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Guidelines and cases histories

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DAMS AND FLOODS

Guidelines and cases 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
Sites : www.icold-cigb.org & www.icold-cigb.net

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AVANT-PROPOS

Au cours des dernières décennies du 20e siècle, des catastrophes naturelles ont gravement affecté la vie humaine en produisant de sérieux impacts sociaux et dommages économiques. Parmi les catastrophes naturelles, on peut souligner celles résultant des crues, qui représentent près de 40 % des impacts socio-économiques. De ce fait, les Nations Unies ont déclaré la période 1990-2000 « Décennie Internationale pour la Réduction des Catastrophes Naturelles » dans leur résolution de 1987, avec l'objectif de réduire la perte de vies humaines, les dégâts matériels et les impacts socio-économiques causés par les risques naturels.

En 1994, la CIGB décida de participer activement aux études entreprises au cours de cette décennie et créa le Comité des Barrages et des Crues, en vue d'étudier la corrélation entre les barrages et les crues, et le rôle des barrages dans l'atténuation des crues et la réduction des dégâts.

Le but principal du présent Bulletin est de démontrer le rôle joué par les barrages dans l'atténuation des crues, en décrivant des cas réels où l'exploitation des barrages au cours de périodes de crues a contribué de façon significative à la réduction des dégâts qui se seraient produits si les barrages n'avaient pas existé. De même, divers barrages en construction, dont le rôle dans l'atténuation des crues est important, sont décrits. En outre, des projets de barrages et leur rôle potentiel dans l'atténuation des crues sont examinés suivant une approche intégrée ; à cet effet, il est nécessaire de combiner et de coordonner toutes les mesures possibles, structurelles et non-structurelles, avec une attention particulière portée sur la mise en œuvre nécessaire d'actions non-structurelles, mais en soulignant le rôle fondamental des barrages dans l'atténuation des crues.

Les membres du Comité des Barrages et des Crues ont préparé ce Bulletin. Ces membres représentent 21 pays, dont certains possèdent un nombre élevé de barrages et ont une grande expérience dans l'exploitation de ces ouvrages pendant les crues. Je remercie vivement tous les membres du Comité pour leur participation active à la préparation du Bulletin, et également l'Ingénieur Espagnol Enrique Velasco pour sa collaboration dans la collecte et l'évaluation des données concernant les crues exceptionnelles sur des sites de barrages. Enfin, au nom du Comité, je tiens à exprimer ma gratitude à l'Ingénieur Robert A. Bank, de l'U.S. Army Corps of Engineers, pour son aide précieuse dans la mise au point du texte anglais du Bulletin.

J'espère que le présent Bulletin servira à démontrer que les barrages ont un rôle essentiel dans la réduction des dégâts dus aux crues, ce rôle étant toujours intégré dans une vision plus holistique et coordonné avec d'autres actions pour une meilleure gestion des crues.

L. BERGA
Président du Comité des Barrages et des Crues

FOREWORD

During the last decades of the 20th century, natural disasters have significantly affected human life by producing grave social impact and economic damages. Among the natural disasters there can be emphasized those produced by the floods, which constitute close to 40 % of the social-economic impacts. For this reason, the United Nations, in the year 1987, created the International Decade for Natural Hazard Reduction, 1990 to 2000, with the objective to reduce loss of lives, material damages and the social and economic impacts caused by natural hazards.

In 1994, ICOLD decided to actively participate in the studies undertaken during this decade, and created a Committee on Dams and Floods in order to study the interrelationship between dams and floods, and the contributions of dams in reducing flood damages.

The principal aim of this Bulletin is to demonstrate the role played by dams in flood mitigation by describing real cases in which the dam operations during flood situations have significantly contributed to the reduction of damages, which would have been produced if the dam had no existed. Likewise, various dams under construction in which the flood mitigation has an important role are described. Additionally, future dam projects and their potential role in flood mitigation are discussed within an integrated approach, in which it is necessary to combine and coordinate all possible measures, both structural and non-structural, with greater attention placed on necessary implementation of non-structural actions, but emphasizing the fundamental role that dams play in flood mitigation.

The members of the Committee on Dams and Floods have prepared this Bulletin. The Committee members are from 21 countries, some of which have the most number of dams and operation experiences during flood situations. I would like to express my appreciation to all of the Committee members for their enthusiastic participation in the drawing up of the Bulletin and also to the Spanish Engineer Enrique Velasco for his collaboration in the collection and evaluation of data of the extreme floods in dams. Finally, and on behalf of the Committee, I would like to manifest our gratefulness to the Engineer Robert A. Bank, of the U.S. Army Corps of Engineers, for their intense and precise work and edition to English of the Bulletin.

I hope that this Bulletin serves to demonstrate that dams have an essential role in the mitigation of flood damages, always integrated in a more holistic vision and coordinated with other actions for a better flood management.

L. BERGA
Chairman, Committee on Floods and Dams

1. INTRODUCTION

Les barrages sont généralement situés sur des cours d'eau et réagissent continuellement aux régimes fluviaux. Une corrélation existe entre les barrages et les crues. D'une part, les barrages modifient l'écoulement des crues et peuvent réduire de manière significative les pointes des crues, atténuant ainsi les dégâts à l'aval. D'autre part, les crues sont susceptibles de causer un risque au barrage dont la sécurité vis-à-vis des crues exceptionnelles doit donc être maintenue.

Étant donné l'importance que les crues ont sur la vie des barrages et les bienfaits de ces ouvrages dans la maîtrise des crues, la CIGB constitua un Comité des Barrages et des Crues au cours de la Réunion Exécutive de Durban (novembre 1994), les missions de ce Comité étant les suivantes :

MISSIONS :

Les crues et leur maîtrise tiennent une place dominante dans le projet, la construction et l'exploitation des barrages. La corrélation entre barrages et crues a été une préoccupation permanente de la CIGB, comme le montrent un grand nombre de Questions des Congrès traitant de ce sujet, ainsi que les travaux de Comités Techniques.

Lorsque le Comité de la Crue de Projet termina sa mission en présentant le Bulletin CIGB n° 82 « Choix de la Crue de Projet », il proposa la poursuite des travaux au sein d'un nouveau Comité, en vue de compléter les études effectuées et publiées par l'UNESCO (Registre Mondial des Crues Exceptionnelles, 1976) et par l'AISH (Registre Mondial des Crues Maximales Observées, 1984).

En outre, dans leur résolution de décembre 1987, les Nations Unies ont déclaré la période 1990-2000 « Décennie Internationale pour la Réduction des Catastrophes Naturelles ». Quarante pour cent environ de toutes les catastrophes à travers le monde étant imputables aux inondations, et compte tenu du rôle fondamental des barrages et des retenues dans la maîtrise des crues, il apparaît essentiel que la CIGB participe activement aux études entreprises au cours de cette décennie.

Aussi est-il apparu opportun de constituer un Comité chargé de traiter de la question des crues en relation avec les barrages. Sa mission portera sur les points suivants :

1) Analyse des crues exceptionnelles survenant dans le monde – en complément des études effectuées par l'UNESCO et l'AISH –, avec une attention spéciale portée à leurs effets socio-économiques.

2) Rôle des barrages dans la maîtrise des crues et la réduction des dégâts (étude de cas).

3) Sécurité hydrologique des barrages : critères fondamentaux et nouvelles orientations dans l'étude de cette sécurité. Évaluation de la sécurité des barrages existants compte tenu des nouveaux critères d'estimation des crues exceptionnelles.

1. INTRODUCTION

Dams are generally situated on the courses of the rivers and interact continually with the fluvial regimes. A mutual interrelationship exists between dams and floods. On one hand, dams alter the flood routing and can significantly reduce peak flows, thereby mitigating downstream flood damages. On the other hand, the floods suppose a risk to the dam, so then dam must be kept safe in the face of extreme floods.

Given the importance that floods have on the life of dams, and the benefits that dams provide for flood control, ICOLD created a Committee on Dams and Floods in the Durban Executive Meeting (November 1994) with the following terms of reference (TOR) :

TERMS OF REFERENCE :

Floods and their control are a dominant factor for the design, construction and operation of dams. Hence, the interrelationship between dams and floods has been a matter of permanent concern to ICOLD, as demonstrated by many Congress Questions dealing with this subject and by the work of Technical Committees.

When the Committee on the Design Flood finished its work by presenting ICOLD Bulletin Nº 82 "Selection of Design Flood", it proposed that research be continued by a new Committee whose task would be to complete the studies carried out and published by UNESCO (World Register of Extreme Floods, 1976) and by IAHS (World Register of Maximum Floods Observed, 1984).

Furthermore, in their resolution of December 1987, the United Nations have declared the years 1990-2000 the international decade for their reduction of natural disasters. As many as 40 % of all fatalities worldwide are caused by flooding, whereas dams and man made reservoirs constitute a fundamental element for the control of floods, ICOLD is challenged to contribute actively to the studies undertaken during this decade.

Therefore it is deemed appropriate to establish a Committee dedicated to the topic of floods related to dams. Its work will cover the following subjects and report on them :

1) Analysis of extreme floods occurring around the world, in completion of the studies by UNESCO and IAHS, and with regard also to their social impact and economic consequences.
2) The role of dams in the control of floods and the reduction of flood damage (with case studies).
3) Hydrological safety of dams : basic criteria for and new trends in its assessment. Safety evaluation of existing dams with regard to the updated criteria for estimating extreme floods.

Les travaux du Comité des Barrages et des Crues présentés dans ce Bulletin sont divisés en trois sections :

1) Étude et analyse des crues exceptionnelles.

2) Bienfaits et rôles des barrages dans la maîtrise des crues.

3) Sécurité hydrologique des barrages existants.

Les crues survenant à travers le monde constituent le danger naturel le plus important, à la fois pour le nombre de victimes et les graves conséquences socio-économiques qu'elles produisent. Ce fait a été également confirmé au cours d'enquêtes du Comité dans plusieurs pays ayant de grandes activités dans le domaine des barrages. Ces enquêtes montrent que, dans la majorité des cas, les crues constituent le risque naturel le plus important, présentant une grande récurrence et conduisant à de graves conséquences socio-économiques. Au cours de la dernière décennie, les dégâts causés par des crues ont également augmenté considérablement, conduisant à diverses initiatives avec coordination des efforts et échange de connaissances. Le but de ces initiatives est d'appliquer les mesures nécessaires et urgentes pour atténuer au mieux les dégâts et impacts causés par les crues.

Les analyses de crues exceptionnelles au niveau mondial remonte à 1984 et furent effectuées par l'Association Internationale des Sciences Hydrologiques (AISH). À cet effet, le Comité a lancé une vaste enquête pour réexaminer et mettre à jour les données concernant les crues exceptionnelles, en tenant compte des données obtenues sur des barrages, en vue d'obtenir une meilleure qualité de résultats. Plus de 400 ensembles de données ont été ainsi obtenus, leur analyse ayant permis de vérifier l'application des courbes enveloppes formulées par Francou-Rodier, bien que leurs valeurs aient été modifiées pour des bassins versants inférieurs à 300 km^2.

Les mesures de protection contre les crues et de réduction de leurs impacts importants seront nombreuses et élaborées suivant une approche intégrée du bassin, étant donné qu'en général la mise en œuvre d'un seul type de mesure conduit à des effets locaux et ne résout pas le problème dans l'ensemble du lit majeur.

Les mesures destinées à prévenir et à réduire les dégâts sont classées en deux grandes catégories : 1) Mesures structurelles qui interviennent dans les phénomènes de formation et d'écoulement des crues (par exemple, conservation des sols et correction des bassins versants, barrages, retenues de maîtrise des crues, levées, digues, ouvrages de dérivation, amélioration des lits, murs de protection contre les crues) et 2) Mesures non-structurelles destinées à prévoir et à atténuer les dégâts causés par les crues (par exemple, zonage, programmes d'utilisation des terres, système d'assurance, réglementation des constructions dans les lits majeurs, systèmes de prévision et d'annonce des crues, et plans d'alerte des crues).

Dans le cadre de ces mesures, on doit souligner les barrages et les retenues qui constituent une mesure structurelle très efficace, car ce sont les seules solutions permettant de stocker d'importants volumes de crues, de modifier de façon significative l'écoulement des crues et de réduire les pointes de crues. Les barrages sont toujours une alternative à prendre en compte pour essayer de résoudre les problèmes de crue, et ce particulièrement dans un grand nombre de cas où les augmentations de population et les concentrations urbaines écartent la prise en considération de mesures non-structurelles. Mais, comme on l'a déjà indiqué, la

The work of the ICOLD Committee on Dams and Floods presented in this Bulletin is divided into three sections :

1) Study and analysis of extreme floods.

2) Benefits and roles played by dams in flood control.

3) Hydrological safety of existing dams.

Floods in the world constitute the most significant natural hazard both in the number of victims and for the grave social and economic impacts that they produce. This fact has also been confirmed in ICOLD Committee investigations in several countries that are the most significant in the field of dams. The ICOLD Committee investigation showed that in the majority of the cases, floods are the most important natural hazard, presenting a great recurrence and giving rise to severe social and economic impacts. Also, in the last decade, flood damages have increased considerably, giving rise to diverse initiatives at a global level to coordinate efforts and interchange knowledge. The aim of these initiatives is to apply the necessary and urgent measures to alleviate the damages and impacts produced by the floods in the best possible way.

