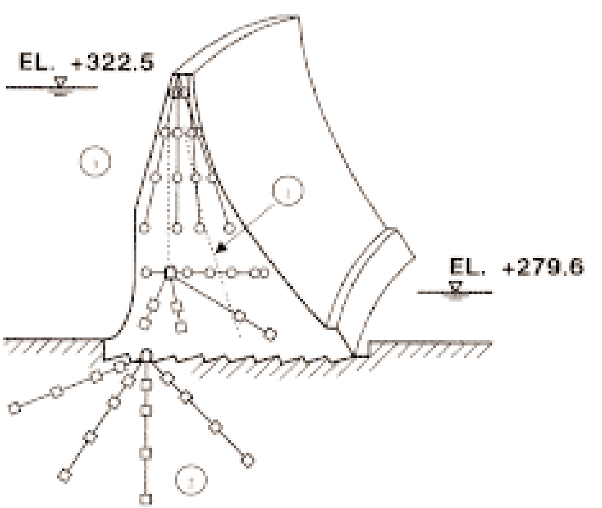


Fig. 3.4.9

Urft Dam. Instruments at cross section IV

1. Drain holes | *3. Thermometers*
2. Piezometers

In 1995 the excavation work began. Because the galleries had to be excavated during full reservoir operation and only 3 m to 4 m distant from the upstream face of the dam, the supervising state administration together with their experts required hand excavation with hydraulic tools. No blasting was allowed. But progress was slow. About 1 metre per day was expected, 0.2 m per day were made on average and the planned construction time would have been exceeded by 15 months. The owner and contractor requested permission for blasting work. The state administration agreed but required additional safety measures, including dense monitoring of the blasting effects to the surrounding masonry and foundation rock. The average progress improved to about 2.2 m per day. Difficulty was found in reducing overbreak in the excavation. (Wittke et al, 1994, Wittke et al, 1995, and Poltczyk et al, 1997)

3.4.2.3 Improvements through Drainage – Drainage Gallery by Tunnelling Machine

Ennepe Dam is a second example of a dam where the stability is improved by additional drainage. As shown on Fig. 3.4.10 it is a 50 m high and 320 m long masonry gravity dam in the Ruhr River catchment, built between 1902 and 1904. Between 1909 and 1912 the dam was raised by adding a 10 m high masonry block on the crest to increase the storage capacity by 2.3 million m^3 to 12.6 million m^3. The reservoir is used for the water supply of about 170 000 people and to empty the reservoir to undertake remedial measures would have been an unacceptable cost. Therefore all remedial measures described below have had to be done during normal reservoir operation.

Fig. 3.4.10

Ennepe Dam

Uplift pressure was not considered in the original design in 1902 and the dam therefore did not meet German Standards. As a preliminary measure the storage level was lowered by 2.3 m and the spillway crests lowered. The state authorities required intensive investigations and rehabilitation. Work was delayed by shortage of money. In 1995/96 a concept was developed to meet the financial and technical requirements and to guarantee for the future, an impoundment up to the original storage level. A drainage gallery was to be excavated at the base of the dam only 3.5 m from the upstream face. Fans of drainage holes are to be drilled from the new gallery, three in the dam and three in the abutment rock (Fig. 3.4.11). The spacing of the fans is to be decided after intensive investigation of the pore pressure distribution when the drainage gallery has been excavated. The gallery shall remain unlined as far as possible in order to improve the drainage of the surrounding rock and masonry.

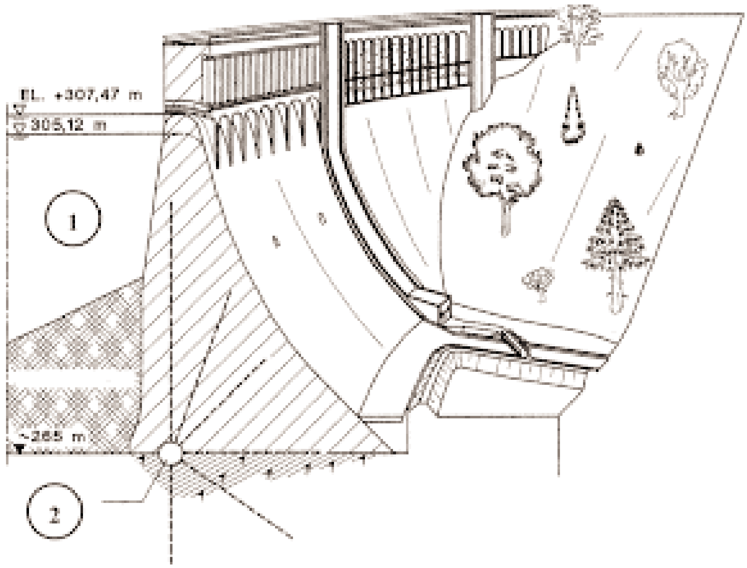


Fig. 3.4.11

Ennepe Dam. Drainage gallery and drainage holes

1. Intze wedge *2. Drainage gallery*

The initial design of the rehabilitation measures also included a grout curtain in the masonry dam and in the rock abutment upstream of the drainage borings. But water pressure tests showed that these were not needed.

At first it was intended to excavate the gallery by blasting. But the understandable concern about blasting work only 3.5 m away from the reservoir body led to the investigation of the use of a full-face tunnelling machine 3.0 m in diameter. The main problem seemed to be and was, the sharply curved line of the gallery. The profile of the gallery was modified to suit the capability of the machine as shown in Fig. 3.4.12

The current concept has incorporated the proposals of the contractor. The position of the starting shaft was his decision. The machine started from a 10 m deep shaft 30 m downstream of right bottom outlet. From there the machine accessed the foundation through a 90 degree curve with a 40 m radius. Fig. 3.4.12 shows the gallery in plan and from downstream, Fig. 3.4.13 in a perspective view. A Robbins tunnelling machine 81-113/3.00 was used with only one pair of grippers. It was possible to use one pair only because of the sharp curved adit axis. On the other hand the diameter to be excavated was so small that only a light machine could be used.

Experience showed that excavating a 40 m radius with such a tunnelling machine in slate and sandstone is at the current limit of technical feasibility. Particular difficulties were encountered when a hard diabase dyke was encountered about 50 m into the curved access tunnel. This had not been foreseen in the geological investigation. The machine was not able to cut the hard diabase. Therefore it was necessary to excavate the 25 m length of the dyke by drill and blast methods. Care was required because this work was carried out between 15 m and 3.5 m from the upstream face of the dam.

After the drill and blast work the machine was returned to the adit. Working in the upper part of the cross section in masonry and in the lower part in the foundation rock, an average daily advance of 4.5 m was achieved with a peak performance of 16 m in 20 hours. In both abutments of the dam the adit is inclined at 30° and this was achieved without problem. After the tunnelling machine reached the left end of the dam it was disassembled, taken to the access adit and reassembled and then it was driven into the adit backwards. Once in place under the dam, the machine excavated the gallery underneath the right wing of the dam. The result is a circular 3.0 m diameter tunnel with smooth walls as shown on Fig. 3.4.13. The tunnel will remain unlined over some 90 % of its whole length. In this state it will act as an effective drainage gallery. From the tunnel, fans of drainage holes spaced at 4 m were bored within a test section. Within this section the uplift pressure is monitored and the data used for the final stability analysis to prove the ability of the dam to resist the increased storage level : up to that envisaged in 1912 (Rißler et al, 1999).

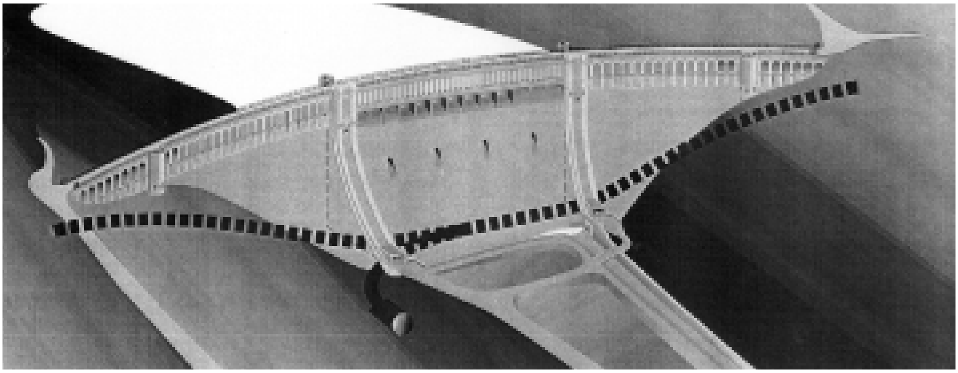


Fig. 3.4.12

Ennepe Dam. Perspective view of dam and galleries

Fig. 3.4.13

Ennepe Dam. Drainage gallery as excavated

3.4.2.4 Improvements by Providing Additional Horizontal Support

Kölnbrein Dam (Obernhüber, 1991) was built between 1971 and 1979. It is a double curvature arch dam, 198 m high and 626 m long, 7.6 m thick at the crest and 41 m at the base. It is one of the slenderest arch dams of the world. The foundation rock consists of gneiss formations, layered at the left abutment, schistose on the left flank of the valley and massive elsewhere. Each rock type has a different bulk modulus. During construction the quality of the rock abutment had been improved by injecting 480 t of cement from 20 000 m of bored holes.

On first filling in 1978, when the reservoir level reached 1890 m, 42 m below the normal full storage level, the seepage into the drains just above the base of the dam increased suddenly. As the reservoir level increased to 1891 m, the seepage increased to 200 l/s. The uplift below the upstream foot of the dam was 100 % of the hydrostatic pressure in the reservoir. The cause was considered to be a new set of inclined cracks in the foundation rock running from about elevation 1750 m downstream at a slope less than 45∞ between the control gallery and base of the dam. The cracks were assumed to be a consequence of high shear stresses in the base of the slender dam. Ad hoc measures were carried out to reduce the high pressure and the seepage :

- grouting with epoxy resin
- a freezing zone had been installed as a temporary measure in the cracked zone at the base of the dam

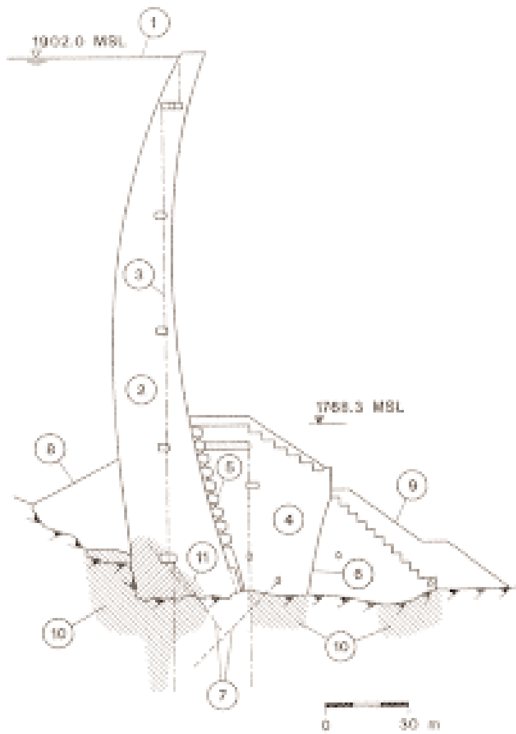


Fig. 3.4.14

Kölnbrein Dam. Cross section after rehabilitation

1. Normal storage level
2. Arch dam
3. Pendulum
4. Supporting block
5. Concrete consoles
6. Longitudinal joint
7. Drains
8. Concrete apron
9. Rockfill
10. Cement grouting
11. Epoxy resin grout

- a concrete apron had been installed on the valley floor upstream of the dam in order to cover the crack zone in the abutment.

Although the desired reduction of uplift and seepage had been achieved, no satisfactory improvement of the dam behaviour had been gained. Therefore the state authorities limited the reservoir level to 22 m below the normal level. A concept for the rehabilitation was developed comprising the following measures :

- As the high shear stresses in dam and abutment were considered responsible for the development of cracks, the lower parts of the dam were to be horizontally supported by a massive concrete block

• Grouting with cement and with epoxy resin was planned to stabilise the cracked zones and to provide a watertight abutment

• Grouting measures were necessary to improve the abutment below the upstream toe, especially for the empty reservoir condition.

The rehabilitation work began in 1989 and was finished successfully in 1994. The adopted solution was to install a 70 m high supporting block of concrete downstream of the dam. Its function is to support the lower third of the dam, but only when the water in the reservoir exceeds a certain level. It was therefore necessary to bring the two elements together only after a certain water load had been applied. The problem was solved by installing 613 flat, 1.2 m square, reinforced neoprene cushions at nine levels at concrete consoles as shown on Fig. 3.4.15. Each cushion is connected to a double steel wedge construction, shown on Fig. 3.4.16, that can be calibrated in height from adits below and above their horizon. In this way it was and remains possible to alter the support provided by each of the wedges. Numerical calculations determine the support to be provided as a function of the reservoir water level. The advantage can also be seen from the arch dam's deformations and seepage. In Fig. 3.4.16 these parameters are shown for the situation before and after rehabilitation (after Heigerth et al, 1994, oral presentation).

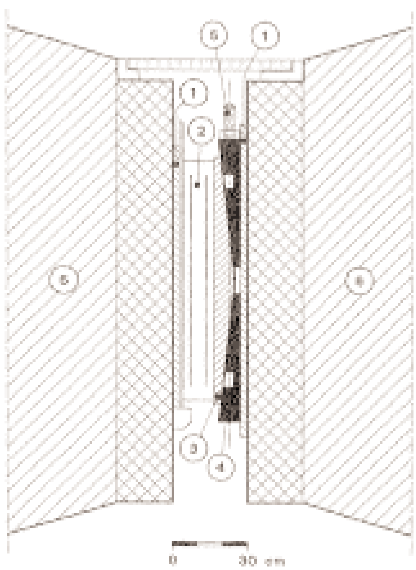


Fig. 3.4.15

Kölnbrein Dam. Neoprene cushion and double steel wedge

1. Steel plate	*4. Sliding wedge*
2. Neoprene cushion	*5. Steel bar*
3. Double steel wedge	*6. Concrete console*

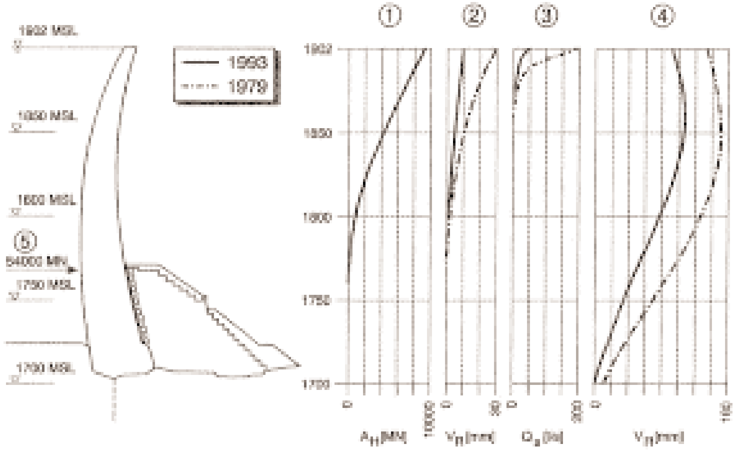


Fig. 3.4.16

Kölnbrein Dam. Deformation and seepage before and after rehabilitation

1. ∑ Forces at the 613 neoprene cushions
2. Radial displacement at the bottom adit
3. Seepage
4. Deformation at full reservoir situation
5. Maximum water load

Vir Dam (Bilik et al, 1998) was built in the early 1950s. It is a concrete gravity dam 66 m high and 390 m long. The foundation rock is dominated by a geological fault running obliquely across it. Where the fault is a short distance downstream of the dam, there was concern for the stability of the blocks. After construction of the dam, a supplementary support system was added. This comprises a strut from the dam to the downstream side of the fault. The strut was loaded by the use of concrete wedges between the strut and the dam. The wedges were forced between the dam and the strut by prestressed rock anchors. The stability of the dam was re-examined in the 1990s and strengthening of the foundation was required.

Other case histories of remedial measures for improving static stability include :

Nature of Rehabilitation	**Location**	**Name of the Dam**	**Reference**
Prestressing	Dam body	Gübsen, Switzerland	Müller, 1985
Increase of cross section by additional concrete upstream and downstream	Dam body	Muslen, Switzerland	Wullimann et al, 1985
Downstream embankment with earth and rock	Downstream	Ternay, France	Lino et al, 1991
Increase the cross section downstream by concrete	Dam body	Ohmatazawa, Japan	Egawa et al, 1994
Prestressing	Dam body	Burt, USA	USCOLD, 1996
Ballast of lean mix concrete	Dam body	Morris Shepherd, built in 1941, USA, 50 m high	USCOLD, 1996
Post tensioned anchors and drainage	Dam body	Leixlip, Pollaphuca, Golden Falls, Ireland	O'Tuama et al, 1989
Wedged struts with prestressed anchors and additional toe weight	Downstream face	Vir, 66 m high, PG, Czech Republic	Bilik et al, 1998

4. EARTH AND ROCKFILL DAMS

4.1 INTRODUCTION

Rehabilitation of an earth and rockfill dam may be necessary for one or more of the following reasons :

- To counter the effects of ageing of the embankment
- To counter the effects of deficiencies introduced at the time of design or construction
- To put into effect the improvements in the understanding of the physical and chemical processes underlying the design and operation of embankment dams.

The separate functions of watertightness and structural stability are served by two different elements in earth and rockfill dams, but each is dependent on the other for the safe performance of the dam. Rehabilitation of one element therefore usually has a beneficial effect on the other. Thus, control of seepage through the embankment may be improved by making the impervious barrier more effective using methods such as grouting, diaphragm walls or sheet piling. Alternatively, improving the drainage of the downstream fill of the embankment controls seepage. Each of these measures also improves the structural stability of the dam in terms of slope stability. The structural stability of the dam is improved by the addition of a berm, adding weight at the toe, or the densification of the foundation. These measures in turn can help to overcome the deleterious effects of the seepage through the dam.

The most important defects in an embankment dam are those that diminish its ability to store water safely, particularly those affecting the watertight element. This is not because the leakage is so excessive that the ability of the dam to store water is seriously impaired, but rather because malfunction of the watertight element can lead to internal erosion of the embankment or to high pore pressures in the downstream fill, which will reduce slope stability. Internal erosion in the embankment or its foundations is potentially life-threatening for the dam.

The major objectives of rehabilitation of an embankment dam is to overcome one or more of the problems listed below :

- Instability associated with internal erosion of the embankment, its abutments or its foundation by leakage of water or seepage from the reservoir or, for example from groundwater within the dam abutments.
- External erosion caused by the failure or distress of the upstream protection element as a result of wave attack or ice action. This can lead to rapid erosion of the upstream fill.
- Slope instability caused by shear failure within either the embankment or foundations, resulting from inadequate shear strength. High pore water pressure within the fill the most important reason for loss of shear strength. A

secondary cause of loss of shear strength is material degradation. It is noted that failure due to slope instability, although important, has rarely led to uncontrolled release of water.

- Insufficient seismic stability resulting from the liquefaction potential of the material in the foundation or in the embankment. Hydraulic fills and foundations on saturated fine sand or silty sand with low relative density are prone to loss of shear strength under seismic shaking.
- Instability due to external erosion of the embankment caused by overtopping of the embankment. This can arise because of insufficient spillway capacity. Rehabilitation usually aims to increase the spillway capacity or raise the dam. Reinforcement of the dam crest and paving of the downstream slope to resist erosion by rare overtopping water have proved to be effective measures on small dams. This aspect of rehabilitation has been dealt with at length in the literature and is not covered specifically in this Bulletin.

This chapter is structured in four major sections following this introduction. In the first, we discuss in more detail the deterioration processes listed above, focusing on key indicators, and the effect of the deterioration on embankments and their foundations. In the second section we discuss techniques for rehabilitating the foundations of embankment dams. In the third we cover the same subject for the embankment itself, dealing with rehabilitation of the watertight element, improvement of the stability of the slopes, repairs to surfaces of the dam face, and finally the loss of bond between concrete elements and the embankment itself. Finally, we discuss the rehabilitation of embankments to meet modern seismic criteria. Throughout the chapter we present appropriate case studies to illustrate the rehabilitation methods described.

4.2 DETECTING AND ASSESSING THE DETERIORATION PROCESSES

4.2.1 Key Indicators

The two key indicators of deterioration of an embankment are deformation and seepage flow. These can be identified by visual surveillance or by instrumentation. Techniques for investigating, monitoring and instrumentation of embankment dams have been reported in ICOLD Bulletins 87 and 104 for example and elsewhere (For example, Charles et al, 1996, Dunnicliff, 1988) and are not repeated in detail in this Bulletin. Each of the key indicators can be expanded as follows :

Deformation

- Excessive rate of general settlement
- Differential settlement
- Increase in the rate of settlement
- Localised settlement or sinkhole in the crest or dam slopes
- Slumping of the upstream or downstream slope associated with shallow or deep seated slope failure.

Seepage flows

- Sudden emergence of seepage or leakage on the downstream slope, valley sides or valley bottom
- Increase in flow rate or turbidity of existing seepage flows
- Marked change in piezometric level within the dam or its foundation
- High pore pressure downstream of the water tight element
- Leakage into outlet tunnel or valve shaft.

The following Table shows a useful correlation between the observed trends in seepage and in piezometric pressure. When both are rising it is advisable to review the situation and where both are falling, stability is improving.

	Seepage increasing	**Seepage decreasing**
Piezometric head increasing	**Unfavourable situation :** rehabilitation may be necessary urgently.	**Unfavourable situation :** it may be necessary to clean or enlarge the drains.
Piezometric head decreasing	**Surveillance required :** risk of internal erosion, hence advisable to check whether there are fine materials in the seepage water.	**Safe situation :** no action necessary.

It is important to understand and treat the cause of the deterioration and not simply to deal with the apparent symptoms. In the following section are described the most frequently encountered mechanisms underlying these initial visual signals.

4.2.2 Mechanisms and Processes Causing Deterioration

4.2.2.1 Deformation

Of the mechanisms and processes causing deformation in embankment dams, those due to slope instability and internal erosion can seriously threaten the safety of the dam. Other causes of movement may be less serious. In this second category are consolidation of fill and strata in the foundation, the deformation due to variations in reservoir level, and seasonal moisture content in the near surface layers of a clay core. These mechanisms may cause small deformations that have no deleterious consequences beyond reducing freeboard and increasing the risk of overtopping. In identifying the mechanism causing the deformation, the engineer should consider that the processes could be interlinked. For example, differential settlements within the dam, caused for example by a benign process such as consolidation, may reduce total stress locally, making hydraulic fracture, leakage and internal erosion more likely.

Movement due to Slope Instability

Embankments can become unstable during construction if excessive pore pressure develops. However, major deep-seated slope instability later in the life of the dam is comparatively rare. Detailed observation of the deformations on the slopes of the embankment may make it possible to identify this cause of movement.

Settlement due to Internal Erosion

Internal erosion, discussed in more detail below, is the most common cause of in-service incidents and failures in embankment dams (ICOLD 1993, Charles 1996). Hydraulic fracturing can initiate internal erosion. Internal erosion may cause sinkholes in, or adjacent to, the crest of the dam. Where the embankment is also undergoing a more widely distributed type of settlement it may be more difficult to determine whether or not the movement is due to internal erosion.

Volume Change in Clay due to Seasonal Moisture Content Variation

Clay soils swell with an increase of moisture content and shrink when drying. Where a clay core is formed of a high plasticity clay substantial surface movements can occur due to seasonal swelling and shrinkage. This type of movement can be identified by its seasonal nature. Such near surface effects may have no significance for the behaviour of the core at depth within the dam and can be distinguished from the monitored settlements by installing settlement stations at an adequate depth, say 1.5 m, below the crest surface.

Consolidation of Foundation Strata

Where an embankment dam is built on a deep deposit of clay, consolidation of the foundation soils may continue at a significant rate over a long period. These movements might be significant in reducing freeboard but have a relatively minor effect on the behaviour of the embankment except where there are large abrupt changes in depth of soils subject to consolidation. Measurements of sub-surface vertical movements can show whether this type of settlement is occurring. Where settlement gauges have been installed through the embankment it is possible to check by topographical levelling of the top of the gauge whether the lowest mark, which is usually set at foundation level, has moved or not.

Mining Subsidence

Large movements caused by mineral extraction can seriously affect dams and reservoirs. In the UK, for instance, such movements have been associated with past and current coal mining. They have been the cause of major construction work (Leggo et al, 1986 ; Hughes et al, 1998).

Consolidation of Fill under Steady State Loading

Steady state loading refers to the condition where there is no change in reservoir level and therefore no change in the external load on the dam. Some settlement may continue to occur even many years after the completion of construction due to secondary consolidation of the fill.

Strain Weakening

Some clays with long mineral chains have residual strengths that are substantially lower than their peak strength. Under permanent loading they may exhibit strain increments that increase with time and affect the deformation of the embankment (Cummins, 1999).

Deformation of Fill Caused by Changes in Reservoir Level

The pattern and magnitude of deformations due to reservoir drawdown depend upon the position of the watertight element (Tedd et al, 1994). Old embankment dams, such as those built in the UK in the second half of the 19th century, have a relatively narrow central puddle clay core and lowering the reservoir level causes an increase in vertical effective stress in the upstream fill and reduces the lateral pressure acting on the core. This results in settlement which is at a maximum at the crest of the dam. In contrast, reservoir drawdown at a dam with an effective upstream watertight element results in a decrease in water pressure acting on the upstream slope and there is a tendency for the upstream slope to heave.

4.2.2.2 Seepage and Leakage Flow (Internal Erosion)

Principle and Causes of Internal Erosion

Internal erosion in embankment dams may not be observed for a long time after construction. It generally originates in design and construction inadequacies. It may occur within the embankment core or in the downstream shoulder. Quite frequently it occurs as concentrated leakage which develops into material transport along leakage paths at the contact of the core or of the earthfill with the foundation. The following causes have been reported :

- cracking of the embankment core in combination with insufficient filters downstream of the core
- internal instability, under seepage, of broadly graded earthfill, for example glacial till
- hydraulic fracture in a cut-off trench under the core
- dispersive clay
- erosion of earthfill at the contact with fissured rock or rock solution cavities
- leaching of soluble mineral salts
- erosion of earthfill materials with open-work gravel.

The term *internal erosion* can be applied to all processes that involve the transport of solid material within a porous medium. In embankment dams internal erosion involves the flow of seepage water removing solid material, usually well dispersed in suspension, from within an embankment or its foundation.

Forms of Internal Erosion

The term *piping* applies to a process that starts at the exit point of seepage and in which a continuous passage or tunnel often referred to as a pipe is developed in the soil by backward erosion. Cohesionless soils, particularly fine sands and silts, are most susceptible. When the pipe approaches the source of water there is a sudden breakthrough and a rush of water through the pipe. It may occur at an interface between different soils when the filter properties of the soil into which the water is flowing are not adequate to retain the particles of soil from which the water is flowing under the existing hydraulic gradient. The hydraulic gradient at the point where the water flows out of the ground is critical but is difficult to predict as it depends on localised weakness in the fill. Piping may start on the embankment slope where it should be relatively easy to prevent by positive design.

In a cohesive soil that is capable of sustaining an open crack, concentrated leaks may occur with erosion of soil particles along the walls of the crack. *Hydraulic fracture* of the core of an embankment by water at reservoir pressure may occur where stiffer granular shoulders support a thin core, not allowing the overburden stress to develop thereby raising the possibility of water at a near reservoir pressure from fracturing the core.

In contrast to piping and hydraulic fracture which are associated with concentrated erosion, a type of mass erosion can occur in soils that are internally unstable. This has sometimes been called *suffosion*. This means there is a mass migration of fines within the pore skeleton of an internally unstable soil. Depending on the overall grading, the coarser skeleton of the material may or may not remain intact. Unlike piping, suffosion usually begins within the mass of the soil. Such internal migration is a particular hazard for heterogeneous, broadly graded cohesionless soils. Earthfill materials may migrate into the voids of gravel in the foundation or rockfill if filter protection is not provided.

Internal erosion in a dam core is often observed by formation of sinkholes in the area of the dam crest. Sinkholes are the result of collapse of cavities formed at depth in the dam by internal erosion. Usually a sinkhole is located at the upstream side of the crest, where the settlements are influenced by the normal location of an upstream filter. When the erosion path in the core reaches the upstream filter, leakage may transport filter material into the leakage path simultaneously forming a cavity in the upstream filter. Collapse settlements propagate upwards in the upstream filter, and finally a sinkhole is formed at the surface of the dam. When the erosion path has propagated all across the core of the dam, the increase of leakage will be apparent.

Bronner et al (1988) report on **Hällby** and **Suorva Dams** in Sweden where sinkholes developed after portions of the moraine core were progressively washed out into widening bedrock cracks under the dam. In one dam the formation of sinkholes was attributed to a progressive erosion of the crack infillings, whereas in the other a progressive solution of the cementitious particles in the grout curtain caused by sulphate rich reservoir water was supposed to have been the main reason for the damage.

If the drainage capacity is high, the flow may increase and cause erosion at the downstream toe of the dam. The erosion at the toe could progress in the upstream direction, and finally lead to failure of the dam. An alternative failure mode may occur if the internal drainage capacity of the dam is not sufficient to transmit the leakage flow. This causes rising pore pressures in the downstream shell, which reduces its stability and may result in slides, which also finally could lead to failure of the dam.

Methods of Investigation of Internal Erosion

The basic methods of investigation are observations of the surface by visual inspection, deformation measurements, and leakage measurements, possibly combined with turbidity, tracers or chemical analyses. These observations are usually supplemented by geotechnical investigation methods ; for example test excavations, boreholes with sampling, piezometers, piezocone and cone penetration test (CPT). Piezocones can provide data on the permeability of the medium. Measurements of water levels in standpipes are preferably combined with temperature profiling in the pipes, particularly in homogeneous dams of relatively high permeability.

During the last decades these traditional methods have often been supplemented by geophysical methods. These methods can be non-intrusive or require boreholes. Examples of non-intrusive methods are ground penetrating radar (GPR), seismic and electric resistivity measurements from the dam surface. Self potential measurements have been performed in some investigations. Electrical conductivity measurements have been used successfully at **Uljua Dam** (Kuusiniemi et al, 1992) to locate areas of anomalous behaviour. Geothermal surveying using thermistors to detect and map subsurface thermal anomalies that may reveal flowing water has also been successfully used (Birman, 1996).

The ground penetrating radar (GPR) technique was used at Eastern **Suorva Dam** to investigate an anomalous increase in porosity in the moraine core. The radar data were generally confirmed by the subsurface mapping resulting from boreholes (Andersson, 1991). Triumf et al (1996) report the successful application of the self-potential method over a period of two years to detect changes in the seepage pattern across the core of the same dam. This method has also been used with success at **Kurkiaska Dam** and **Kalajarvi Dam** in Finland (Pyyny, 1998).

Examples of intrusive geophysical investigations are borehole radar and seismic measurements from boreholes. Crosshole measurements are carried out between two adjacent holes and tomography techniques can be used to define a two dimensional profile of the embankment or of its foundation between the boreholes. Usually, a combination of methods is required to detect inhomogeneities and preferential seepage paths resulting from internal erosion in progress. Exploratory drilling must be carried out carefully using techniques that do not themselves damage the delicate geotechnical balance that is being investigated.

Porjus Dam (see also section 4.3.2) is a 20 m high embankment with a central moraine core and rockfill shoulders founded on rock. A 4 m wide and 3 m deep sinkhole developed suddenly on the upstream side of the dam crest after 18 years of regular operation. A comprehensive investigation programme was immediately initiated. It comprised drilling and sampling of boreholes through the embankment and the rock foundation, installation of pipes for cross-hole radar measurements, as well as temperature and water level readings in order to locate the damaged parts in the dam core. Variation of the radar pulse velocity, combined with temperature measurements, allowed sufficiently precise definition of the extent of the zone to be repaired (Johansson S, 1994).

Laboratory tests, including mineralogy, on soil or rock samples can often be helpful. Nevertheless detection of defects giving a propensity to internal erosion is a specialised application requiring considerable experience. High quality sampling will be needed to identify defects such as a thin layer of sand in a clay core (Charles et al, 1996). If hydraulic fracture of a dam core takes place, the seriousness of this could depend, among other factors, on the erodibility of the clay. Particularly in countries where dispersive clays are common, erodibility tests such as pinhole tests have been developed for identifying dispersive soils (ICOLD, 1990). This type of test has been devised to detect clays that are subject to rapid erosion, and work continues on improving its design.

Other examples of innovative monitoring of dams include :

Nature of the Work	Location	Dam	Reference
Detailed, automated instrumentation. Drilling without flushing water	Core	WAC Bennett, ER, 183 m high, USA	IWP&D (1997)

4.3 REHABILITATION OF THE FOUNDATION AND ABUTMENTS

4.3.1 Introduction

Internal erosion is the most common ageing scenario for the foundation of earth and rockfill dams. ICOLD (1994) reports 64 case histories. Charles et al (1995) have provided useful definitions of the processes involved and a helpful version of these definitions is included in section 4.2.2. Within a rock foundation, the phenomenon may be envisaged as follows. Joints in the foundation rock mass open under the load imposed by the dam, or are disturbed during construction and by the filling of the reservoir. Seepage through the joints in the foundation may increase owing to a range of causes alone or in combination. These include solution of cementitious materials in the grout curtain, solution of minerals in the foundation including for example, gypsum. The internal stability of the material infilling the joints is an important factor and, where the dam is founded on soil, of the foundation material itself. Where the foundation includes erodible materials, fluctuating water level within the reservoir, can, through the mechanical effect of the varying water pressure, accelerate the process. As the process develops, the seepage flow rate may increase to such an extent that the material in the foundation is eroded. This in turn enlarges the flow path and intensifies the erosive process.

This sort of deterioration of the foundation is most frequently detected by the measurement of the seepage flow and the turbidity in the flow. Chemical and isotope analysis of the seepage water or tracers such as dyes or chemicals can help to identify dissolved minerals and to reveal their source (Charles et al, 1997). Rehabilitation measures to deal with excessive leakage and internal erosion through the foundation and cut-off are similar to those used for the watertight element in the embankment. They include :

- Filters designed to prevent or inhibit the flow of particles from the foundation. These are often placed on the foundation – embankment contact and downstream toe of the embankment.
- Drains, galleries or relief wells to adjust to a safe level the hydraulic gradient in the foundation, particularly at the location where the flow emerges.
- Grouting the foundation.
- Installing a continuous diaphragm wall cut-off. This can be formed by excavating a trench or by drilling overlapping holes, or by the technique known as jet grouting.
- Increasing the length of the seepage path by the use of a clay blanket upstream.
- In some circumstances the problems may be so severe that the economic optimum solution is to abandon the dam and to reconstruct it to a different design.

In the section below, case histories are presented to illustrate the use of some of these methods of rehabilitation of the foundation and abutments of embankment dams. The important lessons they reveal for successful rehabilitation of the foundation are :

- the need for careful and detailed planning,
- sometimes extensive temporary works are necessary to give adequate access,
- the need for good site management to react to the conditions giving rise to the rehabilitation, and which can change with great rapidity.

4.3.2 Rehabilitation Measures

4.3.2.1 Filters and Drains

The **Långbjörn Dam** situated in the River Ångermannaälven in Sweden is a 32 m high earthfill dam with a vertical moraine core. The dam is founded on soil and rock and the left abutment is an extension of the dam itself since it consists of a steep sided sandy river bank.

During the first filling of the reservoir excessive seepage and erosion occurred near the left abutment and in 1958 a slide occurred on the downstream side of the abutment. Standpipes installed in the abutment have recorded an increase in ground water pressure ever since the first filling of the reservoir, with temporary lowering as drainage measures were taken. Blankets of filter material and shallow drainage ditches were introduced in 1958, 1966, 1972 and 1986. Despite these a new slide occurred in the area in 1994. Since 1990 sinkholes have been observed in the surface of the abutment, probably as a result of internal erosion and loss of fine material through layers of coarse material in the abutment.

To lower the ground water level permanently, the decision was made to install almost horizontal drains into the abutment from the toe of the slope in the old riverbed. The rehabilitation was carried out in 1995 and 20 drains with a diameter of 100 mm and a length of 25 m were drilled. To further increase the stability of the slope of the abutment a stabilising berm of permeable material was laid out along the toe. The rehabilitation measures are summarised in Fig. 4.3.1.

Standpipe readings after the rehabilitation indicate a considerable lowering of groundwater level in the area and it has since been necessary to install new standpipes because several of the pipes became dry. It can be concluded that the rehabilitation has given even better result than anticipated but it is important to follow up the effect by means of continuous measurements and evaluation (Nilsson et al, 1996).

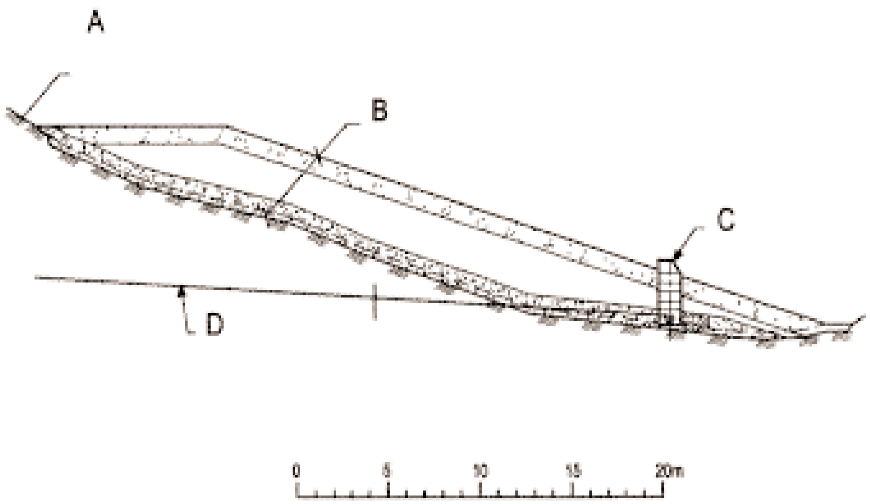


Fig. 4.3.1

Långbjörn Dam. Cross section, horizontal drains and stabilising berm

A. Original profile
B. Stabilising berm
C. Access manhole
D. Drilled drains

4.3.2.2 Grouting of the Foundation

Grouting is usually performed from the crest of the dam, from specially constructed galleries within the embankment close to the contact with the foundation, or from adits driven into the abutments. Special care is required to prevent damage to the embankment materials when drilling through the embankment or from abutment adits to the embankment-foundation contact zone. Well designed grouting can substantially reduce seepage and arrest an erosion process. Repeated treatments are often required either because of continuing deterioration or because the grout may not have succeeded first time in filling all the joints. The choice of the most effective material and measures depends on the problem. Thus the width of gaps and the volume of the region to be filled will determine whether mortar or cement suspension should be used, or whether a chemical grouting techniques is preferable (ISRM, 1995). Where a high strength of the grouted rock is important, for example when grouting fissures in the rock foundation, superplasticizer can be used as an additive. The use of superplasticizer reduces the viscosity and thus improves the ability of the grout to penetrate the joints.

Uljua Dam in Finland is an example of the successful use of grouting. A 13 m high embankment some 10 km long, it has been in operation since 1970 (Kuusiniemi et al, 1992). In 1990 the leakage water observed from fissures in the foundation rock increased in quantity and became muddy. The leakage seemed to be increasing and the decision was made to grout the foundation. Divers found and filled with soil, sinkholes several metres deep in the bottom of the lake near the dam and tracers confirmed that the seepage originated in them. Almost immediately the upstream side of the crest of the dam subsided 3 m over a length of about 7 m. Emergency backfilling of the hole saved the dam from collapse and gave sufficient time for the rehabilitation works to be completed. The reservoir level was lowered by 5 m.