The analyses of extreme floods on a world level date from 1984, and were carried out by the International Association of Hydrological Sciences (IAHS). For this the Committee has conducted an extensive survey to revise and update the extreme flood data, in this case with data evaluated in dams, which present a greater quality. So, more than 400 datasets of extreme floods have been obtained, from the analysis of which the current application of the enveloped curves formulated by Francou-Rodier has been verified, although their values have been modified for basins less than 300 km^2.

The measures to face to the floods and reduce the important impacts that they produce should be many and formulated with an integrated approach of the basin, since, in general, the implementation of only one type of measure usually has local effects and does not resolve the problem in all the flood plain.

The measures to prevent and reduce the damages are classified in two large groups : 1) Structural measures, which interfere in the phenomena of flood formation and flood routing (e.g., soil conservation and correction of basins, dams, flood control and regulating reservoirs, levees, dikes, diversions, channel improvements, floodwalls), and 2) Non-structural measures, which foresee and mitigate the damages produced by floods (e.g., zoning, land-use patterns, system of insurance, building regulations in the flood plains, flood forecasting and flood warning systems, and emergency action plans).

Within this group of measures, dams and reservoirs, which constitute a very effective structural measure, should be emphasized, as they are the only solutions, which permit the storage of large flood volumes, modifying significantly the flood routing, and being able to reduce the peak flow in important values. Dams are always an alternative to be considered when trying to resolve flood problems, and even more so in a multitude of cases where the population increases and urban concentrations preclude consideration of non-structural measures. But, as we have already indicated, the reduction of flood damages should always be established with réduction des dégâts résultant de crues doit toujours être étudiée suivant une approche intégrée, prenant en compte et combinant toutes les mesures possibles. Afin que l'efficacité des barrages de maîtrise des crues soit la plus grande possible, il importe que ces ouvrages soient associés à d'autres mesures structurelles (digues, murs de protection contre les crues, levées, ouvrages de dérivation et autres ouvrages hydrauliques dans la rivière) et également à des mesures non-structurelles (par exemple, utilisation des terres, zonage), et principalement à des systèmes de prévision des crues en temps réel. Ces systèmes constituent un élément essentiel à une gestion adéquate des crues dans le bassin versant au moyen du barrage de maîtrise des crues.

Les barrages et les retenues sont classés en quatre grandes catégories, en fonction du but de la maîtrise des crues :

1) Retenues dont le but unique principal est la régularisation (alimentation en eau, irrigation ou énergie hydraulique), et dans lesquelles l'incidence sur la maîtrise des crues est faible.
2) Retenues à buts multiples dans lesquelles la maîtrise des crues est un objectif important, mais secondaire par rapport aux buts associés au stockage de l'eau.
3) Retenues à buts multiples dans lesquelles la maîtrise des crues est l'objectif principal, associé à d'autres objectifs secondaires.
4) Retenues dont le but unique est la maîtrise des crues et la réduction des dégâts causés par les crues à l'aval.

En outre, dans certains cas, l'objectif de maîtrise des crues concerne une saison spécifique de crue. Dans ces cas, le barrage est exploité pour réduire les dégâts au cours de la saison des crues, et le reste de l'année pour satisfaire aux autres objectifs. En raison de l'influence saisonnière, ces barrages sont classés dans la catégorie des barrages de maîtrise des crues saisonnières.

Tous les barrages présentent des bienfaits dans la maîtrise des crues s'ils sont bien conçus et correctement exploités. Ces bienfaits dépendent du rapport entre le volume de la retenue et le volume de la crue. Cependant, les bienfaits maximaux sont associés aux barrages dont la maîtrise des crues est l'objectif principal. Cependant, il importe de savoir qu'aucune mesure structurelle ou non-structurelle ne réduit le risque de crue à zéro, y compris les barrages de maîtrise des crues qui sont conçus et exploités pour une période de retour de protection aval correspondant à la crue de projet de protection (généralement, entre 50 ans et 500 ans). De ce fait, il faut éviter de croire que les barrages de maîtrise des crues présentent une sécurité totale vis-à-vis de toutes les crues exceptionnelles, et on doit considérer que ces barrages sont extrêmement efficaces vis-à-vis des crues pour lesquelles ils ont été conçus lorsqu'ils sont associés à d'autres mesures structurelles et non-structurelles, comme précédemment indiqué. De ce fait, il conviendrait de changer le terme actuel de « Barrage de Maîtrise des Crues » en « Barrage d'Atténuation des Crues », terme indiquant mieux l'efficacité des barrages dans une réduction significative des impacts et des dégâts résultant des crues, sans une maîtrise totale au cours des crues exceptionnelles.

Ainsi, les barrages sont capables de réduire de façon significative la fréquence et donc les impacts des crues extrêmes qui, en de nombreuses occasions, présentent an integrated approach, which takes into account and combines all possible measures. In order for flood control dams to be the most effective, they need to be implemented together with other structural measures (dikes, flood walls, levees, diversions and other hydraulic works in the river) and also combined with non-structural operations such as land-use, zoning, and principally with real-time flood forecasting systems. Flood forecasting systems constitute an essential element so that the flood control dam can carry out an adequate flood management in the basin.

Dams and reservoirs are classified in four broad categories listed below, depending on the flood control purpose :

1) Reservoirs in which the single principal purpose is regulation (water supply, irrigation or hydropower), in which the incidence in the flood control is small.
2) Multipurpose-reservoirs in which flood control is an important objective, but secondary to purposes associated with water storage.
3) Multipurpose-reservoirs in which flood control is the primary objective, combined with other secondary objectives.
4) Reservoirs in which the single purpose is flood control and the reduction of flood downstream damages.

Furthermore, in some cases, the objective of flood control is related to a specific flood season. In these cases, the dam is operated to reduce flood damages during the flood season and for other purposes during the rest of the year. Because of the seasonal influence, these dams are categorized as flood season control dams.

All dams present flood control benefits, if they are well designed and correctly operated. Flood control benefits are more important according to the relation between the volume of the reservoir and the volume of the flood. However, maximum benefits are obtained from flood control dams with which flood control is the primary objective. Nonetheless, it is important to understand that no structural or non-structural measure reduces the risk of floods to zero, including flood control dams which are designed and operated for a return period of downstream protection, the protection design flood (usually between 50 years and 500 years). Because of this, one must avoid thinking that flood control dams provide total safety in the face of all extreme floods, and consider that these dams are extremely efficient for the floods for which they were designed, when combined with other structural and non-structural measures, as previously noted. For which, it would be convenient to change the actual term of Flood Control Dam to the name of Flood Mitigation Dam, which would better indicate the efficiency of the dams in reducing very significantly the flood impacts and damages, but don't provide total control during extreme floods.

So then, dams are capable of significantly reducing the frequency and hence, impacts of extreme floods, which on numerous occasions present a high recurrence, une récurrence élevée, ce qui, au cours des années d'exploitation des barrages, suppose d'importants bénéfices par rapport aux dégâts qui se seraient produits.

Les barrages à travers le monde ont procuré d'énormes bienfaits dans la maîtrise des crues. Il n'est pas possible de quantifier ces bienfaits globalement, et il n'existe pas d'études au niveau régional ou national. Cependant, il est possible de quantifier les effets des barrages au niveau d'un bassin versant, et quelques cas intéressants, répartis à travers le monde, sont présentés en détail dans le Bulletin pour illustrer les bienfaits de la maîtrise des crues.

Actuellement (2001), 20 % environ des grands barrages existants ont été construits pour la maîtrise des crues, ce but étant unique (8 %) ou l'un des buts principaux. L'augmentation exponentielle des dégâts causés par les crues au cours des dernières années indique que dans le futur il sera nécessaire d'accroître les mesures de prévention et de réduction des dégâts. Ces mesures comprendront l'étude et la construction de nouveaux barrages de maîtrise des crues, associés à des mesures de contrôle de l'occupation progressive des lits majeurs et à des améliorations des systèmes de prévision des crues pour une meilleure fiabilité.

De ce fait, une augmentation du nombre de barrages de maîtrise des crues est prévue dans les vastes programmes de protection contre les crues. Par exemple, le Japon a actuellement 500 barrages de maîtrise des crues avec une capacité totale de retenue de 3 700 hm^3 destinée à cet objectif. Parmi les dégâts les plus importants dus aux crues, certains se produisent au Japon, avec une moyenne annuelle de dégâts de 7 200 millions de dollars. On prévoit que le Japon aura besoin d'environ 400 nouveaux barrages de maîtrise des crues en vue d'augmenter de 2 400 hm^3 environ la capacité de stockage pour la maîtrise des crues.

Une attention particulière est portée aux aspects hydrologiques de la conception des barrages d'atténuation des crues. Ces aspects sont traités dans le chapitre 6 du présent Bulletin « Recommandations relatives à la conception hydrologique des barrages d'atténuation des crues », qui présente des recommandations générales relatives à la conception et à l'exploitation de ces barrages. Deux crues fondamentales seront prises en compte pour les barrages de maîtrise des crues : 1) la crue de projet que le barrage sera capable de supporter avec maintien de la sécurité hydrologique, et 2) la crue de projet de protection pour la réduction de la crue à l'aval, ce qui conditionne la capacité nécessaire pour le stockage de la crue dans la retenue.

Le scénario idéal pour un barrage serait que le volume de la retenue soit suffisant en vue d'assurer une protection aval pour une crue de période de retour supérieure à 1 000 ans. En général, ce scénario n'est pas viable pour des raisons techniques, économiques, sociales et environnementales, et la protection correspond généralement à une période de retour de 50 à 200 ans. Les méthodes classiques d'étude des facteurs hydrologiques et hydrauliques de formation et d'écoulement des crues, et l'évaluation des dégâts pour la conception hydrologique des barrages d'amortissement des crues, présentent d'importantes incertitudes en raison de la nature stochastique du phénomène. Il convient toujours d'effectuer des analyses de sensibilité des méthodes classiques, ou d'appliquer des méthodes basées sur l'analyse de risque, dans lesquelles des analyses probabilistes sont effectuées.

with which over the length of the years of dams operation suppose some important benefits in the face of the damages which would have been produced.

The dams in the world have provided enormous benefits to flood control. It is not possible to quantify these benefits at a global level, nor do studies exist at a regional or national level. However, it is possible to quantify the effects of dams at a basin level, and some relevant cases, distributed throughout the world, are shown and detailed in this Bulletin to illustrate flood control benefits.

At the present time (2001), about 20 % of the existing large dams were built for the purpose of flood control, be it a single purpose (8 %), or as one of its principal purposes. The exponential growth of flood damages in recent years indicated that in the future it will be necessary to increase the measures of prevention and reduction of damages. These measures will include design and construction of new flood control dams, together with measures that control the progressive occupation of the flood plains and improvements to flood forecasting systems for increased reliability.

For this, an increase of the number of flood control dams is foreseen within the extensive flood mitigation plans. For example, Japan presently has 500 flood control dams with a total flood control reservoir capacity of 3700 hm^3. Some of the greatest flood damages occur in Japan, with average annual damages of 7200 M$/year. It being foreseen that Japan will need about 400 new flood control dams to provide an increase of approximately 2400 hm^3 capacity in the flood control reservoirs.

Special relevance is given to the hydrological aspects of designing flood mitigation dams. These aspects have been included in Section 6 of this Bulletin, “Guidelines for the Hydrological Design of Flood Mitigation Dams”, which provides general recommendations for dam design and operation. Two fundamental floods should be considered for flood control dams : 1) the design flood, which the dam should be capable of supporting and maintaining hydrological safety, and 2) the protection design flood for downstream flood reduction, which conditions the necessary flood storage in the reservoir.

The ideal scenario for a dam would be if the volume of the reservoir were sufficient to count on downstream protection greater than a 1000 years return period. In general, this scenario is not viable, because of technical, economical, social and environmental factors, and the protection is typically for the 50 to 200-year return period. The traditional methods of studying hydrologic and hydraulic factors of flood formation, flood routing, and damage evaluation for the hydrologic design of the flood mitigation dams, present important uncertainties, because of the stochastic nature of the phenomenon. It is always convenient to carry out sensitivity analyses of the traditional approaches, or apply methodologies based on risk analysis, in which probabilistic analyses of the uncertainty are conducted.