Geological investigation showed that massive erosion of the foundation and the dam body had occurred at the junction of two water bearing fractured zones in the bed rock. The seismic velocity in the junction area is 3 km/s to 4 km/s and the rock was found by diamond drill coring to be heavily fractured with much core loss. The soil is an extremely erodible silty glacial till. The first muddy seepage was probably material moving from the embankment along a path near the original soil surface. The first exploratory boreholes used air flushing equipment and are thought to have disturbed the foundation when it intersected one of the internal erosion channels. The erosion progressed rapidly upstream causing a sudden and extensive sinkhole. The cross section showing the hypothetical failure mechanism is shown on Fig. 4.3.2. The erosion damage was limited to a 60 m length of the dam, and this was replaced in the dry behind a coffer dam. As the damaged portion of the dam was removed the existence of the erosion channel was confirmed as a tube about 3 m in diameter, sloping down through the core and into the foundation, as shown on Fig. 4.3.2. During the collapse of the dam this tube had filled with some 50 m^3 to 100 m^3 of gravel from the upstream filter. As the rehabilitation work proceeded the gravel was removed carefully as far as possible and the area was sealed by grouting.

The seepage was cut-off by grouting the erosion channel, by constructing a diaphragm wall and by increasing the length of the seepage path. The length of the grouted zone is 585 m and 358 grout holes were grouted using a total of 950 tonnes of cement, more than 5 tonnes of polyurethane, and about 8 tonnes of resin grout. The grout holes were drilled 10 m into rock generally and 15 m in the problem area. Three curtains were drilled. The first two from the crest and the third from ground level after the damaged portion of the dam had been removed. The average spacing of the holes drilled from the crest was 800 mm and in the third curtain 1400 mm. The

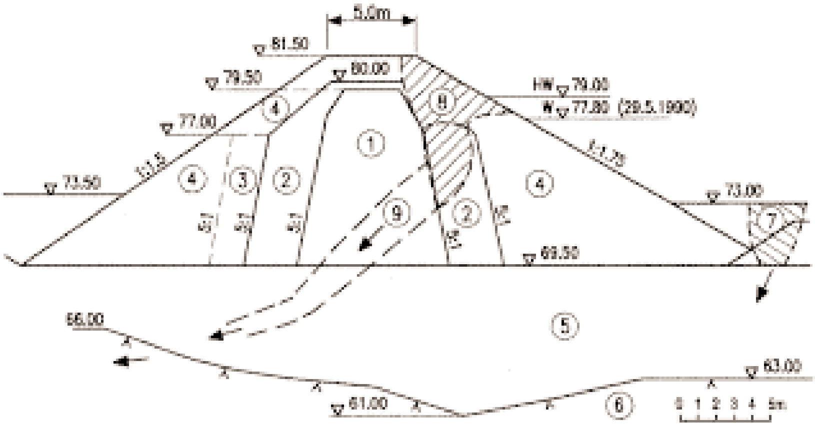


Fig. 4.3.2

Uljua Dam. Cross section of damaged dam

1. Core of glacial till
2. Filter of sandy gravel
3. Coarser filter
4. Rockfill shoulder
5. Glacial till
6. Weathered bedrock
7. Crater
8. Sinkhole
9. Gravel tube

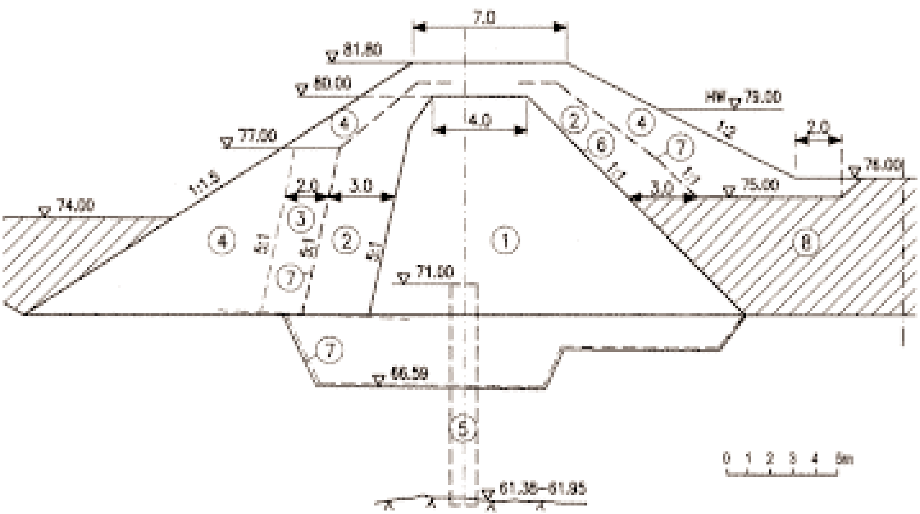


Fig. 4.3.3

Uljua Dam. Reconstructed cross section

1. Core of glacial till
2. Filter of sandy gravel
3. Coarser filter
4. Rockfill shoulder
5. Diaphragm wall
6. Bentonite clay blanket
7. Geotextile
8. Blanket of glacial till

maximum grout pressure used was 600 kPa to 700 kPa. Polyurethane was used in those holes with a significant connection to the surface. Its purpose was to plug the open joints. Grouting of the erosion channel was carried out in two phases : initially a curtain parallel to the dam was grouted with cement grout and this was followed by resin.

A diaphragm wall 10 m long was placed in the most erosive zone. The diaphragm wall comprised 1200 mm diameter cast in situ concrete piles. Their connection to the bedrock was sealed with cement grout. In this critical area the rehabilitated dam section is considerably wider than the original, with glacial till connecting the original core to the coffer dam 60 m upstream. A bentonite layer was placed on the upstream surface of the core to reduce its permeability. The repaired section is shown on Fig. 4.3.3. The repairs to the dam took place in poor weather in which it was not possible to monitor the density of the fill or the pore pressures within it.

Another example of comprehensive grouting treatment of the foundation is **El Chocón Dam**, Argentina (Aisiks et al, 1991). The degradation observed in the foundation was a result of a combination of various adverse natural, design and construction conditions including : solution, particularly of gypsum ; washing of insoluble infilling from joints ; and internal erosion of the dispersive soil in the core. Rising piezometric pressure in the right abutment core contact area and high salt content in the seepage water initiated investigations and led to remedial grouting in both abutments and the abutment core contact. The engineering geology maps prepared during construction of the excavated foundation proved valuable in planning the investigations, developing an understanding of the problem and designing the rehabilitation.

The design for the remedial works required access to the right abutment core contact. This was accomplished by excavating a 107 m deep circular shaft and three levels of horse shoe shaped galleries. These are shown on Fig. 4.3.4. The shaft is 3.4 m in diameter and the galleries are 3 m wide by 2.7 m high. The entire system is lined with 500 mm thick reinforced concrete. Excavation was done with pneumatic hammers because blasting was not permitted. Grouting was done in a zone 12 m ahead of the working face to control the flow of water into the excavations.

The contact between the foundation rock and the core of the dam was highly fractured with joints full of water at reservoir pressure. The core had soft zones. Three zones were identified for grouting : the rock mass, the transition zone, and the rock-core contact. A safe procedure was adopted for the drilling and grouting in the softened core in contact with fractured foundation rock. This is shown on Fig. 4.3.5. Special procedures were developed to control pressures and the flow of material into the boreholes and foundation voids. Drainage was provided. A pressure regulation device and a double gate valve installed at the borehole collar were mandatory when drilling against reservoir pressure. Safe maximum grouting pressure and volume of injected grout were determined by the conditions on site and not by arbitrary general rules. Computers controlled the operating systems to ensure that the design pressures were not exceeded, as well as to monitor, record and direct the grouting. The authors report that they were successful in sealing the open joints system and filling the core cracks that had developed during the first ten years of operation.

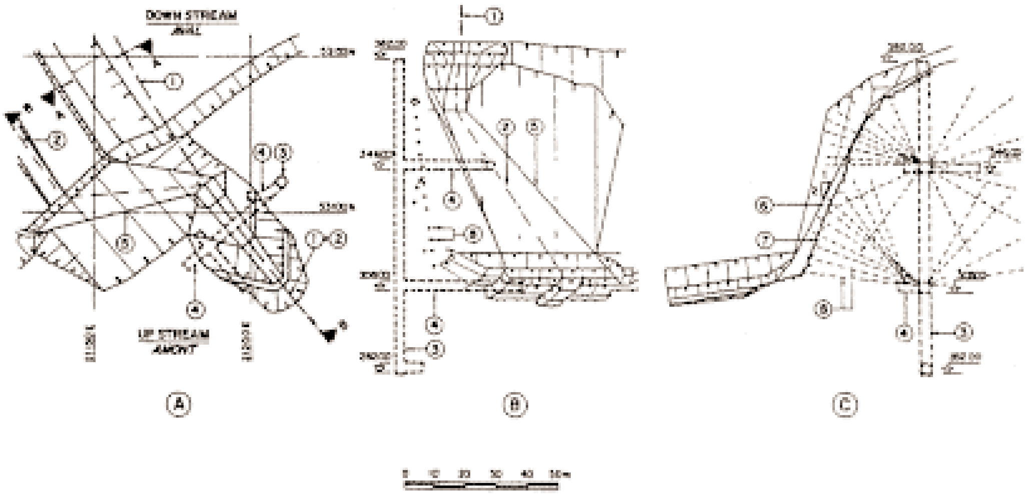


Fig. 4.3.4

El Chocón Dam. Layout of right abutment works

A. Plan
B. Cross section A-A
C. Cross section B-B
1. Dam axis
2. Core axis
3. Shaft
4. Galleries
5. Upstream limit of rock-core contact
6. Design slope of abutment excavation
7. Actual abutment slope
8. Drilled drain hole

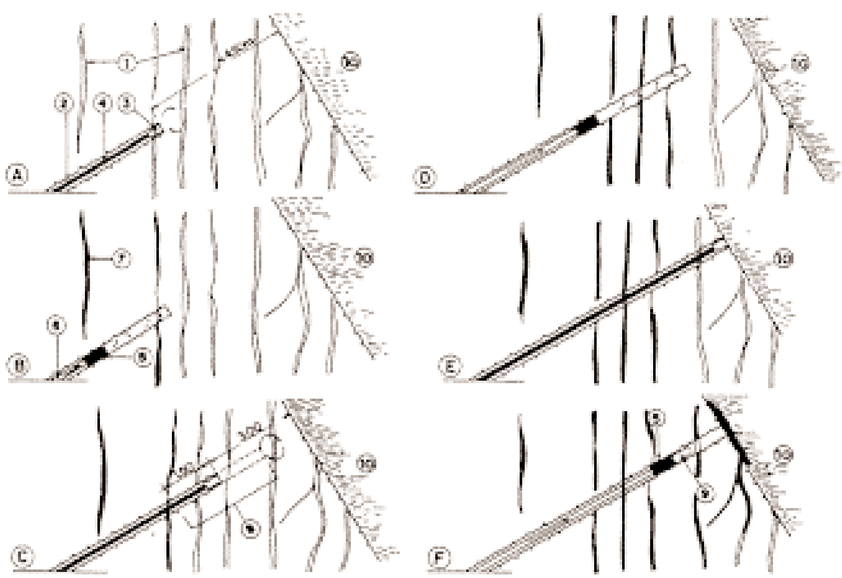


Fig. 4.3.5

A further example of extensive and imaginative grouting of the foundation is **Logan Martin Dam** (Williams et al, 1994 ; Jansen, 1994). The karst forming the foundation of Logan Martin Dam consists primarily of dolomite and chert with minor amounts of sandstone and limestone. The rock is highly fractured from multiple periods of faulting. Both the limestone and dolomite are subject to chemical weathering, which carves conduits along horizontal and vertical fractures providing paths for leakage beneath the dam. In spite of extensive grouting during construction, followed by additional grouting and other remedial treatment of the reservoir floor over almost 30 years following construction completion, degradation of the foundation by washing of infill kept leakage at undesirable levels. Investigations, extended to greater depths, led to the conclusion that the original grouting depths were too shallow. The greatest permeability occurs within two horizons, one shallow the other very deep. The two horizons are connected vertically by fractures that slice through the dolomite. The dam is 30 m high and grouting is being carried on to depths of 152 m below the surface of the rock.

The owner concluded that to treat the foundation satisfactorily he would need intensive long-term grouting with a permanent grouting facility. Grout hole drilling took place from a work berm along the upstream slope of the dam. Casings through the embankment were seated up to 1.5 m into the rock foundation and grouted in place to form a seal at the soil-rock interface. Drilling the deep curtain holes posed problems, such as breaking drill rods in the hard but cavernous rock. To minimise the problem, the holes were drilled using the more durable 114 mm diameter drill rods. A procedure was developed for drilling the various zones of rock types and fracturing, using rotary and percussion with a downhole hammer. A state-of-the art automated, centralised, high-capacity, grout plant was constructed capable of instantaneous response to conditions encountered in foundation cavities with flowing water. The computerised plant can provide up to 40 different mixes. High-speed, colloidal-type mixers in conjunction with high pressure hydraulic piston pumps can grout two holes simultaneously with any combination of aggregates and liquid admixtures to a grout mix of water-to-cement ratios as thick as 0.5 :1 by volume using a superplasticiser. The bulk fillers used to seal and fill cavities included crushed limestone with particles up to 10 mm, light weight aggregates, sawdust, cedar shavings, vermiculite, cotton seed hulls, 13 mm wood chips, and with a dual injection system to add burlap strips and polypropylene sacks to the pumped grout. Liquid admixtures such as antiwash, to reduce bleed water in the grout and retain

---

Fig. 4.3.5

El Chocón Dam. Grouting sequence

A. Drilling through rock mass
B. Grouting the rock mass
C. Drilling the transition zone
D. Grouting the transition zone
E. Drilling to the core-rock contact
F. Grouting the core-rock contact zone
1. Open joint
2. Drillhole
3. Drill bit
4. Drill rods
5. Inflatable packer
6. Grouting pipe
7. Grouted joint
8. Transition zone
9. Lost packer
10. Core

the integrity of the grout slurry in flowing water, liquid calcium chloride and superplasticisers. Sodium silicate was added through a static mixer at the bottom of the packer to preclude gelling in the grout lines and packer pipes. Other materials were tried and considered to be not sufficient effective. The comprehensive grout curtain is scheduled for completion in 2001.

The experience at **Tims Ford Dam** is a useful case study of treatment of the reservoir rim. Since the first filling of the reservoir in the early 1970s, the seepage recorded through the right rim of the reservoir had accelerated markedly, particularly in 1995 and 1996. When the seepage reached some 500 l/s, it was decided to rehabilitate the rim works (Bruce et al, 1998). Tims Ford is a 53 m high compacted rockfill dam with a sloping impervious rolled earthfill core. This Tennessee Valley Authority (TVA) structure is multi-purpose, providing flood control, recreation, water supply and peaking power generation.

The right rim comprises a sequence of limestone formations covered by a red cherty clay resulting from the in-situ weathering of the rock. The limestone beds contain many solution cavities along their bedding planes and vertical joints. The lowest formation is of shale lenses and clay seams. During design, TVA decided to delay treatment of the reservoir rims pending evidence of seepage, see Fig. 4.3.6. Leakage was collected and monitored and found to be responsive to fluctuations in reservoir level. Following the first filling, the TVA planned an orderly programme of work to reduce the flow from all observed leaks.

Grouting was conducted successfully on the left abutment using cement with calcium chloride accelerator and asphalt. Flow from other leaks was also reduced. Leakage increased steadily over the years from Leak 6, which had not been grouted due to the long seepage path, reaching 250 l/s at maximum pool level in 1995. Later that year the rate of flow almost doubled, followed by a large slide on the hillside around the leak. It was reasoned that the annual reservoir fluctuations had resulted in cycles of wetting and drying of the clays and other degenerated material in the features of the limestones. This had made them more susceptible to erosion under the hydraulic gradient at high reservoir levels, thus creating flow paths with larger apertures, and rapidly increasing the overall rock mass transmissivity.

In an exploratory drilling and permeability testing programme at Leak 6, seventeen exploratory holes were percussion drilled, permeability tested in stages, and dye tested to establish flow paths and estimate velocities. Three standpipe piezometers were installed to help gauge the effectiveness of the later grouting. Several dye test connection times of only minutes were encountered to the leak.

A multirow remedial grout curtain was designed. The primary holes of this curtain are shown on Fig. 4.3.7. The holes were inclined at 30 degrees to the vertical to encourage intersection of sub-vertical features and were oriented in opposite directions in the two outside rows. Primary holes in each row were foreseen at 12 m centres, with split spacing to reduce interhole spacings to 3 m. The central, tightening, row was vertical. The grouting was to be executed to seal a 15 m vertical zone and locally deeper if dictated by the stage permeability tests conducted prior to the grouting of each stage.

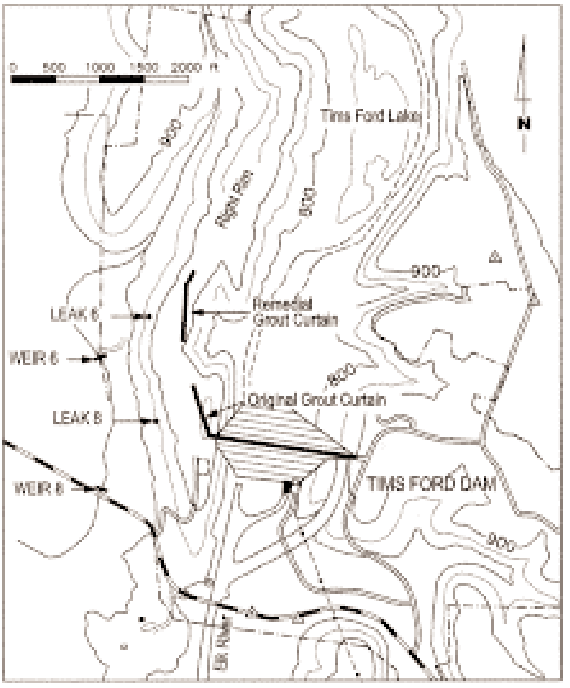


Fig. 4.3.6

Tims Ford Reservoir Rim. Plan of dam and reservoir

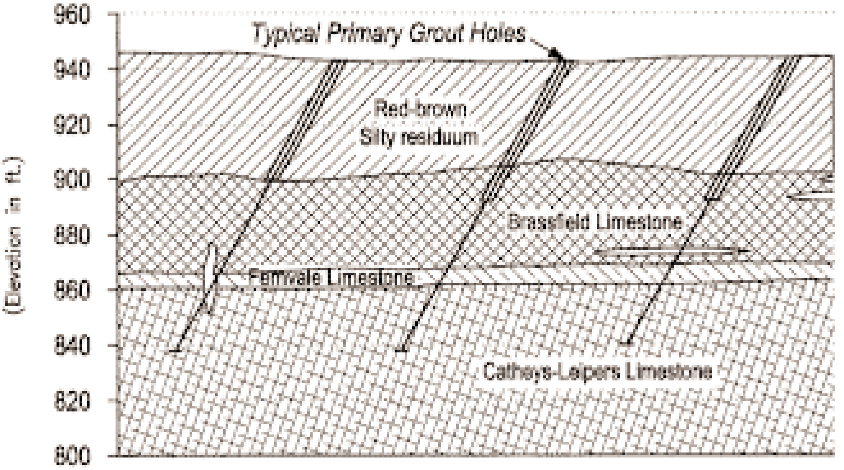


Fig. 4.3.7

Tims Ford Reservoir Rim. Remedial grout curtain

Because high water flow was anticipated the downstream curtain was designed to be grouted with fast-setting hydrophilic polyurethane resin with a 1 to 3 minute set time to provide an initial semi-permanent flow barrier. Holes that did not encounter voids or active flow were to be injected with cementitious grout. Upon completion of the downstream row it was anticipated that the flow would have been reduced, thus allowing the upstream row, followed by the central closure row to be grouted with cementitious grout to form a permanent and durable grout curtain. The grouting was to be performed using upstage methods although poor foundation conditions could locally require downstage grouting in conjunction with the polyurethane resin. The grout holes were to be cased through the overburden from the surface to the top of the curtain. The TVA's goal was to reduce the peak seepage to about 60 l/s and to focus only on the major features. Monitoring and limitations to outflow pH and turbidity were specified to protect the downstream environment. The reservoir was drawn down to reduce hydraulic gradient and flow through the rim. The curtain was constructed by first grouting the far ends, so conceptually channelling the flow through a middle zone which would be sealed last.

Actual field conditions varied in detail from what was foreseen but the following major differences and modifications were made, in full cooperation with the Contractor.

- TVA had to observe strict reservoir drawdown restraints. The rehabilitation programme required the Contractor to use multiple shifts, to drill at times with two rigs and to grout with different mixes simultaneously. Track-mounted diesel hydraulic rigs were used to drill overburden and rock. The overburden casing was installed using rotary drilling techniques and rock drilling was performed with 3.5 inch down-the-hole hammers. A high volume colloidal mixer was fed from bulk cement silos. Two helical screw Moyno pumps were used to grout holes simultaneously. Polyurethane grout was pumped using a high-pressure piston pump (B-10 Rocker Multi-Component). Compaction grout was batched on site, using a two-conveyor, three-component trailer-mounted batch plant, with a hydraulic-driven mixer/conveyor auger. This system of mixing and delivering the grout mix was supplemented by ready mix trucks dispatched from local plants. Specially configured high pressure, double piston pumps were used to pump the grout.
- The outflow at Leak 6 stopped when the reservoir level drawdown approached the leak elevation. As a consequence, much of the grouting was done with no water flowing, thereby largely eliminating the need for the polyurethane grout, and extending the applicability of cement based formulations, which included low mobility mixes.
- Larger than anticipated open or clay-filled features were encountered especially in the upper 6 m of the curtain. For technical, commercial, environmental and programme reasons, these openings were filled with a low mobility grout with a slump between 50 mm and 150 mm containing a water reducing agent and referred to as a *compaction grout*.
- A suite of cement-based grouts were developed to permit the appropriate match of mix design and thickening sequence to the conditions as revealed at each stage by drilling and permeability testing.
- In response to conditions revealed during the treatment, observations of the seepage and further dye testing, extra groups of holes were added at the north

end of the curtain, including 11 orthogonal to the original curtain, to allow specific treatment of key features.

- About 1600 m^3 of compaction grout, 1500 litres of polyurethane, and 600 m^3 cement-based grouts were injected into a total of 250 holes comprising 3350 m of rock drilling in the zone treated.

To monitor the grouting on-site engineers used results from drilling, water tests, calculation of grout hole reduction ratios, and dye testing. This monitoring allowed them to track the development of the integrity of the grout curtain and focus grouting efforts on specific zones along the grout rows. Reduced discharge from the rim leak, groundwater elevations down-gradient from the grout curtain, and headwater elevations confirmed the effectiveness of the grouting. Grout takes closely followed trends observed in the water test data.

Quarternary and closer holes were required fully to treat problem zones. As the contractor began the final stages of the grouting, he performed dye tests to identify connections to the leak and to confirm the required extent of the curtain. By this time, it was apparent that flow was no longer spread out across the rim, but channelled by the grouting process to the north end of the grout curtain. The evaluation of connections, not only by dye tests, but also by observing washout of grout and turbidity in the leak, were instrumental in achieving the success that was attained under flowing conditions in the features of the karstic limestone. Refilling of the reservoir proved that Leak 6 had been reduced to less than one-half of the goal of 60 l/s. Leakage was also reduced at other leak areas. Subsequent monthly inspections of the rim have revealed no new leak, and the previously saturated overburden near Leak 6 is dry. Rim leakage will be monitored over the long term because of the karstic formation containing clay-filled voids that offer the potential for future erosion and increased seepage. Careful investigation, evaluation, design, construction, and evaluation during construction with modifications as needed, all in a well planned and managed program are the keys to a successful rehabilitation.

Other examples of rehabilitation of the foundation by grouting include :

Nature of Rehabilitation	Location	Dam	Reference
Extension of grout curtain and new drainage provision	Foundation	Patana, Finland	Kuusiniemi et al, 1996
Grouting of bedrock and rebuilding of sub-surface drainage	Foundation	Venetjoki, Finland	Kuusiniemi et al, 1996
Foundation grouting	Foundation	Montézic, France	Tournery et al, 1991
Grouting by sleeve tubes of weathered granite foundation	Foundation	Nèpes, France	Lino et al, 1994
Deep grouting of karstic foundation	Foundation	Logan Martin, USA	Williams et al, 1997
Foundation cut-off in residual soil, successful	Foundation	Thika, 60 m high, Kenya	Attewill et al, 1992 Attewill et al, 1994
Remedial grouting	Foundation	Red Rock, USA	Carr, 1996
Grouting from adit to abutment/embankment interface	Abutment	Navajo, ER, 123 m high, USA	Davidson, 1990 Davidson et al, 1988

4.3.2.3 Diaphragm Cut-off Walls

Diaphragm walls have been the chosen method for rehabilitation where the deterioration in the foundation is at too large a scale for grouting to be carried out economically. This is a form of construction for which a large body of experience is available, making it a reliable, though perhaps costly, choice for improving the strength and impermeability of a leaking foundation. It has been chosen to seal open, loose and blocky ground in a remote location where speed was of the essence in securing the dam (Schneeberger et al, 1991). A specification for slurry trench cut-off walls to be used as a barrier to pollution migration contains data useful for the rehabilitation of dams (Doe et al, 1999).

Diaphragm walls are usually constructed through the embankment into the foundation to a depth that will prevent seepage detrimental to the safety of the embankment or undesirable loss of water. Several methods are commercially available for forming the trench, a process that requires a slurry to stabilise the excavation. The major alternative construction methods, single or double stage, are discussed further in section 4.4 in connection with rehabilitation to the core of embankments. The trench may be excavated with a rock mill through the embankment and into the foundation. As soon as it is excavated, each panel is backfilled with concrete to form the wall.

The wall, often 600 mm wide, is usually constructed in panels up to about 4 m long, although other widths are reported as has continuous construction. It is usual to key the wall into the bedrock by a few hundred mm. Care is required to ensure that adjacent panels fit tightly together and that they are accurately vertical. ICOLD Bulletin 51 (1985) gives useful background to the construction techniques and materials used for slurry stabilisation of the excavation. Backfill may be of either rigid or of plastic concrete, depending on the stiffness and long term behaviour of the foundation materials. The following mix was used at Ostrovul Mic in Romania : 700 l water, 300 kg cement, 200 kg bentonite and 10 % sodium silicate (Trandafir et al, 1999).

An alternative approach to forming a diaphragm cut-off is to auger primary large excavations through the embankment, or to excavate it with a clamshell. **Uljua Dam** in Finland is an example of the first of these techniques. These openings may be supported with steel in the embankment and rotary drilled into the rock foundation. The holes are subsequently filled with concrete. Secondary elements are installed by excavating the space between the primary elements with a clamshell and rock chisels under a head of bentonite slurry. A third method is to install overlapping concrete piles, known as secant piles.

Jet grouting is a more recently introduced technique that has also proved capable of improving poor foundations in soft material at a lower cost than for a diaphragm wall. In this technique a hole is drilled with a tricone bit with bentonite. The cement-based grout is then injected under high pressure, as high as 45 MPa, as the drill string is rotated and slowly withdrawn from the hole. Spacing depends on the equipment available and the nature of soil and is usually between 500 mm and 800 mm. Schneeberger et al (1991) provide convincing evidence that adjacent holes can overlap satisfactorily. They report that they keyed the holes 1500 mm into bedrock to ensure that it had been reliably reached.

At **Wister Dam**, USA, a 30 m high earthfill embankment, investigations revealed widespread cracking throughout the dam and foundation overburden and that much of the material was susceptible to dispersive erosion. Part of the rehabilitation consisted in a plastic concrete slurry wall cut-off through the embankment and foundation into the foundation rock. To maintain the integrity of the upper part of the plastic concrete a clay cap was provided on top of the slurry wall (Erwin et al, 1991).

Beaver Dike 1, USA, is one of the auxiliary embankments closing the **Beaver Dam** reservoir (Bruce & Stefani, 1996). During design a graben beneath the dike was identified as a potential source of difficulty. The graben consists of the permeable and highly weathered Mississippian karstic limestone with clay filling known as the Boone Formation. The original design approach to sealing the foundation comprised a grout curtain. Seepage was observed as soon as the reservoir was filled. Ten years later remedial grouting to a greater depth was carried

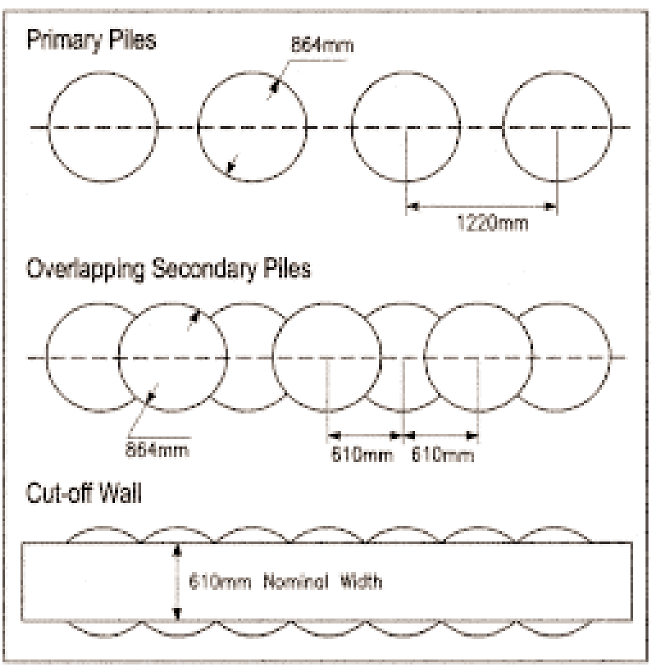


Fig. 4.3.8

Beaver Dike 1. Scheme for construction of the wall

1. Primary piles
2. Overlapping secondary piles
3. Cut-off wall

Fig. 4.3.9

Beaver Dike 1. Drill rigs on crawler cranes out with difficulty against flowing water. Degradation continued with increased seepage. A positive concrete cut-off was proposed using a slurry trench with rock milling technology. Beds of strong relatively fresh rock prevented use of the rock mill and large diameter secant concrete piles were adopted.

The wall was constructed from a work platform built into the upstream face of the dike. The wall is 464 m long and from 24 m to 56 m deep to accommodate the geological conditions. The total wall area is 19 300 m^2. The scheme for construction is shown on the Fig. 4.3.8. In Stage 1 a series of primary piles was drilled and grouted with a tremie. In Stage 2 the intermediate secondary piles were drilled and concreted to complete the cut-off. Careful drilling was required to ensure continuity of the wall and special monitoring was used to control the vertical alignment of the holes by ensuring that the piles did not deviate more than 64 mm at a depth of 56.4 m.

Two drill rigs with 30 m masts mounted on crawler cranes were used to drill the holes as shown on Fig. 4.3.9. The 9 m drill rods had an external diameter of 813 mm and an inner air passage of 302 mm. Each rig could drill 21 m in a single pass using down-the-hole hammers with 864 mm diameter bits. The rotational speed varied between 2 rpm and 10 rpm and depended on ground conditions. The 10 bar compressed air supply to each hammer was from a bank of nine, static electric-powered compressors arranged in two groups of four with one spare. The penetration rates varied from 4.3 m/hr for primary holes to 5.4 m/hr for the secondaries. Some pre-treatment of the ground was done by pressure grouting. Elsewhere in unstable weathered rock the holes were stabilised by grouting or down staging. The seepage through the foundation decreased to 0.3 l/s from over 80 l/s before the rehabilitation works.

At **Twin Buttes Dam** (Dinneen & Sheskier, 1997) the cut-off passes through the embankment into the foundation, which comprises a lean clay rich in calcite locally cemented to varying degrees, coarse grained, highly variable Quaternary alluvial deposits, and flat lying shale and sandstone of Permian age. Attempts had been made to grout the alluvial materials. A number of additional methods were studied before a soil cement bentonite cut-off wall was selected. Backfill alternatives considered were plastic concrete, cement bentonite, soil bentonite, soil-cement-bentonite and a vertical geomembrane. Backfill construction considerations evaluated included batching, segregation and placement methods, workability, quality assurance and control, in-situ strength and permeability, uniformity of construction and horizontal joints. The wall was constructed from a work berm at the upstream toe of the embankment. The length is about 6500 m, the depth up to 31 m and the area of the wall about 130 000 m^2.

Other examples of rehabilitation of the foundation by diaphragm walls and related techniques include :

Nature of Rehabilitation	Location	Dam	Reference
Concrete diaphragm wall through embankment and foundation of karstic limestone	Foundation	Wolf Creek, 79 m high, USA	Fetzer, 1979
Concrete diaphragm cut-off wall through embankment and foundation (highly fractured rock)	Foundation	Fontenelle, 42 m high, USA	Cyganiewicz, 1988
Left abutment treatment by concrete diaphragm cut-off extending into rock foundation	Foundation	Navajo, ER, 123 m high, USA	Davidson et al, 1988 Davidson, 1990
Concrete diaphragm wall through embankment	Foundation	Wells, 49 m high, USA	Kulesza et al, 1994
Early attempts to reduce the seepage and internal erosion augmented by a secant pile diaphragm wall 35 m deep, 140 m long of clay concrete through glacial till	Dam body and foundation	Kureiskaya, 30 m high, 300 m long, ER, Russia	Bellendir, 1999
Plastic concrete diaphragm wall 1 m wide, 40 m to 52 m deep through glacial till to tight geological foundation	Foundation	Meeks Cabin, 56 m high, ER, USA	Gagliardi et al, 1993
Following failure of upstream face and cut-off wall, a secant pile wall 22 m deep and 650 mm diameter, through alluvium to bedrock	Foundation	Ostrovul Mic, 20 m high, 1900 m long, Romania	Trandafir et al, 1999
Bentonite-cement diaphragm wall	Foundation	Saint-Pardoux, France	Lino et al, 1994 Bister et al, 1994
Successful use of bentonite concrete diaphragm and jet grouted cut-off	Foundation	LG-1 coffer dams, 24 m and 20 m high, Canada	Schneeberger et al, 1991
Successful use of overlapping cast in situ concrete piles	Foundation	Uljua, 13 m high embankment, Finland	Kuusiniemi et al, 1991
Successful use of jet grouting to seal the base of sheet piles through the embankment	At interface of sheet piles and granite rock foundation	Dyke 3, Lake Manouane, 10 m high, Canada	Lemelin et al, 1996
Successful use of jet grouting technique	Foundation	Thika, 60 m high embankment, Kenya	Attewill et al, 1992 Attewill et al, 1994
Cement-bentonite cut-off wall and jet grouting columns to stop seepage around abutment	Foundation	Cod Beck, UK	McLeish et al, 1987

4.3.2.4 Removal and Reconstruction

Occasionally partial excavation of the embankment to expose the foundation and then further excavation or replacement of the deteriorated foundation is the selected solution. At **Hackberry Site 1** (USCOLD, 1996) about 215 m of the left abutment was removed. The gypsum bedrock was cleaned as were the voids and cracks visible on the surface prior to their treatment with dental concrete. The entire abutment was capped with 75 mm thick concrete, placed wet just ahead of the replacement fill. The first incident at **Fontenelle Dam** (Cyganiewicz, 1988) was treated by excavating the right abutment, and grouting the foundation before rebuilding the embankment.

The failed **Quail Creek Dike** (Catanach et al, 1991), an embankment dam on a foundation of thin-bedded limestone, dolomite and gypsum, was replaced with an RCC foundation cut-off wall and dam as shown on Fig. 4.3.10. The depth of cut-off was based primarily on engineering judgement and the construction on availability of construction equipment and lowest cost (see Fig. 4.3.11). Important in the choice of dam type was that an embankment dam would require filters that would be expensive and was less certain than an RCC dam. In addition, a grouting gallery would be needed in either dam type and the RCC dam construction would constitute a continuation of the cut-off.

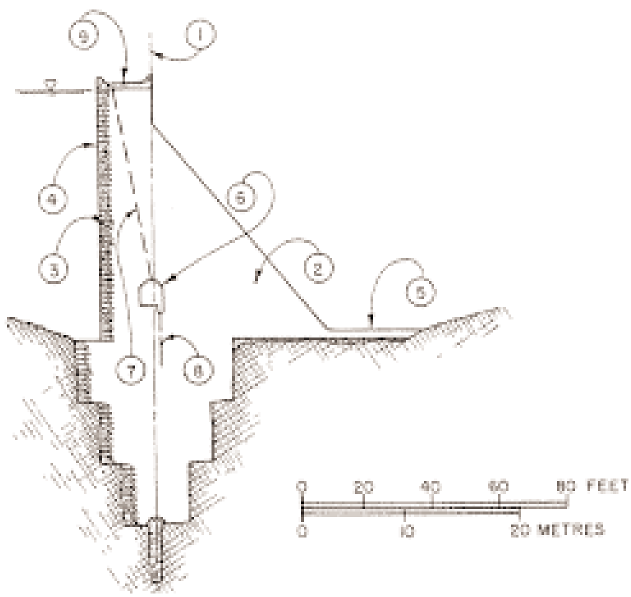


Fig. 4.3.10

Quail Creek Dike. RCC dam section

1. Dam axis at crest
2. Roller compacted concrete (RCC)
3. Conventional concrete
4. Upstream face
5. Apron
6. Gallery
7. Dam drain
8. Cut-off drain
9. Crest

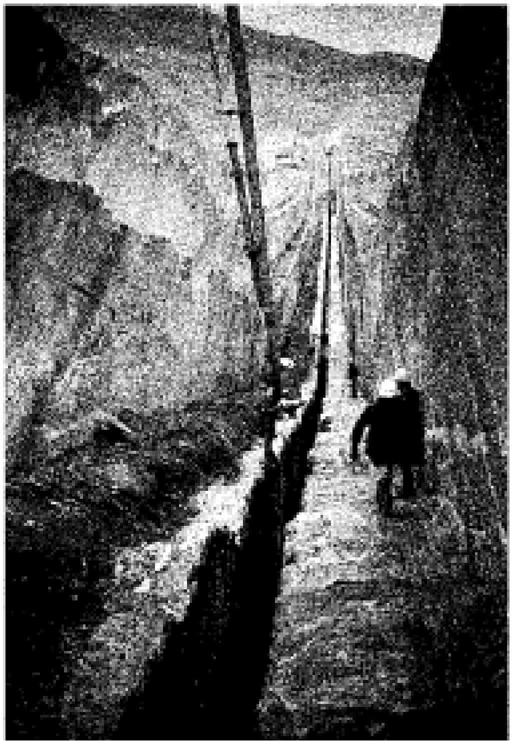


Fig. 4.3.11

Quail Creek Dike. Foundation excavation

The **Penn Forest Dam** (Bingham et al, 1997) experienced a sink hole in the upstream face during first filling. The reservoir was drawn down and the hole repaired. Following the repairs the dam was operated for 26 years under an extensive instrumentation and monitoring programme. A sudden decrease in foundation piezometric level signalled a problem. Investigation revealed deterioration of the foundation and embankment. From several alternatives for rehabilitation, the owner selected a new RCC structure which was constructed into the partially removed embankment. The RCC structure contained a PVC waterproofing element. This allowed full use of the existing appurtenant works.

Fig. 4.3.12 shows the RCC replacement dam with the regraded embankment against the downstream face. The original spillway is on the right abutment, and the original low-level outlet and outlet works on the left abutment. The Figure shows the RCC conveyor and the coffer dam.

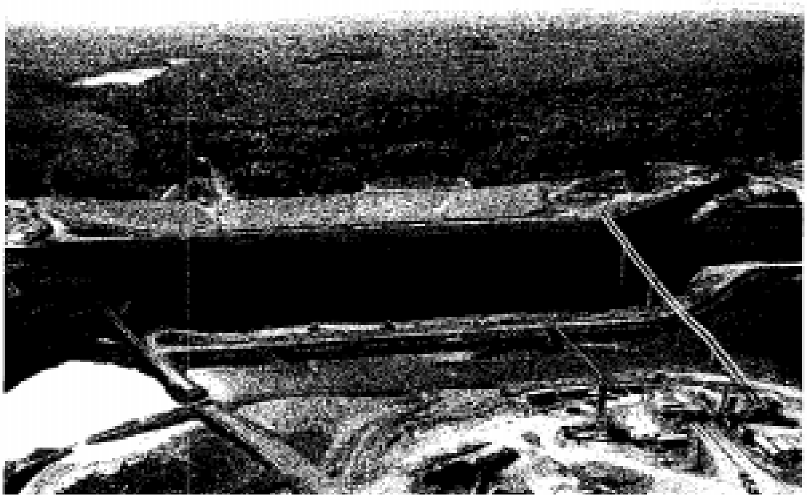


Fig. 4.3.12

Penn Forest Dam. RCC replacement dam

4.4 REHABILITATION OF THE EMBANKMENT

4.4.1 Introduction

We have outlined in section 4.2.2 the mechanisms and processes causing deterioration of the embankment. The rehabilitation works encompass three main categories of work :

- rehabilitation of the watertight element, to reduce seepage and the internal erosion that may accompany it.
- improvement of slope stability, to restore the structural stability impaired by loss of strength or foundation settlement for example.
- rehabilitation of the surface protection, to prevent wave erosion of the embankment or to restore the watertightness of the external facing of dams with an upstream membrane.

In this section we deal with each of these broad subjects in turn.

4.4.2 Rehabilitation to the Watertight Element

Remedial works to the watertight element are normally carried out to reduce leakage through the dam. Leakage volume is not normally so excessive as to impair the economic viability of the reservoir, but the leakage may lead to failure through internal erosion and increased downstream pore pressure, reducing the stability of the embankment slopes. Rehabilitation aims to achieve one of three broad goals : to improve the impermeability of the element ; to control the seepage to prevent internal erosion ; and where these are not viable options, to replace the inadequate construction. Therefore the following methods have been used to rehabilitate the watertight element :

- Improving the impermeability. Methods for this include grouting, installation of an impervious cut-off, such as a diaphragm wall, jet grouting, secant piling or sheet piling.
- Control of the seepage through the element or rehabilitation of a toe drain or a downstream filter.
- Replacement of a faulty section of core and the construction of a new watertight element.

4.4.2.1 Grouting

Grouting is the most common remedial treatment of leakage in embankment dam cores. Pressure grouting was invented and first applied in 1802 by Charles Berigny. It was first used for dam rehabilitation in the United Kingdom for remedial

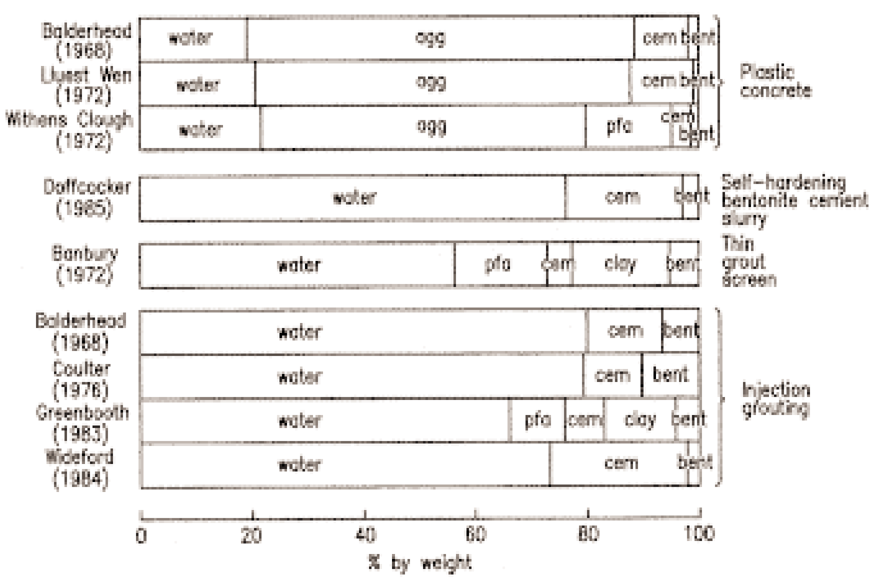


Fig. 4.4.1

Grout and plastic concrete mixes used for repair works on British dams

agg	=	*aggregate*	*cem*	=	*cement*
bent	=	*bentonite*	*pfa*	=	*pulverised fuel ash*

work at the Walshaw Dean Reservoirs in 1911. An authoritative manual on the practical and theoretical aspects of the current state of the art of grouting has been edited by Widmann (1995) and the following is the briefest of summaries of the techniques in use. Suspension grouts of ordinary Portland cement can penetrate gravel and coarse sand while fine cement makes it possible to grout fine-grained materials. Bentonite is often added to cement grout to seal a core of an embankment. A bentonite content of 2 % to 5 % of the cement weight is often required to achieve a stable suspension. In addition to reducing the bleeding : the release of free water as the grout sets, bentonite increases the viscosity and reduces the strength and stiffness of the hardened grout. This is an advantage since the aim is to achieve similar properties to the surrounding undamaged core material. This has proved to be a difficult task (Millmore et al, 1998 and Flemming et al, 1985). Grout and plastic concrete mixes in remedial works performed on some British dams are summarised on Fig. 4.4.1 (Charles, 1989).

Superplasticizer has been used to reduce the viscosity of the grout and to improve its ability to penetrate. Thus superplasticizer has sometimes been used instead of bentonite for grouting of the core in order to improve the penetration of the mix. Chemical grouts have been developed in order to improve grouting of fine grained material. However, concerns over environmental and groundwater pollution have limited the use of chemical grouts. Fine grained cement is often selected as an alternative to chemical grout.

For grouting dam cores the grout mix is often fed by gravity from cased drillholes. The *tube-à-manchettes* technique has also been used to repair the cores of many dams. Here a slotted tube is installed in a cased hole that is backfilled with weak grout before the casing is withdrawn. *Tube-à-manchettes* grouting allows grout to be placed at a selected location. The second advantage of this method is that once the tube is in place, it can be used repeatedly. The weak backfill grout is fractured each time additional grouting is performed. It was first used at the **Bou Hanifia Dam** in Algeria. The method was invented by Ischy in the 1930s and has been described by Glossop (1961). The design of the grout mix is an important part of the rehabilitation work. Ideally the grout should match the parent material in density and shear strength while being satisfactorily penetrative although most grouts are stronger than the parent material in an earth dam. Most importantly the grout should not be erodible. Mixes have been designed using clay, bentonite and cement.

Drilling in a core of a dam is a critical operation and requires attentive specialist supervision. The risk of hydraulic fracture must be considered. Certain embankment locations and conditions pose a higher threat of hydraulic fracture than others. These are :

- Cores with slopes steeper than 0.5 h to 1 v, cut-off trenches, and upstream-inclined cores.
- Close to abutments steeper than 0.5 h to 1 v, where abrupt changes in abutment slope occur, and above boundaries in foundations that sharply separate areas of low and high compressibility.
- Near structures within embankments.
- Cores consisting of mixtures of fine sand and silt.
- Cores with a high phreatic surface.

Improper drilling procedures increase the potential for fracturing. The selection of drilling method should be based on site conditions, availability of equipment and trained personnel. Causes of hydraulic fracture during drilling include : drilling too fast ; placing and removing tools too fast, using drilling mud of the wrong consistency, and the hole squeezing or caving. Drilling methods should include temporary casing advance and in order of preference are as follows :

- Auger
- Reverse circulation percussion or rotary drilling
- Cable tool
- Rotary drilling with clear water
- Rotary drilling using biodegradable mud such as bentonite
- Rotary percussion drilling
- Air foam rotary drilling.

It is essential that drill crews be well trained and aware of the causes of and the problems resulting from hydraulic fracture. The drilling must be stopped immediately when there is an abrupt loss of drilling fluid circulation. Remedial action might include grouting the zone before drilling is continued. Care must be taken to prevent grout penetrating the filters or into drainage holes.

Case Histories of Grouting

Porjus Dam in Sweden is a rockfill dam with a central vertical core of moraine, approximately 22 m high (Johansson et al, 1996 and Johansson, 1994). In 1993 an almost cylindrical sinkhole, with a diameter of some 4 m and approximately 3 m deep, was found at the upstream side of the crest, on the line of the upstream filter of the dam, as shown on Fig. 4.4.2.

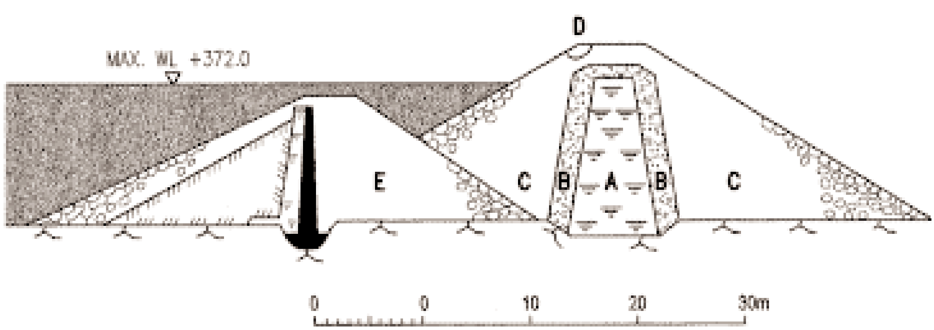


Fig. 4.4.2

Porjus Dam. Embankment section

A. Impervious core
B. Filter
C. Rockfill
D. Surface location of sinkhole, extent not shown
E. Original embankment dam

The plant comprises a power station, constructed in 1914, with a dam. This dam was replaced in 1980 by a new structure just downstream, using the old dam as a cofferdam. The new dam has 3 m thick filters placed on each side of the core. The filter material is a gravely sand with a D_{15} grain size of 1.4 mm, which is coarser than the grain size recommended in current specifications. The field investigation included ground-penetrating radar, drilling with soil sampling, cross-hole radar, temperature and water level measurements. The leakage flow was monitored daily during the investigation period. No turbidity was observed until the repair works began when bentonite was carried into the seepage water.

Analyses showed zones within the core with increased water content and seepage. The damaged part is on the downstream side at a depth of 12 m to 15 m below the crest, and a width of about 12 m along the dam axis. Leakage through the dam core was probably initiated by erosion starting at the boundary between the core and the downstream filter followed by backward erosion (piping) until the erosion path reached the upstream filter and the sinkhole occurred.

The dam was repaired by grouting. To prevent penetration of cement into the filter the grouting commenced with a row of holes in the core close to the downstream filter. Sand (0.4 mm-0.6 mm) was added to a bentonite slurry when these holes were grouted. The portion of the core shown by the investigation to have high water content was then grouted with a cement/bentonite mix. As a third step an attempt was made to inject sand into the filter. The intention was to lower the D_{15} grain size. However, only limited success was achieved.

Evaluation of the repair work confirmed that the grouting considerably reduced the leakage flow. The core areas with high inflow of grout were consistent with the areas with increased seepage. However, although the seepage has now reduced it is still above the original quantity. The seepage rate is measured continuously to detect if internal erosion were to start. Increased seepage will also affect the temperature profiles, which are measured in standpipes. Surveillance will be continued with frequent visual inspections and measurements.

Greenbooth Dam in the UK is a 35 m high, earthfill dam with a puddle clay core, completed in 1961. In March 1983 a shallow depression was noticed in the crest near to the right abutment of the dam. This depression deepened quickly over a few days. One of the first actions was to lower the water level in the reservoir by 9.25 m over an eight day period. Time was too short for a conventional site investigation and use was made of monitoring equipment at the rig. Known as « Enpasol », the equipment recorded parameters of the drilling operation including advance rate, thrust and torque. It identified voids through the puddle clay core and a cavity upstream of the core. Grouting was carried out of the foundation, the shoulders, and then the core by the *tube-à-manchettes* technique using a specially designed grout. The aim was to design a single grout to fulfil four requirements :

- To consolidate the shoulder of the embankment and to fill the cavities under the crest
- To reduce the permeability of the shoulder near the core
- To fill the cavities in the core and later to allow it to be restressed
- As the sleeve grout for the *tube-à-manchettes* process.

It was not possible to match a single mix fully to meet all these requirements. The finally chosen mix was : 1000 litres water, bentonite 50 kg, sulphate-resisting cement 100 kg, fly ash 150 kg, clay 200 kg. The peak shear strength of the mix was about 100 kPa reached within ten days. The strength of the puddle clay in the core was about half this. For use in the shoulder and foundation the cement content was increased to 150 kg which increased the peak shear strength to 450 kPa. In fourteen weeks nearly 1600 m of grout hole were drilled and 440 m^3 of grout was injected. The grout injected was about 4 % of the core volume over the treated area. The dam has performed satisfactorily since the remedial works were undertaken (Flemming et al, 1985).

Sorpe Dam, an earthfill embankment with a vertical concrete core wall was built between 1926 and 1935. At 69 m high it was the highest embankment dam in Germany. The relatively thin core extends from the rock foundation to the crest of the dam. Upstream of the core wall the embankment is made of several zones with material ranging from silty clay close to the wall to coarse stones at the upstream face of the dam. The downstream shoulder consists of pervious rockfill and coarse alluvium. The dam has a double drainage system. Vertical porous concrete pipes are located at a regular spacing in the concrete core wall, while a 1.5 m to 2.5 m thick drain layer just downstream of the concrete wall provides a second line of drainage. In 1951 a substantial increase in leakage was noticed in the inspection gallery at the base of the core wall. The seepage water was muddy and it was feared that the silty clay was eroding, perhaps initiated by cracks in the concrete wall. The water level in the reservoir was lowered and cement grout was injected upstream of the core wall. These measures were not sufficient to bring erosion to an end. The upstream face of the dam had settled locally up to 1.4 m.

Systematic grouting of the concrete core wall and of the silty clay was carried out in 1958/59. The primary drainage system was lost as the drain pipes in the core wall were filled with grout. Replacement drain holes were drilled in the core wall and in the drain downstream of the wall. However these measures were not effective

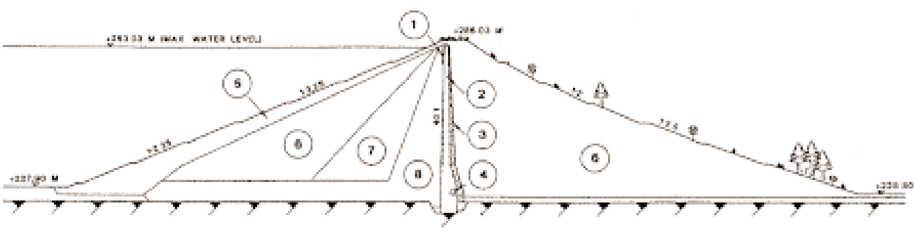


Fig. 4.4.3

Sorpe Dam. Cross section

1. Jet grout curtain
2. Concrete core
3. Drainage layer
4. Inspection and drainage gallery
5. Riprap
6. Rockfill
7. Filter zone
8. Clay

and wet patches and leakage continued to develop in the inspection gallery. Seepage increased sharply when the reservoir level approached the last 5 m below the maximum storage level. Investigation by drillholes in 1992 revealed that the upper part of the concrete core wall was extensively cracked.

The selected solution was to seal the core by installing a jet grouted cut-off upstream of the concrete wall and extending some 10 m below the dam crest as shown on Fig. 4.4.3. The procedure was to cut cylindrical columns in the silty clay using a 400 bar water jet and by injecting in a second phase a cement suspension consisting of one part of water to one part of cement and inert fines (rock flour). The columns were initially installed at 240 mm spacing. This was reduced by a second series of columns and the final spacing was 600 mm.

The rehabilitation work was carried out in 1996-97. The continuous jet grouting wall is 600 m long with a developed area of 6000 m^2. Inclined checkholes indicated that the jetted columns intersected each other and that they were closely bound to the concrete wall. As shown in Fig. 4.4.4 seepage at high water levels in the reservoir has been almost completely eliminated and the treatment can be considered as successful (Heitefuss et al, 1998).

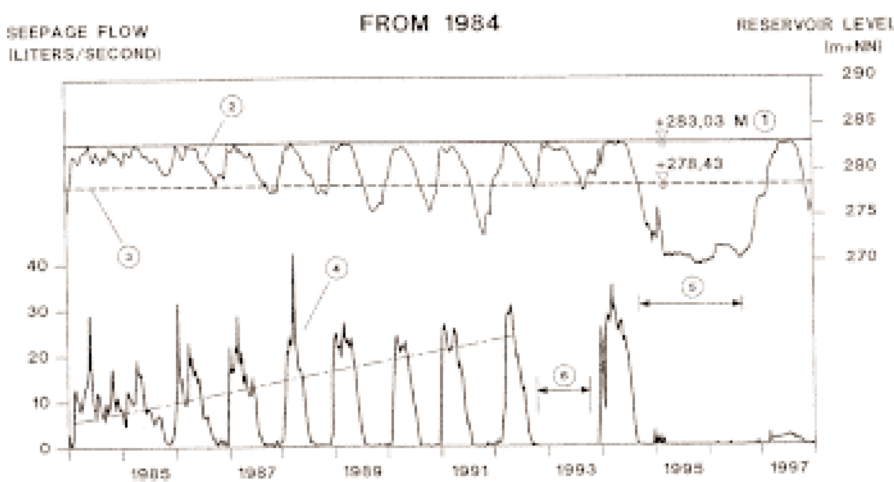


Fig. 4.4.4

Sorpe Dam. Seepage in the inspection gallery

1. Maximum reservoir level	*4. Seepage flow*
2. Reservoir level	*5. Drawdown*
3. Critical reservoir level	*6. No data*

Other examples of rehabilitation of the embankment by grouting include :

Nature of Rehabilitation	Location	Dam	Reference
Grouting with cement-bentonite suspension	Core	Angae, Korea	Kim et al, 1997
Tube-à-manchettes grouting	Core	Sylvenstein, Germany	List et al, 1991
Grouting, diaphragm wall and drainage	Embankment, foundation and abutment	Saint-Pardoux, France	Lino et al, 1994
Grouting with cement-bentonite slurry in the core and cement in the foundation	Concrete core wall and foundation	Sösetalsperre, Germany	Doring, 1986 Schmidt, 1987
Compaction grouting with 20 % pea gravel, 50 % sand, 30 % silt. Improvement reported	Core	WAC Bennett, 83 m high, USA	IWP&DC, 1997

4.4.2.2 Diaphragm Walls

Diaphragm walls using slurry trench techniques have been used to replace leaking cores. Further discussion will be found in section 4.3 in connection with rehabilitating the foundation. Slurry walls have the advantage over grouting of greater certainty that all the defects that are present in the core have been sealed. Two methods have been used :

Single stage method. The technique involves excavation of a trench, typically 600 mm wide under the support of a self-hardening cement bentonite slurry which is left to set to form a low permeability seal. The use of this method in the UK for repair of core has been restricted to small dams with a maximum height of 15 m. Trenches have usually been dug with a hydraulic backhoe type excavator, but rope suspended grabs are also used. Additives can be used to regulate the gelling time. By varying the cement/water ratio, and the bentonite/water ratio the stress-strain properties of the hardened mix can be adjusted to match the surrounding core material. The wall should be impermeable and have stress-strain characteristics similar to the surrounding core. The mix used varies within a wide range. The slurry has a cement/water ratio of between 0.1 to 0.3 and a bentonite content between 30 kg/m^3 and 50 kg/m^3. Fly ash may also be added to replace some cement. Gravel and sand aggregates have been added to make a plastic concrete. This single stage method is considered to be more economical and has proved its practicality and effectiveness in many projects.

Two-stage method. Until recently most diaphragm walls have been constructed using the two stage method. The technique is described in some detail by Xanthakos (1979). In the first stage the excavation is supported by bentonite slurry, which in the second stage is replaced by plastic concrete.

The two stage method is stated to have an advantage over the single stage method because of the stronger and probably more erosion resistant material forming the cut-off. But the self-hardening slurry used in the single stage method forms a wall without joints and the material is plastic and follows the possible deformation of the core. For relatively shallow walls, less than approximately 12 m, backhoes with extended arms are usually the most economic excavation equipment. At greater depths kelly-mounted hydraulic grabs can be used. In the USA walls greater than 100 m in depth have been constructed.

Case Histories of Diaphragm Walls

Balderhead Dam, UK, a 48 m high embankment with a crest length of 925 m. It has a rolled boulder clay core, relatively stiff shale fill shoulders and a concrete cut-off. The top 10.8 m of the clay core has vertical sides. Immediately downstream of the core is a crushed limestone filter which connects with the ground drainage blanket. The filter and the drainage blanket were designed according to standard filter rules.

On first impounding in 1966, just before the reservoir was full, the main underdrain flow increased. Subsequently, localised settlements occurred along the crest and in 1967 two sinkholes formed in the crest. The reservoir was immediately drawn down by 9.2 m and the underdrain flow returned to its previous level. It was established that the main underdrain flow had turned cloudy about a month before the first sinkhole appeared. After drawdown the water became clear.

Exploratory boreholes revealed erosion within the core at several locations; the boulder clay material had become segregated and the finer particles lost by water erosion. The damage was associated with cracking which had been initiated by hydraulic fracture of the core under almost full reservoir pressure (Vaughan et al, 1970). Low stresses in the core were caused by arching across the clay core between the shoulders and possibly by longitudinal strain due to differential settlement across foundation discontinuities. It was also postulated that once the cracks had formed they were kept open by the water pressure and under the low flow conditions the coarser eroded material had segregated in the cracks. On drawdown the seepage paths closed up due to the decrease in water pressure.

Over the central 200 m of the dam, covering the zones of worst damage, the core was repaired by constructing a 600 mm wide diaphragm wall down to the concrete cut-off. The depth of the trench varied from 31 m to 46 m and the panels were 6 m long. The work was carried out with the reservoir level lowered by 10 m. The plastic concrete mix was designed to have a strength and stiffness similar to that of the rolled boulder clay core and be resistant to erosion. The mix proportions adopted are given in the Table below.

	Proportion as % of weight
Ordinary Portland Cement (OPC)	9.7
Bentonite	2.0
Aggregate	69.0
Water	19.3

The permeability of the concrete was in the range 1×10^{-8} m/s to 5×10^{-10} m/s. As well as serving as an additional water barrier, the diaphragm wall was intended to prevent migration of eroded material through the core. The trench was constructed within a tolerance of 0.25 % of vertical. The overall area of the diaphragm wall was 8240 m^2.

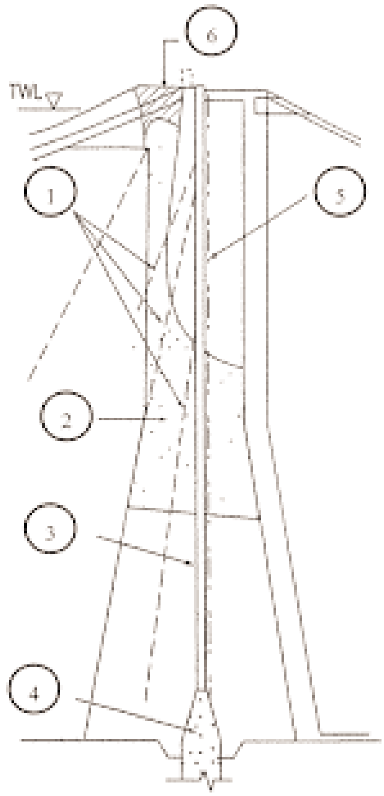


Fig. 4.4.5

Balderhead Dam. Internal erosion and treatment by plastic concrete diaphragm wall

1. Raking grout holes
2. Zone of erosion damage
3. Diaphragm wall
4. Concrete cut-off
5. Grout holes with tube-à-manchettes
6. Sink hole

Other lengths of the core where water losses were observed during investigation drilling were grouted using *tubes-à-manchettes* in order to increase the total earth pressures within the core and thereby resist hydraulic fracture. The grout tubes, with sleeves at 300 mm centres, were installed at 3 m centres. The core was grouted from the bottom upwards, injections being carried out from alternate sleeves and limited to 0.42 m^3 per injection. The grout consisted of 50 % cement and 50 % bentonite with a water/solids ratio of 4. The grout was designed, when set, to be slightly less erodible than the boulder clay core material and to have stress-strain characteristics approximating to those of the boulder clay. Its permeability ranged from $1x10^{-8}$ m/s to $5x10^{-10}$ m/s, the permeability of the intact core being about $1x10^{-10}$ m/s. During the works 5400 m of *tubes-à-manchettes* were installed and 4630 m^3 of bentonite/cement grout were injected. Since rehabilitation the dam has performed satisfactorily for 30 years.

Wolf Creek Dam, USA, is a combination concrete gravity and earthfill embankment structure with a maximum height of 61 m (Fetzer, 1979). The dam was completed in 1951. The homogeneous sandy clay embankment was founded directly on the overburden of sandy clay and sandy silt about 16 m thick. The overburden is underlain by highly solutioned limestone. The embankment design included a cut-off trench near the upstream toe that extended to top of bedrock as shown on Fig. 4.4.6.

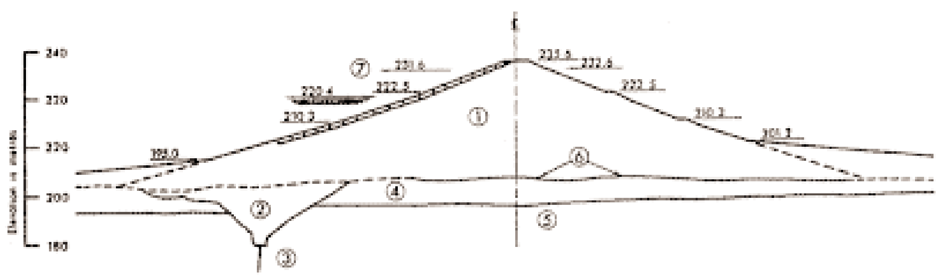


Fig. 4.4.6

Wolf Creek Dam. Cross section

1. Impervious rolled fill
2. Cut-off trench
3. Grout curtain
4. Valley alluvium
5. Rock
6. Drainage blanket
7. Maximum water lever

During construction the cut-off trench was extended to the bottom of a wide solution channel which had a maximum depth of about 16 m below general top of rock. The fill for the cut-off trench was placed directly against the cavity filling of intersecting caves. The project was operated without apparent problem for more than 15 years until a flow of muddy water into the tailrace was observed in late 1967. After a sinkhole developed at the downstream toe of the embankment in March 1968 extensive investigations were undertaken that indicated that the reservoir water was passing under the embankment through the solution channels. Emergency grouting was carried out that stopped the excessive leakage and reduced the piezometric head under the downstream section of the embankment.

Some 300 piezometers were installed to monitor the performance of the dam and its foundation. However studies indicated that piezometers could not be relied upon to forewarn of a pending piping problem and that a permanent blockage of solution channels beneath the embankment was needed to ensure the long-term safety of the structure. A concrete cut-off wall was the most feasible and least costly method of forming a non-erodible permanent blockage of the solution channels.

The wall extends from the end of the concrete section of the dam for 685 m along the upstream edge of the embankment crest. The depth of the wall was determined by a row of close spaced boreholes which were backfilled with a grout mixture of cement, sand and clay. The base of the wall was a minimum of 3 m below the lowest indication of solution activity. The maximum height of the wall was thus 85 m as shown on Fig. 4.4.7.

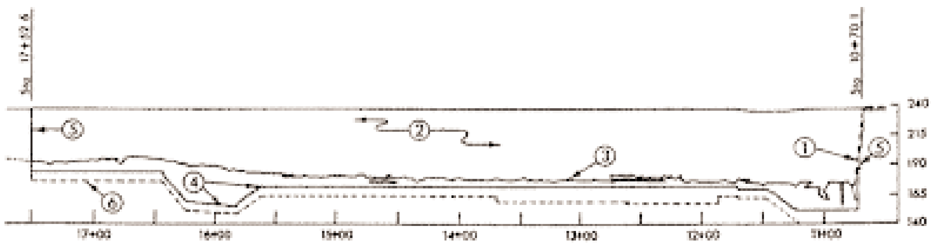


Fig. 4.4.7

Wolf Creek Dam. Profile along embankment diaphragm wall

1. Face of concrete structure
2. Embankment
3. Top of rock
4. Bottom of concrete wall
5. Limit of concrete diaphragm
6. Bottom of grout holes

A separate wall 180 m in length and extending to a maximum depth of 30 m was installed around the switchyard located at the downstream toe of the embankment section to provide protection against tailwater fluctuations. The basic scheme was to construct 660 mm diameter steel cased primary elements at 1.37 m centers and to connect them with bi-concave shaped secondary concrete elements constructed so as to provide a continuous wall with a minimum width of 610 mm. The scheme is shown in Fig. 4.4.8.

Work on primary elements started with excavating a 1.30 m diameter hole to a depth of about 21 m with a clamshell bucket. This work was performed in the dry above the saturation line in the embankment. The hole was then filled with a thin bentonite slurry and 1.20 m diameter temporary steel casing set in the hole. Excavation continued inside the casing as it was sunk to a depth of about 43 m. At

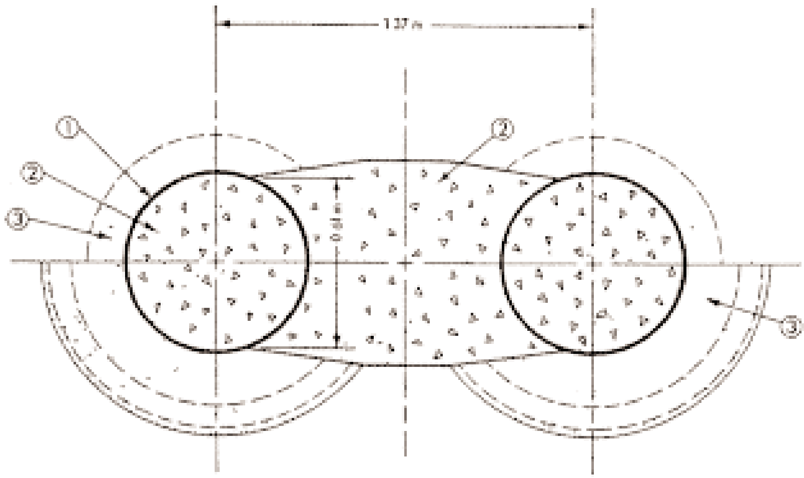


Fig. 4.4.8

Wolf Creek Dam. Plan of primary and secondary wall elements

1. Primary element permanent casing
2. Tremie concrete
3. Weak grout

this depth a 1.05 m diameter casing was inserted into the hole and the smaller casing was then sunk to rock. Verticality of the casing was checked every 9 m with a special plumb bob to determine if the alignment was within the 150 mm tolerance.

A 900 mm diameter hole was drilled into rock with a roller bit. A thin bentonite mud and an airlift were used to remove the cuttings. A 7.5 m deep core hole was drilled at the base of each element to check for cavities. After the core hole was pressure tested and grouted, the 660 mm diameter permanent steel casing was lowered to the bottom of the hole. The casing was sealed at the bottom with a plate and was weighted down as the hole was still filled with bentonite mud. After positioning the permanent casing the surrounding void was backfilled with a weak cement grout as the temporary casings were pulled. Once the grout had set the permanent casing was then filled with tremie concrete.

Excavation for the secondary elements could not start before the concrete in the successive primary elements had set for 20 days. Excavation to top of rock between the primaries was accomplished with a special clamshell having flared wings that fitted the outsides of the two flanking primaries. Excavation in rock was accomplished with a star chisel, an expandable bi-concave chisel and a clam bucket. The excavation was kept filled with a bentonite slurry at all times until it was displaced with the tremie concrete.

The total cost of the rehabilitation amounted to US$ 96.5 million or, for a total area of the wall of 59 500 m^2, US$ 1620/m^2 at 1978 prices.

Other examples of rehabilitation of the watertight element by diaphragm wall include :

Nature of Rehabilitation	Location	Dam	Reference
Single-phase bentonite-cement slurry trench wall	Upper section of core	Luxhay, Somerset, UK	Millmore et al, 1998
Diaphragm wall two-stage method and *tube-à-manchettes* grouting	Puddle clay core	Lluest Wen, Glamorgan, UK	Twort, 1977
Early attempts to reduce the seepage and internal erosion augmented by a secant pile diaphragm wall 35 m deep, 140 m long of clay concrete	Dam body and foundation	Kureiskaya, 30 m high, 300 m long, ER, Russia	Bellendir, 1999
Diaphragm wall two-stage method and *tube-à-manchettes* grouting	Puddle clay core and shoulders of boulder clay	Withens Clough, Yorkshire, UK	Arah, 1975
Excessive seepage and internal erosion overcome by a combination of cement grouting, jet grouting and slurry trench	Foundation	Paso de las Predas, 31 m high, 1706 m long, ER, Argentina	Bustinza et al, 1999
Bentonite slurry trench	Embankment	Little Flint Creek, USA	USCOLD, 1996
Concrete cut-off wall	Core	Mud Mountain, USA	USCOLD, 1996
Plastic concrete slurry wall, combined with other rehabilitation measures	Throughout embankment and foundation	Wister, USA	Erwin et al, 1991

4.4.2.3 Sheet Piling

Sheet piling has been used in the rehabilitation of embankment dams to control leakage in the core. Although the method is cheap and simple, using well established techniques, it has limited applicability. Generally sheet piling has been used to solve leakage problems near the top of embankment with a maximum depth of 5 m.

However, Lemelin et al (1996) has described the installation of sheet piles and jet grouting to rehabilitate a decomposing embankment. **Dyke 3 at Lake Manouane** was originally constructed as a rockfilled timber crib dyke some 10 m high. In 1961 the dyke was raised 3 m and converted into a zoned rockfill embankment featuring an upstream impervious shell, a 1200 mm thick vertical sand filter placed between the impervious earthfill and the original plank facing of the cribs, and a downstream shell consisting of the original timber cribs with additional coarse fill.

Design studies focused on the use of sheet piles driven into the filter to replace the decomposing timber facing. Among the advantages are the structural flexibility of sheet piles and their relative insensitivity to potential long term deformation of the cribs and the short period required for installation. Stiff piles were selected to minimise damage during installation and to give an acceptable service life. A 315 m long section was constructed using 250 pairs of 6 m to 15 m long piles. Particular care was taken to ensure that the piles were vertical and the entire 3100 m^2 of sheet piling was installed in 20 working days. The triangular windows between the base of each pile and the granitic bedrock were sealed using jet grouting at a pressure of about 35 MPa.

Some references to rehabilitation including sheet piling include :

Nature of Rehabilitation	Location	Dam	Reference
Cutoff by corrugated sheet piles driven into the clay fill, of limited effect.	Clay fill	Lockwood, Middlesex, UK	Ray et al, 1982
Sheet pile cut-off through closure section of embankment, together with cement and mud grouting, limited effect. Further rehabilitation required 40 years later.	Earthfill embankment	Wister, USA	Erwin et al, 1991

4.4.2.4 Reconstruction and Replacement of Core Section

Reconstruction of the dam or replacement of the core section may be found to be the most economic and reliable solution after consideration of the alternatives. The opportunity is usually taken to modify the design, for example by widening of the core and using more conservatively designed filters to achieve a higher safety against internal erosion. Replacement of the core in open trench by cement-bentonite mix has been also used in some occasions, especially in the UK (Charles et al, 1996).

A relevant example is **Grundsjön**, which consists of three earthfill dams, of which the highest is 44 m. The structure was completed in 1972 and is located on the Ljusnan River in Sweden. The damage occurred in a 13 m high section. The core of moraine is central, vertical and is surrounded by filter and earthfill. The core is founded on rock and the shoulders on moraine. In 1990 a sinkhole, 400 mm by 500 mm and 1.6 m deep formed in the upstream part of the crest above the zone of the upstream filter. Exploratory drilling through the dam and measurements showed that the core was damaged and it was decided to reconstruct part of the dam (Eurenius et al, 1991). The damaged core is shown on Fig. 4.4.9.

Grouting of the core was considered, but, since it was possible to take advantage of a period of low reservoir level, economics favoured reconstruction. The core was excavated out and replaced over a length of 5 m on each side of the sinkhole. During the reconstruction an erosion path of 0.5 m^2 through the core was discovered. No fines were found in the erosion path, suggesting that the sinkhole was a consequence of internal erosion. The erosion probably started in the downstream face of the core, progressed upstream towards the upstream filter and finally reached the crest of the dam. A drainage system has now been installed downstream of the dam. The instrumentation and monitoring programs have been extended.

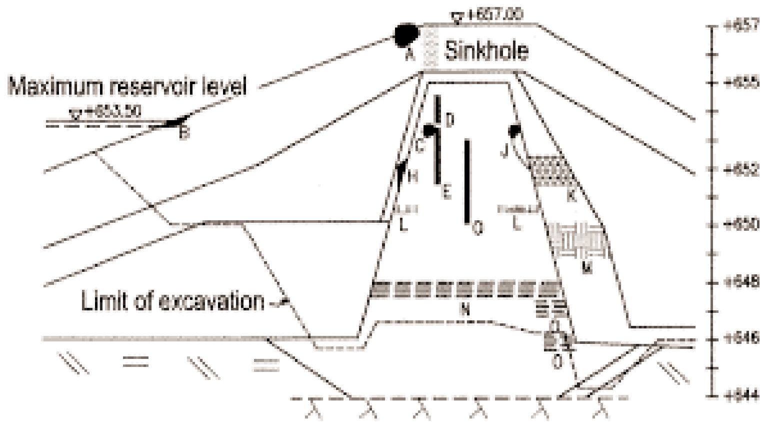


Fig. 4.4.9

Grundsjön Dam. Cross section of damaged core

A. Large stones
B. Washed out fine materials
C. Hard clods of soil
D. Loose material detected by sounding
E. Damaged core detected by infiltration test
H. Core material in the filter
J. Cavity
K. Wet filter
L. Small cavities
M. Filter without fines
N. Pervious material with washed out fines
O. Thin wet layer of large extent

Other examples where the most economic alternative for the rehabilitation of the core was found to be replacement include :

Nature of Rehabilitation	**Location**	**Dam**	**Reference**
Replacement of core with cement-bentonite slurry in open trench	Upper part of core	Little Denny, Scotland UK	Charles et al, 1996
Excavation and replacement of core with puddle clay	Puddle clay core	Castle Howard Yorkshire, UK	Kennard, 1972

4.4.2.5 New Filter and Drain Provisions

As revealed in the case histories summarised here, the chain of events leading to the failure of an embankment dam owing to internal erosion often includes increasing leakage flow, together with local erosion at the downstream toe resulting in unravelling of the slope. Where incidents such as sinkholes and sudden leakages have occurred, weak zones are often found in the dam body itself. These may originate from poor design or construction. An example of a potentially weak zone is a downstream filter that is not in compliance with current standards. Even if leakage has ceased for the time being, future leakage can not be ruled out.

It may not be practicable to repair or replace the filter downstream of the core, which is often inaccessible. A toe drain is sometimes considered as an alternative. The toe drain may be designed to carry a calculated design leakage and, by selecting an appropriate grading, to control the erosion caused by the leakage through the dam. The toe drain can also be incorporated within a berm designed to improve the stability of the embankment. The berm can be designed to be stable under the seepage calculated on the assumption that it is controlled by the hydraulic conductivity of the shell and filter material.

The **Långbjörn Dam** in Sweden is an example of rehabilitation in which new filters and drains are important features of the design. This is a 31.5 m high and 590 m long embankment. The rehabilitation of the abutments is described in section 4.3.

Other examples of rehabilitation in which new filters and drain are provided include :

Nature of Rehabilitation	Location	Dam	Reference
Installation of filtered drains at earthfill-abutment contact	Abutments and dam toe	Soldier Creek, USA	USCOLD, 1996
Drainage and berms at downstream shoulder	Downstream embankment slope	Mitchell's House Lancashire, UK	Rofe et al, 1990
Pressure relief wells and berm at D/S toe	Downstream embankment toe	Reva Yorkshire, UK	Charles et al, 1996

4.4.3 Slope Stability Improvement

Two principal measures are used to improve the stability of embankment slopes. First is the provision of weight berms or blocks, usually involving the placement of fill to flatten the slope or weight at the toe of the embankment. Second is to improve the drainage to reduce the pore pressure in the downstream fill.