L'autre aspect important de la corrélation entre barrages et crues est que les barrages doivent être capables de supporter des crues exceptionnelles, sans rupture. L'expérience montre que les crues constituent un risque majeur pour les barrages, la cause principale de rupture des barrages étant la submersion – près de 40 % des ruptures de barrages. De ce fait, au cours des dernières décennies, dans les règlements et les recommandations de nombreux pays, des propositions ont été faites en vue d'augmenter la sécurité hydrologique des barrages ; actuellement, la crue de projet choisie pour les barrages de risque élevé est la crue maximale probable (PMF), ou une crue de période de retour de 10 000 ans. Cela conduit à une sécurité hydrologique plus grande pour les nouveaux barrages, mais constitue un problème pour les barrages existants. On peut se demander s'il est nécessaire d'améliorer la sécurité hydrologique des barrages existants pour satisfaire aux nouvelles normes, beaucoup plus contraignantes, ou s'il suffit de préserver la sécurité demandée à l'époque de la construction des barrages. En général, la philosophie adoptée dans la plupart des pays est qu'il est souhaitable, et dans de nombreux cas cela est exigé, que les barrages existants présentent la même sécurité que les nouveaux barrages, et qu'ils doivent donc satisfaire aux nouvelles normes relatives à la sécurité hydrologique et s'adapter à la nouvelle crue de projet. Ce souhait d'améliorer la sécurité de barrages existants nécessitera d'importants efforts d'inspection, de révision technique, mais principalement des investissements élevés, dont il faudra, dans de nombreux cas, déterminer les priorités, avec établissement d'un programme général de mesures d'amélioration de la sécurité des barrages. Diverses méthodologies sont disponibles, telles que le classement par ordre de priorité d'un groupe de barrages dans un bassin versant ou une région, et, plus récemment, les techniques basées sur l'analyse de risque.

The other important aspect of the interrelationship between dams and floods is that dams should be capable of supporting extreme floods, without failure. Experience shows that floods constitute a major risk for dams, since the principal cause of failure is overtopping which supposes almost 40 % of dam failures. For this, in the last decades, in the regulations and guidelines of numerous countries, proposals have been carried out to increase the hydrologic safety of dams, and, at the present time the selection of the design flood for the high hazard dams is the PMF (probable maximum flood), or return periods in the order of 10 000 years. This provides greater hydrologic safety for the new dams, but constitutes a problem for existing dams. One may ask if it is necessary to upgrade the hydrological safety of the existing dams to comply with the new, much more demanding standards, or is it sufficient to preserve the safety demanded in the years that the dams were constructed ? In general, the philosophy existing in the greater number of countries is that it is desirable, and in many cases to be required, that the already existing dams should have the same safety as the new dams, and that therefore they should comply with the actual standards relative to the hydrological safety, for which they must accommodate them to the new design flood. This desire for safer, existing dams will include considerable efforts of inspection, technical revision, but principally also supposes some very high investments, for which on numerous occasions it is necessary to determine priorities of the investments and carry out a general plan for the dam safety accommodation. Diverse methodologies are available, such as the ranking score of a group of dams of one basin or of one region, and more recently, the techniques based on the risk analysis

2. CONTENU DU BULLETIN - RÉSUMÉ

La partie principale du Bulletin comprend cinq chapitres. Les chapitres 3, 4, 5 et 6 concernent l'importance des crues dans les catastrophes naturelles, et incluent l'analyse des crues exceptionnelles, et la description et la quantification du rôle joué par les barrages dans la maîtrise des crues et la réduction des dégâts. Des recommandations relatives à la conception hydrologique des barrages d'atténuation des crues sont également présentées. Le chapitre 7 concerne la sécurité hydrologique des barrages existants, le chapitre 8 donnant les références citées dans le texte.

Chapitre 3 - Ce chapitre décrit les impacts sociaux et les dégâts causés par les crues. Les conséquences des catastrophes naturelles sur la vie humaine, les impacts sociaux et les pertes économiques sont analysés. Les risques naturels ont augmenté au cours des dernières décennies et ont affecté près de 10 % de la population mondiale, conduisant à une « moyenne » annuelle de dégâts de l'ordre de 50 milliards de dollars. L'importance des crues dans les risques naturels est décrite. Les crues représentent un tiers environ de toutes les catastrophes naturelles, et un tiers environ du nombre de victimes et des dommages économiques. Le chapitre 3 donne également des données statistiques sur les effets des crues au cours des 20 dernières années.

Le chapitre 3 présente une analyse de l'enquête effectuée par le Comité et concernant les impacts socio-économiques des crues. Elle porte sur les 20 pays les plus représentatifs dans le domaine des crues, et des barrages et des crues, ces pays ayant 80 % environ du total des barrages construits dans le monde. Les résultats de cette enquête concernent 1) les risques naturels les plus importants, 2) l'incidence moyenne des crues les plus importantes, 3) les impacts sociaux, 4) les impacts économiques et 5) les zones exposées à des risques de crues. L'analyse des résultats de l'enquête a conduit aux résultats suivants :

1) Dans la plus grande partie des pays ayant fait l'objet de l'enquête, les crues constituent le risque naturel le plus important.

2) L'incidence des crues est grande ; dans la plupart des cas, la période de retour des crues importantes est inférieure à 10 ans.

3) Au cours des dernières décennies, dans la plupart des pays, le nombre moyen de victimes des crues est inférieur à 20 par an, alors que dans les pays très développés, tels que le Japon et les États-Unis, des chiffres de 100 victimes par an ont été atteints, ce qui résulte principalement de l'occupation très intense des lits majeurs et de l'occurrence de crues soudaines. Le plus grand nombre de victimes, soit environ 85 à 90 % du nombre total, ont été enregistrées en Asie, principalement en Corée, au Bangladesh, en Inde et en Chine, avec des valeurs atteignant, dans ce dernier pays, 3 000 victimes par an, et une moyenne d'environ 15 millions d'habitants affectés par an.

2. ORGANIZATION OF THE BULLETIN - SUMMARY

The principal part of the Bulletin has been organized in five chapters. Chapters 3, 4, 5, and 6 refer to the importance of the floods within natural disasters, and include the analysis of extreme floods, and description and quantification of the role played by dams in the flood control and in the reduction of damages. Also guidelines for the hydrologic design of flood mitigation dams are furnished. Chapter 7 refers to the hydrological safety of existing dams, and Chapter 8 furnishes references given in the text.

Chapter 3 - describes the social impacts and damages produced by floods. The consequences of what natural disasters impose on human life and on the social impacts and economic losses are analyzed. Natural hazards have increased in the last few decades, and have affected almost 10% of the world population, producing some "mean" annual damages of 50B$/year. The significance of floods within the natural hazards is described. Floods constitute about one third of all natural disasters, and include about one third of the number of victims and economic damages. Also, Chapter 3 gives statistical data on the effects of floods in the last 20 years.

Chapter 3 presents an analysis of the survey on the social and economic impacts of the floods, which was executed by the Committee. This included the 20 most significant countries with regard to floods, and to dams and floods, since in total they include about 80% of the large dams of the world. The data of this survey refer to 1) Most important natural hazards, 2) Mean incidence of the most important floods, 3) Social impacts, 4) Economic impacts and 5) Zones of risk to floods. Analysis of the survey data provided the following results :

1) In the greater part of the countries surveyed, floods constitute the most significant natural hazard.
2) The incidence of floods is great, and they frequently recur, with large floods, in the greater part of the cases, recurring in less than 10 years.
3) During the last decades in the greater part of the countries the mean number of victims produced by the floods is less than 20 victims per year, although in the more developed countries such as Japan and USA, values of about 100 victims per year are reached, due principally to the very intense occupation of the flood plains and to the occurrence of flash floods. The greater part of the victims with values about 85 to 90 % of the total of victims are produced in Asia, mainly in Korea, Bangladesh, India and China, with figures in this last country reaching 3000 victims per year, and an average of some 15 millions people affected per year.

4) Dans la plupart des pays, les dégâts causés par les crues sont, en moyenne, inférieurs à 500 millions de dollars/an, les montants étant cependant six fois supérieurs à ce dernier chiffre dans les pays très développés – 3 400 millions de dollars/an aux États-Unis et jusqu'à 7 200 millions de dollars/an au Japon.

Le chapitre 3 indique également les crues les plus importantes survenues depuis 1950 (1950-1998) et décrit les crues les plus fortes observées au cours des dernières décennies : Bangladesh, 1988 ; États-Unis, 1993 ; Pays-Bas, 1995 ; Corée, 1996 ; crues dans les régions de haute latitude ; et crues en Chine. Dans chaque cas, les caractéristiques spécifiques des crues, leurs causes, leur formation et leur évolution, et les impacts les plus importants sont décrits.

Chapitre 4 - Ce chapitre présente une nouvelle analyse des crues exceptionnelles dans le monde. Pour effectuer cette analyse, divers pays firent l'objet d'une enquête en vue d'obtenir les débits de pointe sur divers barrages dans le monde, ayant été exposés à des crues exceptionnelles. L'enquête a permis de rassembler 340 nouvelles données sur des crues exceptionnelles enregistrées sur des barrages. Ces données complètent les études effectuées par l'Association Internationale des Sciences Hydrologiques (AISH) en 1984, ayant servi à formuler les courbes enveloppes de crues au niveau mondial. Les nouvelles données ont été analysées et deux conclusions générales sont présentées dans le chapitre 4 : 1) Les données actuelles ne conduisent pas à une remontée de la courbe enveloppe au niveau mondial ; aucune donnée de crue exceptionnelle n'a dépassé la courbe enveloppe de Francou-Rodier correspondant au coefficient 6,4 de l'équation établie en 1984 - 2) La courbe enveloppe générale ne s'adapte pas bien aux crues exceptionnelles dans les bassins versants inférieurs à 300 km^2; une nouvelle courbe enveloppe semblable à celle de Francou-Rodier est proposée pour ces petits bassins, à laquelle les crues exceptionnelles s'adaptent mieux.

Chapitre 5 - Ce chapitre analyse le rôle des barrages dans la maîtrise des crues et la réduction des dégâts. Les mesures destinées à prévoir et à réduire les dégâts résultant des crues sont décrites et classées en deux catégories : actions structurelles et mesures non-structurelles. Dans le cadre des mesures de protection contre les crues, une attention particulière est portée aux barrages de maîtrise des crues et aux réservoirs régulateurs, qui constituent des solutions structurelles efficaces vis-à-vis des inondations, car ce sont les seules mesures permettant de stocker d'importants volumes d'eau, de modifier l'écoulement des crues et de réduire, de façon significative, les pointes de crues. De ce fait, il est fondamental que les barrages de maîtrise des crues et les réservoirs régulateurs soient conçus et exploités suivant une approche intégrée à l'intérieur du bassin considéré globalement, et associés à d'autres mesures structurelles et non-structurelles, en particulier le développement d'un zonage, les programmes d'utilisation des terres, les systèmes de prévision des crues en temps réel. Les termes « maîtrise des crues » et « atténuation des crues » sont discutés. Le terme « maîtrise des crues » conduit à penser que les barrages et les retenues sont capables de maîtriser les crues, en totalité, et d'assurer une protection totale des habitants et des villes à l'aval. Cette perception n'est pas réaliste, en particulier dans le cas des crues exceptionnelles. Pour cette raison, il conviendrait de changer le terme généralement utilisé aujourd'hui de « barrage et retenue de maîtrise des crues » en « barrage et retenue d'atténuation des crues ». Ce dernier terme est plus réaliste et indique que les barrages sont capables de réduire

4) In the greater part of the countries the mean damages caused by the floods are less than 500M$/year, although in the very developed countries the figures are six times superior that number, with 3400 M$/year in the USA, and up to 7200 M$/year in Japan.

Chapter 3 also presents the most important floods since the 1950's (1950-1998),and it describes the most important floods of the last decades : Bangladesh 1988, USA 1993, Netherlands 1995, Korea 1996, floods in high latitude countries, and floods in China. In each case, the specific characteristics of the floods, their causes, formation and evolution, and the most significant impacts are described.

Chapter 4 - presents a new analysis of extreme floods in the world. To conduct this analysis, various countries were surveyed to obtain data of peak flows at different dams in the world, which have been exposed to extreme floods. The survey resulted in 340 new data of extreme floods at dams. These data complete the studies carried out by IAHS in 1984, which served to formulate the envelope curves of floods at a world level. The new data were analyzed, and two general conclusions are presented in Chapter 4 : 1) With the present data an upward movement is not observed in the enveloped curves at a world level. No extreme flood data have surpassed the Francou-Rodier envelope curve of 6.4 established in 1984 ; and 2) The general envelope curve does not adapt well for the extreme floods in basins less than 300 km^2. A new envelope curve similar to that of Francou-Rodier is proposed for these smaller basins, to which the extreme floods are better adapted.

Chapter 5 - analyzes the role of dams in the flood control and in damage reduction. The measures to predict and reduce flood damages are described and classified in terms of structural actions and non-structural measures. Within the measures in the face of floods, special reference is made to flood control dams and regulating reservoirs, which constitute efficient structural solutions to flooding, as they are the only measures that can store significant amounts of water, modify the flood routing, and significantly reduce peak flow. For this, it is fundamental that flood control dams and regulating reservoirs be designed and operated with an integrated approach within the basin as a whole, and combined with the other structural and non-structural measures, in particular to develop zoning, land-use patterns, and flood forecasting systems in real time. Flood control and flood mitigation terms are discussed. The term flood control induces one to think that dams and reservoirs are capable of controlling all the floods, in total, and create a perception of total protection to the downstream inhabitants and townships. This perception is not realistic, especially in the case of extreme floods. For this reason it would be convenient to change the terminology generally used today from "flood control" to "flood mitigation" dams and reservoirs. The term flood mitigation indicates more realistically that dams are capable of significantly reducing flood damages , but cannot totally eliminate them, or reach a zero risk even in very extreme floods.

de façon significative les dégâts résultant des crues, mais ne peuvent les éliminer en totalité, ou conduire à un risque zéro dans le cas des crues très exceptionnelles.