Giffaumont Dam, France, also called the **Marne Dam**, is the largest of a series of eight embankment dams with a total length of 18 km built in the late 1960s to regulate the Seine River. The dam has a maximum height of 18 m and a length of 3000 m. The homogeneous embankment with internal filter was constructed from the locally available material, a silty clay. The slopes of the dam vary from the top (1V :2.5H) to the base (1V :4H). The downstream slope is covered with grass. The upper part of the upstream face is protected by a 60 mm thick pervious bituminous layer on a 300 mm sub-base of sand and gravel, while the lower part is covered with a 600 mm thick rockfill layer.

The high natural water content of the silt and clay caused difficulties during construction. Local slides occurred above water level during operation, usually after rain. Investigations showed that the condition of the material varied from the surface towards the interior of the dam. The plastic fill material has a plasticity index of between 30 % and 40 %. Its moisture content is 30 % to 35 % at the surface against 25 % to 30 % 3 m below the surface. This moisture content is always higher than the plastic limit and the water content at compaction.

In 1980/81 an increasing number of slides on the downstream slope coincided with crest deformations and increase of the pore water pressure in the downstream shell. The problem appeared to be caused by surface water infiltrating and increasing the pore pressure in the upper part of the embankment. The plastic clays forming the embankment have dilating properties, they are sensitive to cycles of thawing-freezing and wetting-drying which tend to create cracks in the dam body. On the upstream face, covered by a bituminous layer, bulging of the lower section was observed at several locations. In 1984 a 40 m long slide occurred after a long rainy period. Additional investigations indicated that the bituminous concrete was intact but that the underlying material had a weak consistency (Bister et al, 1994).

Corrective measures are shown on Fig. 4.4.10. They consist of :

- installing a minimum thickness of 2 m of stabilising fill on the downstream slope. This was a granular non-cohesive and free draining material and was placed at a constant slope of 1V :3H.
- placing a 1 m thick rockfill layer directly on the upstream slope .

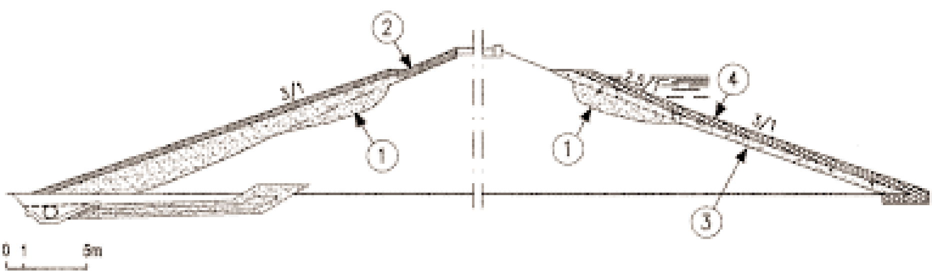


Fig. 4.4.10

Giffaumont Dam. Cross section of the rehabilitated embankment

1. Sand and gravel — *3. Small rock fragments*
2. Loam — *4. Rockfill*

Ajaure Dam in Sweden is a 46 m high rockfill dam with a central impervious core of glacial till supported by filter zones on the upstream and downstream side. The shells are made of rock from the excavation for the appurtenant structures and the powerhouse. The central high part of the dam is founded on rock, while the abutments are founded on glacial till. The crest has been subsequently raised by 1 m leading to slopes of 1V :1.35H to 1.40H in the upper 12 m of the dam. Below this level the downstream slope is inclined at 1V :1.8H. The dam, shown on Fig. 4.4.11, was completed in 1966 (Nilsson et al, 1991).

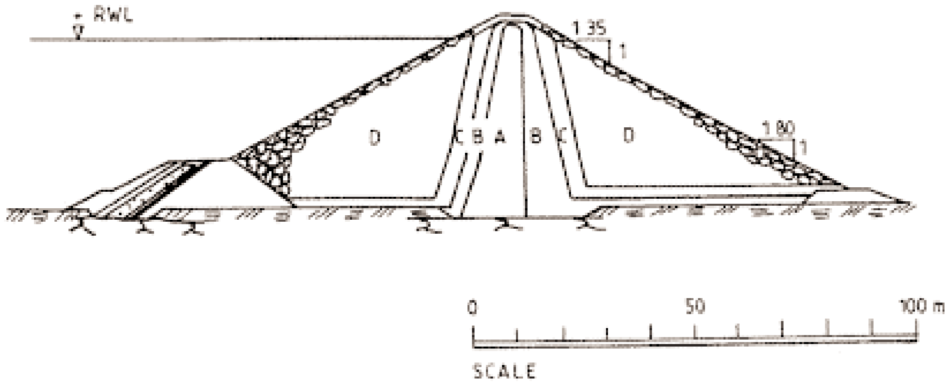


Fig. 4.4.11

Ajaure Dam. Typical cross section

A. Impervious core *C. Coarse filter*
B. Fine filter *D. Rockfill*

About 20 years after construction it was noted during an engineering inspection that the records from an observation point along the crest of the dam showed continuing horizontal movements with no decreasing trend. Observations were intensified. An analysis of the readings indicated that while the rate of vertical settlement of the crest had decreased to about 1 mm to 2 mm per year, 10 years after impounding the horizontal displacements were continuing at a rate of 7 mm/year downstream. The readings showed also that the reservoir level significantly influenced the horizontal deformations.

Field investigations were initiated at this stage in order to provide basic data to analyse the reasons for deformations, and for the design of possible rehabilitation measures. Test pits were excavated at the downstream toe of the dam and along the crest. The rockfill comprises schists and gneiss with a high content of mica. The shear strength of the minus 60 mm material was determined in a large shear apparatus. Crushing occurred to a large extent during testing. The results of these tests established the weakness of the material at high shear stress, as well as the exceptionally low shear strength of the material compared with other published data.

Stability analyses performed with the results of the shear strength tests indicated a factor of safety of 1.17 for the most critical surface. This compares with 1.27 assumed during the design. The shear stresses in situ were therefore close to the shear strength. It was concluded that this situation, combined with the cyclic loading from the reservoir, resulted in the progressive crushing of the rockfill and that crushing was causing the horizontal deformations.

Remedial measures comprised incorporating a 13 m wide berm at about dam mid-height on the downstream side. The outer slope has an inclination of 1V : 2H.

The minimum factor of safety was increased by about 10 percent as shown on Fig. 4.4.12.

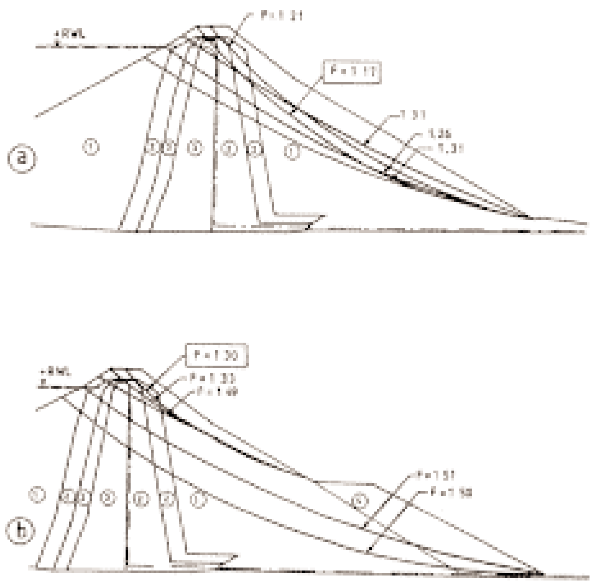


Fig. 4.4.12

Ajaure Dam. Influence of d/s berm on factor of safety

a. Original cross section
b. Cross section with stabilising berm
1. Rockfill
2. Filter, friction angle 35°
3. Glacial till, friction angle 30°
4. Stabilising berm

4.4.4 Rehabilitation of Surface Protection

4.4.4.1 Upstream Protection

On the upstream slope the damage may be associated with the action of waves, fracturing of the stones making up the riprap, or freeze-thaw phenomena. The last of these has been described by ICOLD in Bulletin 93 for **Sveg Dam** in Sweden (Cederwall et al, 1991). The damage can occur abruptly, leading to a loss of function of the protection and a consequent loss of freeboard (Carlyle, 1988). Alternatively the damage can be progressive, allowing repair to occur without loss of function. Erosion of riprap on the upstream face is a potentially serious condition and if left untreated can lead to the destruction of the dam under the effect of waves. Surface erosion is straightforward to detect visually during routine surveillance.

Photogrammetry has been found to be a useful tool for monitoring localised movements on embankments (Hopkins et al, 1990). The options that appear to be considered most frequently in the repair of riprap slopes include :

- Strengthening the existing protection by reconstruction into larger panels by concreting
- Increasing the size and thickness of the riprap

The use of asphalt in various forms has found application on a number of dams for upstream slope repair of stone pitching (Smith, 1998). The longevity of these methods still has to be assessed.

Megget Dam, UK, is a gravel fill embankment 570 m long and 56 m high with a central asphalt core. Damage to the riprap protecting the upstream face has occurred at an unacceptable frequency since first filling the reservoir in 1983 (Gallacher et al, 1998). The long fetch generates waves in the reservoir that have been shown to exceed the design wave height. The alternatives considered included upgrading the existing rip rap, grouting the existing riprap, flattening the upstream slope, provision of blockwork protection and a floating breakwater. A comparison of these alternatives showed that bituminous grouting of the rip rap in a grid pattern was the preferred option. Extensive testing proved the effectiveness of the method.

Pattern grouting of the existing face was adopted as shown on Fig. 4.4.13, obtaining the benefits of void filling while allowing effective drainage. A nominal 2.5 m grid of grouted strips was used, where the grouted material represented about 64 % of the total surface area. The full depth of the 1200 mm thick rip-rap was grouted and additional grouting carried out within the grid of vertical and horizontal strips, to ensure that all significant stones were secured. The strips were grouted in "stages" to achieve uniform void filling throughout the layer and to control the spread of grout, such that adequate drainage areas remain.

The bituminous grout is composed of a basic mastic asphalt with bitumen of 100 Pen, limestone filler and sand to which 10 mm aggregate is added to form the grout mix. The mastic asphalt was designed such that the viscosity at 140°C would lie between 800 poise and 1200 poise. Three grout mixes were used. Mix 1 was a thin mix for the vertical strips, mix 2 was a thicker mix for the horizontal strips and for spot grouting. Mix 3 was a very thin mix with no aggregate content and was used for the first stage of grouting in areas where the quantity of small stones within the rip-rap reduced penetration. The mixes were as shown below.

Grout composition	Mix 1	Mix 2	Mix 3
Bitumen	14.8 %	14.3 %	21.5 %
Filler	16.3 %	16.4 %	19.6 %
Sand	48.9 %	49.3 %	58.9 %
10 mm aggregate	20.0 %	20.0 %	0 %

The vertical strips were grouted in three passes, and the horizontal ones in two. Penetration of the full 1.2 m rip-rap thickness was always achieved. The average grout application rates were 680 kg/m^2 for the vertical strips, 340 kg/m^2 for the horizontal strips and 50 kg/m^2 for the spot grouting. Grout performance tests were performed in addition to standard material production tests. They included small inclined plate tests, large box stability tests, adherence test and in-situ grout penetration, void filling and spread tests.

A major band of thick silt deposits was exposed on the upstream face as the reservoir was drawndown. This would prevent satisfactory grout penetration and had to be removed before grouting could proceed. This was achieved by jetwashing using a steel manifold with six 25 mm nozzles positioned over the dam face by crane, and fed by a submersible pump sited within the reservoir.

Fig. 4.4.13

Megget Dam. Placing bituminous grout

The grouting works were completed in October 1998 and were achieved at a cost of approximately US$ 90/m^2.

The stone pitching protection on the upstream face of **Carron Dam** embankment in the UK has been repeatedly damaged and maintenance was becoming necessary on an annual basis (Hay-Smith, 1999). Following major damage in the winter of 1996/97, a permanent solution was sought. The existing pitching, laid on a 1V :3H slope, is 300 mm thick, bedded on 150 mm filter layer. The typical stone size is approximately 300 mm wide by 500 mm long. The design solution was to reuse the existing pitching blocks, but to increase the effective block size by bedding a number of stones together in concrete to form panels.

The design minimum effective size of stone required was 0.49 m^2 and preferably greater than 0.79 m^2 for the existing bedding conditions. The existing pitching protection was inadequate to withstand extreme events. The effective size of the stones was improved by bedding several stones together in a concrete panel. The panels were 450 mm deep, bedded on 100 mm thick broken stone drainage layer and a layer of geotextile filter fabric. The panels were finished by grouting the stones using a sand/cement high strength grout. Gaps between the panels were maintained to allow drainage. This was achieved by using a 10 mm pre-formed drainage membrane that acted as a joint former. Approximately 1000 m^2 of the upstream slope protection were upgraded in this way, at a cost of around US$ 175/ m^2. Construction was completed in December 1997.

4.4.4.2 Crest, including the Wavewalls

On the crest and on the downstream slope the erosion may be the result of spray over the dam from breaking waves, overtopping or wave slop over the crest, or heavy rainfall.

The aim of the rehabilitation on the crest and on the downstream face will be to increase the resistance of the surface to the flow of water. Options that appear to have been considered for repairs of these surfaces include :

- cast in place reinforced concrete
- roller compacted concrete
- soil cement
- articulated concrete block revetment systems.

The work of CIRIA in the UK has been helpful in setting design criteria for these. See for example Hewlett et al (1985) and Hewlett et al (1987). Case histories include :

Nature of Rehabilitation	Location	Dam	Reference
Strengthening panels with concrete	Upstream face	Kielder, UK	Carlyle, 1988
Increase rip rap size and thickness	Upstream face	Foremark, UK	Carlyle, 1988
Asphalt upstream face replaced by cast concrete face over the upper part	Upstream face	Willes Meadow, UK	Axton, 1996
Protect lowered crest with RCC, additional fill on downstream face and flatter slope	Crest Downstream face	South Prong, USA	USCOLD, 1996
813 mm to 914 mm rip rap placed on 1 on 4 slope	Downstream face	Nameless Valley Ranch, USA	USCOLD, 1996

4.4.5 Loss of Bond between Concrete Structure and Embankment

Deterioration of embankments adjacent to concrete structures may be associated with differential movements in the contact zone. A consequence of this is increased seepage rates at the interface between the two structures, and internal erosion. Displacements in the transition zone between the embankment and the concrete structure may be the result of settlement of the embankment material due to poor compaction, or settlement of the foundation due to inadequate treatment. Such settlements often result in arching in the fill causing a reduction of the effective stress, and hydraulic fracture. This may lead to the development of preferential seepage paths in the form of cracks or more permeable layers in the fill or along the face of the concrete structure, promoting internal erosion and failure if the filter-drainage systems are insufficient. Swedish experience of internal erosion in embankment dams (Nilsson, 1998) describes 16 cases of sinkholes in the embankment close to the contact with concrete structures.

The deterioration can usually be detected by means of visual observation, geotechnical investigations such as sounding and sampling, settlement readings and measurements of increasing seepage rates. Remedial measures that have been adopted are listed below :

- Reconstruction of the embankment with a new design to avoid arching. Recommendations for construction of dams adjacent to concrete structures include (a) wider excavation in foundation (b) wider impervious core in combination with adequate compaction and moisture control, (c) slightly sloping concrete structure at the transition surface to the fill to promote more effective contact as a result of compaction of consecutive layers, (d) exclusion or reduction of length of sheet pile wall, (e) use of coating of concrete surface and sheet pile wall with asphalt, (f) wider rather than narrower filters, (g) conservative drainage provisions capable of discharging sudden leakage.

- Reduction of seepage by grouting of foundation or the embankment
- Construction of a downstream berm for stabilisation against sliding and improvement of drainage capacity

Increased monitoring is recommended once the deterioration process has been noticed and assessed, in order better to establish when rehabilitation measures should be undertaken.

The 32 m high **Stenkullafors Dam** has a cross-sectional design similar to that of many Swedish embankment dams. It has a central, thin core of glacial till and alluvium filters of sandy gravel upstream and downstream of the core. The left and right flanks of the embankment are divided by the concrete intake and spillway structure, as shown in Fig. 4.4.14. Adjacent to the concrete structure, on both the left and right side of the concrete structure, there is steel-sheet piling in the centre of the core with a length of 10 m in plan. The piling is founded on bedrock and was erected prior to the construction of the dam.

In 1987, some four years after impoundment, a sinkhole was observed in the left-hand crest of the dam, adjacent to the concrete structure on the upstream side of the piling. It had an area of approximately 2 m^2 and a depth of 200 mm. Field investigations comprised penetration testing, tube sampling and installation of standpipes. During the investigation, a similar sinkhole appeared on the right side of the dam, and consequently the investigation was extended to include this area.

The investigations revealed very soft material in the core adjacent to the concrete structure, and also that the hydrostatic head in the core close to the downstream filter was higher than would be expected. In addition to these soft areas, very soft layers of material 200 mm to 100 mm thick were found at different depths in the core also beyond the end of the sheet piling. At the time of the investigation there was no signs of leakage, but sampling showed that fine material had been washed away, probably due to previous internal erosion and leakage. However, from excavations in the core it was concluded that the sinkholes, which initiated the investigations, were in all probability caused by inadequate filter between the riprap and the core material at the crest of the dam, and not by internal erosion.

It was concluded that leakage could possibly occur in the future and that further erosion of core material could follow. In order to give a satisfactory factor of safety against slides in the downstream slope, and to increase the erosion resistance in case of excessive leakage, a stabilising rockfill berm was placed along the downstream toe in 1990. Additionally, measuring weirs of the Thompson type giving continuous readings, were constructed downstream of the left and right side of the dam.

In 1998 a safety evaluation of the **Stenkullafors Dam** was carried out, which found no sign of further internal erosion. Furthermore, the seepage through the dam was stable. Today, there are no plans for additional rehabilitation of the weaknesses in the core discussed above. However, this may be reconsidered if the seepage measured in the weirs downstream of the dam increases or fails to remain stable. (Nilsson et al, 1991).

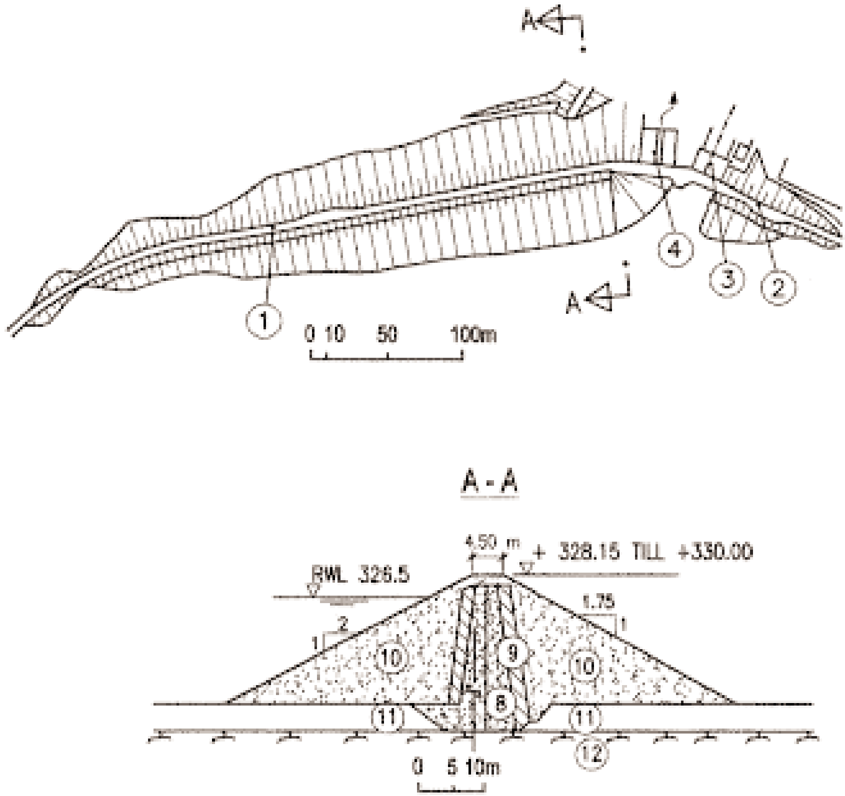


Fig. 4.4.14

Stenkullafors Dam

1. Left dam	*8. Core*
2. Right dam	*9. Downstream filter*
3. Power station	*10. Shoulder*
4. Spillway	*11. Natural ground*
	12. Bedrock

The 49 m high **Roxo Dam** in Portugal was completed in 1968. It consists of a buttress, a concrete gravity and an earthfill dam. Strong progressive settlements have been observed in the embankment in the contact zone with the concrete structure. The settlements have been monitored by visual observation and surveying methods. The instrumentation of the dam includes piezometers, slope indicators and vibrating wire pressure cells. The remedial works comprised the installation of several series of drainage holes and the construction of drainage gallery under the embankment, which was later grouted with a special sealing grout. The settlements continue at a slower rate and an extension of the concrete gravity section into the embankment is planned (ICOLD, 1983).

Other examples of rehabilitation following loss of bond between concrete structures and embankment include :

Nature of Rehabilitation	Location	Dam	Reference
Slurry trench	Embankment at concrete abutment	Arbon, Spain	ICOLD, 1983
Reconstruction	Embankment adjacent to spillway	Lake Cawindilla, Australia	ICOLD, 1983
Excavation, filling of seepage paths with clayey soil	Embankment adjacent to spillway	Staviste, Czechoslovakia	ICOLD, 1983
Backfilled trench to spillway	Embankment adjacent	Virginia Ranch, USA	ICOLD, 1983

4.5 UPGRADING OF EMBANKMENT DAMS FOR SEISMIC REASONS

4.5.1 General

Historically, few dams have been significantly damaged by earthquakes. Worldwide, only about a dozen are known to have failed completely as the result of an earthquake. These dams were primarily tailings or hydraulic fill dams, or relatively old small earthfill embankments of perhaps inadequate design. A few embankment dams of significant size have been severely damaged, some of which experienced near total failure, and were replaced (USCOLD, 1984, 1992, 1999).

Worldwide, the current performance record appears outstanding, based on the limited number of failures. However, few dams have been shaken by earthquakes of duration and intensity sufficient to jeopardise their structural integrity. Most existing dams have not been tested by levels of ground motion equivalent to the Design Basis Earthquake. Conversely, a few dams have experienced significant damage under shaking substantially less demanding than what should have been, considered in their design. Their performance has generally been closely related to the nature of the materials used and the construction procedures. Most well-built earthfill dams are believed to be capable of withstanding substantial earthquake shaking with no detrimental effects. Dams built of compacted clayey materials on clay or bedrock foundations have historically withstood extremely strong levels of ground motion, even when obsolete or inefficient compaction procedures were used. In contrast, older embankments built of insufficiently compacted sands and silts, and tailings dams, represent nearly all the known failures, primarily as a result of the potential for liquefaction of these materials and that of the foundation materials.

Rehabilitation or upgrading of an embankment dam may be justified after a risk assessment including the hazard rating, influenced by the proximity of the dam to sources of seismic activity, the materials and construction of the embankment, and the materials in the foundation.

Earthquake shaking may cause susceptible materials to lose sufficient strength to initiate slope failure or undesirable deformation of the embankment. Shaking may cause deformation, settlement and lateral movement, owing to consolidation of either foundation or fill materials, or both. Differential deformation of adjacent materials, abutments and structures may develop fissuring, which could allow piping to initiate. Settlement might be sufficient to allow overtopping. Rehabilitation measures have been designed and constructed to ameliorate the effects of a future earthquake and to preclude catastrophic consequences.

4.5.2 Rehabilitation Measures

The following is paraphrased from a published presentation of methods available to improve the resistance of embankment dams (not including tailings dams) to seismic effects. The methods were collated from a survey of completed and active projects (Marcuson et al, 1993). In order to rehabilitate an earth dam to prevent potential seismic instability, one must either change the engineering properties of he dam and foundation, modify the geometry of the existing dam, or both.

Upstream and downstream berms and buttresses are used to increase the effective overburden pressure on a problem material and thus decrease its liquefaction potential. This increase in overburden pressure causes a small amount of consolidation and thus improves the void ratio. Berms and buttresses are also used to increase the length of the probable failure surface, provide a counterweight to limit movement, and maintain a remnant section. The effectiveness of a berm is generally limited to a zone that is about as deep as the berm is thick. Addition of a berm or buttress does not reduce the factor of safety during day-to-day operation, and its favourable effect is verifiable. If coarse-grained soil or rock is available, berms and buttresses can, perhaps with difficulty, be constructed on the upstream shell without lowering the pool.

Excavation and replacement removes the problem material, replacing it with a material that is not prone to liquefaction. Excavation and replacement offer relative assurance that what was designed has actually been constructed in the field. It is often expensive and operationally difficult. Dewatering is almost always required and usually the reservoir level must be lowered significantly. This approach is most useful when the problem material is near the surface. The excavation-and-replace method can also be used after an earthquake to repair shallow cracking.

When excavation and replacement are ruled out, in-situ densification can be used to decrease the potential for liquefaction by decreasing the void ratio of the problem material. The method includes vibrotechniques, dynamic compaction, compaction grouting, and displacement techniques. In-situ densification is most effective when the material to be improved is close to the surface and has limited fines content. To date this approach has not been used under an existing dam except where most of the embankment over the foundation zone to be densified could safely be temporarily removed. Foundation densification is unlikely to be uniform and could adversely change the dam-foundation interface. Cracking might occur, which could increase the risk of piping. Verification of the degree of improvement, and of the spatial variability of the improvement is required as a follow-on measure.

Similar to in-situ densification, in-situ strengthening forms a composite material that is strong enough to ensure stability. Displacement piles, stone columns and methods of deep soil mixing are examples. Some of these methods may also cause consolidation and increase the strength of the soil around the feature, but this increase in strength is generally ignored in stability analyses. In-situ strengthening is generally most effective when the material prone to liquefaction is confined to a relatively thin layer, but deep soil mixing can be implemented for thick deposits. Caution is needed when the technique is used under an existing structure because of the possibility of differential settlement. Conventional grouting of the foundation through the embankment has not been used for the purpose of strengthening for two reasons. One is the possibility of hydraulic fracture in the embankment. The other is that it is not easy to determine to what extent the grout has penetrated the zones needing improvement.

An increase in freeboard may be used when the seismic analysis indicates that the dam is marginally stable or only small earthquake-induced deformations are probable. This approach would be intended to decrease the probability of overtopping associated with settlement or slumping of the crest.

Drainage provides for relief of seismically-induced pore water pressure. Techniques include strip drains, stone columns, and gravel trenches. Gravel trenches can be used to intercept migrating, elevated pore pressure plumes. Care must be used in placement of stone columns and gravel trenches so that stone crushing is minimised and permeability remains high. The spacing between stone columns should be sufficiently small to assure dissipation of induced pore pressures during the earthquake, and to prevent the occurrence of high hydraulic gradients that could carry large amounts of fines into the gravel drains. Ideally, stone columns should be flushed periodically to maintain their performance. It is inevitable that extra drains added for rehabilitation will reduce the length of seepage flow paths, which in turn will increase the hydraulic gradients under static pool conditions. Hence, even if the drains are designed as filters with respect to the adjacent material, the static safety of the dam will be somewhat reduced and more water will seep through the dam.

Because available documentation provides no record that a rehabilitated earth dam has been subjected to strong earthquake shaking, we have no field validation of the current techniques. Good judgement requires that nothing be done to increase the likelihood of failure during normal static operating conditions, such as increasing the likelihood of failure by piping, or hydraulic fracture. One must also verify that any intended densification or drainage is achieved. Verification of densification is usually achieved by the use of a test section where standard penetration tests (SPT), cone penetration tests (CPT), or shear wave velocity measurements are compared before and after treatment. If the technique is found successful in the test section, the same before and after index tests may then be used to verify densification of the improved zone.

4.5.3 Selected Case Histories

The **Matahina Dam** is located on the Rangitaiki River in the North Island of New Zealand. It is an 80 m high rockfill dam and is part of a run-of-river hydro electric plant constructed in the 1960s. The reservoir is aligned along the Waiohau

Fault. In 1987 the dam was damaged by a magnitude 6.3 earthquake on the nearby Edgecumbe Fault. A subsequent reappraisal of the Waiohau Fault in relation to the safety of the dam and reservoir concluded that the fault was active and that in the event of future fault movement within the dam foundation, the existing dam would breach rapidly. The dam was strengthened to enable safe reservoir operation in the event of movement of fault traces within the dam foundation. The seismic design of the original dam was in accordance with the practice at that time and allowed for a 0.15 g lateral loading. The dam zones had a minimum thickness of 2.4 m in recognition of the proximity of the Waiohau Fault, as shown below in Fig. 4.5.1 (Gillon et al, 1998).

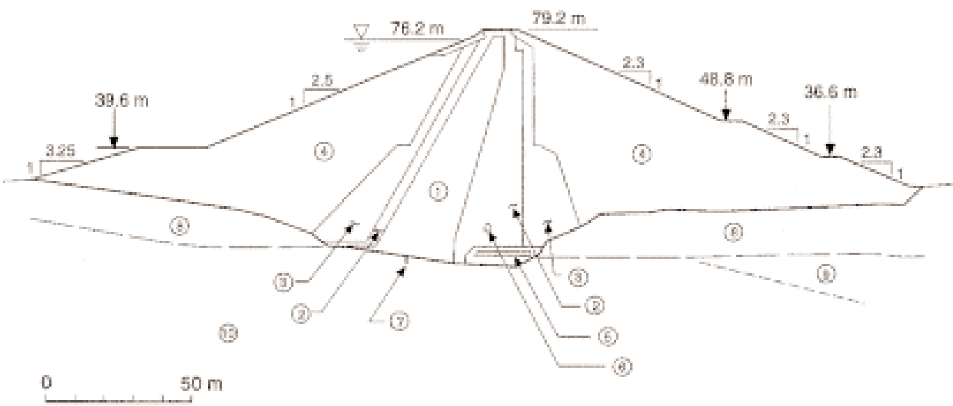


Fig. 4.5.1

Matahina Dam. Original dam prior to upgrading

1. Core
2. Inner transition
3. Outer transition
4. Rockfill shoulders
5. Drainage blanket
6. Inspection gallery
7. Cut-off wall
8. Alluvium
9. Ignimbrite
10. Luke's Farm beds

The main trace of the 80 km long Waiohau Fault passes some 300 m west of the left dam abutment. A state-of-the-art reappraisal of the fault, following the 1987 event, and its relationship to the dam concluded that the dam did not meet widely accepted safety standards. Several Waiohau Fault strands pass in an upstream-downstream direction through the dam foundation.

The core material is a residual sandy clayey silt to silty clay derived from weathered greywacke. It was placed dry of optimum to high densities and thus, it is stiff and susceptible to cracking under shear at low confining pressures, and it is readily erodible. The inner transition consists of the finer strippings of unwelded ignimbrite and has a sandy silt grading. It was compacted to high densities and has a high unconfined compressive strength, to the extent that an inspection gallery was tunnelled through this material without excavation support. The outer transition was built with the finer strippings of the hard-welded ignimbrite and grades from a sandy gravel to a gravelly silt. Its gradation is not filter-compatible with that of the inner transition of the core. The rockfill shells consist of quarried, approximately equidimensional, hard ignimbrite blocks ranging in size from 150 mm to 900 mm.

In view of the characteristics of the embankment materials, it was concluded that the dam lacks a reliable filter and that the core and inner transition have a high vulnerability to cracking, internal erosion, and piping. Relatively small shear displacements are likely to cause cracking of the core. Since the inner transition is also likely to sustain open cracks, and the outer transition does not provide an adequate filter, the core and inner transition can readily pipe into the rockfill shell or the open-jointed ignimbrite. The past performance of the dam has shown that this has occurred.

Since its construction in the early 1960s, the dam has experienced two serious piping incidents that required substantial repairs. The first occurred in 1967 during first filling when a large temporary increase in flow from the drainage blanket was observed (Sherard, 1973). Shortly thereafter, an erosion cavity was detected downstream of the core in the right abutment area and the lake was lowered. Exploratory pits revealed that shear deformations induced by benches in the abutment had led to cracking of the core and inner transition. The core was repaired by placing a 600 mm thick plastic concrete patch backed by a granular filter and by grouting the damaged area with a cement-bentonite grout.

The second incident occurred in connection with the 1987 Edgecumbe earthquake. Strong seismic shaking at the dam site during the earthquake caused deformations of the dam shoulders and increased leakage into the drainage galleries. Investigations of subsidence at the crest indicated that cracking and erosion of the core had occurred near the left abutment. The dam was repaired in 1988 by reconstructing the core and transition zones at both abutments. This included reconstructing the abutment core contact to a smooth profile, replacing the core and inner transition zones with a low strength plastic core material and cohesionless filter compatible materials, and providing special drainage measures within the repair zones.

In view of the possible consequences of a dam breach, and in accordance with international standards, the dam is classified as a very high hazard dam. The performance criteria used to evaluate the dam required that it withstand the Safety Evaluation Earthquake (SEE) without uncontrolled release of the reservoir, albeit with some damage. The SEE was defined as the controlling maximum earthquake that could affect the dam, except that the SEE ground motions and fault displacements need not have an annual probability smaller than 1/10 000.

In view of its potential for generating strong ground motions and fault surface displacements at the site, the maximum earthquake on the Waiohau fault was selected as the controlling maximum earthquake which could affect the dam. This earthquake is estimated to be a magnitude M 7.2 event with an 80 km rupture length. The corresponding ground motions at the site are characterised by 84th-percentile-level free-field peak horizontal and vertical accelerations of 1.25 g and 1.35 g. Fault surface displacements are characterised by 3 m of oblique slip (84th percentile level) which might occur on any of the traces beneath the dam or possibly be distributed between them. For design, it was assumed that the full displacement could occur on any of the traces beneath the dam.

Given the history of cracking and internal erosion under more modest loads and displacements at the Matahina Dam, it is clear that such movements would result in major cracking and the consequent initiation of accelerated piping and internal erosion. Design studies demonstrated that breaching could develop sufficiently quickly that reservoir drawdown was precluded as a possible mitigation measure. The studies also showed that foundation displacements, due to smaller movements and earthquakes on the Waiohau Fault than the SEE event could result in a dam breach.

A variety of rehabilitation and mitigation options was considered. Strengthening of the dam with a downstream buttress which would also resist leakage ranked highest in terms of performance reliability, construction feasibility, environmental impact, and cost, and was selected for implementation.

The principal criterion for design of the strengthening was that the dam must withstand the SEE without catastrophic release of the reservoir. However, damage during the earthquake is to be expected. The total release flow rate from the reservoir, including leakage through the damaged dam, should not exceed the post-earthquake downstream river channel capacity. In addition, the dam should be capable of passing the 20 year flood (500 m^3/s) after the SEE without overtopping, even if the appurtenant facilities were to be out of service. The design against overtopping should consider potential vertical offset on the fault, earthquake-induced settlements, and subsidence due to erosion cavities within the embankment.

The buttress design included features to control post-earthquake leakage through the dam and prevent internal erosion of the core and inner transition from leading to piping of the dam or collapse of the dam crest. As shown in Fig. 4.5.2 the

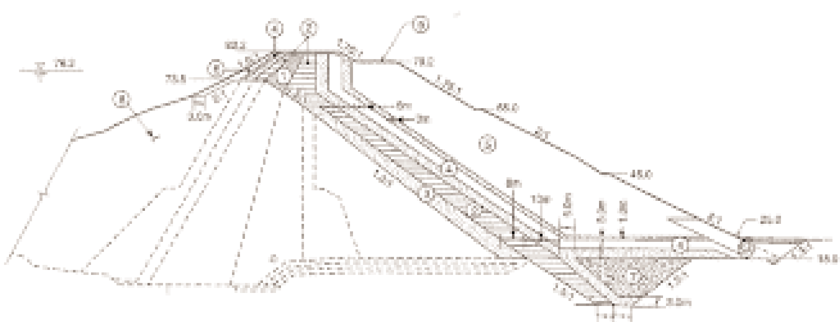


Fig. 4.5.2

Matahina Dam. Rehabilitated dam

1. Core	*5A. Large rockfill*
2. Filter	*6. Rip rap*
3. Transition	*7. Alluvial backfill*
4. Drain	*8. Existing embankment*
5. Rockfill	*9. New roadway*

buttress incorporates filter, transition and drain zones, an enlarged rockfill shell, and a zone of oversize rockfill at the toe. The filter and transition zones consist of cohesionless materials that will not sustain an open crack and will run into and fill any cavities that may tend to develop as a result of the fault displacements. The filter is designed to minimise migration of the core or inner transition materials and to limit flows through the buttress even if the core is breached. The rockfill shell is sized to provide stability during and after the SEE and possible aftershocks. The design assumed that the core might breach and full reservoir head could develop under the filter immediately after the SEE. The drain is designed to limit the development of a high phreatic surface within the rockfill so that stability can be maintained. The oversize rockfill zone will prevent erosion at the dam toe during through-flow as discussed below.

The minimum horizontal width of the sloping filter, transition and drain zones is 5 m each. This width is about twice the estimated horizontal component of fault displacement during the SEE. Likewise, the thickness of the level horizontal transition and drain layers at the base of the buttress is 3 m each, which is about twice the estimated vertical component of fault displacement during the SEE.

The excavation of the existing downstream shell and the location of the buttress were laid out to minimise the size of the buttress required for stability, the amount of buttress overlap required at the left abutment, and the volume of core material that can migrate into the rockfill shell. To prevent piping of materials through the alluvium beneath the buttress, a wide cut-off was excavated through the alluvium as seen in Fig. 4.5.2.

Fig. 4.5.3

Matahina Dam. Work in progress

Rehabilitation of Matahina Dam commenced in June 1997 (Fig. 4.5.3) and consisted of partial excavation of the downstream shoulder of the dam and its replacement by a buttress. The spillway gates have also been strengthened. Reconstruction was completed in late 1998.

For smaller dams it is often neither practical nor economical to mobilise the type of equipment necessary to perform in-situ densification of the foundation or the embankment. If the level of water in the reservoir can be lowered sufficiently, then the defective materials can be removed and replaced. The most economic rehabilitation might be to reconstruct the dam. An example is **Dulce Dam** in the USA, a homogeneous earthfill embankment with a maximum height of 8.5 m and a length of 216 m (Jacoby, 1998). It was constructed in 1903 and was raised in 1910 and additionally in 1936. The dam has a high hazard rating because of the potential for loss of life and property downstream should it fail. An evaluation confirmed that the dam and foundation were prone to liquefaction. In addition, the spillway was inadequate and seepage was not controlled. Removal of the dam and its replacement using modern methods was judged to be the most economic means of rehabilitation.

The original dam was breached, while downstream precautions were taken to control the released water. The embankment material was completely removed and stockpiled. The foundation was excavated to a depth of up to 20 m in alluvium to remove the material prone to liquefaction. The excavated material was stockpiled. The foundation was backfilled with a selected impervious material forming a vertical water barrier, random fill supporting the barrier and a downstream filter, drainage blanket. The new construction is shown in Fig. 4.5.4.