Le chapitre 5 introduit également le concept d'efficacité hydraulique, très utile pour l'obtention de données quantitatives en vue de comparer les diverses solutions et combinaisons de digues et retenues en termes d'atténuation des dégâts. Les problèmes de conception d'un grand barrage sur la rivière principale près de la zone à protéger, ou de divers petits barrages situés dans la zone amont du bassin versant et sur les affluents de la rivière sont examinés. La conclusion est qu'en général une plus grande protection est obtenue avec des retenues situées près de la zone où les dégâts résultant des crues doivent être atténués.

Une partie importante du chapitre 5 est consacrée à la présentation et à l'analyse de divers cas significatifs où des barrages ont joué un rôle important dans l'atténuation des crues. Ces cas sont classés en trois groupes :

1) Cas réels de barrages en exploitation.
2) Barrages en construction, où l'atténuation des crues constitue un rôle important.
3) Grands projets de barrages d'atténuation des crues.

La première partie décrit l'efficacité de retenues de maîtrise de crues, situées sur des rivières au Japon (Rivière Tone et Rivière Chikugo), en Espagne (Bassin de l'Èbre), Tunisie (Région de Kairouan), Corée (Rivière Han), États-Unis (Fleuve Mississipi et District de Conservation de Miami), Norvège (Rivière Glomma) et Brésil (Rivière Parana). Les effets des barrages de maîtrise des crues sont quantifiés afin d'indiquer leur efficacité au cours des nombreuses périodes de crues. En général, ces cas étudiés concernent des plans de maîtrise des crues dans des zones étendues avec d'importants problèmes de crues, qui combinent les effets des retenues, des barrages, des digues et de la canalisation des rivières, avec l'exploitation en temps réel des systèmes de prévision des crues. Généralement, la période de retour de la crue de projet de protection, dans ces divers cas, varie de 35 à 200 ans, avec des situations particulières où elle atteint 500 à 1 000 ans. Dans les cas présentés ici, les effets des mesures de réduction des crues étaient importants, avec des valeurs de réduction des crues de pointe variant entre 25 % et 82 %. Pour de nombreuses crues, la réduction de la crue de pointe dépassait 50 %.

La partie consacrée aux barrages en construction, où l'atténuation des crues constitue un rôle important, présente les exemples suivants : Barrage des Trois Gorges sur le fleuve Yangtze en Chine, Barrage Seven Oaks en Californie (États-Unis), Barrages Cerrillos et Portugues à Porto Rico, Rivière Kitakami au Japon, et Bassins Segura et Jucar en Espagne. Un des buts principaux du barrage des Trois Gorges sur le fleuve Yangtze en Chine est la protection contre les crues sur le tronçon Jingjian, sur le cours moyen du Yangtze. Le niveau actuel de protection au moyen de digues correspond à une crue de période de retour de 10 ans qui, avec la construction du barrage, passera à 100 ans et pourra atteindre 1 000 ans si on y associe les opérations d'atténuation des crues au Lac Dongting. Le barrage Seven Oaks en Californie (États-Unis) protége plus de deux millions de résidents en Californie du Sud. Les barrages Cerrillos et Portugues, à Porto Rico, ont déjà réduit de façon significative les dégâts résultant des crues produites par les hurricanes Hortense et Georges. Enfin, les étapes finales de plans de maîtrise des crues sont

Chapter 5 also introduces the concept of hydraulic effectiveness, which is useful for obtaining quantitative data to compare different alternatives and combinations of dikes and reservoirs in terms of damage mitigation. Also, the problems of designing a large dam on the main river close to the area to be protected or various small dams located on the headwaters of the basin and on the tributaries of the river are discussed. This section concludes that, in general, greater protection is obtained with reservoirs located close to the area where flood damages are to be mitigated.

An important part of the chapter 5 is dedicated to presenting and analyzing various significant cases in which dams have played an important role in flood mitigation. These cases are grouped into three sections :

1) Real cases of dams in operation.
2) Dams under construction, in which flood mitigation has an important part.
3) Large future projects in flood mitigation dams.

The first section describes the efficiency of flood control reservoirs on rivers in Japan (Tone River and Chikugo River), Spain (Ebro River), Tunisia (Area of Kairouan), Korea (Han River), USA (Mississippi River and Miami Conservancy District), Norway (Glomma River), and Brazil (Parana River). The effects of the flood control dams are quantified and indicate their efficiency during numerous flooding events. In general, these cases studied refer to flood control plans in ample areas with significant flood problems, which combine the effects of the reservoirs, dams, dikes and river canalization with the real-time operation of flood forecasting systems. Generally, the return period of the protection design flood of these cases varied from 35 to 200 years, with singular situations reaching as high as 500 to 1000 years. In the cases presented here, the effects of flood reduction measures were significant, with values varying between 25 % to 82 % in the reduction of the peak flow. For numerous flood situations, the reduction of peak flow surpassed 50 %.

The section of dams under construction in which flood mitigation has an important role describes the Three Gorges Dam on the River Yangtze in China, Seven Oaks Dam in California, USA, Cerrillos and Portugues Dams in Puerto Rico, Kitakami River in Japan, and the Segura and Jucar Basins in Spain. One of the main purposes of the Three Gorges Dam on the River Yangtze in China is flood protection on the Jingjian stretch in the middle Yangtze. The actual level of protection with dikes is for a flood of return period of 10 years, which, with the construction of the dam, will move to 100 years and could reach 1000 years if it is combined with operations of controlled flooding of the Dongting Lake. The Seven Oaks Dam in California, USA, protects more than two million residents in Southern California. The Cerrillos and Portugues Dams in Puerto Rico have already significantly reduced damages from floods produced by Hurricanes Hortense and Georges. Finally, the final stages of Flood Control Plans are presented, namely that of the Kitakami River in Japan, and of the Segura and Jucar Basins in Spain.

présentées, à savoir ceux concernant la rivière Kitakami, au Japon, et les bassins Segura et Jucar en Espagne.

Ce chapitre se termine par la présentation de projets de barrages d'atténuation des crues au Japon, aux États-Unis, en Chine et en Espagne.

Chapitre 6 - Ce chapitre présente des « Recommandations relatives à la conception hydrologique des barrages d'atténuation des crues ». Le but de ce chapitre est de développer des concepts fondamentaux et de présenter des recommandations générales et techniques qui apporteront une aide à l'ingénieur de projet dans la conception hydrologique des barrages d'atténuation des crues. Ces barrages sont classés en trois grandes catégories : 1) Barrages dont le but unique est la réduction des dégâts résultant de crues, 2) Barrages à buts multiples dans lesquels l'atténuation des crues est le but principal, et 3) Barrages dans lesquels l'atténuation des crues est un but important, mais est secondaire par rapport aux autres objectifs, tels que l'alimentation en eau, l'irrigation ou la production d'énergie.

Les principes généraux relatifs aux barrages de réduction des crues sont formulés et les objectifs des retenues de maîtrise des crues sont décrits. En outre, on décrit les concepts de base des plans de maîtrise des crues, comprenant généralement un système de retenues souvent complétées par des digues et des murs de protection contre les crues.

Une partie importante des recommandations présentées dans le chapitre 6 concerne la conception hydrologique des barrages d'atténuation des crues, basée sur deux critères fondamentaux : 1) la crue de projet ou la crue de sécurité pour assurer la sécurité hydrologique du barrage et 2) la crue de projet de protection pour assurer la réduction des crues à l'aval. Des recommandations relatives au choix de la crue de projet, généralement basées sur la classification des risques, sont présentées. Des recommandations générales sont également données pour la détermination des périodes de retour de la crue de projet de protection dans les zones rurales et urbaines, qui varient généralement entre 20 et 200 ans. Dans les cas de grandes cités, et si les considérations sociales et environnementales sont importantes, des périodes de retour de 500 ou même 1 000 ans peuvent se justifier.

Les méthodes hydrologiques classiques utilisées dans la conception des barrages d'atténuation des crues sont décrites. La nouvelle méthode d'analyse basée sur le risque est également présentée. Cette analyse est une méthode d'étude qui tient compte explicitement des incertitudes dans les données techniques. Cette méthode comprend des analyses probabilistes d'incertitude, à partir desquelles l'application de l'analyse basée sur le risque dans le processus de conception est formulée.

Enfin, les principes généraux et la méthodologie de développement et de réalisation des systèmes de barrages d'atténuation des crues, dans le cadre des plans de protection contre les crues, sont décrits. Cette description comprend également les facteurs à prendre en considération pour le choix du type d'évacuateur de crue (évacuateur avec vannes ou sans vanne), ainsi que les consignes d'exploitation du barrage au cours des grandes crues.

Chapitre 7 - Ce chapitre traite de la sécurité hydrologique des barrages existants. Il se réfère aux récentes données statistiques sur les ruptures de barrages, qui indiquent que la submersion est la cause de rupture la plus fréquente (36 %), la

This chapter is concluded with the presentation of future flood mitigation dam projects in Japan, USA, China, and Spain.

Chapter 6 - presents "Guidelines for the hydrologic design of flood mitigation dams". The aim of this chapter is to develop basic concepts, make recommendations, and suggest general and technical guidelines that will assist the design and operational engineer in developing the hydrology of flood mitigation dams. The flood mitigation dams are classified into three general categories : 1) Single-purpose dams for flood damage reduction, 2) Multipurpose dams in which flood mitigation is the main objective, and 3) Dams in which flood mitigation is an important objective, but is secondary to purposes associated with other objectives, such as water supply, irrigation or hydropower.

The general principles for dams as flood reduction measures are formulated, and the objectives for flood control reservoirs are described. In addition, the basic concepts of flood control plans are described, which generally include a system of reservoirs often supplemented by means of dikes and floodwalls.

An important part of the guidelines in Chapter 6 refers to the Hydrologic Design for flood mitigation dams which is based on two fundamental conditions, given the hydrologic criteria for the design : 1) The inflow design flood or safety check flood to assure hydrologic dam safety, and 2) The protection design flood for downstream flood reduction. Recommendations for the selection of the design flood, generally based on dam hazard classification, are presented. Also, general recommendations are given for determining protection design flood return periods in rural and urban areas, which in general range between 20 and 200 years. In extraordinary cases with major cities, and if the economic, social and environmental considerations are favourable, return periods of 500 or even 1000 years may be justified.

Hydrological methods using the traditional formulation approach for designing flood mitigation dams are described. The new methodology of Risk-Based Analysis is also presented. Risk-based analysis is a method of performing studies that explicitly takes uncertainty of technical data into account. This method includes probabilistic uncertainty analyses, from which the application of the risk-based analysis in the design process is formulated.

Finally, the general principles and the methodology to develop and implement flood mitigation dam systems within flood control plans are described. This description also includes the factors to consider for selecting the type of spillway (gated versus ungated spillways), together with operation rules that should govern the functioning of the dam during the occurrence of major floods.

Chapter 7 - deals with the hydrological safety of existing dams. This chapter refers to the latest statistical data of dam failures, which shows that overtopping is the most frequent cause of failure (36 %), with the greater part of these failures due plus grande partie des ruptures par submersion survenant sur des barrages en remblai. De ce fait, la sécurité hydrologique est un élément essentiel de la sécurité des barrages, le critère de risque minimal devant être établi lors de l'évaluation de la crue de projet. Les critères utilisés depuis les années 1950 pour le choix de la crue de projet sont décrits, ainsi que leur évolution depuis cette date. Les critères de première génération étaient basés sur des considérations empiriques et générales, et étaient applicables à tout barrage en toute situation. Les critères de deuxième génération étaient principalement basés sur la classification des barrages suivant les risques à l'aval. Les conditions réelles dans divers pays sont établies, et les critères généraux et recommandations pour le choix de la crue de projet sont donnés, en se basant sur la classification des risques et sur la crue de projet et la crue exceptionnelle ou la crue de sécurité. Le chapitre 7 recommande que des mesures et des normes de sécurité maximale soient adoptées pour les barrages présentant un risque élevé. Les mesures de sécurité actuelles et les critères standard sont examinés, et leurs limitations analysées. Le chapitre présente également les nouvelles tendances dans l'évaluation de la crue de projet basée sur la méthode de l'analyse de risque. Il est indiqué que ces nouveaux critères basés sur l'analyse de risque sont encore dans une phase de développement et que leur utilisation a été limitée jusqu'à maintenant à quelques cas, sans une application généralisée.

Une partie de ce chapitre est consacrée à la description des principaux concepts et à la terminologie concernant la méthode, basée sur le risque, de détermination de la sécurité hydrologique des barrages. Les quatre principales phases de la procédure d'évaluation du risque : identification du risque, estimation du risque, mesures de réduction du risque et prise de décision, sont décrites, ainsi que les méthodologies d'étude basées sur les arbres d'événements. Les critères de base concernant les risques vis-à-vis de la vie sont également développés sous deux aspects, risque individuel et risque social, avec une analyse des conditions actuelles et l'indication des critères proposés dans divers pays.