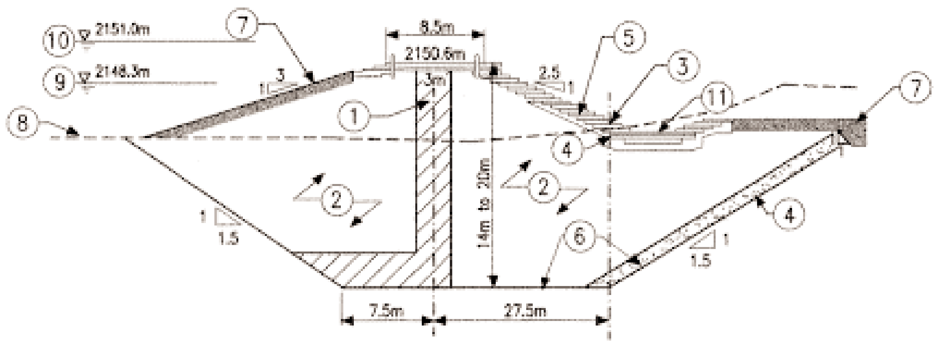


Fig. 4.5.4

Dulce Dam. Typical embankment section

1. Impervious fill	*7. Riprap on bedding*
2. Random fill	*8. Existing foundation surface*
3. Roller compacted concrete	*9. Service spillway crest*
4. Sand filter, drain	*10. Overtopping level of design flood*
5. Weep holes with plastic pipes	*11. Stilling basin*
6. Excavation limits	

The crest and the downstream slope were armoured with roller compacted concrete to give additional spillway capacity. This, together with rehabilitation of the concrete service spillway and a new outlet works, was completed in one construction season.

Other examples of dam and foundation upgrading for seismic reasons :

Nature of Rehabilitation	Location	Dam	Reference
Stabilising berm	Upstream dam toe and slopes of left abutment ridge	Clearwater Lake, 48 m high, USA	USCOLD, 1996
Concrete flood control structure	D/S of existing dam	Graham Lake, 13 m high, USA	USCOLD, 1996
Rockfill buttress	D/S face of dam	Milner, 26 m high, USA	USCOLD, 1996 Pujol-Ruis et al, 1991
Dynamic compaction and berm Stone columns	Foundation area at u/s toe of dam Toe of d/s slope	Mormon Island, 50 m high, USA	USCOLD, 1996
Removal and replacement of foundation material, berm and toe drain	D/s area of dam	O'Neill Forebay, 27 m high, USA	USCOLD, 1996
Stabilising berm	D/s toe of dam	Pineview, 41 m high, USA	USCOLD, 1996
Construction of rolled fill bolsters, foundation excavation and backfilling, compaction grouting	D/s area of dam	Pinopolis West, 23 m high, USA	USCOLD, 1996
Liquefaction within the shell and foundation ; strength loss in silty clay foundation and core. Prestressed concrete piles installed to form a strengthened zone.	U/s hydraulic fill	Sardis, 36 m high, USA	Stacy et al, 1996
Removing foundation material, dynamic compaction and compacted backfill	D/s toe of dam	Steinaker, 49 m high, USA	USCOLD, 1996
Successful upgrade : weighting berms on downstream face, high efficiency foundation pressure relief wells	D/s face and d/s foundation	Mardi, 26 m high, 425m long, ER, Australia	Gan et al, 1999
Reconstruction of dam on a foundation densified by dynamic compaction and soilcrete columns by the soil mixing wall method	Entire foundation	Jackson Lake, 12 m, USA	Farrar et al, 1990
Non structural, extensive emergency action plan. D/s flat, sparsely populated swampland	D/s warning system	Santee Cooper, 15 m, USA	Gotzmer, 1994

4.6 REHABILITATION OF UPSTREAM FACES

4.6.1 Introduction

A major benefit from the point of view of rehabilitation is that an upstream watertight element is readily available. All that is required is to lower the level of the water in the reservoir to give immediate access. Indeed, techniques are being developed to allow rehabilitation to be carried out under water (Scuero et al, 1998). Many materials have been used to provide an impermeable upstream face to an embankment dam including concrete, asphalt, timber, steel, clay and geomembranes. The most frequently used, at least in recent times, are asphaltic concrete and reinforced concrete.

4.6.2 Asphaltic Concrete Faces

Asphaltic concrete has been particularly popular as a lining to the upper pond in pumped storage plants, where the frequent and rapid rate of rise and fall of the water level is an important ageing factor. There are several causes of deterioration of asphaltic concrete faces. Older mixes are prone to oxidation and brittleness under the influence of atmospheric oxygen combined with sunlight and high temperatures. Such a brittle material is less able to resist the fluctuating stress in circumstances where the reservoir level is variable. Similar observations can be made on asphaltic concrete facings lacking a protective bituminous mastic seal coat which normally requires re-application every 10 to 15 years. Early designs incorporated two superposed impervious bituminous layers and air bubbles sometimes developed between them at the interface and can lead to significant deformation and cracking of the asphalt surface. Vertical joints between strips carried out by finisher equipment are prone to more rapid deterioration if they have not been carefully executed.

Rehabilitation of asphaltic concrete facings falls into one of three categories : surface repairs, layer repairs major replacement of one or more of the layers of the facing.

4.6.2.1 Rehabilitation of Asphaltic Concrete Faces

Work in the first category is relatively common and involves local patching and repair of cracks and application of new mastic seal coat. The materials required are usually brought premixed and applied with hand tools. Patching first requires the surface to be milled, followed by drying the exposed surface and applying a tack coat of emulsified bitumen. The next stage is to place and compact the impervious bituminous concrete. The repair is completed by applying a coating and lapping it into sound material. Cracks are repaired by milling the surface on each side of the crack, routing and blowing the crack clean, heating the milled area, applying the tack coat, caulking the joint with hot bituminous concrete and applying the seal coat. More extensive repairs use similar techniques on a larger scale and usually require specialised equipment. If the investigations of an asphaltic lining reveal a general deterioration of the asphalt, local patching will be of little use. The joints between the old brittle asphalt and the patches are likely to re-open even if carefully executed. The only long term solution would be a general renewal of the dense layer. Fig. 4.6.1 shows milling equipment being used to remove part of the impervious layer. Total removal is usually performed by caterpillar excavators.

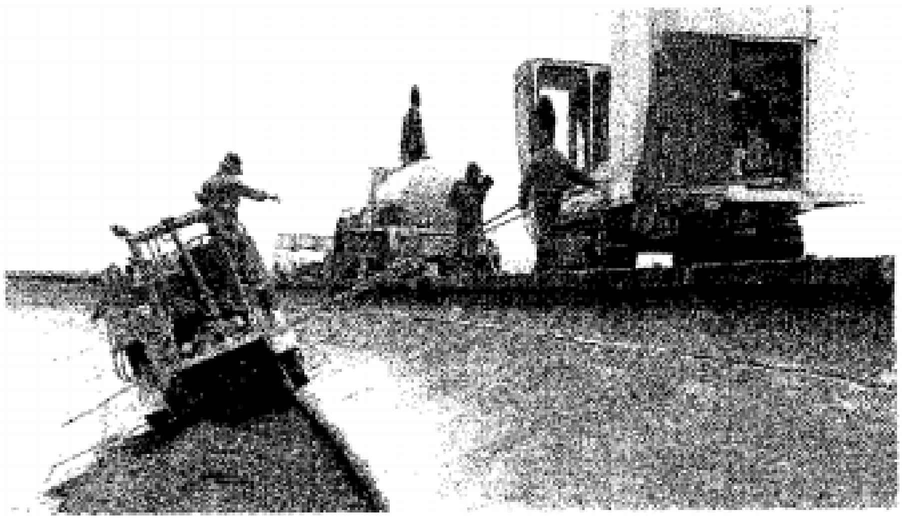


Fig. 4.6.1

Milling equipment for a partial removal of sealing layers

Scuero et al (1998) report that the **Sa Forada** embankment dam in Italy was rehabilitated using an exposed geomembrane system. The PVC membrane was used to protect the bituminous concrete face that had deteriorated after 20 years of service. The essential features of such a system are that it must be effectively drained, and that it should be engineered in detail. **Moravka Dam** in the Czech Republic is 39 m high and was completed in 1966 (Brezina et al, 1998). Deterioration of the upstream bituminous concrete face required additional layers of bituminous concrete. These tended to slide over the original layers below and leakage continued. A PVC membrane was placed directly on to the existing concrete face after removing the additional layers. The membrane remains exposed and is anchored to the bituminous concrete by a mechanical anchoring system. This form of rehabilitation is discussed in more detail in Chapter 3 in connection with concrete dams.

4.6.2.2 Rehabilitation with Asphalt Concrete Linings

Asphaltic concrete technology has been successfully applied not only for the rehabilitation of old asphaltic linings but also of concrete faces. Usually the existing asphaltic lining does not need to be removed completely ; it can provide a suitable base layer for the new sealing. Thus, reconstruction can be limited to the dense layer, which reduces costs in three respects : lower cost of demolition and for the reconstruction of the new sealing, as well as, often most important, a reduced shutdown period due to shorter construction time.

Design and Material

In principle the same design criteria are applicable as for new asphaltic concrete linings (Geiseler, 1996). The standard design for a complete lining consisting of a binder layer and an impervious (dense) asphaltic layer. The binder layer is placed on a non-bituminous drainage or filter layer. Where appropriate a second dense layer and a drainage layer are applied which provide better seepage control.

The asphaltic concrete for the dense layer is composed of crushed aggregates, manufactured and natural sand, mineral filler and bitumen. The grain size distribution is well graded, with a maximum of about 11.2 mm. The composition of the aggregates and the bitumen content are crucial for the behaviour of the asphalt and its sealing properties. It is important to point out that each application for rehabilitation work requires a specific mix design which is adapted to the local conditions and also the envisaged placing technique. It has to be established by laboratory and in-situ tests.

Protection of the asphalt sealing against the ultra violet rays of the sun and consequently against ageing of the bitumen is provided by a mastic seal coat. In most cases hot mastic is used which consists of 2/3 filler and 1/3 bitumen and, if required fibrous additives for the stabilisation of the mastic on steep slopes.

If the rehabilitation is limited to the impervious layer, the existing dense layer is partly milled off to provide a sound asphalt surface for the new lining. Where there are two superposed dense layers, it is important that the top layer is milled off completely and the second layer partly so, in order to avoid the existence of a bondless interface between these old layers. The milled area has to be cleaned and prepared with a bonding agent before the new dense layer is placed. Often the renewal of the dense layer can be limited to the zone of fluctuating water level. Fig. 4.6.2 shows a standard joint design for the connection of the existing to the new dense layer. After placing of the new layer, the joint is re-heated with an infra red heater and re-compacted with vibro tampers.

Placing Equipment

Placing of the new asphaltic layers can be carried out by a conventional dam finisher machine which might travel vertically or horizontally according to local conditions. For reservoirs and canals, a bridge finisher has been used with success. It paves the entire slope in one or two lanes and thus avoids vertically joints. If required it can also produce the curves in the transition to the base sealing and to the crest. Individually adjustable screed segments make sure that the minimum layer thickness will also be achieved at each location even on the uneven slopes of old sealings.

Milling Equipment

Conventional milling machines used for road work require minor adaptation for them to work on the steep slopes of a dam. Usually milling is executed in vertical

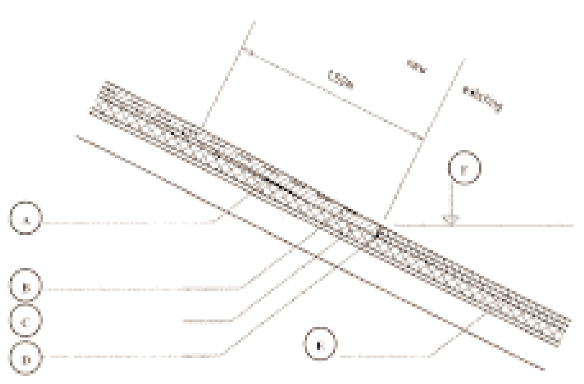


Fig. 4.6.2

Transition joint from existing to new dense layer

A. Existing dense layer milled off to approx. 1.5 cm
B. New dense layer
C. Existing binder layer
D. Joint filled with mastic
E. 2 dense layers (existing)
F. Rehabilitation zone

strips from the top to the bottom. The milling machine is held in position by a winch wagon on the crest. For special applications as for the curved transition zone at the top of the crest or the connection joint to an existing asphaltic layer, horizontal milling might be advantageous.

Case Histories

Vertical Placing with Dam Finisher

The upper reservoir at **Markersbach** pumped storage plant in Germany is contained by a 2600 m long ring-shaped rockfill dam 16 m to 26 m high. The lower reservoir is formed by a 55 m high rockfill dam with a straight crest across the Mittweida Valley (WPDC, 1990). The fluctuation of the water level in both reservoirs is between 15 m and 20 m and occurs several times per day. Bituminous concrete sealing membranes were applied on the upstream face of the lower dam (about 25 000 m^2), on the water side of the upper dam (about 95 000 m^2) and on the base of the upper reservoir (about 350 000 m^2). About four years after commissioning, cracks were observed in the face of the upper reservoir. Repairs were carried out using several bituminous mixtures. Further damage was observed during the following years and it was concluded that the quality of the original sealing had been questionable. Aggregates with low frost resistance had been used and it appeared that errors had occurred during construction of the sealing layers. Despite these problems no leakage was observed at the upstream reservoir.

Several years of unsuccessful temporary repairs followed. Finally the supervising authority recommended that the owner repair the damaged sealing. The nature of the work at the upper dam meant that the plant had to be put out of operation. It was therefore decided to replace the sealing layer of the lower dam at the same time, even though no major damage had been observed there. The rehabilitation work had to be carried out as quickly as possible because of the importance of the 1050 MW peaking plant to the grid. It was completed in four months, from May to August, and encompassed the following works :

The top sealing layer at the Lower Dam was removed by milling as shown on Fig. 4.6.3 over an area representing two-thirds of the upstream face. A new single dense bituminous layer 80 mm thick was placed in vertical strips by equipment retained by a mobile winch on the crest of the dam. The layer was compacted by vibratory rollers. A 2 mm thick mastic coating was then applied.

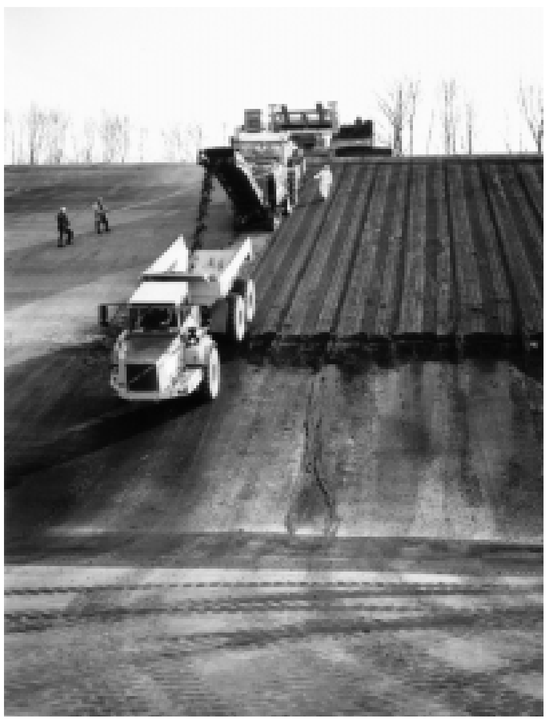


Fig. 4.6.3

Markersbach Lower Dam. Milling the asphaltic concrete

The repair sequence at the Upper Dam was similar to the Lower Dam, except that the new dense layer was placed in two horizontal stages, starting with the bottom half of the ring dam. This procedure enabled partial operation of the pumped-storage plant while the repair was being carried out to the top half of the dam. The equipment placing the bitumen travelled horizontally. The finisher equipment pre-compacted the dense bituminous layer and final compaction was achieved immediately afterwards by three vibrating rollers connected to mobile winches on the crest.

The base of the Upper Reservoir was cleaned and inspected. It was found to be in a satisfactory condition, not requiring any repair work.

A section through the surface layers of the lower dam is presented in Fig. 4.6.4. At the upper dam the section is similar, except that the dense layer rests directly on the 80 mm thick supporting layer, without drainage and lower dense bituminous layers. The mineral content of the new dense bituminous layer for both dams consists of natural and crushed sand (up to 2 mm), gravel (2-5 mm, 5-8 mm, 8-11 mm, 11-16 mm) and limestone dust filler. The bitumen has then been added to the dry mix in the ratio 7.4 to 100. The maximum mixing temperature reached 195° C.

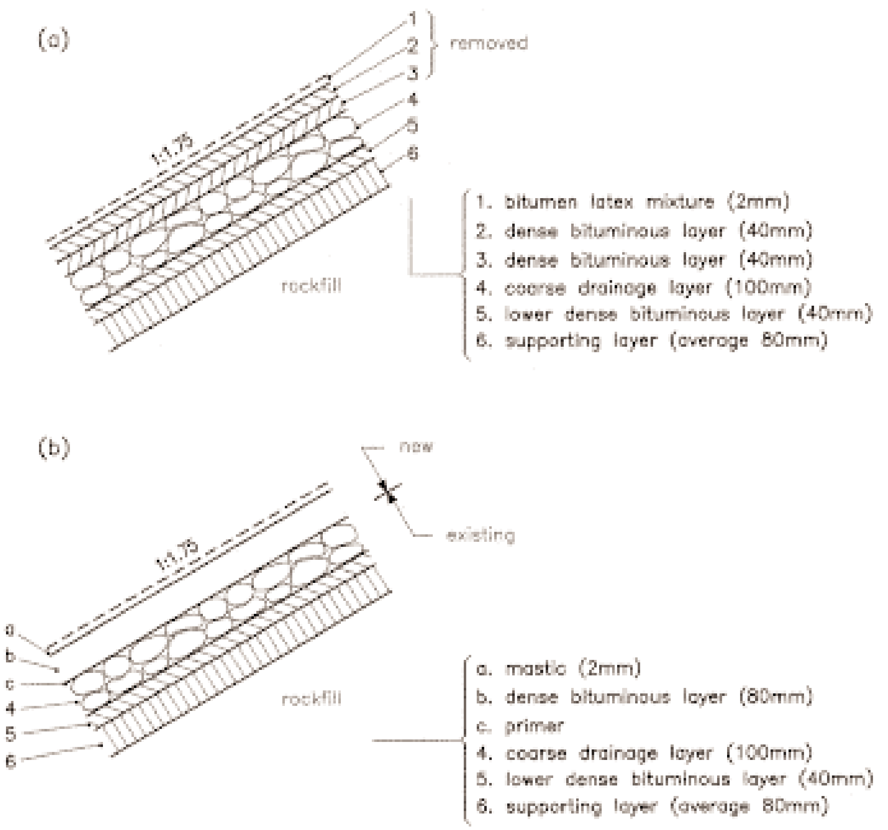


Fig. 4.6.4

Markersbach Lower Dam. Section through the surface layers
(a) before the repair and (b) after the repair

Horizontal Placing with Dam Finisher

Hardap Dam with a 40 000 m^2 asphaltic concrete face impounds Namibia's largest reservoir. After more than 30 years of operation, the sealing element, which consisted of a binder and a dense asphalt layer revealed major defects including surface cracks, open joints at the spillway structure and surface deterioration owing to the mastic protection being missing. A first rehabilitation attempt failed in 1993/94 owing to an inadequate placing technique. A new lining was placed successfully in 1995 by a specialist contractor.

The rehabilitation work consisted of the milling of the existing dense asphalt layer and partial replacement of the binder layer in damaged areas. It also included the complete renewal of the joints adjacent to the wing walls of the spillway. As part of the preparation work, trial mixes and in situ trial placements were carried out to establish the optimum mix design for the asphaltic concrete. Placing of the new dense layer was carried out by a paver with high compaction screed (duo tamper). The upper end of the slope was confined by a 2 m high concrete parapet wall. Consequently, and in order to minimise hand-placing of asphalt, the paver travelled in a horizontal direction, starting at the bottom of the slope. The hot asphalt was transported in a supply car from the crest to the paver. Two steel drum rollers were used for final compaction to achieve a void content of less than 3 %. The machines working on the slope surface with an inclination of 1 on 1.7 were held in position by winch wagons on the crest. The horizontal joints between the lanes were reheated by infra-red heaters and recompacted with manually operated vibro tampers. Finally, an asphalt mastic seal coat was placed to protect the asphalt lining against aggressive ultraviolet radiation. The rehabilitation project was finished within two months (Frohnauer et al, 1996 (a)).

Overlaying Existing Concrete Sealing

The 16 km long headrace canal (Alz canal) for a 50 MW powerplant near Burghausen in Germany was constructed between 1916 and 1922. Based on a structural investigation, a rehabilitation strategy was established in the early eighties. It envisaged two shutdown periods between 1984 and 1987 for extensive rehabilitation work, which included a new asphalt lining for a 1440 m long section of the canal.

The investigation showed that the existing concrete slabs were a suitable base for the asphalt lining. With respect to the steep slopes with an inclination of 1 on 1.25 and in order to ensure sufficient adhesion of the new asphalt lining, horizontal grooves were milled into the existing concrete surface. For the jointless placing of the two asphalt layers (60 mm drain binder and 60 mm dense asphalt), a special bridge finisher was developed to meet the demands of high quality and the construction time of six weeks (Waechter et al, 1988).

Complete Replacing of Concrete Sealing

In 1997 a rehabilitation project was carried out for a section of the Isar Canal near Munich. The owner would not accept any reduction of the flow area by an overlaid asphalt lining. Therefore, the existing concrete slabs had to be removed. The new asphalt lining was placed on a lean concrete layer which also served as a drain for seepage control. Placing of the two asphalt layers and also of the concrete layer was carried out with two bridge finishers. Because of the steep slopes (1 on 1.25) the asphalt was stabilised by adding mineral fibres. The asphalt surface was coated with a cold spray applied mastic. All sealing work of the 2.2 km canal section was completed within 6 weeks (Frohnauer et al, 1998).

Rehabilitation Work during Partial Drawdown of Reservoir

Thirty years after construction, the asphaltic concrete sealing of the **Geesthacht** pumped storage reservoir was partially replaced in 1985/86 (Haug et al, 1987). In the zone of the fluctuating water level an increasing formation of cracks and open vertical joints was observed. These defects indicated a progressive ageing of the asphalt due to a combination of chemical and mechanical effects. Atmospheric oxygen under the simultaneous influence of ultraviolet radiation led to a hardening of the bitumen. The ductility of the sealing was reducing over the long term. The daily deformation induced by filling and emptying of the reservoir were progressively producing more cracks. The elastic deformation had an important impact on the sand embankment and was considered to be the main ageing factor.

The base sealing layer and also the lower part of the slopes which were permanently submerged were still in good condition. The rehabilitation work was restricted to the upper slope section on a width of 25 m. The rehabilitation measures included the milling of the two 35 mm dense layers, with only 15 mm left in place. The new 80 mm thick asphaltic concrete lining was placed in one layer with a horizontal travelling bridge finisher, thus avoiding vertical joints. By using polymer modified bitumen the mechanical behaviour of the asphalt under repeated dynamic loading was expected to be improved.

A total of 50 000 m^2 of asphalt sealing was placed in two stages each of 6 weeks. The work was carried out under partial drawdown of the water level and continuing operation of the power plant.

Total Replacement of the Concrete Sealing by an Asphalt Lining

The upper reservoir of the pumped storage power scheme of **Reisach Rabenleite** was built in 1955 with a slope sealing of unreinforced concrete slabs. A few years after commissioning the first cracks appeared and progressively affected the entire sealing. Frequent repair work allowed the continued operation of the reservoir, but the rate of deterioration of the concrete increased. In search for a long term technical solution, in-situ tests were carried out with an asphaltic sealing overlaying the existing concrete face. But the results were not satisfactory : cracks re-appeared in the asphalt sealing after a short time period. They were caused by the unsuitable support conditions of the substructure. Based on these tests, it was decided to remove the entire concrete sealing and to construct a new asphaltic concrete lining. After demolition of the concrete slabs, the slopes of the embankment were re-shaped with a re-alignment of the transition from the base sealing to the slope sealing. On the compacted slopes a drainage layer was built, stabilised with 50 kg cement per m^2. The new asphalt sealing was placed on top of the drainage layer. The 80 mm thick binder layer and also the dense layer of the same thickness were placed by a bridge finisher in two horizontal strips of 21 m width each.

The existing bottom sealing of the reservoir, consisting of three layers of bituminous roofing sheets reinforced with glass fibre fleece was also completely reconstructed. After removal of the old sealing, a gravel drainage layer with separate drain fields for seepage control was built. The individual seepage pipes were connected to a control gallery which was constructed approximately through the axis of the reservoir, under the bottom sealing. A 60 mm binder layer and a 80 mm dense asphaltic layer were placed by high compaction finishers (Frohnauer et al, 1996 (b)).

4.6.3 Concrete Faces

Concrete faced rockfill dams have been used successfully in many circumstances since the middle of the twentieth century. Earlier designs were based on dumped rockfill, this being replaced in the 1970s by compacted rockfill. Rehabilitation has been necessary where the concrete face has been unable to accommodate the settlement of the rockfill within the embankment. The excessive settlement of the rockfill may have a variety of causes, including lack of stiffness and also the effects of lack of free draining characteristics. Problems encountered include cracks in the concrete face, water bar failure and spalling at joints. It has been frequently pointed out that this design of dam is inherently safe, provided that the rockfill is free draining.

Courtright Dam, 100 m high in the USA, built in 1958, was one of the last of the dams to be designed using dumped rockfill. Early in 1970s the dam began to show distress including open, offset joints resulting in unacceptable leakage. Reconstruction of the upstream face was undertaken (Kollgaard et al, 1988).

The 71 m high **Gouhou Dam**, a concrete faced gravel structure in China is an example of the failure of a dam of this type (Chen, 1993). Immediately the dam was completed in 1989, when the water was still 22 m below the normal retention level, a concentrated flow appeared on the downstream slope near the toe and started to scour the embankment material. The scouring was repaired and the leak appeared to stop. An inspection following an earthquake in 1990 revealed no cracking or leakage. Further concentrated flows appeared later in 1990 and were repaired during a period in which the water level was lowered. In 1993 the reservoir reached its highest level. Within a day, water was seen gushing out of the downstream face. Witnesses reported a thunder-like noise and saw water splashing and stones rolling down the central part of the dam. It was found that after compaction the rockfill contained between 24 % and 60 % of minus 5 mm particles, making the embankment not free draining. The failure of the dam was attributed to water leaking through cracks in the slab of the crest wall when the reservoir level was only 300 mm above the level of the cracks.

Repairs to damaged concrete faces have been attempted using both geomembranes (Brezina et al, 1998) and also as described above using asphaltic concrete. See, in addition, Flemme et al, (1990). The geomembrane form of rehabilitation is discussed in more detail in Chapter 3 in connection with concrete dams. It will become increasingly important to be able to carry out repairs of this nature under water.

4.6.4 Other Faces

Steel faced dams require periodic recoating of the steel to resist corrosion. Sometimes cathodic protection is appropriate.

Examples of rehabilitation involving work on upstream watertight elements include :

Nature of Rehabilitation	Location	Dam	Reference
Repair of the asphalt face and protection by rip-rap	Upstream face	Lake Bostal, Germany	Kast et al, 1996
Remove damaged coat and replace it with one without joints	Upstream face	Geestacht, Germany	Kuhlmann, 1996
Repair of blisters	Upstream face	Winscar, 53 m high, 520 m long CFRD, UK	Wilson et al, 1998
Asphaltic membrane to repair cracked concrete face slab	Upstream face	Martin Gonzalo, 55 m high CFRD, Spain	Flemme et al, 1990
PVC membrane	Upstream face	Moravka 39 m high, 396 m long ER, Czech Republic	Brezina et al, 1998
Renewal of 80 mm thick sealing layer	Upstream face	Porabka-Zar, Poland	Fiedler et al, 1996
Replacing a three layers sealing by a single 70 mm dense layer	Upstream face	Hennetal Main Dam, Germany	Ruhrverband, 1998

5. APPURTENANT WORKS

5.1 INTRODUCTION

The scope of this chapter is the rehabilitation of appurtenant works of dams where the rehabilitation is required primarily due to the effects of ageing. The appurtenant works include spillways and outlet works, together with their gates and the associated mechanical and electrical control equipment.

Of particular importance to this chapter is the body of replies to ICOLD's Question 71 that was dealt with at the Durban Congress (ICOLD 1994). The General Report by Pinto will be of particular assistance to those engineers tasked with managing the rehabilitation of the appurtenant works of dams. Much, but not all of the damage to appurtenant works tends to be brought about by the effects of flowing water. Corrosive processes affect some elements and these should have been designed to facilitate replacement. A particular example is the pipes installed through old embankment dams in the UK.

The circumstances leading to rehabilitation will be discussed and then case histories of remediation in each of the major appurtenant works will be presented.

5.2 CHANGES IN KNOWLEDGE OR STANDARDS

Research and experience leads to increase in knowledge of the loads imposed and the behaviour of elements of dams. This increased knowledge can reveal that the dam will not behave safely in some circumstances. The owner may no longer view this as acceptable and in these circumstances the dam may require remedial works.

A frequent reason for rehabilitation is to increase the capacity of the spillway to pass a larger flood than was originally anticipated. Rehabilitation may also be called for where experience on other dams reveal a weakness or a risk of failure of a component common to both. In the same way, alterations to an operating procedure might be the justification for remedial work. Some national design guidelines or standards relate the required level of security to the consequences of failure of the dam. Hence, changes to land use downstream, by building housing for example, might justify rehabilitation.

Examples of rehabilitation to spillways following a review of capacity using modern criteria are described below and include **Coolidge Dam**, **Theodore Roosevelt Dam** and **Shepaug Dam,** all described by USCOLD (1996).

Circumstances leading to rehabilitation are normally detected through regular dam safety reviews. At regular intervals the design assumptions for the dam should be examined to ensure that they remain valid.

Risk analyses sometimes reveal that the level of risk posed by a dam or an important element of the appurtenant works is too high or that the consequences of failure have increased due to changed circumstances downstream. Because of the high cost of modifications and rehabilitation it is becoming increasingly common for owners to evaluate hazards and risk before embarking on rehabilitation works. The techniques in use for critical items, and also for the whole project are briefly described in Chapter 2 of this Bulletin. This may lead to a reduction in the extent of rehabilitation required or it may, in high hazard circumstances, lead the owner to adopt a solution with a lower risk of failure than is normally accepted.

5.3 PRINCIPAL CAUSES OF DETERIORATION

5.3.1 Local Scour

The term scour usually describes the effects of fast flowing or turbulent water on natural materials including rock and soil. The effect of impact, turbulence, and friction is to generate hydrodynamic forces against the faces exposed to the flow. Experience has shown that these forces are often not well understood or their magnitude underestimated. As a result hydraulic structures are sometimes under-designed in this regard and suffer significant damage as a result.

Erosion or undermining downstream of the structure is a consequence of local scour. Spillway structures older than 25 years with a history of frequent operation at high flows are particularly susceptible. Structures on sand or soil foundations are susceptible to the erosion of the foundation.

Unlined spillways have been successfully used, resulting in considerable cost saving. The potential erodibility of rock can sometimes be determined by precedent. However, erodibility is dependent on the geotechnical properties of the rock mass, hence close study of the geology is essential to ensure stability where the rock foundation adjacent to a structure may be eroded. The technique of calculating stream power and comparing this with the Kirsten index appears to provide a useful tool for assessing erodibility (Van Schalkwyk et al, 1994). The internal erosion of the foundations of appurtenant works is discussed in Chapter 4.

5.3.1.1 Detection and Monitoring

Scouring is a complex phenomenon, and there are at present no analytical and experimental methods for forecasting scouring phenomena definitively. Periodic visual or sounding surveys and occasional diver inspections are useful in determining the extent and development of the scoured area. These inspections may have to be scheduled for periods when the discharge or reservoir level is decreased. On some structures changes to normal operations may be required to permit the surveys.

Piezometers are sometimes useful in revealing the effect of the increased flow through a foundation in which erosion has occurred but measurement of local piezometric pressure alone is rarely sufficient to detect flow changes.

5.3.1.2 Effect on Safety and Performance

In the most serious cases, erosion downstream can undermine an important structure, causing structural collapse.

5.3.1.3 Rehabilitation Measures

Rehabilitation measures against erosion include two broad approaches. First to remodel the works to modify the pattern of flow, avoiding the damaging concentration of flow. Second is to fill the erosion cavities with material more resistant to the eroding process.

5.3.2 Erosion by Abrasion

Solid particles in the flowing water or water subject to wave action causes erosion by abrasion in hydraulic structures. The extent of damage is a function of velocity and turbulence in the flow as well as the hardness of the abrasive material and the quality and nature of the surface being abraded. Hydraulic jump stilling basins and bottom outlets are particularly vulnerable to abrasion damage. The example of **Chaudiere Ring Dam**, Canada, described by Mirza et al (1994) is outlined below. Asymmetric flow is a major contributor to the problem, exemplified by the case study of **Uljua Dam**, (Kulkarni et al, 1994), described briefly below. As also mentioned below, the operating procedure for **Ohmatazawa Dam**, was modified so that it operated without using gates, although these had been included in the original design (Egawa, 1994).

Three principal sources of abrasive material are reported. In scour and low-level outlets sediment originates in the reservoir. Abrasion in energy dissipaters has frequently been caused by rock drawn into the stilling basin from downstream by reverse currents. A third source is material that finds its way into stilling basins, tunnels or pipelines by other means, such as, construction or maintenance debris, objects thrown by visitors or fallen rock from side slopes.

There are reported cases of damage from abrasion at a discharge much smaller than the design discharge, see for example, **Seyhan Dam** (Orhan, 1994) described below, especially where a high tailwater level causes an hydraulic jump to form on a sloping chute approach to the horizontal floor of the basin. There is a particular problem immediately downstream of flip buckets where the low flow is not thrown clear and abrades the foundation immediately downstream of the structure.

Some dissipation basins designed to form a hydraulic-jump tend to draw rock and sand from the downstream channel back into the basin, and continue to circulate the material rather than eject or sweep it from the basin. This circulation of sand and rocks is similar to the action of a ball mill, causing severe erosion of floors, side walls, floor blocks, and the energy dissipating teeth. The depth of erosion may reach meters (ICOLD, 1994).

Damage to low-level outlets and temporary diversion outlets has been reported in the conduit lining, gate and valve parts, and pipes. Particularly vulnerable are outlets used for diversion during construction, for reservoir sediment release, or for outlets designed for the control of reservoir sedimentation.

Once damage to concrete or steel surface has started, the abrasion will accelerate with each operation of the spillway or bottom outlet unless the abrasive materials are removed. Hydraulic cavitation may also be triggered by the abrasion damage, greatly increasing the rate of destruction.

5.3.2.1 Detection and Monitoring

Regular inspection of stilling basins and low-level outlets is the only reliable means of detecting the extent of damage. Underwater inspections may be required, see for example the account of the work at the **Potomac River No. 5 Dam** (McClain et al, 1994) described below. Exploratory drilling may sometimes be helpful in determining the depth of damage. Abrasive damage may develop rapidly under high flow conditions.

5.3.2.2 Effect on Safety and Performance

Abrasion action causes pitting of a concrete surface, first eroding the cement paste surrounding aggregates. Sometimes the exposed aggregates are damaged or detached from the surface. As the process develops the concrete may be eroded down to the reinforcing steel. The process can, over time, lead to a catastrophic failure.

On metal surfaces the abrasive action can result in insufficient material remaining to perform the design intent. The abrasive action often causes pitting which in high velocity flow will result in cavitation and rapid removal of the surface.

5.3.2.3 Repair of Damaged Surfaces

Rehabilitation options for structures suffering from abrasion damage fall into three major categories : repair of the damaged surfaces, redesign to prevent the flow conditions responsible for the damage, and improved operating techniques.

Abrasion damage can be repaired and minimised by constructing flow surfaces of special concretes or resistant materials such as stainless steel. Natural materials are often useful, particularly blockwork of good quality igneous rock (Kogovek, 1997). However, these solutions are expensive and do not eliminate the cause. In the design of these facilities it is important to exclude the abrasive content of the flow as far as possible. This is often impracticable in rehabilitation. Except for diversion flows, most reservoir outlets do not carry significant amounts of abrasive material in their releases and in outlets designed for diversion it is common to provide for full functionality after an appropriate allowance for abrasion.

Because of their high cost, the use of special concrete and other resistant materials is recommended primarily for the repair of highly eroded areas under severe attack. Silica fume in conventional concrete is an effective means of improving the resistance to erosion by surface abrasion. This extremely fine silica powder results in a harder and stronger cementing paste in the concrete. Paste or mortar in concrete is susceptible to erosion by wear. Good quality hard aggregate will resist wear better than normal mortar. The combination of high quality aggregate in silica-fume-modified concrete results in a harder and more durable material better suited to severe erosion environments.

The performance of calcium aluminate cement and calcium aluminate aggregate are reported by Cabiron (1996) and Cabiron et al (1998). Cylinder compressive strengths of 50 MPa in 24 hours are possible and the resulting material has shown in tests to be an effective repair material with good adhesion and durability under severe abrasion coupled with high water velocity.

Toyoda et al (1991) report test results that show that the resistance of a range of materials to attack by gravel. These showed that even high strength concrete was eroded much faster than stainless steel and indicates that in the most severe cases more expensive solutions may be warranted.

5.3.3 Erosion by Cavitation

Cavitation is one of the most frequent causes of deterioration of high head spillways and outlet works. Cavitation occurs in flowing water when a reduction of pressure within the water leads to a change of phase from liquid to vapour. The process starts at sites known as gas nuclei. The gas nuclei may grow rapidly, forming visible cavities in the fluid. As the local velocity increases, the pressure decreases proportionally, and it may reach a critical value at which the vapour cavities become unstable. When the cavities move into a higher pressure zone, they collapse. The sudden collapse of the cavities generates intense pressure shock waves, which produce noise and surface damage. The pressure bursts may reach thousands of MPa. The cavitation damage itself may produce a region of reduced pressure leading to further cavitation.

In hydraulic structures under high-velocity flows, pressure reduction is mostly related to changes in local velocity caused by boundary irregularities. In high-head outlets, the cavitation potential can be reduced by providing adequate back pressure. The most vulnerable area for outlets is the region where pressure flow changes to free flow, such as downstream from a control gate or valve that discharges into a free-flow conduit, tunnel or a chute.

5.3.3.1 Detection and Monitoring

Cavitation risk is usually based on the evaluation of the critical cavitation index, and its comparison with the cavitation number for the flow. Empirical expressions for the calculation of critical cavitation index are available in the literature. Such expressions were obtained mainly by the evaluation of laboratory tests on flows across different types of surface irregularities including gate slots for example. The cavitation number for the flow depends for open channels or unpressurised tunnels mainly on the vapour pressure, slope of the chute bottom to the horizontal, radius of the vertical bend and water depth normal to the flow (Jansen, 1988). The cavitation number for the flow through gates depends mainly on vapour pressure, water pressure and flow velocity at the critical location.

The potential for cavitation damage can be evaluated for existing hydraulic structures by measuring the pressure profile and by the comparison of local pressure with the water vapour pressure. Such field tests are applied during the phase of the design of remedial measures for damaged structures. For practical purposes, the critical pressure for the onset of cavitation may be taken as the vapour pressure of the fluid. If the pressure within the flow fluctuates, there may be increased risk of cavitation, even though the mean pressure is well above vapour pressure. As a general rule, if the velocity of the stream entering a stilling basin exceeds about

Fig. 5.3.1

Typical cavitation damage

20 m/s, the flow near baffle blocks may cavitate. Cavitation will be a serious possibility when the velocity exceeds approximately 25 m/s. Tunnel spillways have been shown to be particularly susceptible to cavitation and care needs to be taken with both changes of grade and concrete finish to avoid creating zones of low pressure. Aeration devices are now generally included in spillways where velocities may exceed 20 m/s. The cavitation in the stilling basin by **Pit 6 Dam**, USA, has been discussed by Cassidy (1994). See also the accounts of rehabilitation at **Glen Canyon Dam** by USCOLD (1996) and at **Karun I Dam** by Fouladi (1994).

5.3.3.2 Effect on Safety and Performance

The effect of cavitation on concrete and steel surfaces can be unexpected, rapid and disastrous. Within a single event spillways have been destroyed and valves or outlet works rendered inoperative. An extreme example of the destructive force and speed of cavitation is found in the outlet at **Tarbela Dam** in Pakistan (Lowe et al, 1979). The flow velocity at which cavitation damage became significant was approximately 47 m/s in the tunnels.

5.3.3.3 Rehabilitation Measures

Aeration of high-velocity flow has been demonstrated to be the best method for the prevention of cavitation. Thus design of new structures should include aeration provisions to prevent or minimise cavitation damage. Existing structures that have been damaged by cavitation erosion may be retrofitted with aeration devices. In retrofitting structures with aeration slots, concrete may have to be excavated in their construction. However, concrete excavation can be minimised by the use of ramps to create a space for introduction of air.

The use of smooth walls and cavitation-resistant covering, for example high strength concrete, fibre concrete, masonry or steel can assist in avoiding cavitation problems. Zhang Jinsheng (1994) has described work of this nature at **Sanmexia Dam** in China. However it is better to eliminate the source of the cavitation than to try to prevent damage that the problem causes. The use of surface protection, even stainless steel armouring, has not been effective in preventing cavitation damage for devices installed to increase turbulence. Streamlining these devices, often useful for other parts of spillways and bottom outlets, of course, would be counterproductive.

In areas subject to erosion by cavitation and abrasion, the use of special concrete or other materials should be considered. Concrete made of calcium aluminate aggregate and cement has been shown to be effective in resisting the effects of cavitation on concrete.

Fibre reinforced concrete contains between 0.5 % and 1.5 % by volume of steel or polypropylene fibres. This amounts to approximately 2 million fibres per cubic metre of concrete. These fibres are effective in increasing the toughness and tensile strength of the concrete. Impact resistance and fatigue strength are also improved. However, fibre reinforced concrete is less effective than special concrete or blocks of igneous rocks in resisting erosion, especially under a high water flow velocity. This has been attributed to the grinding action of the particles entrained in the water, coupled to flexure of the fibres leading to local damage to the surrounding concrete. Under cavitating conditions the evidence is conflicting as to whether fibre reinforced concrete is effective in reducing damage.

Patching can be used to correct damage from cavitation or erosion, initial tolerance errors, form bolt holes, and lift join imperfections. The process needs detailed attention. In recent years, excellent materials and procedures have been developed, and some of the good old ones described below have been re-recognised. The patching material must be correct for all requirements of each situation, and it must be properly prepared, applied, and cured. Mirza et al (1994) report on a series of tests of repair materials and methods.

There is no simple solution for repairs. In some circumstances the solution is to use concrete of the same quality as the surrounding material, and held in place monolithically. For this to be successful it is imperative to use a mix that is similar to the original material in aggregates, cement and water content. This is to achieve a repair that has the same texture, moduli and thermal coefficient, and which can act monolithically with the base. However, even similar concrete placed after the original concrete has gone through its drying shrinkage can pull away from the base material as it cures. The designer may have to vary the properties of the patch material to account for this.

It is not necessarily true that high compressive strength means a better material. Crushing by compression is seldom the mode of failure in an environment of cavitation or erosion. Failure is more often related to the dimensional stability, tensile capacity, fatigue endurance, strain capacity, and continuity of the repair material with parent material.

Epoxy resin is an excellent repair material and yet many repairs have subsequently failed. Investigation of failures has shown that the epoxy did not fail, but the repair system did. That is, the epoxy itself held up well and bonded to the concrete, but a separation occurred just below the glue line. Differing shrinkage or thermal properties can contribute to this failure. It also can be caused by vapour or water pressure building up beneath the epoxy, causing it to spall off with the failure in the weaker concrete matrix just beneath it. In other cases the epoxy repair has resisted the cavitation forces but transmitted them to the base material without redistributing them sufficiently for the base concrete to withstand them.

Monomers and polymers provide possibilities for repair work in concrete (Lampa, 1994). This type of material can also be used in original construction as insurance, to provide high resistance to damage in areas where cavitation is known to be possible, or where expensive consequences are expected if damage did occur. In new construction, the monomer can be soaked into the hardened concrete after moisture is force-dried out of the capillaries. It then is polymerised or solidified in situ. The resulting strength of the concrete and its resistance to cavitation can be significantly increased. The repair of **Libby** and **Dworshak Dams** used epoxy, fibre reinforced concrete and polymerized concrete (Regan et al, 1979).

5.3.4 Obstruction by Solids in the Flow

A common problem with overflow spillways and low-level outlets is the obstruction of the discharge by debris. This scenario has the most serious consequences when the spillway or the low-level outlet become inoperable. Trashracks can be damaged and the operation of gates and valves impaired by the debris.

Floating blockage is the problem for overflow spillways, especially where the openings are covered by, for example a road bridge as at **Palagnedra Dam**, described below. Logs can catch at gates or in gate or stoplog slots and prevent operation. Floating debris can also damage gates by physical impact.

Low level outlets may be blocked not only by timber, but also by sediment. See for example the accounts of rehabilitation at **Holmstyes Dam** in the UK by Dyke et al (1998) and **Alloz Dam** in Spain by Uceda et al (1996). Clogging is more likely in small outlets that are infrequently operated. Long periods without operation may allow the openings to become permanently blocked and lead to the loss of the facility. Silt accumulation in gate slots is a frequent nuisance. Valves that block the flow passage, such as butterfly valves and cone valves, appear to suffer more from blockage than valves or gates that expose the whole cross section.

5.3.4.1 Detection and Monitoring

The blockage of a spillway is often detected visually. Low-level outlet blockage is detected by the failure of the outlet to work. The frequent exercise of valves will assist in detecting this problem.

Siltation within a reservoir can be monitored by the survey of “silt lines” or by the observation of siltation at selected points. This is normally done using a Global Positioning System (GPS) to locate the points and sounding or sonar to measure the depth. Obtaining sufficient accuracy to monitor low siltation rates has been difficult.

It is important to monitor catchment conditions to predict when debris load is likely to be a problem. In areas where forest fires can occur these can add significantly to the problem.

5.3.4.2 Effect on Safety and Performance

In the most serious cases the spillway capacity may be reduced below the design requirement so as to endanger the dam from relatively small inflows.

Siltation or submerged debris may render bottom outlets inoperable and prevent the lake from being lowered in an emergency.

5.3.4.3 Rehabilitation Measures

Rehabilitation measures against the effect of blockage include two broad approaches ; to remove the solid material or to prevent it blocking the opening. Routine maintenance is an essential component and a risk assessment is recommended. This approach is briefly described in Chapter 2 where reference to more specialist text is made.

5.4 REHABILITATION OF OUTLET WORKS

5.4.1 Introduction

The operation of a dam depends on its outlet or control works to achieve its purpose of water supply, water regulation or power generation. Typically each outlet consists of an intake structure through which the stored water enters the outlet, a tunnel or a pipeline to convey the water and a powerhouse inlet valve or pressure dissipating valve.

Most of the operating components of outlet works (trash screens, control valves, control systems) have a significantly shorter life than other components of a dam. This is normally recognised at the design stage and facilities are incorporated to simplify replacement or repair. In some structures the replacement method envisaged by the designer required the water level to be lowered, often considerably. The loss of water resource, diversion potential or revenue caused have encouraged many owners to seek solutions which do not require the water level to be lowered during the repairs.

Bottom outlets or scour outlets where the intake is under a considerable depth of water pose a particular problem because of the magnitude of the loss of resource, the time required to lower and refill and sometimes the impracticability of doing this with uncontrolled inflows. A particular problem is the economic cost of the loss of use of the water when the reservoir is emptied. This has prompted imaginative rehabilitation solutions to outlet works in which the work is done with the reservoir full.

Particular care is required in planning and carrying out work by divers. Even a water flow velocity of 1 m/s or 2 m/s can make conditions unacceptably hazardous.

5.4.2 Outlet Tunnels and Conduits

Perhaps the largest outlet tunnel repair was that undertaken at **Tarbela Dam** where in Tunnel 2 alone 382 000 m^3 of low strength concrete had to be placed before a new lining in the tunnel could be commenced (Chao, 1980).

In other places rehabilitation of outlet conduits has involved installing a small diameter sleeve within the existing outlet and grouting the annulus. Virtually all technologies used in water main replacement may be used successfully. It is important to provide sufficient support to the liner to resist the external water pressure.

The design of high-head outlet conduits needs care. For long conduits, the downstream loss will ordinarily produce the required back pressure to prevent cavitation, but for short conduits, gate passages frequently must be enlarged or exit constriction provided to produce appropriate pressure conditions. When conduits are to be operated at partial gate openings, special care should be taken to provide streamlined shapes because back pressure will not be provided when the conduits flow partly full.

5.4.3 Bottom Outlets

The cavitation on bottom outlets will be minimised by the application of some or all of the following design features :

- Improvement of the shape of water passages. Examples are streamlining of conduit entrances, increasing the amount of offset and decreasing the rate of taper downstream of gate slots, or using larger bend radii.
- Increasing the pressure by raising the hydraulic grade line at areas of disturbed flow, which may be accomplished by flattening any downward curve, restricting the exit end of the conduit, or increasing the cross-sectional area in such localities as gate passages to decrease the velocity and increase the pressure.
- Introducing air at low-pressure areas not only to raise the pressure, but to provide air bubbles in the flow that will inhibit the formation of cavitation pockets and cushion the effects of their collapse.

Proper design of the rehabilitation works reduces the probability of major problems occurring. Large clear openings are required ; Pinto (1994) recommends a minimum opening of 3 m^2 for a bottom outlet. Radial gates or slide gates are preferred (Lefranc et al, 1994). Trashracks, if they are to be used, should have an opening of 60 % to 90 % of the opening. If hollow jet valves are used narrow trash rack spacing is necessary to prevent the valves themselves from blocking.

At **Holmstyes Dam** in the UK the intake to the outlet works was reconfigured by installing a 90 degree bend to raise the intake 2 m clear of the reservoir bed (Dyke et al, 1998).

The bottom outlet of the **Alloz Dam** in Spain was found to be unserviceable. It was submerged by up to 5 m depth of sediment and to remove it it was necessary to use underwater pneumatic drills, so hard had the consolidated silt become. Once the outlet was exposed it was possible to remove the non-functioning valve and to replace it with a 1500 mm diameter Howell-Bunger regulating valve downstream of the dam. The regulating valve is protected by a guardgate 1200 mm by 1000 mm. The valves are installed in new pipework that is welded to the original pipework through the dam. No alteration was reported to the inlet geometry to prevent the outlet becoming clogged again (Uceda et al, 1996).

5.4.3.1 Operation and maintenance

Clogging of low-level outlets can be averted by sensible operation procedures in which the outlet valves or gates are routinely exercised and the accumulation of debris near the outlet is removed. It is sometimes possible to flush debris through the openings. Emptying the reservoir is not usually an economic option and underwater work by divers is required. Detailed collaboration with specialised divers is required particularly where there is danger of underwater mudslides, or when they are working at depth.

Minimising debris is accomplished by complete removal of vegetation from the bed of the reservoir before it is filled and periodic use of dredgers to remove sediments near the intake and the outlet. It also includes the use of dredgers to remove sediments near the intake of the outlet. Combined with management of the catchment to prevent debris from entering the lake, this can reduce the risk of blocking of the outlet.

Valves and Gates

The rehabilitation of the **Möhne Dam** provides a valuable case history describing the replacement of the low level outlet valves underwater (Campen et al, 1994, Rißler, 1996, 1998). The masonry dam, shown on Fig. 5.4.1, is of the Intze type and has four bottom outlets each of 1400 mm diameter with a capacity of 23 m^3/s. Each outlet has the same fundamental design, shown on Fig. 5.4.2.

The two upstream wedge gate valves in each outlet are controlled through a dedicated valve tower. Water enters the valves at the base of the towers through inlet culverts as shown on Fig. 5.4.2. It passes through 1400 mm diameter pipes within the dam to two further valves. In the outer two outlets these are needle valves, while in the two outlets nearer the centre of the dam sluice gates control the flow into vertical nozzles in the downstream compensation lagoon.

The reservoir plays an important role in augmenting the flow in the River Ruhr for the supply of water to 5 million people. The outlets were replaced, even though no failure had occurred, to prevent strategic damage to the water supply system should they fail. To avoid taking the reservoir out of service the work was done underwater.

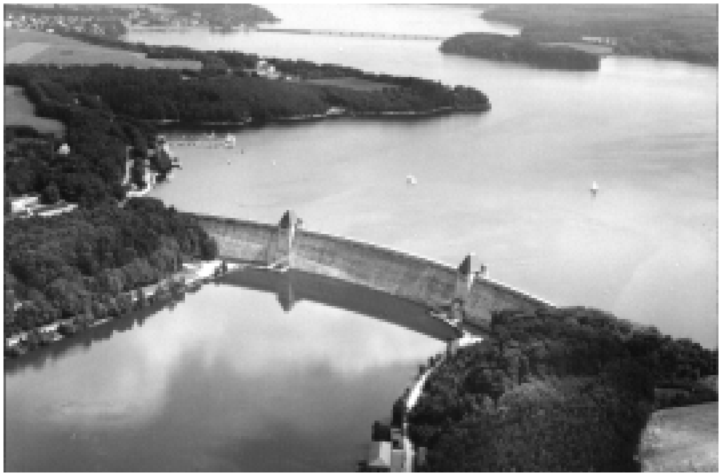


Fig. 5.4.1

Möhne Dam

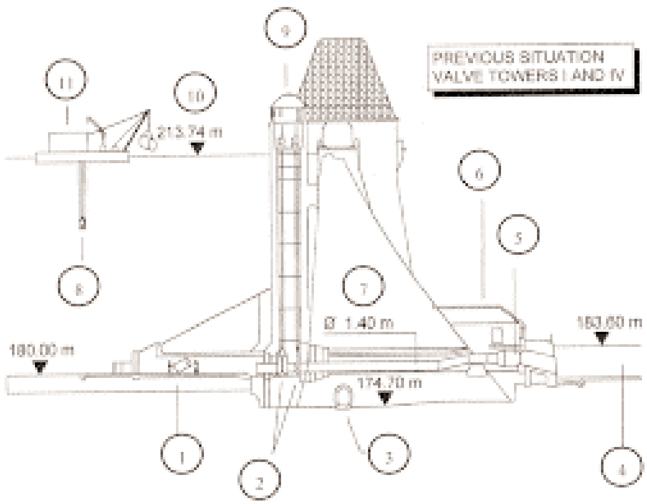


Fig. 5.4.2

Möhne Dam. Valve tower and bottom outlet

1. Installation of steel pipe
2. Old valves
3. Control gallery
4. Compensation lagoon
5. Needle valve
6. Valve house
7. Conduit
8. Entrance for divers
9. Valve tower
10. Normal water level
11. Diver's pontoon

It was not possible to arrange emergency gates upstream of the valve towers without considerable work. Inspections by divers showed that the culverts under the Intze wedge were in poor condition. It was decided therefore to rehabilitate the masonry culverts and to provide a support for the emergency gates by installing steel liners within the culverts. The liners are 14.2 mm thick and are embedded in concrete, an operation that required considerable ingenuity to accomplish. The downstream ends of the liners are wrapped in deformable material to allow relative movement. The work was all done by divers working at a depth of up to 35 m.

Following the wartime damage, two of the outlet culverts under the Intze wedge were strengthened with concrete as shown on Fig. 5.4.3. The design of the culvert liners had to take account of the narrow entrance. Divers manipulated the pipe sections so that the spigots engaged and the joint was tightened against the o-ring using a winch operated from the surface. Final adjustment was made by screws in plates welded to the pipes.

In the first stage of the work all four culverts were lined and backfilled with concrete. Two of the bottom outlets had to remain serviceable at all times. Fig. 5.4.4 shows the factory installation of a pipe. Fig. 5.4.5 shows an end piece of pipe with buoyancy attached to simplify the manipulation of the pipes underwater. Fig. 5.4.6 shows the concrete work and Fig. 5.4.7 shows one of the emergency gates.

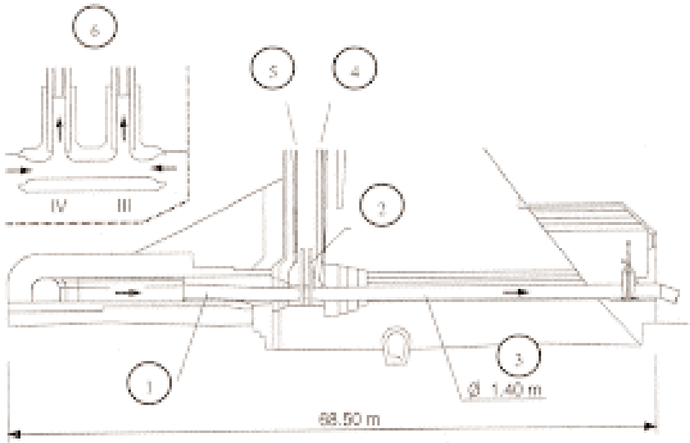


Fig. 5.4.3

Möhne Dam. New valve (bottom outlet III)

1. Relining bottom outlet tunnels
2. New hydraulic sluice gate
3. New conduit
4. Vent pipe
5. Glass fibre reinforced plastic pipe
6. Hammer-shaped entrance tunnels to bottom outlet

Fig. 5.4.4

Möhne Dam. Test installation

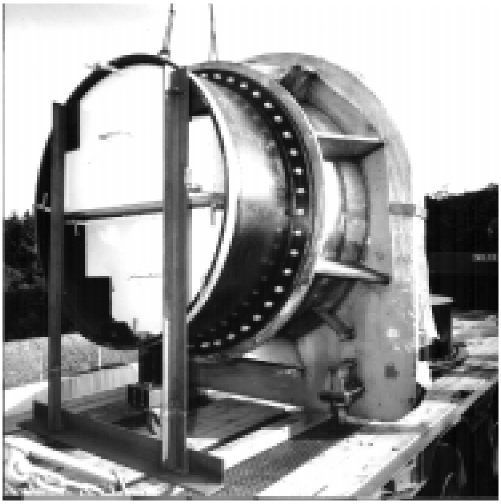


Fig. 5.4.5

Möhne Dam. End piece with flotation

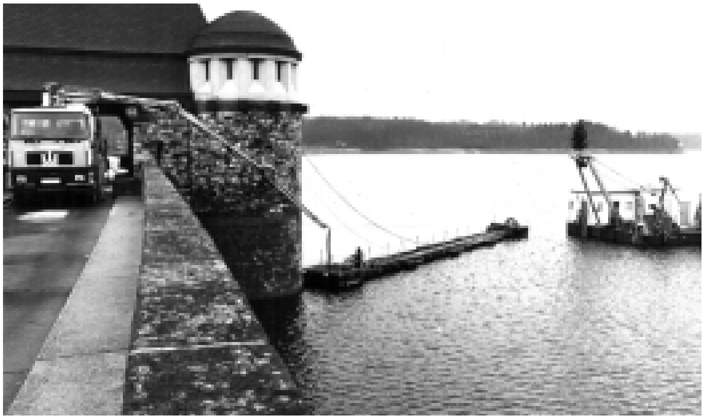


Fig. 5.4.6

Möhne Dam. Backfilling of pipes

Fig. 5.4.7

Möhne Dam. Emergency gate

The diving work was planned carefully. A special platform was constructed, shown on Fig. 5.4.2, in co-operation with the divers themselves. A steel pipe extends 9 m below the support boat. The water in the tube can be displaced with compressed air, allowing the diver to get out of the water under pressure but dry. An elevator takes him to the pressure lock from which he moves to the decompression chamber. For emergency use there is a mobile pressure chamber that allows the diver to be transferred to hospital in less than 30 minutes.

Each valve tower was lined with glass fibre reinforced epoxy resin shells backfilled with concrete. Two new massive hydraulically operated sliding gates in each tower replaced the originals. Both gates can operate against full reservoir head and close against flowing water. An opening in the pipe between them allows the tower to fill with water.

The 80 year old oval body wedge gate at the downstream end of two of the outlets were in good enough condition to be retained, only replacing the operating gear. The needle valves in the other two outlets were found to be too badly damaged to repair.

Other case histories for the rehabilitation of low level outlets include :

Nature of Rehabilitation	Dam	Reference
New low level outlet	Dobra, 53 m high, 234 m long, VA, built in 1953, Austria	Heigerth et al, 1994
New low level outlet	Möll, 93 m high, 164 m long, VA, built in 1952, Austria	Heigerth et al, 1994
New low level outlet	Hochwurten, 50 m high, 260 m long, TE, built in 1978, Austria	Heigerth et al, 1994
New high pressure gates	Tase, Japan	Sasaki, 1992
Renew gates	Dreilagerbach	Anon, 1994
Repair and redesign of the gate system	Picote, Portugal	Lemos, 1973
New gates installed : concrete added for stability	Muslen, Switzerland	Wullimann et al, 1985
New bottom outlet operating system	Rempen, Switzerland	Zurfluh, 1985
New bottom outlet system	Rhodannenberg, Switzerland	Zurfluh, 1985
Outlet geometry improved, steel lining, crack monitoring	Miranda, Portugal	LNEC, 1980
Stainless steel liners anchored to the masonry	Roseires, Sudan	Denny, 1998
Repairs to granite masonry	Sennar Dam, Sudan	Sims, 1998
Modification of valve design to improve passage of debris	Tolla, France	Lefranc et al, 1994
Excavation of sediment above low level outlet	Alloz, Spain	Uceda et al, 1996
Divers installed temporary bulkhead under 35m water	Infierno, 46 m high, Spain	Valenzuela B, 1994
Divers installed temporary bulkhead under 51m water	Guadalhorce, ER, Spain	Valenzuela B, 1994
Following laboratory study, cavitation repaired by high strength concrete	Sanmexia, 106 m high, ER, China	Zhang Jinsheng, 1994

5.5 REHABILITATION OF SPILLWAYS

5.5.1 Introduction

The safety of the dam depends upon the effective functioning of the spillway during emergency. Hence regular examinations and thorough maintenance are essential if failure of the structure is to be prevented and costly repairs are to be avoided.

Seyhan Dam in Turkey, built in 1956, is an example of erosion that became so serious as to threaten the safety of the dam itself (Orhon, 1994). The dam was designed with a service spillway with a terminal flip bucket. When the spillway discharged small amounts over long periods during its first years of operation, a hydraulic jump occurred on the spillway chute channel causing severe structural damage. Clogged drainage underneath the spillway slab contributed to the generation of uplift pressures at the damaged area.

A second erosion mechanism was also observed. The water spilling over the flip bucket at low flows eroded the riverbed just downstream the spillway and endangered the foundation of the structure.

5.5.2 Improving Flood Capacity

Over time the estimates of maximum floods have increased and the required spillway capacity has similarly increased. It is not the purpose of this Bulletin to discuss the determination of greatest flood that must be accommodated, but we are concerned with the methods of increasing the ability of the dam to sustain floods. The flood capacity is often related to the Probable Maximum Flood (PMF) which is the resulting flood when the Probable Maximum Precipitation (PMP) is routed through the catchment. The assumptions used to determine the design flood are continually being questioned.

Faced with the requirement to make their dams safe in conformity with current practice, owners are increasing the flood capacity of their dams. The common themes that emerge from a study of the available modern case histories show that the following work is typical :

- dams are raised to contain the higher water level ;
- greater stability measures are required to accommodate higher flood water levels, such measures include cable anchors and mass gravity structures ;
- better data collection methods to give advanced warning of adverse conditions and to monitor the response of the dam and reservoir.

Other possible methods of dealing with a requirement to pass higher flows include :

- reducing the reservoir operating level to increase flood storage ;
- the construction of additional (secondary) spillways ;
- the modification of catchment flood characteristics by constructing flood detention devices or even an upstream dam ;

- strengthening the crest and downstream face of the embankment to allow some overtopping (Hewlett et al, 1987).

Modern flood discharge requirements are sometimes significantly greater than those adopted in the original design. Rigorous modern principles require the acquisition of much additional basic data and the solution of many site-specific problems, in relation to hydrology, dam stability, mechanical and electrical equipment and control regulations. Cascades of dams are common in some countries including the United Kingdom. It is usual to consider the effect on the whole valley of revised design floods. It may be possible to make economies by increasing storage and attenuation at one reservoir, thereby avoiding enlarging spillways at those downstream.

Particular care must be taken with gated structures that adequate provision has been made to ensure that gates may be opened in sufficient time during a flood event when normal power may not be available. Operator training is important in achieving a safe response to a major flood. Communication systems have often failed in an emergency emphasising the need for effective training to ensure that operators can work effectively in isolation.

Examples of dams where spillway upgrades have been required are listed below.

Coolidge Dam, USA, is a multiple-dome reinforced concrete dam 76 m high, high hazard, modified 1992-1995. Investigation showed that the dam was unable to pass the PMF without overtopping. During the design flood the left abutment proved to have marginal stability. It was considered probable that the already deteriorated spillway chutes would fail during high flows. There was inadequate instrumentation to assess behaviour.

An early warning system was installed in April 1991. It comprises automatic telemetering to the Coolidge Control Centre of flow, deformations and high piezometric levels. The data can also be transmitted to USBR's Denver Office. The left abutment was stabilised using a massive gravity buttress and extensometers were installed to monitor the subsequent movement. The abutment was protected from overtopping with a reinforced concrete cover. The spillway chutes were rehabilitated (USCOLD 1996).

Theodore Roosevelt Dam, USA, is a thick-arch masonry dam 85 m high, completed in 1911, high hazard, modified in 1991-1996. The passage of the revised PMF would overtop the dam by up to 5 m, with damage to both spillways and the 36 MW power station at the toe. Reservoir releases would also overtop three major downstream dams. The existing dam was raised 23.5 m using a single-curvature arch configuration with massive thrust blocks on both abutments. The total volume of the dam was increased by 270 000 m^3. Two 6 m by 9.1 m top-seal radial gates on each abutment provide a spillway capacity of 4300 m^3/s (USCOLD 1996).

Carron Dam, UK, is a masonry gravity dam (and earthfill) 13 m high, completed in 1938 and modified in 1985-87. The central gravity section was found to require rehabilitation because of high uplift pressures. The overflow capacity was also inadequate. Fissure grouting was conducted beneath gravity section and the gravity section was also grouted. Pressure relief drains were installed and the overflow and adjacent abutment sections of gravity dam reshaped with reinforced concrete, 58 rock anchors were installed.

Shepaug Dam, USA, is a concrete gravity dam 43 m high, completed in 1955 and modified in 1988. It was classified as a high hazard because of its likely effect if it were to fail. It was determined to be unstable at a revised PMF : The new PMF had an outflow of approximately three times the original design flow with a flood elevation 3 m over the non-overflow section. Approximately 100 post-tensioned anchors were installed in the dam (USCOLD 1996).

5.5.3 Protection of Abutment

The abutments of a dam may also need to be strengthened to resist the higher water loads associated with a modern revision of the required spillway capacity. The consequences to the valley slopes of the overtopping water must be considered.

The repairs at **Gibson Dam** (USCOLD 1996) described below are typical. This is a gravity-arch concrete dam 61 m high in the USA, completed in 1929 and modified in 1982. It is classified as a high hazard dam. Flooding on June 8 and 9, 1964 overtopped the dam. Overtopping exceeded 1 m over the parapets and lasted for about 20 hours. The intent of the modification was to allow safe overtopping of the dam during floods greater than the 1 % annual probability event. The modification included preparation of abutments, installation of rock bolts, and placing concrete caps to protect the rock downstream from the abutments from detrimental erosion during overtopping. In addition splitter piers along the downstream parapet provided aeration during over-topping and minimised the risk of damage from unstable flow.

Morris Shepherd Dam, USA, is flat-slab-and-buttress concrete and embankment dam, 47 m high, completed in 1941 with a high hazard classification. It was modified in 1988. Evidence of downstream movement of the main dam was identified in 1987. The horizontal movement, which was measured to be approximately 110 mm, was found to originate from a slide along a weak seam in the clay shale foundation. Further investigations also found high uplift pressures in the impervious and weak foundation rocks. Cracks were found in the transition beam along four bays that allowed water to enter the foundation. Both the concrete and the embankment sections would be overtopped during the peak of the PMF.

Grouting of cracks and other parts lead to a reduction in seepage and a decrease in the piezometric pressures. Ballast was added to provide additional weight and the height of concrete core wall in the embankment increased. A new emergency spillway was constructed and the service spillway was reconditioned (USCOLD, 1996).

5.5.4 Debris in Spillways

Log Booms. Godtland et al, 1994, have carried out a study of the conditions under which drifting trees will clog an overflow spillway and to determine the anchor forces required to hold back tangles of trash from reaching the dam crest.

Remodelling the Spillway. Where the design of the spillway is such that it does not easily pass the volume of floating debris in the reservoir it is appropriate to redesign the structure. Openings must be ample. Flap gates are appropriate for passing large floating debris. Radial gates perform well in this regard.

An example is **Palagnedra Dam** in Switzerland. This is a 72 m high arch-gravity dam 120 m long built in 1950 to 1952. It forms a fjord-like reservoir over 2 km long. The spillway was situated at the crown of the dam, and comprised 13 openings each 5 m by 3 m. The intermediate piers serve as supports for the bridge of a main road. The design capacity of the spillway of 450 m^3/s was based on flow measurements from 1927-1935, which showed a peak value of 325 m^3/s. This was found inadequate during the construction of the dam and during the following years when floods were measured that exceeded the design flood. For instance in 1975 the flow was 700 m^3/s. On the 7 August 1978 enormous damage was caused in the surrounding area by an exceptional flood. The inflow to the reservoir was 900 m^3/s. The water level in the reservoir exceeded the permitted value by about 2 m, reaching the roadway of the bridge passing over the dam. A large quantity of floating timber was carried into the reservoir and there to the dam where the tree trunks got stuck in the spillway inlets, and formed a barrier. This is shown on Fig. 5.5.1. As a result the water level rose to 3 m above the road bridge. Because of the sudden breaching of one or more natural dams of driftwood which had formed in the river bed upstream of the reservoir a peak flood wave at the entrance to the reservoir of over 3000 m^3/s and an overflow of 1800 m^3/s to 2000 m^3/s were estimated.

Fig. 5.5.1

Palagnedra Dam. Crest with up to 3 m cover of wood

After this event the discharge capacity of the spillway was reassessed. Analysis of the 37 years of records showed the 100 year flood to be 1050 m^3/s and the 1000 year flood to be 1800 m^3/s. Because of the possibility of a flood wave following the collapse of a temporary dam upstream of floating timber, the design flood was set at 2200 m^3/s. The two wing walls of the spillway were raised by 4 m and the bridge over the dam was replaced with a new bridge with 60 m clear span 25 m downstream. The maximum permitted reservoir level was raised by 3.25 m (Zurfluh, 1985). The new design is shown on Fig. 5.5.2.

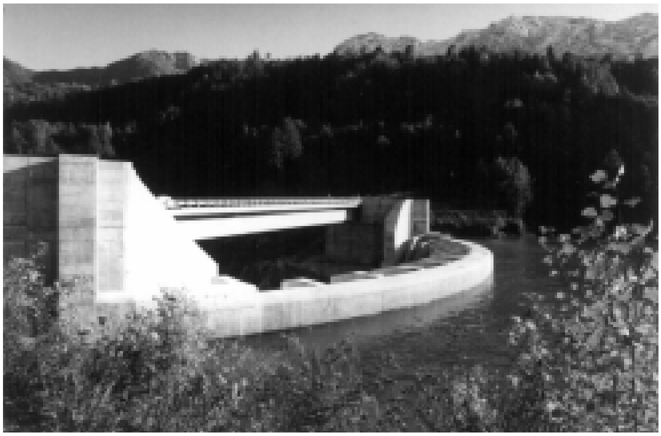


Fig. 5.5.2

Palagnedra Dam. New crest with nappe interrupter

5.5.5 Operational Issues

The operation of gated spillways is complex and demanding. CFGB (1998) gives useful advice on the assessment of spillway safety and on the key items to be monitored in a surveillance programme.

*Simplification***.** It is generally worthwhile to reconsider the whole operating philosophy of the structure instead of doing partial repair work. At **Ohmatazawa Dam** in Japan (Egawa, 1994) fundamental renovation design has successfully incorporated cost-effective and lasting repair techniques and materials into the structural upgrading work. It has been possible to compensate for some of the renovation cost through a considerable reduction of future operation and maintenance costs. The dam, built in 1916, is a concrete gravity structure 18.5 m high with stone-pitched facing. With the elapse of about 70 years since its completion, scouring of the downstream apron, wear on the spillway gates, and deterioration elsewhere had become apparent.

In the renovation the spillway was redesigned to operate without gates. This was done for ease of future operation and maintenance and was done following an economic analysis of alternative solutions taking full account of the operational requirements of the plant.

Not only because of the abrasion damage, but also because of the need to provide additional security against the effect of earthquake it was decided to add concrete to the downstream face. Extensive testing and thermal analysis was done to determine the distribution of shear stress of the joint between the new concrete and the existing dam body, and how this relates to the expected strength. The original stone-pitched facing, removal of which would greatly extend the repair work period, was retained as far as possible to minimise the disruption in power generation operation due to renovation construction. Pumped concrete was used and the mix was carefully optimised to facilitate the placing of the concrete while ensuring its long-term strength. Fig. 5.5.3 and 5.5.4 from the original paper show the dam before and after the renovation.

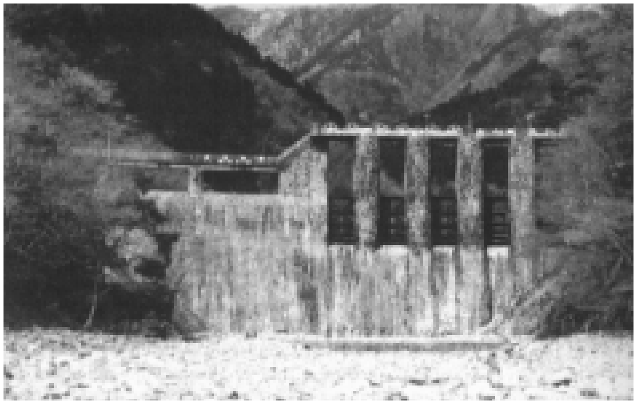


Fig. 5.5.3

Ohmatazawa Dam. Downstream view, before renovation

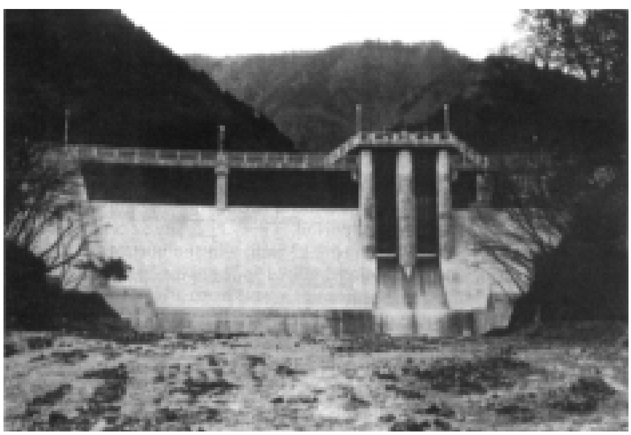


Fig. 5.5.4

Ohmatazawa Dam. View of rehabilitated dam

5.5.6 Spillway Chutes

Spillway chutes are subject to damage from cavitation, erosion and abrasion. Inadequate wall height can lead to overtopping during extreme events and this water may undermine the chute or cause other damage. As the repairs for erosion and abrasion are similar to outlets and dissipating structures, often without the difficulties of working under water, this section deals with damage due to cavitation. A useful introduction is provided by Pinto (1994).

The principal defence against cavitation damage is to provide for smooth flow with no areas of reduced pressure. However at high velocities very small construction defects or slight service damage can initiate cavitation. To minimise the possibility of damage, aeration is provided. The small air bubbles in the flow can expand to reduce the pressure loss at irregularities and can provide a cushion for the impact jets if cavitation nuclei do implode.

The design of an aeration system requires answers to three questions :

- The location of the first aerator
- The volume of air to be entrained at the aerator
- The spacing between aerators to maintain a given protection level.

The answer to the first question is a cost balance. The higher quality of concrete placing and surface finishing, the lower the risk of cavitation damage. With a smooth surface, the first aerator can be placed further downstream, and fewer might be needed, but the construction cost is higher. Gradual deviations of the order of 1 mm in 1 metre can be achieved, but require exacting control and skilled workers. The designer should apply such a tolerance only where it is really needed, and this is likely to be where the velocity is high. A problem with surface tolerance, which is often overlooked, is how it is to be achieved. On flat work or sloped and curved surfaces that can be trawled, overworking the concrete during placement can give a better tolerance by removing undesired irregularities, and provide what at first appears to be a better finish. However overworking will result in a softer surface, with a greater potential for scaling, and micro-cracking, which often develops into surface crazing. Grinding to remove excess undulations and abrupt offsets can also be detrimental because it removes the matrix above aggregate particles that tends to hold them in place. These can be plucked out more easily than those with cement matrix over them. The problem is less severe when angular crushed aggregates are used.

Grinding leaves an initially polished surface, while the surrounding surface has a formed or hand-finished texture. Water flowing across such an area at high velocity will change suddenly from one boundary condition to another and back again. With time, a sandblasted and coarse texture has been seen to develop downstream of ground areas. This appearance is typical of the first stages of cavitation damage.

Questions about the amount of air entrained at an aerator and the spacing between aerators along the spillway chute are directly related to the performance of the device proper. Generally, where velocities exceed approximately 25 m/s to 30 m/s, cavitation may be a serious problem. For top quality smoothness, velocity up to approximately 35 m/s may be accepted without the need for aeration slots. However, there is no guarantee that the quality of surface finish will last. Therefore, installation of aeration provisions for cavitation damage control is prudent because the cost of installation is small when compared with the cost to repair a severely damaged spillway chute.

Ramps or steps inserted on the chute surface are the simplest and the most practical devices used to cause natural aeration of water flow near a solid boundary. The sudden discontinuity in the bottom alignment creates an air-water interface along which the high-velocity water drags air in an intense mixing process. A transverse gallery or a recess can be added in order to improve conditions for air admission to the cavity below the jet, and to improve aerator effectiveness.

5.5.7 Dissipating Structures

Significant damage to piers and aprons in the **Chaudiere Ring Dam** in Canada (Mirza et al, 1994) was the result of continued abrasion. This was caused by the erosive nature of the silt-laden flow passing through the gate openings. This sluicing was necessary to remove sediment from the reservoir and was a routine aspect of the operation and maintenance of the control dam. The damage on the downstream side of the piers is similar to the damage found on the aprons ; the depth of the abrasion varied from about 25 mm to 100 mm.

Improper operation of the spillway may lead to severe scour due to return currents, as illustrated in **Ujjani Dam** in India (Kulkarni et al, 1994). The spillway had operated almost every year during the last 13 years since completion in 1980. Although most of the discharges have been relatively small compared to the design outflows the operation of the spillway has been impaired because the operators did not follow the published operation schedule of the gates. They operated gates on one side of the spillway only.

There was a reverse roller action bringing debris into the bucket resulting in considerable damage by erosion of the bucket, teeth, apron, and downstream riverbed. The bucket was dewatered and inspected immediately after the severe flood. Reinforcement had been exposed at the damaged areas of bucket concrete, the teeth, and the apron downstream of the roller bucket. The cladding plates on the lip of the main bucket and on the subsidiary apron had broken away in several locations.

Improved Operating Techniques

The severe erosion by abrasion of hydraulic-jump-type stilling basins has caused many dam owners to perform underwater inspection and removal of debris as a routine maintenance procedure. For small stilling basins it may be practical to provide basin covers and stoplog slots to facilitate dewatering, inspection, cleaning and repair.