Enfin, le problème de la sécurité hydrologique des barrages existants est analysé. L'adoption de nouvelles normes et règles, et la prise en compte progressive de fortes crues nécessitent une sécurité plus grande pour les nouveaux barrages, mais posent un problème lorsqu'elles sont appliquées à des barrages existants. En général, la philosophie est qu'il est souhaitable d'obtenir la même sécurité hydrologique dans les barrages existants que dans les nouveaux barrages. Les nouveaux critères concernant le choix des crues de projet seront pris en considération dans l'évaluation de la sécurité des barrages existants et dans leur adaptation aux nouvelles règles. Des cas présentant un certain relâchement de ces critères sévères sont analysés, en plus des différentes méthodologies relatives à l'évaluation de la sécurité hydrologique des barrages existants. Ces méthodologies comprennent le classement par ordre de priorité et l'application de techniques basées sur l'analyse de risque. On indique également que les évaluations basées sur l'analyse de risque sont un complément aux critères classiques plutôt qu'une solution alternative.

Chapitre 8 - Ce chapitre contient les références mentionnées dans le Bulletin, celles-ci étant classées suivant les divers chapitres.

to overtopping occurring in embankment dams. Because of this, hydrological safety is an essential element of dam safety, for which the criterion of minimum risk must be established in the evaluation of the design flood. The criteria used since the 1950s for the selection of the design flood are described, and the evolution of that criteria since then. The first-generation criteria were based on empirical and general considerations, and applicable to any dam in any situation. Second generation criteria were principally based on the classification of dams according to the downstream hazards. The actual conditions in different countries are developed, and the general criteria and recommendations for the selection of the design flood are given, based on the dam hazard classification and on the consideration of the design flood and the extreme flood or safety check flood. Chapter 7 recommends that maximum safety measures and standards be adopted for high hazard dams. The present safety measures and standards criteria are discussed their limitations are analysed. This chapter also presents the new trends in the design flood assessment based on the risk analysis approach. This information indicates that these new criteria based on risk analysis are still in the development stage and that their use has been limited so far to some cases and without full application.

A part of this chapter is dedicated to describing the principal concepts and terminology of the risk-based approach to determining hydrological safety of dams. The four principal phases of the risk assessment process, risk identification, risk estimation, risk action, and decision, are described together with the methodologies of study based on event trees. Also the basic criteria for the risk to life are developed both from the individual perspective, and from the social point of view, with an analysis of actual conditions and the criteria proposed in different countries.

Finally, the problem of hydrological safety at existing dams is analyzed. The adoption of new standards and regulations, and the progressive consideration of large floods requires greater safety for new dams, but presents a problem when applied to existing dams. In general the philosophy is that it is desirable to reach the same hydrological safety in the existing dams as in the new dams. The new criteria for the selection of design floods should be taken into account in the safety evaluation of the existing dams, and in their accommodation to the new regulations. Cases which provide a certain relaxation of these strict criteria are analyzed, in addition to the different methodologies for evaluating the hydrological safety of the existing dams. These methodologies include the rankings score and the application of techniques based on the risk analysis. It is also pointed out that evaluations based on Risk analyses serve to complement the traditional criteria rather than being an alternative.

Chapter 8 - contains the references aforementioned in the Bulletin, which have been organized and detailed following the different chapters.

3. SOCIAL IMPACTS AND DAMAGES PRODUCED BY FLOODS

3.1. INTRODUCTION

Natural disasters significantly affect human life and produce serious social impacts and important economic losses. From 1970 to 1985, 657 catastrophic disasters were related to hurricanes, earthquakes, and floods occurred, which represent an average of 40 disasters per year due only to these three natural phenomena. These disasters have affected more than 220 million people, or almost 5 % of the world population, with economic losses of some 20 million $ per day [1, 2, 3].

The number of major natural disasters in the world in terms of significant damage (1 % or more of total annual GNP), affected persons (1% or more of total population), and numbers of deaths (100 or more) are increasing with time, as deduced from IDNDR studies [2, 4, 5]. Fig. 3.1 illustrates the increases in these disaster variables as they have occurred from 1963 to 1992.

The number of significant flood disasters are greater than any others, in terms of significant damage and number of deaths (32 % in relation to the significant damage, and 26 % in relation to the number of deaths) (Fig. 3.2), when compared to other types of disasters. Fig. 3.2 also indicates that the numbers of significant flood disasters in terms of persons affected is also high (32 % in relation to the affected people). Fig. 3.3 shows the increasing number of catastrophic floods in the last four decades for all of the variables described above. All natural disasters produce substantial economic losses with mean present day losses of some 50 billion dollars per year, which have significantly increased in the last four decades (Fig. 3.4), having produced in the latter years natural catastrophes with grave social and economic impacts (Table 3.1).

Table 3.1
Most significant natural disasters from 1988 to 1998

DATE	YEAR	EVENT	REGION	DEATHS	ECONOMIC LOSSES US $ million
December	1988	Earthquake	Armenia	25 000	14 000
September	1989	Hurricane Hugo	Caribe. USA	61	9000
Jan-Feb	1990	Winter gales	Europe	230	15 000
June	1990	Earthquake	Iran	40 000	7000
Summer	1991	Floods	PR China	3074	15 000
September	1991	Typhoon Mireille	Japan	62	6000
August	1992	Hurricane Andrew	USA	74	30 000
July-Aug	1993	Floods	USA	38	15 600
January	1994	Earthquake Los Angeles	USA	59	13 000
January	1995	Earthquake-Kobe	Japan	6300	50 000
July-Aug.	1998	Floods	PR China	4150	30 000
October	1998	Hurricane Mitch	Central America	21 000	4000

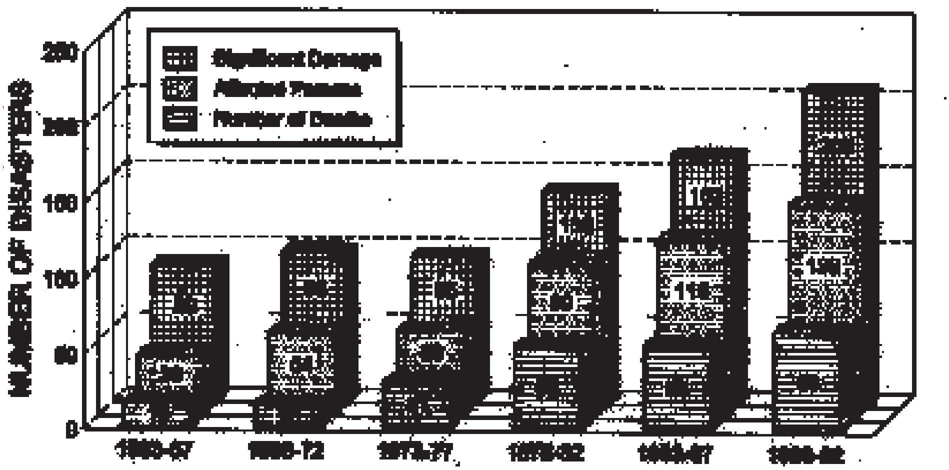


Fig. 3.1
Major disasters around the world, 1963-1992
Significant disasters based on damage, affected persons, deaths

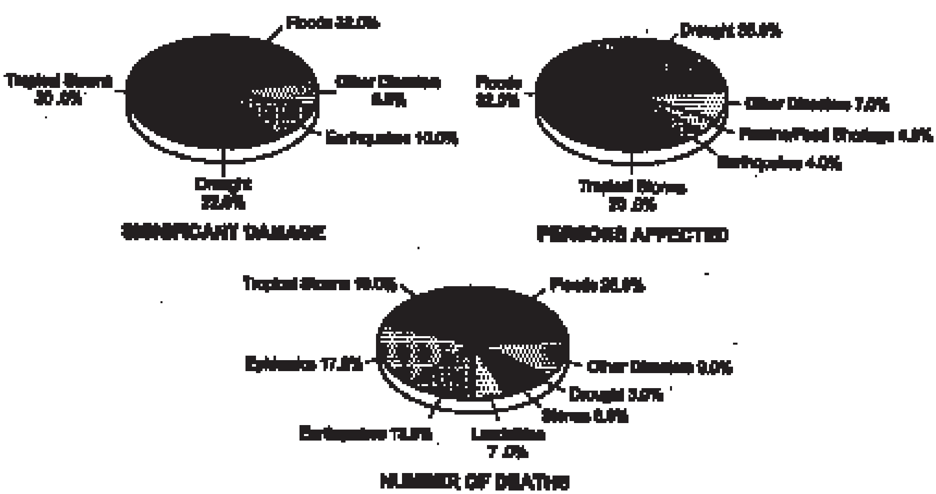


Fig. 3.2
Major disasters around the world, 1963-1992
% of significant disasters by type, based on: damage, persons affected, deaths

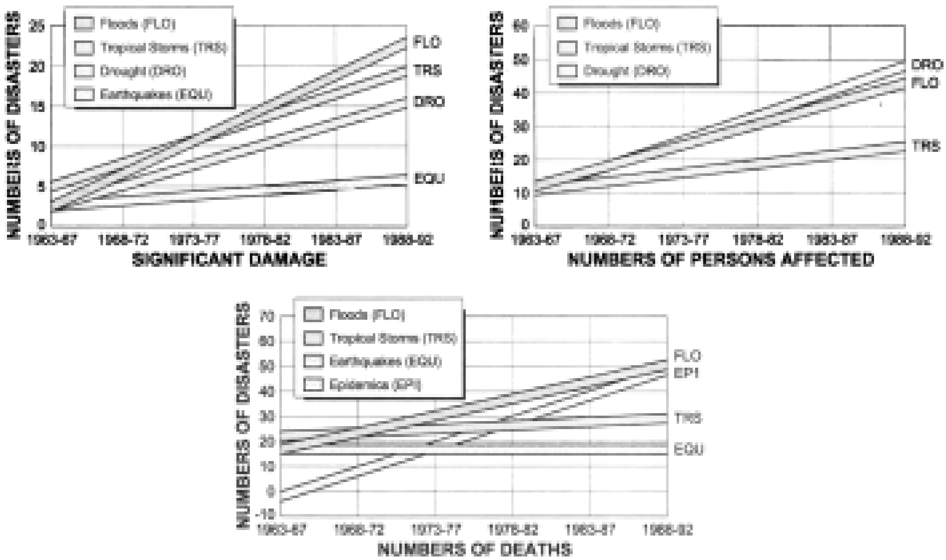


Fig. 3.3
Major disasters around the world
Trends: Most significant disaster types by category

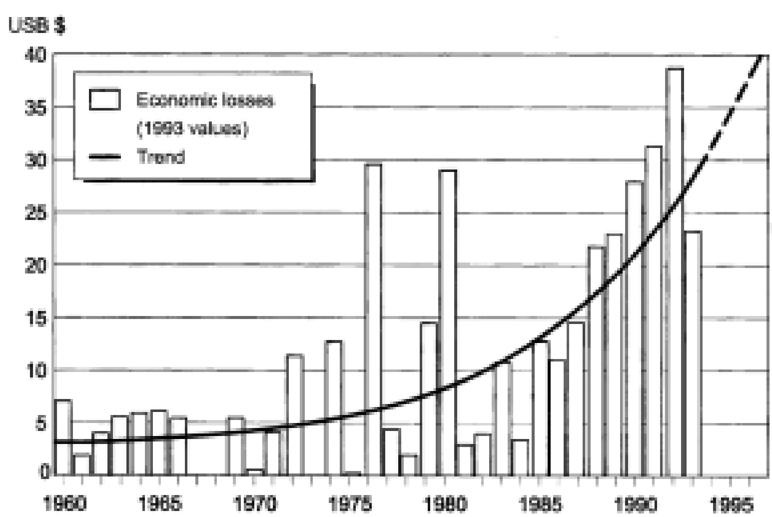


Fig. 3.4
Great natural disasters, 1960-1993
Economic losses

Regarding statistics that refer to floods, in 1988, UNDRO published data relative to 162 floods, which had produced more than 10 deaths and/or 1 M $ of damage, during the period 1980 through 1985 [2]. These floods caused approximately 33 500 deaths, affected 184 million people, and left more than 19 million homeless. The greater part of the loss of lives, almost 95 %, occurred in developing countries, where economic impacts are also greater, with which they can affect and even prevent the emergence from the country's actual economic development situation.

The impacts of floods have increased in the recent years, and so the statistical data refers only to great floods (floods which produce damages to the order of one billion dollars and/or fatalities to the order of thousands) that occurred during 1986 to 1996 are shown in Table 3.2

Table 3.2

Great floods 1986 - 1996 [6]

YEAR	FLOODS	DEATHS	ECONOMIC LOSSES US$ million
1986	2	4684	4135
1987	6	2241	5932
1988	1	3000	1695
1989	1	12	509
1990	0	0	0
1991	1	3074	18 426
1992	1	2000	1179
1993	3	65	22 062
1994	2	1910	17 955
1995	1	28	3675
1996	1	2700	24 000
TOTAL	19	19 714	99 568
Average per year	1.7	1794	9052

All these data show that floods disasters are one of the most important of natural disasters and have the most impact on the population.