Outlets should be operated to produce symmetrical flow within the stilling basin, and, to the extent that it is practical, flow patterns that produce back eddies into the basin should be avoided. The owner is encouraged to restrict public access to the vicinity of the stilling basin to prevent people throwing rocks and other debris into the basin. Consideration should be given to the use of configurations that tend to be self-cleaning under operating conditions. Designers should include self-cleaning characteristics along with other considerations in selecting basin types. The design of stilling wells should include covers, gratings, and sills to prevent the entry of rocks.

Design measures that should reduce abrasion damage include paving the transition length of the outlet channel, where the water flows from the stilling basin to the outlet channel. It is recommended that end sills be designed carefully to prevent the recirculation of abrasive materials and to encourage them to sweep from the basin. A hydraulic model is useful to determine that this is the way the basin will function.

Rehabilitation Measures

The objective of rehabilitation work is to recover the original function or integrity of the damaged parts. Usually the work is carried out as soon as practicable. Repair work of the **Ujjani Dam** for example was carried out during the following dry season. Exposed reinforcement in the bucket concrete and teeth was brought back to shape by welding in place new reinforcement. The cladding plates were put back in the original position and anchored.

A common theme in the rehabilitation of eroded structures is the replacement of the eroded concrete and its replacement with high strength material. Often a relatively thin layer of concrete is used that is usually reinforced and also anchored to the sound concrete beneath. Important issues include the minimum thickness of the repair concrete and the most effective way of attaching it to the undamaged material. In particularly severe conditions engineers have chosen to use erosion resistant liners such as stainless steel and blocks of resistant igneous rock as referred to earlier in this Chapter.

Repair work can be carried out either in the open or underwater, depending upon the circumstances, the nature of the damage, the program and the funding available. Underwater repairs have the considerable benefit of allowing the damaged project to continue to operate while the repair is carried out. However the cost and complexity of the work is increased. It will always be appropriate in considering the alternatives to carry out an economic analysis that takes account of the value of the product of the project, be it water for irrigation or electricity. It will also be necessary to carry out a thorough risk analysis too. One case history is described below for each approach.

Rehabilitation after Dewatering

Repair work in the **Seyhan Dam** spillway needed to be carried out while the spillway was not operating (Orhon, 1994), and therefore had to be completed before the arrival of the next flood. The damaged concrete was excavated, cleaned and steel mesh reinforcement installed and high dosage concrete was placed to form a replacement concrete slab. The concrete slab was provided with an effective anchorage.

Underwater Repair

The **Potomac River Dam No. 5** is a 6 m high stone masonry dam, built in 1836 in the USA (McClain et al, 1994). In 1991, cavernous voids undermining the dam were discovered during an underwater inspection. Although water flow could be diverted away from the dam by cofferdam work, a significant tailwater pool at the dam toe still existed and the damaged area was under as much as 6 m of water. It was necessary therefore to dewater the stilling basin to carry out a comprehensive assessment of the extent of damage and the need for repair of the concrete surface. The voids were formed by the erosive force of the Potomac River continuously flowing over the dam crest and impacting along the downstream dam toe. The geological setting of the foundation bedrock had also an important bearing on the void formation. The loose and weathered rock underlying the dam was progressively undercut and eroded away. The voids destroyed the structural support for the dam, and if left uncorrected could have precipitated a sudden, localised collapse of the dam.

The repair work was performed underwater to avoid the expense and time required to construct a cofferdam followed by dewatering. Furthermore, dewatering could possibly have increased the potential for collapse by causing further disturbance to the dam. The repairs consisted of filling the voids with concrete pumped from the dam abutment staging area through a slick line that was placed underwater along the toe of the dam. Sketches showing repair of a typical void are shown on Fig. 5.5.5. The backfill concrete was anchored into the bedrock and also grouted at the dam base interface. The engineers responsible concluded that pumped concrete placement offered more versatility than other methods for underwater repairs.

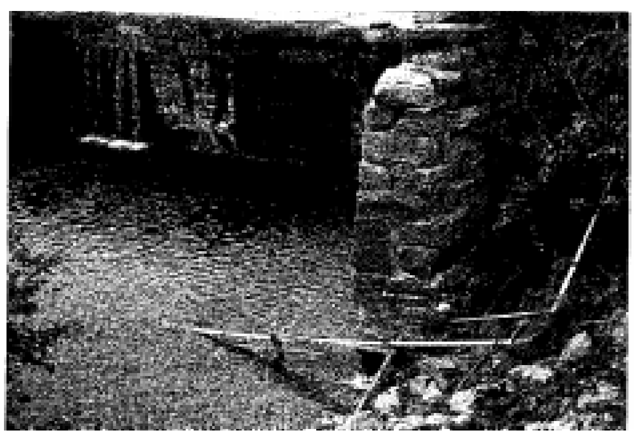


Fig. 5.5.5

Potomac River Dam. Deterioration and concreting

Viluiskaya 648 MW hydro power project was constructed in the Russian arctic in 1978 (Belladir, 1999). The associated dam is a 75 m high rockfill embankment 600 m long. The spillway chute is lined with concrete panels 8 m square and 1500 mm thick, anchored to the bedrock. During a flood approaching the design flood undercutting of the foundation by about 2 m occurred and four of the panels were badly damaged. The damage to the concrete was reported to have been caused by settlement over the eroded foundation together with the effect of the turbulent flow in the spillway and the abrasive effect of the entrained sediment. The selected rehabilitation was to fill the eroded hole in the foundation with concrete and to replace the damaged slabs.

The area to be repaired was under 10 m of water whose temperature was about 2°C. The concrete was placed underwater using a tremie pipe. The concrete mix chosen is as follows :

cement	450 kg
sand	960 kg
coarse aggregate	810 kg
water	200 litre
accelerating additive	11.25 kg
plasticiser	900 g
water-cement ratio	0.44 maximum
slump	180 mm to 200 mm

This mix proved workable ; after 30 to 45 minutes the slump had reduced to 140 mm to 150 mm. The temperature of the mix was 10°C on leaving the plant. Placed at about 18 m^3/h, with an air temperature of about 6°C the temperature of the slab reached a maximum of 40°C after 4 days. The concrete has subsequently performed well. Trial grouting with cement revealed a concrete slab with no cavities.

Other case histories for remedial measures for spillways include

Nature of Rehabilitation	Name of the Dam	Reference
New spillway construction at the crest	List, Switzerland	Wullimann et al, 1985
Compressed air caissons, repair to abraded areas by PVC, copolymers, epoxide mortar	Gezhouba, China	Zhang Jinsheng, 1994
Concrete apron and stilling pond	Pollaphuca, Ireland	O'Tuama et al, 1989
Concrete apron and stilling pond	Golden Falls, Ireland	O'Tuama et al, 1989
Concrete apron and stilling pond	Leixlip, Ireland	O'Tuama et al, 1989
Concrete apron and stilling pond	Inniscarra, Ireland	O'Tuama et al, 1989
Concrete apron and stilling pond	Carrigadrohd, Ireland	O'Tuama et al, 1989
Concrete apron and stilling pond	Cliff, Ireland	O'Tuama et al, 1989
Concrete apron and stilling pond	Cathaleen's Falls, Ireland	O'Tuama et al, 1989
Concrete apron and stilling pond	Ardnacrusha, Ireland	O'Tuama et al, 1989
Concrete apron and stilling pond	Parteen Weir, Ireland	O'Tuama et al, 1989
Weighting of spillway slab to resist uplift	Kotri, Pakistan	Padgett et al, 1998
Design and construction of bellmouth spillway	Rhodannenberg, Switzerland	Zurfluh, 1985
Extreme cavitation and erosion damage on both tunnel spillways	Glen Canyon, 216m high, USA	USCOLD, 1996
Spillway flipbucket destroyed by cavitation	Karun I, Iran	Fouladi, 1994

5.6 REHABILITATION OF GATES AND OTHER DISCHARGE EQUIPMENT

5.6.1 Introduction

Mechanical and electrical plant is an essential component of a dam project. Indeed it could be argued that this equipment provides the essential control for which the project was built. However the life of mechanical and electrical plant is usually no more than about 30 or 40 years. Therefore its rehabilitation plays an important part in the rehabilitation of dams. The civil works can be realistically expected to have a useful life of well over 100 years with appropriate maintenance and we can expect that the mechanical plant will be rehabilitated more than once in the life of the project. The problems experienced by hydraulic equipment that limit its life are caused by the combined effects of corrosion, erosion and poor maintenance. Corrosion protection systems have proved inadequate in many instances, notably to the equipment itself, but also to the lifting and control devices.

Rehabilitation may be appropriate before the gate or valve itself needs attention. Vibration of these items of equipment has been reported and is the subject of specialist texts ; see for example ICOLD (1996). Self induced opening of gates has been reported in computer-controlled systems (Rojar et al, 1994). The incidents with gates and valves highlight the importance of testing, maintenance and good operating procedures.

5.6.2 Detection and Monitoring

The need for rehabilitation is frequently indicated by poor performance : gates and valves fail to operate effectively, becoming jammed or vibrating badly (ICOLD 1994). Loss of stiffness and structural failure is a threat when corrosion reaches an excessive level. The Table below is typical of measurements on the 40-year-old gates at **Kotri Barrage** in Sindh Province in Pakistan. The gate is of riveted construction with the skin plate supported by two bowstring girders and it was clear from visual inspection that considerable loss of metal had occurred. The Table shows the measured percentage thickness of steel remaining (Padgett, 1998) as an average for the gates. Measurements were made with an ultrasonic measuring instrument.

Member	Lowest individual gate %	Mean of all gates %
Upper bowstring d/s chord	83	90
Upper diagonal	79	86
Upper bowstring u/s chord	71	78
Lower bowstring d/s chord	42	83
Lower diagonal	32	49
Lower bowstring u/s chord	50	78

At **Möhne Dam** in Germany (Campen & Mantwill, 1994) the low level outlet gates were found by inspection to have suffered from graphitic corrosion and to be leaking unacceptably.

Strain gauges and related instruments can provide useful data in some circumstances, particularly where excessive stress or strain has to be quantified.

5.6.3 Effect on Safety and Performance

Failure of outlet gates and discharge equipment will not as a rule lead to the failure of the dam. It will however make it impossible for the structure to fulfil its designed role. It may not for example be possible to divert water for irrigation, power or water supply. The reservoir may not be able to be filled or, more likely emptied under control.

The consequences of poorly operating equipment is that it may not be possible to protect the dam in time of distress. An aim, used in France, is to restore the capacity of low level outlets, so that in an emergency the operators can reduce the pressure on the dam by 50 % in 8 days (Combelles et al, 1985).

5.6.4 Rehabilitation

This case history illustrates the major approaches to the rehabilitation of gates and valves : when the work is done in the air as opposed to Möhne Dam where much of the work was carried out underwater.

Rehabilitation in the Open. The study of the Kotri Barrage gives valuable insight into the rehabilitation of large hydraulic gates. This is the lowest of the barrages on the Indus River. Commissioned in 1955, it provides irrigation water to lower Sindh and supplies potable water to the cities of Karachi and Hyderabad. Although designed for a head differential across the barrage of 9.45 m, this was limited to 7.32 m for several reasons and rehabilitation was needed to remove the stringent operational difficulties. These difficulties were compounded by the realisation that erosion and corrosion were damaging the sluiceway gates, removing up to 15 % of steel thickness annually. The failure of a gate on a barrage immediately upstream, causing the loss of the pond and serious loss of water for irrigation had an important impact on the planning of the rehabilitation work.

There are 44 vertical lift gates at Kotri, each 6.4 m high and weighing 44 tonnes. The skinplate is supported by two horizontal bowstring girders disposed about the centre of pressure. The gates are lifted by manual winding through a spur reduction gearbox to wire ropes and are fitted with a counterweight. The vertical built in parts are of cast iron. The water pressure on the gates is transferred to the structure through stony rollers. The original side seal was by circular steel staunching bars and no bottom seal was provided. The protective coating was red lead oxide covered by a coating of pitch petroleum and lime called locally “khankie mixture”.

The owner carried out annual measurements of metal loss on each sluice gate which confirmed that rehabilitation was needed. The measurements confirmed that the gates were losing steel at a rate that if continued could lead to failure within five years. The Table above shows that the lower girder had suffered more than the upper one.

Five strategies were considered to provide an acceptable life for the barrage.

- Continue with the current maintenance regime.
- Implement a new and modern paint system.
- Strengthen the gates.
- Replace those gates in the worst condition.
- Replace all the gates.

The last option was chosen because none of the others would continue the life of the barrage with sufficient security. An immediate problem for the engineers was that the barrage had been constructed without the means to isolate each sluiceway gate. There was no provision in the original design for stoplog gates. Fig. 5.6.1, shows a view of the upstream face of the dam. Not only were there no stoplogs available, there were no slots provided in the masonry either. It was necessary to devise a method of isolating and dewatering each sluiceway. There were several specific problems to be overcome.

- The masonry facing to the piers is rough, making sealing difficult.
- The bottom seal would have to be within a short distance of the sluiceway gate because the upstream glacis slab had not been designed to resist unbalanced uplift pressure.
- The isolating device would have to be positioned from upstream because there was little access from the road bridge.

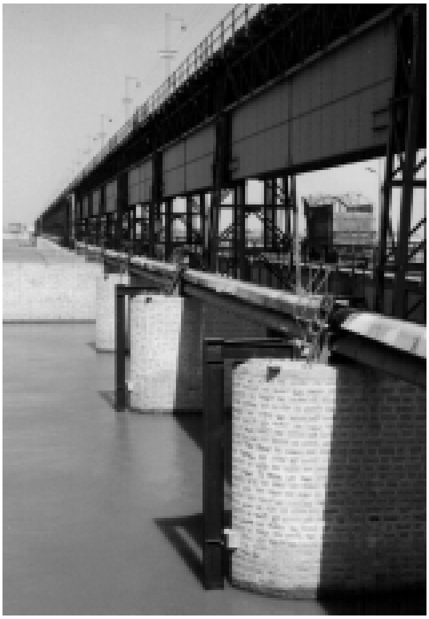


Fig. 5.6.1

Kotri Barrage

- The hydrostatic load from the device must be carried through the barrage structure itself, probably the piers.
- The isolating device must be capable of operating over the full range of upstream water level.

The solution adopted was to design and construct a self-contained buoyant bulkhead gate, shown on Fig. 5.6.2. The bulkhead gate is floated into the space between the piers on either side of the gate. This is controlled using a pontoon and a tug. The pontoon is fitted with side spacers to centralise the bulkhead. The bulkhead is positioned to engage side arms on to load transfer fenders previously positioned on each pier. Once in position the ballast of the bulkhead gate is adjusted so that the gate moves from its horizontal attitude to float vertically. The ballast is adjusted further until the bulkhead sinks on to the sill of the gate opening. Side seals are then extended using hydraulic rams and adjusted against the masonry of the pier faces. Finally the space between the bulkhead gate and the gate to be replaced is pumped dry.

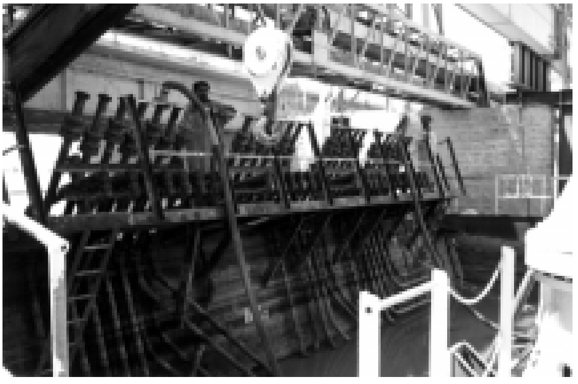


Fig. 5.6.2

Kotri Barrage : Bulkhead gate

The pontoon was used as a service barge for the bulkhead gate, carrying dewatering pumps, air compressors, diesel generators, cranes and other equipment to help to position the bulkhead gate. At Kotri three bulkhead gates were provided with a single pontoon and tug servicing all three. Important aspects of the rehabilitation contract included requiring the contractor to do the detailed design of the bulkhead gate and its associated equipment and also to contribute to the effective training of the owner's staff in the effective use of the equipment.

The rehabilitation of the barrage gates was completed successfully in 40 months. The replacement of each gate was achieved over a fifteen-day cycle. Erection in the dewatered spillway was achieved by bolting together five prefabricated sections and then seal welding the upstream side of the joints. All the elements of the existing gates were replaced including the gate hoist, ropes and stony roller trains. However the existing hoist machinery and counterweights were found to be generally in good condition.

Other case histories for remedial work to gates and valves include :

Nature of Rehabilitation	Name of the Dam	Reference
Modified operation procedure to correct excessive vibration of low-level valves	La Vinuela, 96m high, ER, Spain	Rodriguez A, 1994

6. RESEARCH AND DEVELOPMENT

6.1 THE ROLE OF R&D IN DAM REHABILITATION

Dam engineering has for thousands of years been based on empirical knowledge. Dam building is similar to other fields of construction, extrapolating from successful results, retreating after failure. Scientific analysis is a relative newcomer. The number and size of dams has increased significantly during the last hundred years with the accelerated development of water resources. Even modern dams represent the development of engineering design and construction strategies that have been implemented over a rather long period. For successful rehabilitation an understanding of this evolution is necessary.

Damming water is a continuous struggle. Dams may fail if the flow of water over, or the seepage through or under the dam is not controlled. Dams are an unusual type of construction having to accept large horizontal loads to be redirected and transferred down either into the foundation mainly by means of the weight of the dam or into the abutment by means of arching. Every dam site and foundation is unique and often not sufficiently well known. Each dam has unique topography, hydrology, ecology and geology.

A characteristic of dams is the often catastrophic consequences following dam failure. This puts safety in focus as the ultimate objective for dam construction. Risk management and environmental impact mitigation must be properly integrated in design, construction and operation of dams. Rehabilitation is crucial for restoring fundamental dam performance where there are deficiencies in design or construction or when the dam does not fulfil its required primary purpose.

Dam engineering as a modern technology emerged during this century, particularly after the Second World War. It is based on the disciplines of engineering geology, soil science, soil and rock mechanics, hydrology and hydraulic engineering, structural engineering, construction material technology and earthquake engineering. Principles of design, construction and operation of dam structures have developed parallel with the expansion of knowledge within these disciplines. Important contributions to such development were obtained by means of tests and monitoring of dams in operation and by studies of scaled physical as well as mathematical models. We now have a basic understanding of the physics of the performance of different types of dam, which also provides the theoretical platform for developing rehabilitation technology.

The number of dams that have shown deficiencies in their performance is significant and there is a tendency for an increased rate with ageing. There is a need to develop tools for systematic and qualified monitoring of dams for safe and reliable performance as the basis for planning of their operation, maintenance and rehabilitation.

Development of technology for rehabilitation of dams is a demanding task that must be based on comprehensive and qualified knowledge of a wide spectrum of processes related to dam design, construction and overall performance. Measures taken to restore proper dam functioning are critical in the sense that a successful result has to be assured. That means that we must have full insight into the rehabilitation procedure itself and its effect on dam performance. Evidently research and development of rehabilitation technology has an important role to play in dam engineering. Research in dam engineering has a wider and probably even more important role as quality assurance of educational efforts at our engineering universities as part of the professional training of academic personnel.

A particular approach that can be useful in improving quality in rehabilitation measures is risk analysis. Risk analysis is the use of available information to estimate the risk to individuals or populations. We may assume that rehabilitation is carried out following detection of problems as a result of monitoring and inspection at the dam site. Failure may occur although there was no evidence of a problem at the time of inspection : malfunction may be evident without previous warning during operation. Risk analysis can also be used to assist with the production of refurbishment schedules enabling cost-effective reduction of identified failure modes occurring. It promotes a systematic approach in the management of maintenance, repair and rehabilitation.

Applied research in engineering should be initiated and communicated effectively with the engineering community so that both research environments and practice will benefit. Dam rehabilitation normally follows a safety assessment such as provided by SEED-analysis (Safety Evaluation of Existing Dams). These two linked activities are important for a qualified management of water resources infrastructure. The linkage emphasises the potential benefits of co-operation both on national and international levels between dam owners, contractors, consultants and universities and research institutes engaged in water resources engineering research.

6.2 REHABILITATION OF DAM PROJECTS

Continuing research and development relevant to dam rehabilitation can be classified into three broad areas :

- Research to improve the understanding of the behaviour of the dam and the mechanisms that lead to deterioration and ageing to enable the appropriate remedial works to be carried out. This would include, monitoring instrumentation and observational techniques, investigation and in-situ testing techniques, and both physical and numerical and analytical modelling techniques.
- Research into the magnitude and effects of extreme natural events such as floods, earthquakes and extreme temperatures. This should include the effects of possible climate change.
- Research directed to assessing the performance and reliability of existing remedial methods and developing new rehabilitation methods, including processes and materials.

Each of the above areas is often interrelated. The object of this chapter is to identify where research is being undertaken and where research is required in the future for both the rehabilitation of ageing of dams and newly constructed dams.

6.3 MONITORING AND INVESTIGATION TECHNIQUES

Effective surveillance and monitoring can provide vital evidence of deterioration and unsatisfactory behaviour, and the need for rehabilitation of the appropriate type. The key indicators of dam performance are movement, leakage and pore pressure changes. Numerous types of instrument are currently available to measure these parameters, however, most are discontinuous in terms of both time and space. They can be costly to install, monitor, and maintain, and interpretation can be ambiguous. The level of monitoring and the scale of an investigation following unsatisfactory behaviour or an incident will depend upon factors such as the economic value of the dam, hazard posed by the dam and the size and location of the dam. Developments in monitoring methods need to address the requirements for all types of dam that pose a hazard. To date developments have tended to concentrate on the larger, high consequence dam.

Significant developments in geotechnical instrumentation occurred in the 60s and 70s with emphasis in the 80s and 90s on data acquisition systems. Developments are being made in the use of fibre optics, resistivity, temperature measurements and ground penetrating radar. Improved understanding of dam performance is developing as more realistic computer simulations are made of real dam behaviour under different loading conditions, dam geometry and deficiencies in construction.

The following are considered to be the areas where developments are taking place and could reap benefits, and areas where research is required :

- Developments are required to produce low cost, reliable, maintenance free instruments that can detect anomalies.
- Non-intrusive geophysical methods. There has been a steady development and application of geophysical techniques to geotechnical and ground water studies, and their use in dam investigations has become more common. The developments have mainly been in interpretation resulting from increase in computer power. Interpretation of data can often be difficult or ambiguous. Purposes for which techniques have been developed include :

    a) detection of leakage path

    b) determining the depth of the phreatic surface

    c) identification of localised features, voids and defects within the dam

    d) measurement of embankment properties and their spatial variation.

- Developments of in-situ testing methods, such as the piezocone to determine geotechnical properties of embankment fill have increased in application in recent years, particularly in the cores of dams. Increasing concern particularly in the seismic stability of some older structures may lead to greater emphasis on measuring the in-situ geotechnical parameters using such methods. Validation of such methods against conventional borehole investigation methods requires continued study.
- Development and implementation of efficient automatic measurement and data management systems, perhaps taking advantage of modern developments in satellite communications (Klebba et al, 1999 ; ASCE, 2000).

6.4 MODELLING OF DAMS AND THE PERFORMANCE OF THEIR COMPONENTS

Physical and numerical modelling are likely to be beneficial with respect to rehabilitation in several areas. Research in these areas will be helpful.

Modelling studies associated with rehabilitation assess the development of the deterioration and support the design and implementation of the rehabilitation measures. The deterioration of a dam may be due to ageing or to the effect of exceptional events. Therefore the models to evaluate the development of deterioration should take into account time effects and non-linear behaviour of the materials. Coupled models are needed in studies of the thermo-mechanical behaviour of concrete and masonry, and the hydro-mechanical behaviour of rock mass, soil, concrete under static and dynamic loads.

Simulation studies are helpful in the design and implementation of rehabilitation measures. Further research is necessary into the structural effects of repair works such as grouting, prestressing or blasting. The interfaces between old and new materials as in a masonry dam repaired with concrete elements, joints between new and old concrete and interfaces between embankment and concrete elements require simulation studies to improve the design solutions.

In-depth studies are needed into the physical and chemical phenomena involved in the deterioration of rock, soil, concrete and masonry. The characterisation and modelling of these phenomena and the assessment of their evolution in the course of time is a topic that requires further research.

The effect of earthquakes on concrete and masonry dams and appurtenant works is the subject of current research. Further guidance is needed on the seismic assessment, dynamic behaviour and rehabilitation of gates and intake towers.

Modelling of the sedimentation in reservoirs and its management and control will become increasingly important.

Physical modelling of the mechanisms of internal erosion and the erosion resistance of the materials used in rehabilitation including grouts and slurries is needed to develop understanding of the phenomena, and to provide guidance for rehabilitation.

Research related to studies of the effect of flood waves following a hypothetical dam break is needed to analyse from a geotechnical point of view the effects of such extreme events on the infrastructure and landscape. This will include prediction of large scale erosion and associated disturbances effecting the progressing flood wave as a basis for mitigation and restoration measures.

6.5 REHABILITATION OF EMBANKMENT DAMS

Certain components of embankment dams are fundamental for the overall function and therefore must have a high degree of reliability. Thus the integrity and stability of each zone of the dam is vital. In particular the water barrier, the filters within zoned dams, the erosion protection of the upstream and downstream slopes of the dam, the downstream drainage facilities, the foundation and the boundary between dam body and adjacent structures laterally are all important. Thus research into the rehabilitation of embankment dams should be directed towards the matters described below.

Research into the behaviour and rehabilitation of internal cores is required in two areas. First, the effects of strain softening, collapse on wetting and liquefaction deserve to be better understood. Second, the development of new methods for securing the watertight element. This might include the development of grouting systems, diaphragm wall methods and materials including deep mixing of soil with cementitious materials.

The feasibility of installing new filters into an existing dam is of interest.

Failures in upstream slope protection are common. The use of asphalt in a range of roles has received attention. Long term assessment of these methods is required, including both the functional and environmental aspects.

Upstream watertight membranes are potentially vulnerable. Research is needed into methods of repairing asphalt and concrete membranes.

A common theme to many of these research topics is the development of new materials. From the perspective of 2000, there appears to be scope for the introduction of further high performance materials including resins, geomembranes and perhaps others.

6.6 REHABILITATION OF CONCRETE AND MASONRY DAMS

A concrete dam should be founded on a rock mass able to support the forces and hydraulic gradients related to its construction and operation. The strengthening of the foundation includes its consolidation and sealing – normally by grouting – and drainage. The interaction between the dam body and the rock mass defines the relationship between external and internal forces, and the seepage through dam and foundation. The hydraulic gradient across the dam varies with reservoir water level. Hence variations in water level promote a dynamic stress on the system that may initiate cracking, deformation and movement of joints in both dam body and foundation. Thus research into the rehabilitation of concrete and masonry dams should be directed towards the matters outlined below.

The control and prevention of chemical reactions within concrete that reduces its strength or durability. An example of the kind of reaction to be avoided in future is alkali silica reaction (ASR).

Particularly older dams show cracking that can be extensive. Research is needed into appropriate methods of repair that will remain durable, even in the presence of acidic water or freezing and thawing conditions. New materials for grouting and strengthening may prove to be appropriate and economic.

Much progress has been reported recently in the use of geosynthetic membranes to seal the upstream face of deteriorating dams, even for the work to be done underwater. It seems probable that further research in this area will reap benefits.

The long term control of uplift remains an urgent area of concern.

Drainage systems are often in need of upgrading and rehabilitation. Pressure relief installations can be blocked, reducing the stability owing to increased uplift pressure. It may be relevant to carry out research into methods of clearing or reconstructing pressure relief systems. By using innovative technology of horizontal drilling and trenchless installations of pipes and drainage systems, a wide field of new applications and new rehabilitation concepts for concrete and embankment dams could be provided. Further research is necessary to overcome the lack of experience with this technology in dam engineering.

6.7 REHABILITATION OF APPURTENANT WORKS

The life span of electrical and mechanical equipment for operation and control of dam projects is shorter by a factor of at least three than the main structural components of the dam. Mechanical wear, environmental impact and especially corrosion are major reasons for this, in addition to poor maintenance. More refurbishment work is required on the appurtenant elements than on the main structure of the dam. Gates and their guides, which appear to be in good condition when inspected in-situ, frequently have unacceptable wear on guide paths or roller trains. There may also be important elements that we cannot readily access to inspect. They can be difficult to inspect without their removal or the installation of cofferdams. Particularly where the design has not provided slots for stop logs, underwater inspection and repair will be necessary and research and development into the practicalities of this will be useful.

Critical elements for the safety of a dam, such as a radial gate, may fail without warning during operation. A failure of this kind exemplifies and verifies the need for detailed assessments of the condition of different components in a spillway system. The operation of gates and valves must be tested regularly. The gates, intake and discharge channels have to accommodate design flows during a certain time interval without causing damage in the spillway system or erosion of the dam. Special emphasis should be given to the performance of older dams in this respect.

Much of the structural damage to appurtenant structures results from the effects of cavitation, scour and abrasion. Research will be valuable with the development of this sort of damage, and of robust and economic ways of repairing it.

There are also other structures associated with a dam that may require rehabilitation along with the main dam structure. These include gate-houses, fish passes and spillway structures which are sometimes independent of the dam. It is common to incorporate enhanced spillway capacity to comply with more stringent flood requirements required today. These are all elements were it may be valuable to carry out research.

The operation of dam projects can be improved by using modern information technology and more efficient components like sensors and signal systems within a comprehensive automatic control system. To improve performance and reduce risk for failures, risk analysis is a valuable tool. There is a particular potential in analysing performance and reliability of systems based on electrical and mechanical components with relatively well defined properties.

6.8 ENVIRONMENTAL IMPROVEMENTS

Environmental views today have both local and global perspectives. Dams in combination with hydropower generation have a benign impact on the global environment and are therefore advantageous compared to fossil energy production that notably produces green-house gases. The local impact has a wider range. Many older structures such as dams and mines are historically associated and provide positive environmental effects with areas for recreation. They are today important landmarks in the development of our society.

Adverse effects on the local environment that can be related to dams might have been avoided with our current knowledge of ecology. This implies a research need to improve the environment around old dams and hydropower plants which is certainly useful also in design of new hydropower schemes.

Research would also be useful with the perceived environmental benefits of existing dams.

7. LIST OF REFERENCES

AISIKS E G, PALLARES J J, TIPPING E D & VARDE O A, 1991. Special Techniques for Corrective Treatment of Right Abutment at EL Chocon Dam. 17th ICOLD Congress, Vienna, Q65 R65, pp 1169 to 1190.

ANDERSSON P M, 1991. Borehole radar as a tool for detecting of dam ageing. 17th ICOLD Congress, Vienna, Q65, Contribution 14.

ARAH R M, 1975. Investigations, problems and remedial works at Withens Clough. Proceedings of BNCOLD Conference on *Inspection, Operation and Improvement of Existing Dams*, Paper 5.5 University of Newcastle-upon-Tyne.

ASCE, 2000. Guidelines for instrumentation and measurements for monitoring dam performance. To be published by Energy Division, Reston VA, USA.

ATKINSON J H, CHARLES J A & MHACH H K, 1989. Examination of erosion resistance of clays in embankment dams. Quarterly Journal of Engineering Geology vol 23, pp 103-108.

ATTEWILL L J S, GOSDEN J D, BRUGGEMANN D A & EUINTON G C, 1992. The construction of a cut-off in a volcanic residual soil using jet grouting. *Water Resources and Reservoir Engineering*, 7th Conference of the British Dam Society, published by Thomas Telford, London.

ATTEWILL L J S & MOREY J, 1994. The use of jet grouting for the cut-off of Thika Dam, Kenya. XIII International Conference on Soil Mechanics and Foundation Engineering, New Delhi, India.

AXTON M, 1996. Willes Meadow Reservoir Wave Protection Works, Scheme No N61-OV-A25 Completion Report, Jan, 1996 Unpublished report by Severn Trent Engineering.

BABTIE SHAW & MORTON, 1992. Glenkin Reservoir Remedial Works in Draw off Tunnel. Private communication to Dumfries & Galloway Regional Council.

BAECKER, E & KOEHNE, H D, 1985. Die Staumauer der Oleftalsperre ; Konstruktion und Ausfuehrung der Ertuechtigung. Deütscher Beton-Verein.

BALDOVIN G, BALDOVIN E, FIAMBERTI A, 1994. Rehabilitation and waterproofing of the upstream facing of Ceresole Reale dam. ICOLD Congress, Durban, Q68 R87.

BEAK C, FINDLAY J W & AIKMAN D I, 1997. Experience of Failure Mode, Effect and Criticality Analysis on UK hydro power schemes. *Hydropower '97*, Trondheim.

BELLENDIR Y N, 1999. Rehabilitation of embankment dam of Kureiskaya Hydro Power Plant. Private Communication.

BERCHTEN A R, 1985. Repairs of the Zeuzier arch-dam in Switzerland. 15th ICOLD Congress, Q 57 R 40.

BINGHAM B, HOLDERBAUM R & SHERMAN W, 1997. Recycling Penn Forest Dam. *Civil Engineering*, ASCE, August.

BILIK J & SATRAPA L, 1998. Reconstruction of the Vir Dam on the Svratka River, Southern Moravia, Czech Republic. INCOLD Symposium. *Rehabilitation of Dams*. 66th ICOLD Annual Meeting, New Delhi, November.

BIRMAN J H, 1996. Thermal Investigation and Monitoring of Leakage through Dams. Proceedings Association of State Dam Safety Officials, Seattle, pp 599-602.

BISCHOF R, 1993. The Efficiency of Repair Work on old Concrete Dams in Switzerland. Technical Symposium on Maintenance of Older Dams, Chambéry, Vol. 1, pp 211-228.

BISTER D, BONNET P, BURDIN J & CHARVOZ B, 1993. Maintenance des Barrages en Béton – Techniques et Coûts. Colloque Technique sur la Maintenance des Vieux Barrages, Chambéry, France.

BISTER D, BONNET P, DENIS B, DE BEAUCHAMP R, DUBOIS P & GOGUEL B, 1991. Contribution au suivi des barrages en béton Français sujets à gonflement ou retrait – application à des ouvrages adultes (Chambon, Vouglans) et au béton jeune (cas du BCR). 17th ICOLD Congress, Q65 R7.

BISTER D, FRY J-J, COSTAZ J, HOUIS J, DUPAS J-M, DEGOUTTE G, LINO M, & RIZZOLI J-L, 1994. Reassessment of earth-and-rockfill dams safety, Case Histories. 18th ICOLD Congress, Durban, Q68 R43.

BOWLES D S, ANDERSON L R, GLOVER T F & CHAUHAN S S, 1998. Portfolio Risk Assessment : A Tool for Dam Safety Risk Management, Eighteenth USCOLD Annual Meeting and Lecture, Buffalo, New York, 8-14 August.

BOWLES D S, ANDERSON L R, GLOVER T F, TARBOX G S, WAITE R B & YIN AU-YEUNG P E, 1990. Evaluation of Dam safety at a series of hydropower dams including risk assessment. *The Embankment Dam*, 6th Conference of the British Dam Society, published by Thomas Telford, London, ISBN 0 7277 1647 6.

BRE, 1991. An Engineering Guide to Seismic Risk to Dams in the United Kingdom. Report BR210, B R E Garston WATFORD UK, ISBN 0 85125 5108.

BRE, 1988. Alkali Aggregate Reactions in Concrete, Digest 330, HMSO London.

BREZINA P, SATRAPA L & SKONAN T, 1998. The reconstruction of the collapsed bituminous concrete facing the Moravka Dam, Czech Republic. Indian National Committee on Large Dams Symposium. *Rehabilitation of Dams*, 66th ICOLD Annual Meeting, New Delhi, November.

BRIGHTON R, & LAMPA J, 1991. Ageing of B C Hydro's dams. 17th ICOLD Congress, Vienna, Q65 R30.

BRONNER N, FAGERSTRÖM H & STILLE H, 1988. Bedrock cracks as a possible cause of leakage in two Swedish dams. 16th ICOLD Congress, San Francisco, Q61 R55.

BRUCE D A, HAMBY J A & HENRY J F, 1998. Tims Ford Dam, Tennessee : Remedial Grouting of Right Rim. Proc of the 1998 Conference, Association of State Dam Safety Officials, Las Vegas, Nevada, pp 823-835.

BRUCE D A & STEFANI S, 1996. Rehabilitation of Beaver Dam. A major seepage cut-off wall. Ground Engineering Vol 29 June, 1996. pp 40 to 45.

BUDWEG F M G, 1998. Some Particularities of Dam Rehabilitation Projects. INCOLD Symposium *Rehabilitation of Dams*, 66th ICOLD Annual Meeting, November, New Delhi, India.

BUSTINZA J A, PUJOL A & SCHEFER J C, 1999. Paso de las Predas Dam – Repair of foundation problems. Symposium *Dam Foundations Problems and Solutions*. 67th ICOLD Annual Meeting, Antalya, 23 September, pp 385-396.

CABIRON J-L, 1996. Evaluation of abrasion resistant materials in hydroelectric structures. From laboratory tests to on site appraisal after 5 years. SWEDCOLD Symposium *Repair and Upgrading of Dams*, Stockholm.

CABIRON J-L & LAVIGNES, 1998. Research on the use of ALAG concrete for hydraulic abrasion and cavitation protection in dam structures. Indian National Committee on Large Dams Symposium. *Rehabilitation of Dams*, 66th ICOLD Annual Meeting, New Delhi, November.

CAMPEN F & MANTWILL H, 1994. Preventative overhaul and refurbishment of the bottom outlet works of an 80 year old masonry dam performed without emptying the Reservoir. 18th ICOLD Congress, Durban, Q71 R3.

CARLYLE W J, 1988. Wave Damage to Upstream Slope Protection of Reservoirs in the UK. Proceedings of 5th BNCOLD *Conference on Reservoir Renovation*, Manchester.

CARR P, 1996. Remedial grouting phase II Red Rock Dam, Pella 1A. *Dam Safety '96*, Association of Dam State Safety Officials, pp 511-518.

CASSIDY J J, 1994. Cavitation problems at Pit 6 stilling basin. 18th ICOLD Congress, Durban, Q71 Contribution 6.

CATANACH R B, HALL R B, O'NEILL A L & VON THUN J L, 1991. Quail Creek Dike Failure, Failure Mechanisms, Redesign and Construction. 17th ICOLD Congress, Vienna, Q66 R58.

CAVALCANTI, A J C T, SILVEIRA, J F A & DEGASPARE, J C, 1991. Alkali-silica reaction and remedial measures at Moxoto powerplant. 17th ICOLD Congress, Vienna, Q65 R8.

CEDERWALL K & NILSSON A, 1991. Description of three deterioration scenarios. *Hydraulic Engineering no 49*. Royal Institute of Technology, Stockholm.

CFGB, 1976. Infiltrations dans les barrages d'Electricité de France, constatations, interprétation et traitement. 12th ICOLD Congress, Q45 R20.

CFGB, 1994. Réévaluation et amélioration de la sécurité des barrages en maçonnerie et en béton. 18th ICOLD Congress, Durban, Q68 R44.

CFGB, 1998. Practical guidelines for improvement of dam safety during floods. CFGB, Barrages et Réservoirs, number 8, June. See also summary in 20th ICOLD Congress, Beijing, Q79 R29.

CHAO P C, 1980. Tarbela Dam – problems solved by novel concrete. Civil Engineering, ASCE, December, pp 58-64.

CHARLES J A, 1989. Deterioration of clay barriers. Case histories. Proceedings of ICE Conference on Clay Barriers for Embankment Dams, London.

CHARLES J A, TEDD P & HOLTON I R, 1995. Erosion in clay cores of British Dams, *International Conference on Research and Development in the Field of Dams*, Swiss National Committee on Large Dams, Crans Montana, September, 1995.

CHARLES J A, TEDD P, HUGHES A K & LOVENBURY H T, 1996. Investigating embankment dams. Building Research Establishment Report, London, 82 pp.

CHARVOZ B, 1995. Barrage de la Girotte – Étanchéité voûte 11-12 par membrane CARPI. EDF Dept. Sepr. Dom.

CHEN Z, 1993. Breaching of the Gouhou Concrete Face Sand and Gravel Dam. International Symposium on *High Earth and Rockfill Dams*, Beijing.

COMBELLES J, GOUBET A, LLOPIS N, & PACCARD M, 1985. Mesures destinées à améliorer la sécurité des ouvrages hydrauliques des barrages. 15th ICOLD Congress, Q59 R46.

COMBELLES J, 1991. Ageing of Dams and Remedial Measures, General Report, ICOLD, Vienna, Q65.

Comité National Suisse des Grands Barrages, 1982. Comportement anormal du barrage-voûte de Zeuzier (Suisse). Wasser, Energie, Luft, édition spéciale à l'occasion du XIVe Congrès CIGB, Rio de Janeiro.

CONTESSINI F, RAMACCIOTTI R & GVAZZI P, 1967. La détérioration des parements de certains barrages des Alpes. Observations et mesures prises pour leur réfection. 9th ICOLD Congress, Istambul, Q34 R48.

COOPER B W, 1991. Past and current remedial measures at Nepean dam. 17th ICOLD Congress, Vienna, Q65 R62.

CUMMINS P, 1999. Private Communication.

CYGANIEWICZ J M, 1988. Construction of a cut-off wall at Fontanelle Dam. 16th ICOLD Congress, Q61 Contribution 15, pp 227-230.

DAUTZENBERG W, & SAGE, F, 1994. Neue Schale vor alter Mauer. Beton 44 H.12, pp 734-737.

DAVIDSON L & McDANIEL T, 1988. Installation of a Deep Cut-Off Wall at Navajo Dam. ICOLD, Q61 Contribution 16, pp 230-238.

DAVIDSON L, 1990. Performance of the concrete diaphragm wall at Navajo Dam. USCOLD annual meeting, New Orleans, *Dam Foundation Engineering*, 10th annual lecture series.

DE BEAUCHAMP T, 1994. Concrete swelling and reinforcement of measures at the Chambon Dam. *Intl Jl on Hydropower & Dams*. March.

DEMMER W, LUDESCHER H, 1985. Measures taken to reduce uplift and seepage at Kölnbrein dam. 15th ICOLD Congress, Lausanne, Q58 R81

DENNY M, 1998. Refurbishment of deep sluices at Roseires Dam. *Dams & Reservoirs*, vol 8 no 1, British Dam Society, London.

DIAZ–CANEJA RODRIGUEZ J I, 1994. Evaluation and increase of safety for dams in service in the basin of the Duoro River. Remedial works. 19th ICOLD Congress, Durban, Q68 R30.

DIETHELM W, 1983. Reparaturarbeiten an der Staumauer Schöni des Kraftwerkes Oberalp. Wasser, Energie, Luft, Vol. 4.

DINNEEN E A & SHESKIER M, 1997. Design of soil-cement-bentonite cut-off wall for Twin Buttes Dam. 17th Annual USCOLD Lecture, San Diego.

DOE G, JEFFERIS S & TEDD P, 1999. Specification for the construction of slurry trench cut-off walls. Thomas Telford, London. ISBN 07277 2625 0.

DÖRING M, 1986. Abdichtung der Sösetalsperre. Wasser und Boden 86/9.

DUNNICLIFF J, 1988. Geotechnical instrumentation for monitoring field performance. New York, Wiley, 577 pp.

DYKE T & WILLIAMS P J, 1998. Rehabilitation of Holmstyes Reservoir. 10th British Dam Society Conference *The Prospect for Reservoirs in the 21st Century.* Thomas Telford, London.

EGAWA K, HIBINO Y & YOSHIDA T, 1994. Renovation of 70-year-old Dam on Reinforcement and Application of Gateless Spillway. 18th ICOLD Congress, Durban, Q68 R56.

ERIKSSON H, 1994. Investigation and rehabilitation of the Storfinnforsen dam. 18th ICOLD Congress, Durban, Q68 R19.

ERWIN E & GLENN J, 1991. Major Rehabilitation of Wister Dam. *USCOLD Newsletter,* March 1991.

EURANIUS J, 1991. Unpublished VBB report.

FANELLI G C, 1998. Plastic membrane limits seepage at dam. Hydro Review world-wide, May.

FARRER J A, WIRKUS K E & McCLAIN J A, 1990. Foundation treatment methods for the Jackson Lake modifications. USCOLD Annual Meeting, New Orleans, *Dam Foundation Engineering*, 10th annual lecture series.

FEDDERSEN I, 1992. Instandsetzung der Staumauer der Edertalsperre (Reconditioning of the Eder Dam). Wasserwirtschaft 82 2.

Fédération Internationale des Ingénieurs-Conseils, 1992. Conditions of Contract for Works of Civil Engineering Construction. FIDIC PO Box 86, CH1000 Lausanne 12.

FEINER A & ZICHNER G, 1967. Sealing of the Agger Dam. 9th ICOLD Congress, Istanbul, Q34 R11.

FETZER C A, 1979. Wolf Creek Dam Remedial Work/ Engineering Concepts, Actions and Results. 13th ICOLD Congress New Delhi, Q49 R5.

FIEDLER K, PTASZYNSKI Z & SZYMANSKI A, 1996. Repair of asphaltic concrete sealing of the upper reservoir of the Porabka-Zar pumped-storage plant. SwedCOLD Symposium *Repair and upgrading of Dams*, Stockholm, Sweden, June 1996.

FLEMME W, BAZTAN J A & FUNKE, 1990. Repairs to the Martin Gonzalo Dam. Water Power & Dam Construction, Vol. 42, No. 11, November.

FLEMMING J H & ROSSINGTON D T, 1985. Repair of Greenbooth Dam following localised settlement in the crest. 15th ICOLD Congress, Lausanne, Q59 R55.

FLÖGL H & STAUBLE H, 1991. Experience with the Supplementary Installation of an Elastic Diaphragm in the Rock of Schlegeis Arch Dam. 17th ICOLD Congress, Vienna, Q66 R41.

FOULADI C, 1994. Karun I chute spillway damage and construction of a new flip bucket. 18th ICOLD Congress, Durban, Q71, Contribution 7.

FROHNAUER R & TORKUHL C, 1996(b). New Asphaltic Lining for the Rabenleite Upper Reservoir. Strabag Brochure No. 51, Asphaltic Concrete for Hydraulic Structures, Cologne.

FROHNAUER R & TORKUHL C, 1996(a). The refurbishment of Namibia's Hardap Dam. Hydropower & Dams Vol 3, July.

FROHNAUER R, SCHMID R & WALLENBURGER A, 1998. Bauabwicklung der Instandsetzungsarbeiten am Werkkanal Mittlere Isar. Symposium *Planung und Realisierung im Wasserbau*. Publication 82 of the Versuchanstalt für Wasserbau und Wasserwirtschaft, Technical University of Munich.

FUJISAWA T, 1994. Rehabilitation of the Hohnen-Ike Dam under Redevelopment Project. *The Dam Digest* No. 599, published by the Japan Dam Foundation.

GAGLIARDI J & ROUTH R, 1993. Geotechnical modification at Meeks Cabin Dam. ASCE Speciality Conference on *Geotechnical Practice in Dam Rehabilitation*, Raleigh, NC.

GALLACHER D, DOAKE R M & HAY-SMITH D, 1998. Remedial works to upstream face protection, Megget Reservoir. *The prospect for Dams in the 21st Century*. 10th British Dam Society Conference. Thomas Telford, London.

GAN J K & McDONALD L A, 1999. Mardi Dam – A case study of a dam on permeable foundations. Symposium *Dam Foundation Problems and Solutions*. 67th ICOLD Annual Meeting, Antalya, Turkey, 23 September 1999, pp 397-419.

GEISLER W D, 1996. Introduction to the Technology of Asphalt Linings for Storage Reservoirs. Strabag Brochure No. 51. Asphaltic Concrete for Hydraulic Structures, Cologne.

GERBOTHE D, JÄGER W & JÜNGEL E, 1994. The Rehabilitation of Carlsfeld Dam. 18th ICOLD Congress, Durban, Q68 R7.

GIL GARCIA A & CAJETE BALTAR J, 1991. Study on the interaction aggregate– paste in the concrete at the San Esteban Dam, 17th ICOLD Congress, Vienna, Q65 R4.

GILG B, INDERMAUR W, MATHEY F, PEDRO JO, AZVEDO MC & FERREIRA M J E, 1987. Special design of Steno arch dam in Greece, in relation with possible fault movements. ICOLD Symposium, Beijing.

GILLON M D, EVERITT S & MEJIA L, 1998. Rehabilitation of the Matahina Dam for foundation fault trace displacements. Symposium on Rehabilitation of Dams, 66th ICOLD Annual Meeting, New Delhi, India.

GIOVAGNOLI M, ERCOLI F & SCHRADER E, 1996. Concepcion Dam design & construction. Problems and their solutions. *Conference on Geomembranes*, Laboratório Nacional de Engenharia Civil, Lisbon.

GLOSSOP R, 1961. The invention and development of injection processes, Part II Geotechnique 11(6), pp 255-276.

GODTLAND K & TESAKER E, 1994. Clogging of spillways by trash. 19th ICOLD Congress, Durban, Q68 R36.

GOSSCHALK E M, HINKS J L, JOHNSON F G & JARVIS R M, 1991. Overcoming the Build-up of Stresses, Cracking and Leakage in Mullardoch Dam, Scotland. 17th ICOLD Congress, Vienna, Q65 R26.

GOTZMER J W, 1994. Santee Cooper's comprehensive emergency action plan : a non-structural solution to seismic instability. USCOLD 14th annual lecture series, Phoenix, *Dam Safety Modification and Rehabilitation*.

GOUVENOT D, BOUYGE B, FRANCQ J, JENSEN A & MEYERFELD M, 1991. Perte d'étanchéité par vieillissement du corps des barrages. Pratiques en développement d'actions correctives. 17th ICOLD Congress, Vienna, Q65 R40.

Groupe de Travail du Comité Français des Grands Barrages, 1979. Réparations et entretien des barrages entraînés par le vieillissement ou un accident. 13th ICOLD Congress, New Delhi, Q49 R23.

HARLAN R C, ONKEN S, SCUERO A M & V ASCHETTI G L, 1998. The underwater installation of a drained geomembrane system on Lost Creek Dam. Proceedings of CANCOLD Conference, Nova Scotia, Canada.

HAUG W & REICHENBACH W, 1987. Observations of Time-Dependent Changes of the Asphaltic Concrete Sealing of the Pumped Storage Reservoir Geesthacht and Execution of the Re-engineering Works. Wasserwirtschaft, 77, Jahrgang, Heft 7, June.

HAWS E T, SNOWDEN H N & HORSWILL P. Survival with A S R : 22 Years at Val de la Mare Dam. 18th ICOLD Congress, Q68 R.68.

HAY-SMITH D, 1999. Private communication.

HEIGERTH G, MELBINGER R, OBERNHUBER P & TSCHERNUTTER P, 1994. Assessing and Improving the Safety of Existing Dams in Austria. 18th ICOLD Congress, Durban, Q68 R58.

HEIGERTH G & STÄUBLE H, 1994. Two Austrian examples for cracks in concrete dams and the repair works. 19th ICOLD Congress, Durban, Q68 R13.

HEITEFUSS C H, KNY H J & RUHNKE H, 1998. Die Sanierung des Sorpe Dammes mit einer HDI-Dichtwand. Wasserwirtschaft 88 (11), pp 562-565.

HEITFELD K H, 1961. Beispiel der Verkarstung fur den Talsperrenbau (am Beispiel der Hennetalsperre im Einzugsgebiet der Ruhr) – Journal Karst– und Hölenkunde, 2 iXXII, München.

HEITOR A M, & PEDRO J O, 1993. Pre-stressed piers for high radial gates. Fratel spillweir dam. ICOLD Congress, Madrid, Q43 R11.

HERBERT D M, LOVENBURY H T, ALLSOP N H W & READER R A, 1995. Performance of blockwork and slabbing protection for dam faces. Report SR345, HRL Wallingford, UK.

HEWLETT H W M, BOORMAN L A & BRAMLEY M E, 1987. Design of reinforced grass waterways. Construction Industry Research and Information Association Report 116, London.

HEWLETT H W M, BOORMAN L A & BRAMLEY M E, 1987. Guide to the design of reinforced grassed waterways. Construction Industry Research and Information Association Technical Note 120, London.

HINKS J L, BURTON I W, PEACOCK A R & GOSSCHALK E M, 1990. Post-tensioning Mullardoch dam in Scotland. Water Power & Dam Construction, Nov, 1990.

HOEG K, 1996. Performance Evaluation, Safety Assessment and Risk Analysis for Dams, Symposium Repair and Upgrading of Dams, Stockholm, Sweden, June 1996.

HOFFMANN H G, 1985. Die Ertüchtigung der Olef-Staumauer. Wasserwirtschaft, 75. Jahrg, Vol 7/8, pp 308 – 316.

HOPKINS J K, WICKHAM D B & STIRLING D M, 1990. The use of close-range photogrammetry for reservoir embankment monitoring, *The Embankment Dam.* Proceedings of the 6th Conference of the British Dam Society, paper 24 published by Thomas Telford, ISBN 0 7277 1647 6.

HOPKINS J K & WICKHAM D B, 1988. Repairs to Downstream Face of Haweswater Dam. Technical note 3, BNCOLD Symposium, University of Manchester, Sept 1988.

HUGHES A K & BEECH C J, 1998. Ireland Colliery Reservoir : A reservoir created by deep mining. *The Prospects for Reservoirs in the 21st Century*. 10th British Dam Society Conference. Thomas Telford, London.

ICOLD, 1983. Deterioration of dams and reservoirs. Report prepared by a team of Portuguese engineers of the "Laboratório de Engenharia Civil". LNEC, Lisbon.

ICOLD, 1985. Filling materials for watertight cut-off walls. Bulletin 51.

ICOLD, 1990. Dispersive soils in embankment dams – Review. Bulletin 77.

ICOLD, 1994. Ageing of dams and appurtenant works. Review and recommendations. Bulletin 93.

ICOLD, 1996. Vibrations of hydraulic equipment for dams. Review and recommendations. Bulletin 102.

IDEL K H & BECKMANN J, 1977. Alterung und von Talsperren. Vorträge des Seminars Talsperren– und Dammschäden – Ursachen und Sanierung – Publications, Inst. Foundation Engineering, Soil Mech., Rock Mech and Waterways Construction, RWTH Aachen.

IDEL K H & RIßLER P, 1982. Einflußer Sanierungsmaßnahmen auf die Möhnemauer. Wasserwirtschaft, 72. Jahrg., Vol 3, pp. 130-134.

Institution of Structural Engineers, 1992. Structural Effects of Alkali-Silica Reaction, Technical guidance on the appraisal of existing structures. London, ISBN 1 874266 02 6.

INTERNATIONAL WATER POWER & DAM CONSTRUCTION, 1997. WAC Bennet. Mind over matter, August.

JACOBY S R, 1998. Replacement of Dulce Lake Dam. Proceedings Association of State Dam Safety Officials, Las Vegas, Nevada, pp 561-572.

JANSEN R B, 1988. Advanced Dam Engineering for Design, Construction and Rehabilitation. Van Nostrand Reinhold, New York. ISBN 0-442-24397-9.

JOHANSSON B, JOHANSSON S & NILSON R, 1996. Investigations and repair of the embankment dam at Porjus Powerstation, SwedCOLD Symposium *Repair and Upgrading of Dams*, pp 218-227. Stockholm.

JOHANSSON S, 1994. Monitoring during ongoing internal erosion and after restoration of the embankment dam at Porjus Powerstation 18th ICOLD Congress, Durban, Q68 Prepared Contribution 5.

KAST K, 1996. Ageing and rehabilitation of an asphalt concrete faced embankment dam. SwedCOLD Symposium *Repair and Upgrading of Dams*, Stockholm, Sweden, June 1996

KEBBA J M & HULTMAN K L, 1999. Automated data acquisition systems for performance and safety monitoring of large dams. International Conference on Dam Safety and Monitoring. Three Gorges Project, Hubei, China.

KENNARD M F, 1972. Examples of the internal conditions of some old earth dams. Journal of Institution of Water Engineers, 26 (3), pp 135-154.

KIM S K, LIM H D & KIM K W, 1997. Leakage and stability problems of Angae Dam. 19th ICOLD Congress, Florence, Q73 R26.

KOGOVEK B & PIRC H, 1997. Dam on the Sava River for Nuclear Power Plant Krško-Monitoring and Maintenance. 19th ICOLD Congress, Florence, Q75 R5.

KOLLGAARD E B & CHADWICK W L, 1988. Development of dam engineering in the United States. Pergamon ISBN 0-08-034685-5.

KOVARI K, 1985. Detection and Monitoring of Structural Deficiencies in the Rock Foundation of Large Dams. 15th ICOLD Congress, Lausanne, Q56 R36.

KRAUTHAN H & RENNER J, 1989. Vorbereitende Arbeiten für die Sanierung einer Intze-Mauer, aufgezeigt am Beispiel der Brucher-Talsperre. Wasserwirtschaft 79, Vol 7/8, p. 378.

KRÖLL A, 1991. Pack Dam overall repair after 50 years of operation. 17th ICOLD Congress, Vienna, Q65 R17.

KULESZA R, PFLUEGER K, GARDNER C & YAN P, 1994. Investigations and design of diaphragm wall for seepage control at Wells Dam. 14th Annual USCOLD Lecture Series, pp 223-299.

KULKARNI D N, SHUKLA S Y & DESHPANDE V M, 1994. Deterioration of Spillway of a Large Dam in Maharashtra State, India. 18th ICOLD Congress, Durban, Q71 R4.

KUHLMANN W 1996. Experiences in the field of reinstatement works on hydraulic asphalt sealing structures according to examples of execution. SwedCOLD Symposium *Repair and Upgrading of Dams*, Stockholm, Sweden, June 1996

KUUSINIEMI R, 1991. Internal Erosion of the Uljua earth dam. 17th ICOLD Congress, Vienna, Q65 Contribution no 15.

KUUSINIEMI R & LOUKOLA E, 1996. Repairing works on earth dams owned by the State of Finland. SwedCOLD Symposium *Repair and Upgrading of Dams*, Stockholm, pp 1-12.

KUUSSINIEMI R, POLLA J & RATHMEYER H, 1992. Internal erosion at the Uljua earth dam, Water Power & Dam Construction, March 1992.

LAMPA J, 1994. Repairs at Hugh Keenleyside Dam. 18th ICOLD Congress, Durban, Q71 Contribution 9.

LEFRANC M, BATTISTEL R, CAULT J B & GOGUEL B, 1994. Exemples d'intervention sur des ouvrages d'évacuation. 18th ICOLD Congress, Durban, Q71 R26.

LÉGER P, TINAWI R, BHATTACHARJEE S S & LECLERC M, 1997. Failure mechanisms of gravity dams subjected to hydrostatic overload : influence of weak lift joints. 19th ICOLD Congress, Florence, Q75 R2.

LEGGO P J, WARREN C D & BLAKEWAY D, 1986. Application of isotope hydrology to groundwater problems in engineering geology. (Ed. J C Cripps, F G Bell & M G Culshaw.) *Proceedings 21st Annual Conference of the Engineering Group of the Geological Society on Groundwater in Engineering Geology, Sheffield*, September 1985, London, Geological Society, pp 169-176.

LEMELIN D & JOBIN H, 1996. Rehabilitation of a rockfilled timber crib dyke. SwedCOLD Symposium *Repair and Upgrading of Dams*, Stockholm, Sweden, June 1996.

LEMOS F O, MARTINS H F, PEIXEIRO L C & LEITE D O, 1973. An accident with a big tainter gate of a spillway. 11th ICOLD Congress, Madrid, C15.

LINO M, DEGOUTTE G, GOGUEL B, CHASSARD G, SAINT MARCEL A, GOUVENOT D & DELMAS F, 1994. Steps taken to improve the watertightness of dams and their foundation and to reduce the risk of internal erosion. 18th ICOLD Congress, Durban, Q68 R46.

LINO M, SALEMBIER M, ANTOINE F, ROUAS G, CLERDOUET D & SAINT-MARCEL A, 1991. Vieillissement de quelques barrages Français très anciens. Pratique de leur réhabilitation. 17th ICOLD Congress, Vienna, Q65 R1.

LIST F & STROBL T H, 1991. Changing of tightness in the grout curtain and core at the Sylvenstein Dam by ageing. 17th ICOLD Congress, Vienna, Q65 R72.

LNEC, 1980. Observation of Miranda Dam. Crack movement in the gallery at the elevation 510.5m (in Portuguese). Internal Report, Lisboa, Portugal.

LOWE III J, BANGASH H D & CHAO P C, 1979. Some Experiences with High Velocity Flow at Tarbela Dam Project. 13th ICOLD Congress, New Delhi, Q50 R13.

MARCUSON III W F, HADALA P F & LEDBETTER R H, 1993. Seismic Rehabilitation of Earth Dams. ASCE Special Publication N° 35, Geotechnical Practice in Dam Rehabilitation.

MATSUHUIJ R, HIRAP T & UMEMI T, 1993. Improvement Work of the Magaribuchi Dam. *Engineering for Dams* No. 84, published by Japan Dam Engineering Centre.

McCLAIN J, CARTER B & HENNIG C, 1994. Underwater Repairs for Potomac River Dam No. 5. *Dam Safety '94*. Proceedings, Association of Dam Safety Officials, pp 339-347.

McLEISH W P, APTED R W & CHIPP P N, 1987. Geotechnical remedial works for dams : some UK experiences. Proceedings of International Conference on Structural Faults and Repairs, London, pp 181-194.

MEHMEL A, 1964. Verstärkungsmaßnahmen an der Oleftalsperre. Der Bauingenieur, 39. Jahrg., Vol 6, pp. 231-237.

MEJIA L, FORREST M, BISCHOFF J, GILLON M D & EVERITT S, 1998. Strengthening of Matahina Dam for fault displacement. Proceedings of ASDSO Annual Conference, Las Vegas, pp 503-519.

MILLMORE J P, STABLES J F & SHANNON, F E, 1998. The Rehabilitation of Luxhay Dam, Somerset. *The Prospects for Reservoirs in the 21st Century*. 10th British Dam Society Conference. Thomas Telford, London.

MIRZA J & DURAND B, 1994. Repair Mortars for Controlling Abrason-Erosion Damage on Dam Surfaces. 18th ICOLD Congress, Durban, Q71 R12.

MONARI F & SCUERO A M, 1991. Ageing of concrete dams : the use of geocomposites for repair and future protection. 17th ICOLD Congress, Vienna, Q65 R42.

MONFORT L, MILLET J C, FABRE J P, BORDES J L, EUVRARD D, RICARD C & ROYET P, 1991. Éléments de méthodologie pour la détection et l'analyse du vieillissement illustrés par des exemples. 17th ICOLD Congress, Vienna, Q65 R23.

MUELLER R, 1985. Comité National Suisse des Grands Barrages Chapter 6.2 : Structural Restoration Guebensee.

MUELLER R & POUGATSCH H, 1985. Comité National Suisse des Grands Barrages Chapter 6.2.2-Zeuzier.

NILSSON A & MIKAELSSON J, 1996. Groundwater Pressure Increase and Internal Erosion at Langbjorn Dam, SwedCOLD Symposium *Repair and Upgrading of Dams*, Stockholm pp 503-519.

NILSSON A & NORSTEDT U, 1991. Evaluation of ageing processes in two Swedish dams. 17th ICOLD Congress, Vienna, Q65 R2.

O'TUAMA F F & O'MAHONY B O, 1989. Hydro Electric Dam Safety – Upgrading of ESB Dams. The Institution of Engineers of Ireland.

OBERNHUEBER P, 1991. Remedial works for the Kölnbrein Dam : Design and Analysis. 17th ICOLD Congress, Vienna, C7.

ORHON M & BILGI V, 1994. Deterioration of Spillway of Seyhan Dam. 18th ICOLD Congress, Durban, Q71 R10.

PACELLI W A, ANDRIOLO F R & SARKARIA G S, 1993. Treatment and performance of construction joints in concrete dams. Water Power & Dam Construction, November 1993.

PADGETT I E & MORRISON K F, 1998. Gate replacements at Kotri Barrage, Pakistan. To be published.

PAOLINA R, APPENDINO M, BALDOVIN E, BERRA M, BIANCHINI A, CARABELLI E, POSTA U & VIELMO I, 1991. Deterioration problems for concrete and masonry dams. 17th ICOLD Congress, Vienna, Q65 R43.

PEDRO J O, FLORENTINO C A & GOMES A S, 1979. Observation methods for controlling the safety and deterioration of concrete dams. ICOLD Congress, New Delhi, Q49 R25.

PEDRO J O, MENDES A, ROCHA AFONSO J J & ROLO F M, 1994. Behaviour of concrete and masonry dams under severe climatic conditions. The cases of Covão de Ferro, Lagoa Comprida and Cova do Viriato dams. 18th ICOLD Congress, Durban. C24.

PEDRO J O, MASCARENHAS A T, SOUSA L R, RODRIGUES L F, SILVA H S & CASTRO A T, 1998. Rock Mass Foundation Deterioration of Portuguese Concrete Dams. Some Case Histories. Report to the European Research Office, USA Corps of Engineers. LNEC, Lisboa.

PENDSE M D, HUDDAR S N & KULKARNI S Y, 1998. Rehabilitation of Koyna Dam – A Case Study. Indian National Committee on Large Dams Symposium. *Rehabilitation of Dams*, 66th ICOLD Annual Meeting, New Delhi, November 1998.

PHILLIPS P & SHANNON F E, 1994. Recent Investigations at Wimbleball Dam, Somerset. *Reservoir Safety and Environment*, 8th British Dam Society Conference, Thomas Telford, London.

PINTO N, 1994. General Report on Question 71. 18th ICOLD Congress, Durban.

POLTCZYK H, WITTKE W & KÜPPER H J, 1997. Sprengtechnischer Votrieb eines Kontrollganges in einer Talsperre unter Stau. Tunnelbautaschenbuch 1997, Glueckauf Verlag.

POUGATSCH H, 1982. Unexpected behaviour of a large arch-dam in Switzerland. 14th ICOLD Congress, Q52 R40.

PRIETO F B & PEREZ J A G, 1994. Experiences in the application of new technologies to the El Atazar dam. 18th ICOLD Congress, Durban, Q68 R29.

PUJOL-RUIS A, KOLLGAARD E B, BREWER G W & EAKIN J H, 1991. Rehabilitation of Milner Dam. 17th ICOLD Congress, Vienna, Q67 R23.

PUYDEBOIS H & DURAND P, 1995. Barrage des Saints-Peyres – Réfection du parement amont. COGECH 1995, E D F.

PYYNY J, 1998. Private communication from Kemijoki Oy.

RAJAR R & KRYZANOWSKI A, 1994. Self induced opening of spillway gates on the Mavcice Dam – Slovenia. 18th ICOLD Congress, Durban, Q71 R8.

RAY W J F & BULMER T, 1982. Remedial works to puddle clay cores. Proceedings of 2nd BNCOLD Conference, Keele, pp 27-44.

REGAMEY J M & HAMMERSCHLAG J G, 1995. Barrage de l'Illsee – Assainissement. SwissCOLD Conference *Research and development in the field of dams*, Crans Montana, Switzerland.

RIßLER P, 1996. Ertüchtigung von Talsperren, Special 'Forum Staedtehygiene' (IFWW) Patzer Verlag, Berlin, Hannover.

RIßLER P, 1994. Rehabilitation of existing gravity dams in Germany with respect to safety philosophy and economy. 18th ICOLD Congress, Durban, Q68 R5.

RIßLER P & HEITEFUSS C, 1999. Adaptation of the Ennepe Dam Germany to the Established Technical Standards. *Int Journ on Hydropower and Dams,* vol 6 issue 2.

RIßLER P, 1998. Talsperrenpraxis. Oldenbourg Verlag, Germany

RODRIGUEZ A J A, 1994. Resonance phenomena in the bottom outlets of La Vinuela Dam. 18th ICOLD Congress, Durban, Q71 C3.

ROFE B H, WICKHAM D B & HOPKINS J K, 1990. Mitchell's House reservoirs – investigations, monitoring and remedial works. *The Embankment Dam*. Proceedings of 6th Conference of British Dam Society, Nottingham, pp 145-149.

ROST M & T KLÜBER, 1992. Umfang und Ergebnis der technischen Überprüfung der Schwarzenbachtalsperre. Wasserwirtschaft 74, Vol 6, p.249.

RUHRVERBAND, 1998. Hennetalsperre. Sanierung des Hauptdammes. Information leaflet, Essen, Germany.

SASAKI I, 1992. Modification of the guard gate sheet of outlet conduit, *Engineering for Dams*, No. 67, Japan Dam Engineering Centre.

SCHMIDT M, 1987. Die Sanierung der Sösetalsperre. Wasserwirtachaft 77/6.

SCHNEEBERGER CE, PIGEON Y, LEVAY J & BONCOMPAIN B, 1991. The use of slurry walls and jet grouting techniques to repair existing cofferdams at LG-1. 17th ICOLD Congress, Q65 R29.

SCHOBERL P, 1996. Crack sanitation at Zillergrundl arch dam. SwedCOLD Symposium. *Repair and Upgrading of Dams*, Stockholm.

SCHÖBERL P, DUBER H & DÖPPER H, 1993. Kraftschlüssige Rißinjektion an der Gewölbemauer Zillergründl (Force-locking Crack Injection at Zillergründl Arch Dam). *Grouting in Rock and Concrete*. Balkema, Rotterdam.

SCHRADER E, 1980. Repair of waterstop failures. Proc. of the American Society of Civil Engineers, Vol 106, No. EY2, October.

SCUERO A M, 1996. Synthetic geomembranes and their role for a better use of water resources. *Conference on Geomembranes*, Laboratório Nacional de Engenharia Civil, Lisbon.

SCUERO A M & VASCHETTI G L, 1998. A drained synthetic geomembrane system for rehabilitation and construction of dams. *The Prospects for reservoirs in the 21st Century*. 10th British Dam Society Conference. Thomas Telford, London.

SHERARD J L, 1973. Embankment Dam Cracking. Embankment Dam Engineering, Casagrande Volume, Wiley.

SHEWE L D, 1997. Successful remedy for the failed initial refurbishment at Hardap Dam. Hydropower and Dams, Vol. 4, No. 6.

SHIBATA I & SANO T, 1991. Effects of ageing and subsequent repairs of dam concrete at Sasanagare Dam. 17th ICOLD Congress, Q65 R21.

SILVA MATOS D & LIBERAL O, 1993. Pracana powerplant refurbishment and uprating. An uncommon and challenging project. *Uprating and refurbishment of hydro powerplants – IV*. Water Power & Dam Construction Conference, Florence.

SILVA MATOS D & LIBERAL A O, 1993. Pracana Power Plant Refurbishment and Uprating : an uncommon and challenging project. Water Power & Dam Construction, *Uprating & Refurbishing Hydro Power Plants*, Florence.

SILVANO R, FRONGIA F, MONDADA A & PIAZZA A, 1997. Repairworks at Flumendosa arch dam. 19th ICOLD Congress, Florence, Q75 R39.

SILVEIRA A F & MORA RAMOS J, 1994. Strengthening and raising of Lagoa Comprida dam. ICOLD Congress, Durban, Q70 R14.

SIMS G P, 1991. Contribution 2, Section A ; Alkali-aggregate reaction (AAR) : reaction process ; effects on the structures ; interpretation of these effects ; mathematical models ; aggregate reactivity tests ; remedial measures. 17th ICOLD Congress, Vienna, Q65, Vol 5, pp 193-196.

SIMS G P, 1994. Ageing of masonry dams. Proc Instn Civ Engrs Wat, Marit & Energy 106. March, 61-70.

SIMS G P, 1998. Personal Communication.

SIMS G P & EVANS D E, 1988 Alkali-silica reaction : Kamburu spillway, Kenya, case history. Proc. Inst. Civil Engineers, Part 1, Dec., 1213-1235.

SINNIGER R, 1991a. Ageing of Dams – Swiss experience. 17th ICOLD Congress Lausanne, Q65 R10.

SINNIGER R, 1991b. Concrete properties and behaviour of an old gravity dam. 17th ICOLD Congress, Lausanne, Q65 R14.

ST JOHN T, NICHOLLS R A & SENIOR K W, 1998. Grouting the puddle clay core at Barrow No 3 Reservoir, Bristol. *The Prospects for reservoirs in the 21st century*. 10th British Dam Society Conference. Thomas Telford, London.

STACY S T, FORREST T W, FLEMMING R L & LEDBETTER R H, 1996. Seismic Investigation and Remediation of Sardio Dam. USCOLD 16th annual lecture series, Seismic Design and Performance of Dams, Los Angeles.

SWISS NATIONAL COMMITTEE ON LARGE DAMS, 1982. General Paper. 14th ICOLD Congress, Rio de Janeiro, GP–RS8.

TADA S & SHIBATA K, 1991. Kuromata dam. Leakage prevention work. 17th ICOLD Congress, Vienna, Q65 R58.

TADDEI B & POUPART M, 1996. Reinforcement of the Chambon Dam. SwedCOLD Symposium *Repair and Upgrading of Dams*. Stockholm.

TEDD P, CHARLES J A, HOLTON I R & ROBERTSHAW A C, 1994. Deformation of embankment dams due to changes in reservoir level. Proceedings of 13th International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, pp 951-954, New Delhi.

TEDD P, DUTTON D P M & HOLTON I R, 1998. Investigating internal erosion at Brent Dam. *The prospects for reservoirs in the 21st Century*. 10th British Dam Society Conference. Thomas Telford, London.

TERRACINI C, 1970. Behaviour of Alpe Gera gravity dam in its early operation stages. Comparison between the measurements in situ and the analytical and experimental results. ICOLD Congress, Montreal, Q39 R48.

THOMPSON D M & SHUTTLER R M, 1976. Design rip rap slope protection against wind waves. CIRIA Report 61, London.

TOURNERY J F, MONNET R, CASTANIER G, BONAZZI D, PERARDEL G, GANDAIS M, JENSEN A, COUTURIER B & MARIOTTI G, 1991. Cas complexes d'étanchement de fondations hétérogènes. Transactions of 17th ICOLD Congress Vienna, Q66 R73.

TOYODA T & TAKASU S, 1991. Erosion control for sediment flushing facilities. 17th ICOLD Congress, Vienna, Q65 R20.

TRANDAFIR S, TILICE A G & DASCALESCU N, 1999. Remedying solutions for sealing system of peripheral embankments of Ostrovol Mic reservoir located on permeable foundation. Symposium *Dam Foundations Problems and Solutions*. At ICOLD 67th Annual Meeting, Antalya, 23 September 1999, pp 363-369.

TRIUMF C A & THUNEHED H, 1996. Two years of self-potential measurements on a large dam in northern Sweden. SwedCOLD Symposium, *Repair and Upgrading of Dams*, Stockholm, pp 307-315.

TSOLODAYI C M, MAZVIDZA D Z & GURUKUMBA K, 1996. Securing spillways at the Kariba Dam. *International Water Power & Dam Construction*, January 1996.

TWORT A C, 1977. The repair of Lluest Wen Dam. Journal of Institution of Water Engineers and Scientists, 31 (4), pp 269-279.

UCEDA J L, GINES R & HERAS E, 1996. Conditioning, repair and completion of the Alloz Dam. SwedCOLD Symposium *Repair and Upgrading of Dams*. Stockholm, June.

UMANA J & PAUTRE A, 1973. Dispositifs d'étanchéité au large dans des conditions géologiques inhabituelles. 11th ICOLD Congress, Madrid, C12.

USCOLD, 1984. Bibliography on performance of dams during earthquakes.

USCOLD, 1992. Observed performance of dams during earthquakes.

USCOLD, 1996. Anthology of Dam Modification Case Histories, ISBN 1-884575-09-9, April 1996.

USCOLD, 1999. Observed performance of dams during earthquakes, Volume II.

VALENZUELA BENAVIDES J, 1994. Underwater Repair of the Bottom Outlets of the Dams of the South of Spain. 18th ICOLD Congress, Durban, Q71 C2.

VAN SCHALKWYK A, JORDAAN J M & DOOGE N, 1994. Erosion in unlined spillways. 18th ICOLD Congress, Durban, Q71 R37.

VUORINEN J, 1969. On the behaviour of hardened concrete during freezing. The State Inst. For Technical Research Finland, Publication 145, Helsinki.

VICK SB G & STEWART R A, 1996. Risk analysis in dam safety practice. ASCE geotechnical speciality conference *Uncertainty in Geologic Environment : from theory to practice.*

WAECHTER H, HAUG W & HAHN, 1988. Specialist construction plant developed for Alz Canal refurbishment. Water Power & Dam Construction, December.

WALLISCH U, 1993. Renewal and Modernization of Gmünd Dam. *Symposium Uprating and refurbishment of hydro powerplants IV. Int.* Water Power & Dam Construction, Firenze.

WASSER UND SCHIFFAHRTS VERWALTUNG DER BUNDESREPUBLIK DEUTSCHLAND (WSB), 1994. Edertalsperre.

WATER POWER & DAM CONSTRUCTION, 1990. Bituminous sealing layers replaced at Markersbach. Editorial paper, Vol. 42, No. 8, August.

WEAVER K, 1991. Dam Foundation Grouting. ASCE Report, New York.

WIDMANN R (editor), 1995. Commission on Rock Grouting, ISRM.

WILLIAMS B E & ROBINSON R L, 1994. Grouting to great depths (33 year battle against leaks beneath Logan Martin Dam) *Civil Engineering*, December. Also 18th ICOLD Congress, Durban, Q68 R81.

WILSON A C & ROBERTSHAW A C, 1998. Winscar Dam : Investigations and repairs to asphaltic concrete membrane. The Prospects for Reservoirs in the 21st Century. 10th British Dam Society Conference. Thomas Telford, London.

WITTKE W & GREB H, 1995. Die Sanierung der Urfttalsperre. Wasserwirtschaft 85 7/8.

WITTKE W & GREB H 1994. Überprüfung der Standsicherheit der Urfttalsperre. Wasserwirtschaft 84 11.

Work Group of the Portuguese National Committee on Large Dams, 1991. Some case studies of ageing scenarios control in Portuguese concrete dams. ICOLD Congress, Vienna, Q65 R57.

Work Group of the Portuguese National Committee on Large Dams, 1985. Cracking and repair works at Cabril dam. ICOLD Congress, Lausanne, Q57 R21.

WULLIMAN E & AMMANN E, 1985 Comité National Suisse des Grands Barrages, Chapter 6.4.1 – Muslen.

XANTHAKOS P P, 1979. Slurry Walls. McGraw-Hill Book Company

YANASE K & MATSUMOTO K, 1989. Renewal of the Dam serving for sixty years after construction (Rehabilitation of the Ogakura in Nagasaki Prefecture). *The Dam Digest No. 532*, published by the Japan Dam Foundation.

YORKSHIRE WATER SERVICES Ltd, 1995. Boothwood Reservoir, Reservoirs Act 1975 : Certificate under Section 10 (6) as to the carrying out of Safety Recommendations.

ZHANG JINSHENG, 1994. Deterioration of the bottom outlets in Sanmexia Project and abrasion of the spillways for Gezhouba Project. 18th ICOLD Congress, Durban, Q71 Contribution 12.

ZURFLUH E, 1985a. Comité National Suisse des Grands Barrages, Chapter 6.3 : Improvement of Spillways and Outlets – Palagnedra.

ZURFLUH E, 1985b. Comité National Suisse des Grands Barrages, Chapter 6.3.4 – Rempen.

ZURFLUH E, 1985. Contributions to Swiss Dams-Monitoring and Maintenance, Swiss National Committee on Large Dams.

Imprimerie de Montligeon
61400 La Chapelle Montligeon
Dépôt légal : Mars 2001
N° 20209
ISSN 0534-8293