So then, in spite of the progress in the knowledge of the natural hazards, and in particular of floods, the damages caused by the natural catastrophes continue to increase. To address this issue, the UN decided in 1987 [7] to create the International Decade for Natural Hazard Reduction (IDNHR), for the decade 1990-2000, with the objective of using concerted international action to reduce loss of lives (especially in developing countries), material damages, and the social and economic disorders caused by the natural hazards. Among the essential elements of the activities of the IDNHR the following points stand out [8] :

1. Greater emphasis in planification and preventive measures.
2. Adoption of integrated actions (structural and non-structural) to reduce the impact of disasters.

3. Establishment of forecasting and alarm systems compatible with the technology and culture of countries.
4. Development of a social conscience of the necessity of the reduction of the impacts.

Without any doubt, dams have a very important role in the development of the activities for the reduction of the damages produced by the floods.

3.2. ANALYSIS OF SURVEY ON THE SOCIAL AND ECONOMIC IMPACTS OF FLOODS

A survey was conducted to determine the social and economic impacts of the floods in the countries represented on the Committee, and others in which the floods produced catastrophic damages. Data were obtained from Argentina, Australia, Bangladesh, Brazil, Canada, China, France, India, Indonesia, Ireland, Italy, Japan, Korea, Netherlands, Norway, Russia, South Africa, Spain, Sweden and the USA. Although the survey sample is not total, it constitutes a significant representation in the field of dams, as close to 80 % of the total of the world's large dams are located in these 20 countries.

The data from survey responses were analyzed and the results are summarized in the following sections : 3.2.1) Most important natural hazards in each country, 3.2.2) Mean incidence of the most important floods, 3.2.3) Social impacts, 3.2.4) Economic impacts, 3.2.5) Zones of risk of floods.

3.2.1. Most Important Natural Hazards

The most significant natural hazards for each country are shown in Table 3.3, in which can be observed the great significance of the floods in the countries studied, which represent in 65 % of the cases the most important natural hazards in those countries. Likewise, in 90 % of the cases, floods constitute the first or the second most significant natural hazard.

Table 3.3

Most significant natural hazards

NATURAL HAZARDS	SIGNIFICANCE	
	FIRST	SECOND
Floods	13	5
Storms (Winds, Surges)	3	4
Earthquakes	2	1
Cyclones	1	2
Volcanic Eruptions	1	–
Droughts	–	6
Landslides	–	2

3.2.2. Mean Incidence of Most Important Floods

The mean incidence of important floods was 7.2 years (7.2 ± 7.9, n = 19), with an interval between 1 and 35 years. Also in 16 countries (85 %), the number of years between important floods was less than 10 years, and in 13 countries (68 %) this value was less than 5 years. These values represent a great incidence and strong recurrence of the presentation and perception of important floods.

3.2.3. Social Impacts

The evaluation of the "mean" number of victims per year caused by the floods is shown in the Table 3.4, although one must be take into account that occasionally occurring extreme floods produce a much higher number of victims, with much higher punctual effects.

Table 3.4
"Mean" number of victims per year

VICTIMS	COUNTRIES
0 – 10	Argentina, Australia, Brazil, Canada, France, Ireland, Italy, Netherlands, Norway, South Africa, Sweden, Russia.
10 – 20	Spain.
50 – 100	Indonesia, USA.
100 – 150	Japan.
> 150	Korea (250), Bangladesh* (200), India (1500), China (2000-3000).

* Includes only floods caused by rivers and simultaneous local rainfall. Flooding caused by storm surges and by cyclones and resulting in many victims has not been included.

In most of the cases, the mean number of victims per year is lower than 20, whether it be due to the implementation of structural and non-structural measures in the face of the floods. Nevertheless, there are outstanding, the cases of the USA and Japan (94 and 114 victims, respectively), both very developed countries with intensely occupied flood plains, which together with the presence of flash floods gives rise to these figures. Even so, these figures have been slowly decreasing in the last few years, because important preventive measures were used. The greater number of victims due to floods is produced in the Asian countries.

In regard to the "mean" number of people affected per year by floods, Argentina, Brazil, Japan, the Netherlands (whose numbers may be distorted because of the 1995 flood when about 200 000 people were evacuated), Russia, and the USA have more than 10 000 people affected by floods. In Indonesia, floods affect more than 400 000 persons, and in India, China and Bangladesh, figures can reach a mean of more than 15 million people per year.

3.2.4. Economic Impacts

Table 3.5 shows the evaluation of the mean annual damages produced by the floods in various countries under study.

The high amount of damages observed in very developed countries such as the USA and Japan are due to the high economic value of the properties and structures installed in the areas prone to floods.

Also, very important are incidents in underdeveloped countries, where the amount of the damages and the social disruption caused by the floods could become a limiting factor in development.

Table 3.5
Mean annual damages produced by floods

COUNTRY	DAMAGES (M $/per year)
Brazil, France, Ireland, South Africa, Sweden	< 10
Norway	27
Argentina	30
Canada	100
Bangladesh*	135
Netherlands	150
India	240
Australia	320
Russia	380
Korea	500
Spain	600
Italy	800
China	3000
USA	3400
Japan	7200
* Only floods caused by rivers	

3.2.5. Flood Risk Zones

Table 3.6 shows the percentages of the area and of the population in the flood risk zones in the different countries in which it has been possible to obtain information. In various cases, information of this type does not exist at a national level, and in any case the figures are only approximate.

Table 3.6
Percentage of areas and population affected by floods

COUNTRY	% AREA	% POPULATION
Norway	1.0	–
South Africa	2.0	–
France	3.0	–
Sweden	0.2	0.5
Argentina	1.0	1.3
Brazil	6.0	8.0
Netherlands	6.0	8.1
Canada	2.0	10.0
Indonesia	–	30.0
India	12.0	10.0
Italy	20.0	30.0
Japan	10.0	50.0
Korea	98.0	99.0

It can be observed from the data given above that countries with greater affected areas and populations are those which suffer more impacts due to the floods, and only in the cases in which ample structural measures and flood forecasting systems have been developed, has it been possible to progressively lessen the damages.

3.2.6. Most Important Floods since 1950

The most important floods in the second half of this century were selected from information obtained from the survey. These floods are listed in Table 3.7.

Table 3.7
Most important floods since 1950

COUNTRY	YEAR	VICTIMS	DAMAGE M $ without inflation
Australia	1974	14	220
Argentina	1966	0	226
	1983	0	664
Bangladesh	1974	28 000	–
	1988	1655	1300
Brazil	1983	–	120
	1992	–	40
Canada	1954	81	500
	1996	0	1000
China	1954	33 000	1350
	1975	30 000	800
	1998	4150	30 000
France	1987	23	–
	1992	30	–
India	1959	8000	70
	1978	1220	340
Japan (Typhoon)	1959	5098	22 500
Korea	1990	179	726
Netherlands	1993	0	254
	1995	5	1650
Norway	1987	0	40
	1995	1	240
Russia	1994	36	295
South Africa	1987	380	–
	1995	170	–
Spain	1962	973	25
	1982	38	3000
Sweden	1973	1	–
	1985	0	35
USA	1955	187	715
	1972	118	7000
	1993	38	15 600

Of these catastrophic floods listed in Table 3.7, several cases in different countries listed below, are discussed in the following sections (3.3 to 3.8) :

– Bangladesh	1998
– USA	1993
– Netherlands	1995
– Korea	1996
– High latitude countries	
– China	

3.3. THE BANGLADESH FLOOD OF 1988 [9-12]

3.3.1. Introduction

Most of Bangladesh is located within the flood plains of three great rivers (the Ganges, the Brahmaputra and the Meghna), their tributaries and distributaries (Fig. 3.5). The three rivers drain into a total catchment area of about 1.6 million square kilometres, of which 9 % lies within Bangladesh.

In summer, the warm, moist air of the monsoon sweeps up the Bay of Bengal from the Indian Ocean, producing some of the highest recorded rainfalls in the world over Bangladesh and the upstream catchments of the major rivers, particularly in the Indian states of Meghalaya and Assam as well as over northeastern Bangladesh.

The annual rainfall over the area varies greatly in both time and location. Seventy to eighty-five percent of the annual rainfall occurs during the monsoon season from May to September. The average annual rainfall within Bangladesh varies from about 1100 mm in the extreme west to a maximum of 5500 mm in the northeastern corner of the country. Average annual rainfall in the Himalayas, Khasia and Jaintia Hills ranges generally up to about 5000 mm annually and up to 10 000 mm locally.

The overall upstream catchment of the three main rivers entering Bangladesh is 1.504 million square km. The area of Bangladesh is 144 000 km^2, i.e. 9.57 % of upstream catchment.

The Brahmaputra starts rising in March due to snowmelt on the Himalayas which causes a first peak in May or early June. It is followed by subsequent peaks caused by the heavy monsoon rains over the catchment up to the end of August. The response to rainfall is relatively quick, resulting in rapid increases of water level. Six to ten days will elapse from a rainfall in the upper catchments until the corresponding peak is felt within Bangladesh.

The Ganges starts rising gradually in May-June to a reach a maximum peak sometime in August. High water levels are normally sustained until mid-September.

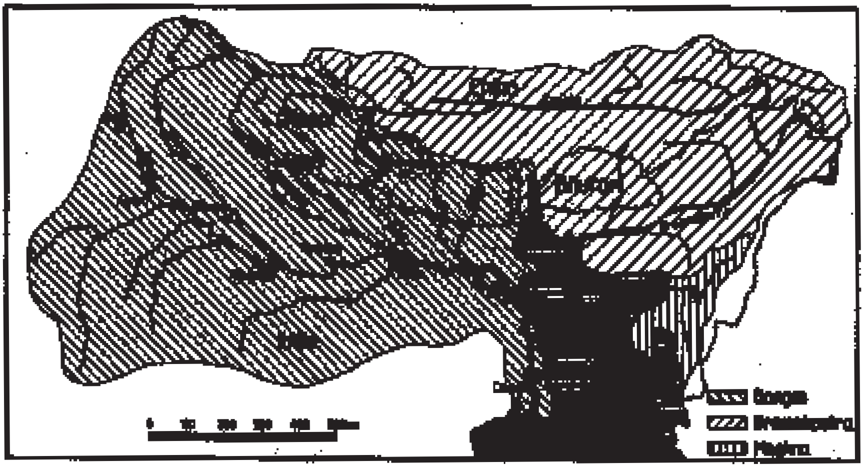


Fig. 3.5
Catchment basins of the rivers Ganges, Bramahputra and Meghna

The Meghna may not attain its annual peak until August-September. The upper Meghna carries only about 10 % of the flow in the Ganges and Brahmaputra, and this flow is generated by rainfall within Bangladesh and in adjacent areas in India in approximately equal proportions.

The total volume of runoff in the Ganges, the Brahmaputra and the Meghna is determined by the net precipitation over all catchments (Fig. 3.5). It is subject to short and long-term variations.

3.3.2. Flood Control Options

Embankments along upstream reaches of the Ganges and the Brahmaputra will remove parts of the flood plain storage. This storage removal will lead to higher peak levels and steeper rise of downstream water levels during floods. Further construction of embankments along upstream reaches of the main rivers will lead to increased peak discharges and water levels inside Bangladesh. This would put additional emphasis on the need for flood control structures along downstream reaches of the major rivers. Furthermore, if the deposition of sediment on upstream flood plains is prevented by embankment development then the sediment load transferred to downstream areas may well be increased.

Upstream reservoirs, which are operated so as to store flood runoff, would have some effects of reducing at least early flood season runoff. Reservoirs operated for other purposes than flood control would tend to be filled during the earlier part of the monsoon season, and would thus probably modify the flood peak.

Bangladesh is located a long distance downstream of potential sites for major reservoirs, and reservoirs therefore do not represent a promising flood control option. Huge storage volumes would have to be created, the dams would have to be operated specifically for flood control purposes, and deciding when to start storing monsoon inflows would be difficult. Furthermore, significant runoff is generated by rainfall in the substantial catchment area downstream of possible reservoirs.

3.3.3. Types of Floods in Bangladesh

While the monsoon dominates the rainfall pattern in Bangladesh, the flooding is a result of a series of factors. These factors include the rapid rate of runoff, the location and effect of the confluence of the major rivers inside the country, the extremely flat topography of the area, and the influence of tides and surges in the Bay of Bengal.

In Bangladesh, the following types of floods are normally encountered :

- Flash floods in the eastern and northern rivers
  These floods are characterized by a sharp rise followed by a relatively rapid recession a few days later often causing high flow velocities that damage crops and properties.

- Rain flood due to high intensity rainfalls
  The very high local rainfall intensities and long durations in the monsoon season will often generate runoff volumes in excess of the local drainage capacity.

- Monsoon floods from the major rivers
  Overbank flood spills of the major rivers and their distributaries cause the most extensive flood damage in Bangladesh. With the onset of monsoon all the major rivers start rising. The major rivers generally rise slowly and the period of rise may extend over several weeks. Most extensive flood damages are caused when the three major rivers peak simultaneously.

- Floods due to storm surges in the coastal area
  The coastal area of Bangladesh consists of large estuaries, extensive tidal flats, and low-lying islands. Storm surges generated by tropical cyclones cause widespread damage to life and property. Cyclones are most common during the pre- and post-monsoon periods (April-May and October-November respectively), and have not been known to coincide with monsoon flood peaks.

In recent years, as a result of unplanned construction of rural roads and flood embankments, drainage has often been impeded which has aggravated the flooding conditions.

The normal sequence of floods starts with flash floods in the eastern hill streams during the pre-monsoon period in the months of April and May. Onset of the monsoon generally occurs in June and the Meghna and the Brahmaputra reach flood peaks during July and August, whereas the Ganges normally peaks during August and September.

Heavy flooding occurs if the peaks of the Ganges and the Brahmaputra coincide ; this may happen during August-September.

3.3.4. The 1988 Flood

In 1988, Bangladesh experienced one of the worst floods in living memory. Of the country's 64 districts, 53 were inundated to varying degrees, which means approximately 90 000 km^2 (63 % of the country), and almost half of the total population (125 million) were directly affected by the floods. In the rural areas, damage to crops, livestock, housing, roads, railways, water control works, social and other services was severe. Flooding reached unprecedented levels in Dhaka, which added to the difficulty of Government in coping with the national emergency. Some 1655 lives were lost while damages amounted to US $ 1300 M.

It must be noted that the Brahmaputra and the Ganges are two of the world's most difficult rivers on which to carry out discharge measurements. Hence, great precision in flood range ratings should not be expected, and flood frequencies and trends derived from flood analysis should be interpreted with corresponding caution. The uncertainties involved in extrapolating frequency plots to return periods significantly longer than the data periods (30 to 50 years for key stations) should also be acknowledged.

Table 3.8
Flood flow frequencies

River	Brahmaputra	Ganges	Padma	Meghna	Teesta
Station	Bahadurabad	Hardinge Bridge	Baruria	Bhairab Bazar	Kaunia
Return Period (yrs)	Flood Peaks in m^3/s × 1000				
MAF	65.5	51.3	95.3	14.0	5.1
5	73.1	58.2	106.3	15.9	6.1
10	79.3	63.7	115.3	17.5	7.1
20	85.3	69.1	123.9	19.0	8.4
50	93.0	76.0	135.0	20.9	9.7
100	98.8	81.1	143.4	22.3	10.6
200	104.6	86.3	151.7	23.8	11.5
1987 Flood Peaks :	74	76	113	15.1	8.81
1988 Flood Peaks :	98.6	72.3	132	19.8	6.87
1987 Return Period (yrs)	6	50	8	4	27
1988 Return Period (yrs)	97	31	39	30	7
Trend in 1000 m^3/s/year :	0.14	0.28	1.0	0.09	0.03

Table 3.8, which indicates flood flow frequencies for the main rivers and the Teesta, enables the 1987 and 1988 flood flows to be seen in the perspective of recorded experience. This Table shows floods that would be expected for a range of

frequencies based on a data set including the 1988 data. Table 3.8 also indicates the frequencies of the 1987 and 1988 floods on the basis of these data set. It can be seen that the 1988 floods had return periods in the order of 30 years for the Ganges, 7 years for the Teesta, 100 years for the Brahmaputra, and about 40 years for the Padma.

In contrast to the largely internally generated (i.e. by local rainfall) floods of 1987, the severe 1988 floods were generated by intensive rainfall that extended over North and Northeast Bangladesh, India, Nepal and Bhutan, with the most intense local rainfall concentrations in Assam, Meghlaya, Bhutan and Arunachal Pradesh. The flood peak of the Brahmaputra was the highest ever recorded. The flood peak of the Ganges was also high, but most significantly, the two peaks unusually coincided, with devastating effects on the Padma downstream of the Brahmaputra/ Ganges confluence. Very large overbank areas along the Brahmaputra, Ganges and Padma were flooded to an unprecedented extent, and Dhaka was seriously affected for the first time. An exceptionally high flood on the Meghna added to the flood congestion in the Lower Meghna.

The floods of 1988 started when some rivers in the southeastern hill basins crossed danger levels in early May. Subsequently, another wave of flash floods occurred in June, peaking around 27th. Flood levels, however, were lower than the corresponding 1987 flood levels. Rivers in the Meghna basin were highly flooded in early July due to heavy rainfall in the Meghalaya. The flood was of a very high intensity and exceeded the peak levels of 1974 and 1987. The storm crossed over the Meghalaya, entered the Assam valley and caused flooding in the rivers of the Brahmaputra basin with the result that the Teesta, Dharla and Brahmaputra crossed their danger levels.

The Brahmaputra remained in flood stage from early July and had two other peaks on 10 July and on 30 July prior to the catastrophic flood of August-September (Fig. 3.6 and Fig. 3.7). Whereas the Ganges normally peaks some weeks after the peak of the Brahmaputra, there was a close synchronisation of a late and extreme peak on the Brahmaputra and a moderately high peak on the Ganges (time difference about three days). The effect of this synchronisation resulted in a very serious flood situation.

The highest recorded flood levels were exceeded at 10 out of 34 water level stations monitored. The 1987 peak levels were exceeded at 22 stations. The entire flood plain of the Ganges, Brahmaputra and Meghna river systems of the country, including Dhaka, Narayanganj and Tongi, was inundated. Most cities in the flood plains were affected to varying degrees, depending on local conditions.

For the Brahmaputra, the flood peak flow at Bahadurabad was estimated to be 98 600 m^3/s, and its peak level of 20.62 m PWD was 0.3 m higher than the previously recorded maximum level of 1958. The estimated Ganges flood peak at Hardinge Bridge was 72 300 m^3/s, which is not much less than the 1987 highest recorded flood. As a consequence of the high floods on both the Brahmaputra and the Ganges and the close synchronization of their peaks, the Padma flood peak of 132 000 m^3/s at Baruria was also the highest on record, though for a rather short record period of 21 data years since 1966. The Meghna flood at Bhairab Bazar was estimated to be 19 800 m^3/s, which is also the highest on record in a relatively short data period (20 years).

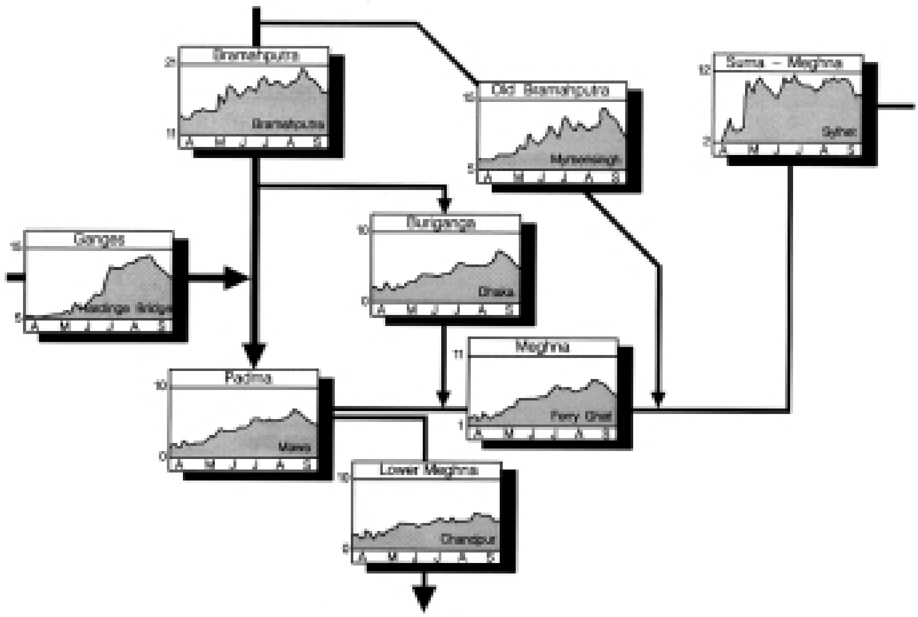


Fig. 3.6
1988 flood hydrographs [9]

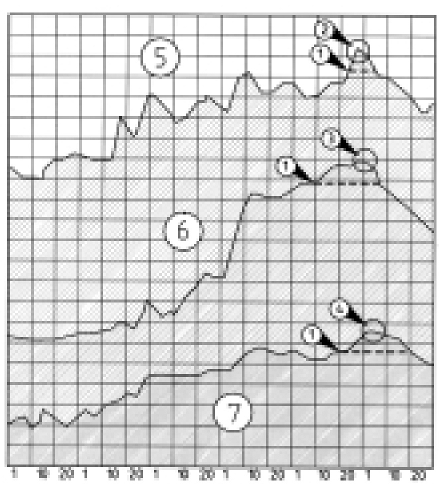


Fig. 3.7
1988 hydrographs of major rivers compared [9]

1) Danger level
2) August 30, 1988
3) Sept. 2, 1988
4) Sept. 7, 1988
5) River Bramahputra at Bahadurabad
6) River Ganges at Hardinge Bridge
7) River Meghna at Meghna ferry crossing

An indication of the flood extent as derived from NOAA images is shown in Fig. 3.8. The Figure has been compiled from the only cloud-free NOAA images for 15 and 24 September.

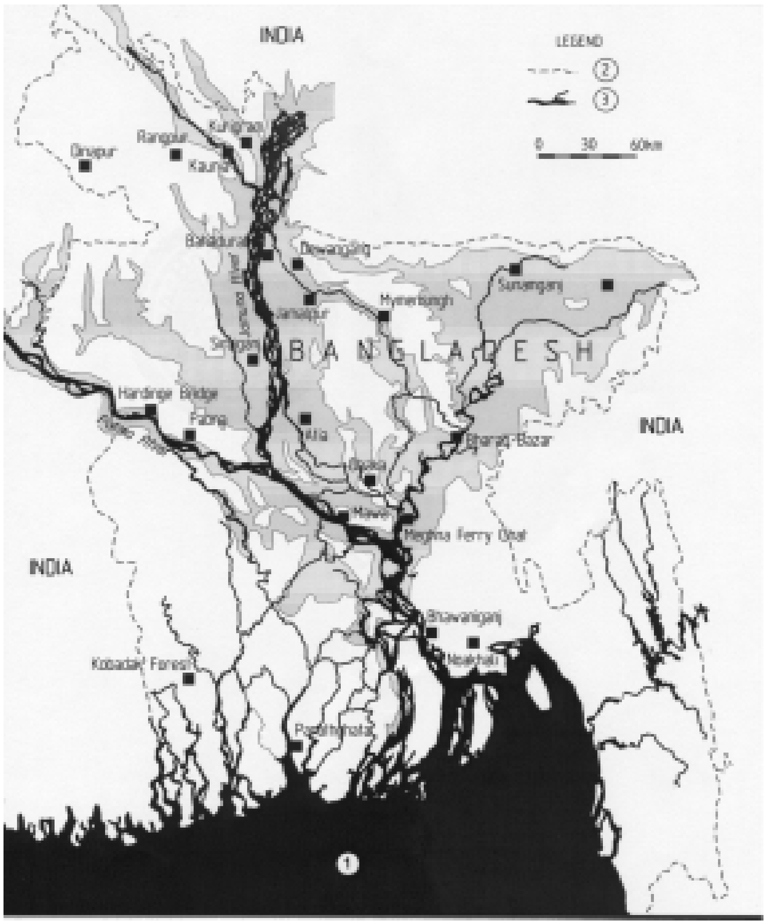


Fig. 3.8
Flood extent [9]

===== Flooded area from 14 and 24 Sept. 1988 as derived from NOA-10 satellites ; Peak flooding extent 1-3 Sept. not shown

■ Location of key rainfall stations and river gauging stations

1) Bay of Bengal
2) International Boundary
3) River

3.4. THE GREAT FLOOD OF 1993 IN THE UPPER MIDWEST, USA [12-13]

3.4.1. Introduction

In 1993, one of the most devastating floods in the history of the United States occurred in the upper Midwest portion of the country (Fig. 3.9). The cause of this tragic flood was the repetitive occurrence of heavy precipitation over a long time period. The resulting runoff overwhelmed natural channel capacities and many flood protection facilities, and resulted in untold damage and the loss of 38 lives. A brief description of the upper Mississippi River basin and the impacts of the 1993 Flood on the people and property located therein is provided below.

The outlet of Lake Itaska in north central Minnesota is the source of the Mississippi River. From this point, the river falls 445 m as it flows in a southerly

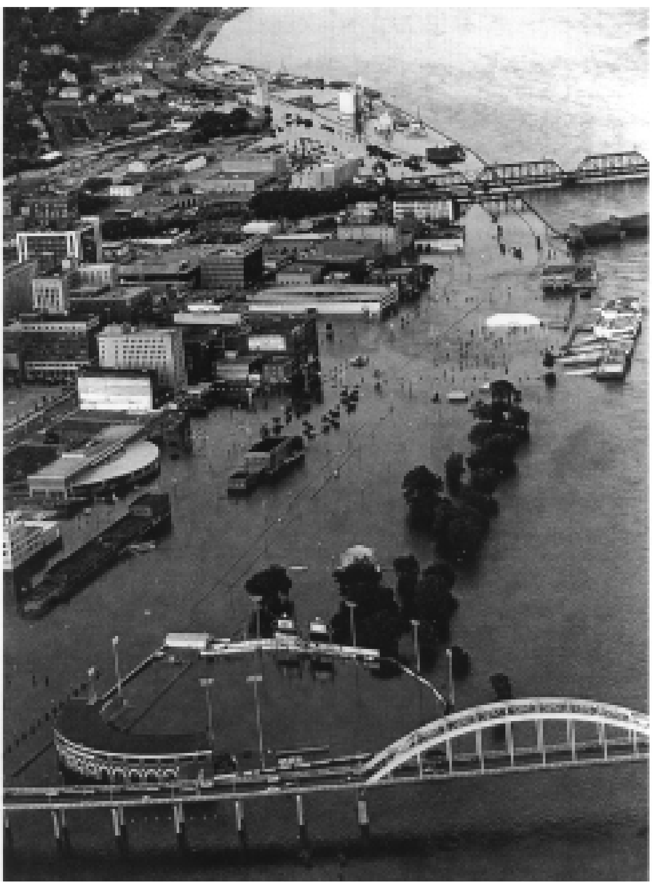


Fig. 3.9
Mississippi River at Davenport, Iowa. View of Davenport looking upstream. July 1993.
Flooded athletic field in foreground
(Photo courtesy of the "Quad City Times" Newspaper. Davenport, Iowa)

direction for 3735 kilometers to the Gulf of Mexico. The Mississippi River drains an area of 3.25 million square kilometers, which is equivalent to 41 percent of the area of the forty-eight contiguous United States. The portion of the Mississippi River drainage above the confluence with the Ohio River is referred to as the upper Mississippi River basin (Fig. 3.10). The upper Mississippi River basin drains 1.85 million square kilometers of land lying in all or part of 13 states. Its principal tributary is the Missouri River, which joins the Mississippi River at St. Louis, Missouri, and drains 1.37 million square kilometers, including 25 100 square kilometres in Canada. The climate of the upper basin is highly variable, ranging from semi-arid climates in Colorado and Wyoming to humid-temperate climates in the Great Lakes region. Because of this climate variability, the Missouri River contributes only 42 percent of the long-term average annual flow at St. Louis, even though it drains 74 per cent of the upper basin.

3.4.2. The Flood

Beginning in November 1992, above-normal monthly precipitation occurred throughout the entire upper midwestern portion of the United States. At many locations in the upper Mississippi River basin the November-July time period was the wettest nine months ever recorded. This region of the United States usually experiences high runoff early each spring as a result of snowmelt. In the spring of 1993, the volume of mountain and plains snowmelt runoff was extremely large, which resulted in some minor flooding but, more importantly, established very wet conditions in April, which is the beginning of the high water season in this part of the country. During April and May, many areas in both river basins received unusually high amounts of precipitation that caused significant local flooding and prolonged the basin-wide saturated conditions. Between April 1 and July 25, areas of western Kansas, southeastern Nebraska, southern Iowa, and northern Missouri experienced well-above-normal rainfall. Normal amounts for that period range from 250 mm to 305 mm. Recorded amounts during the spring of 1993 ranged from 457 mm to as much as 940 mm. The large rainfall amounts were caused by a very unusual weather pattern, created by a stable upper-level atmospheric circulation pattern with a deep trough to the west of the upper Mississippi valley and a persistent ridge of high pressure that had set up over the southeastern United States (Fig. 3.11). This high-pressure ridge kept the Southeast U.S. very hot and dry, while effectively directing each storm over the flood-affected area as it progressed across the country.

The large amounts of precipitation, which fell on the already saturated ground, resulted in record runoff into the upper Mississippi and lower Missouri Rivers. Initially, the heaviest flooding occurred in the uppermost reaches of the upper Mississippi River basin, causing much damage along the Minnesota River and the mainstem Mississippi River. Subsequently, the high-pressure ridge shifted south and west bringing persistent rainfall to the states of North Dakota, South Dakota, Nebraska, Kansas, Missouri, Iowa and Illinois. As a result of the well-above-normal amounts of rain, major flooding, exceeding all previous records at many locations, occurred throughout all or parts of nine states in the region. New record high water levels were established on the Mississippi River at Davenport and Keokuk, Iowa, Hannibal and St. Louis, Missouri, and on the Missouri River at St. Joseph and Kansas City, Missouri. Typical data [13] are shown in the following Table.

Table 3.9

River	Location	Drainage Area	Peak Discharge
Mississippi R.	Keokuk, IA	31×10^3 km^2	12 310 m^3/s
Missouri R.	Kansas City, MO	1256×10^3 km^2	15 000 m^3/s
Mississippi R.	St. Louis, MO	1805×10^3 km^2	29 730 m^3/s

The extent and magnitude of the 1993 Flood was almost incomprehensible (Fig. 3.12). The United States Geological Survey (USGS) [13] reported that records were set at 41 streamgauging stations spread across the nine states affected by the flood. The peak discharge at 45 USGS gauging stations was estimated to have exceeded the one-percent annual chance value. River levels exceeded flood stage at over 500 National Weather (NWS) river forecast points and record flooding occurred at 95 forecast points throughout the flood-affected region. The duration of the flood was also significant, with many locations on the major rivers remaining above flood stage for several weeks and a few for as long as five months. A stage hydrograph of the Mississippi River at St. Louis, for the 1993 calendar year is shown on Fig. 3.13. A discharge hydrograph of the Mississippi River at St. Louis, for the period 1 June to 29 September 1993, is shown on Fig. 3.14. Note in Fig. 3.14, the decrease in stage and flow at St. Louis that occurred in mid-July, which resulted from overtopping and subsequent breaching of several large levees on the mainstem Mississippi, upstream of the Missouri River confluence.

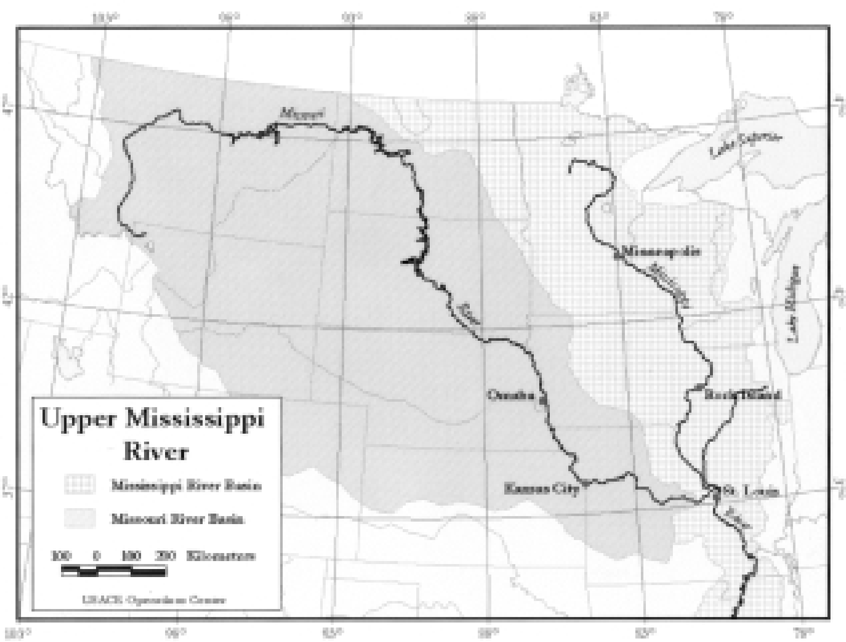
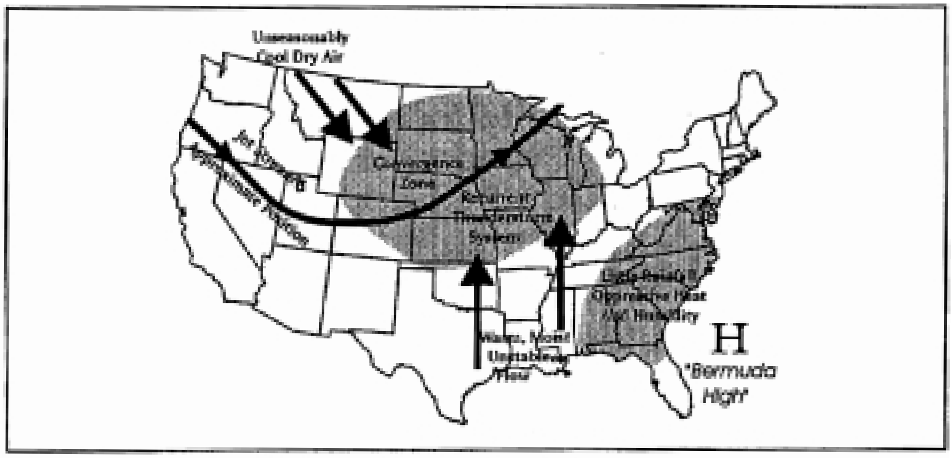
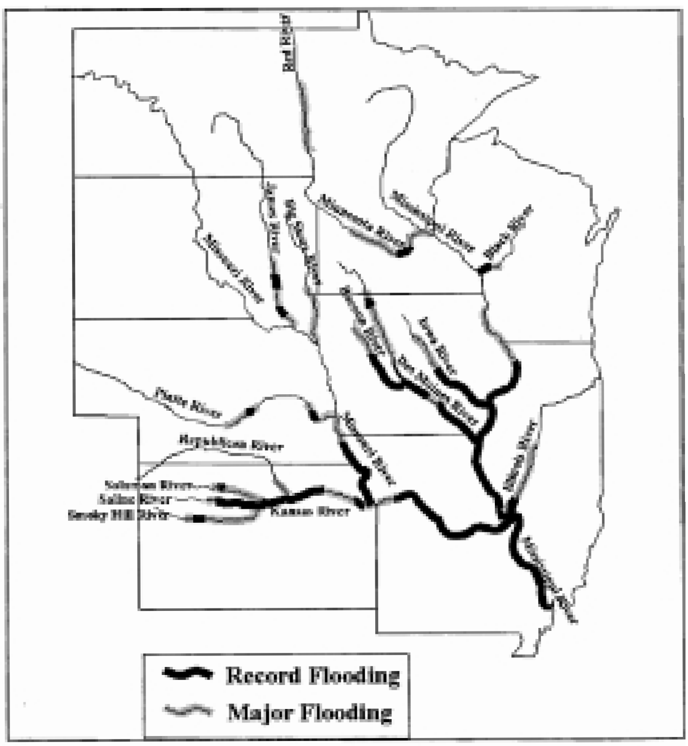


Fig. 3.10
The Upper Mississippi River basin



Source: U.S. Department of Commerce, NOAA, National Weather Service.

Fig. 3.11
Weather pattern. June-July 1993



Source: U.S. Department of Commerce, NOAA, National Weather Service.

Fig. 3.12
Areas flooded in 1993

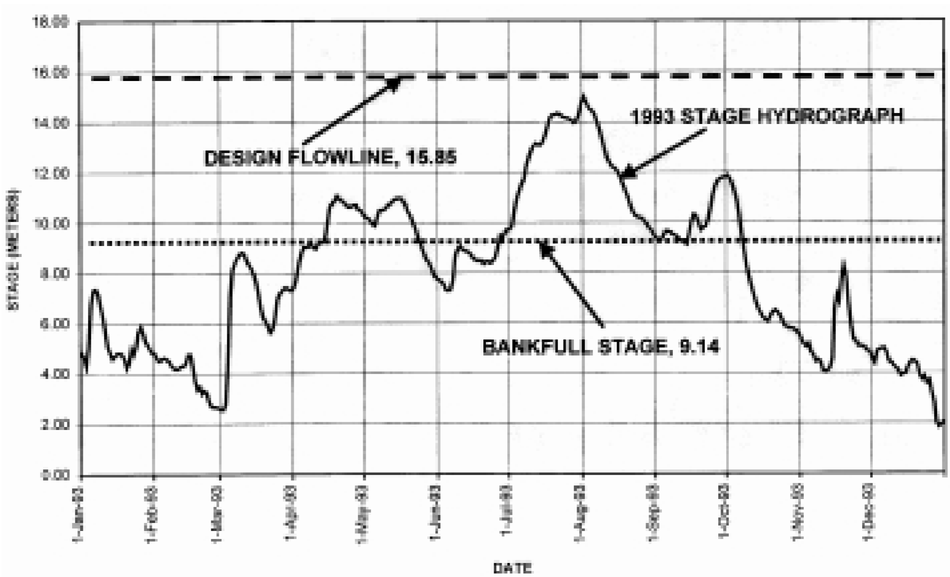


Fig. 3.13
1993 Stage hydrograph, St. Louis, MO

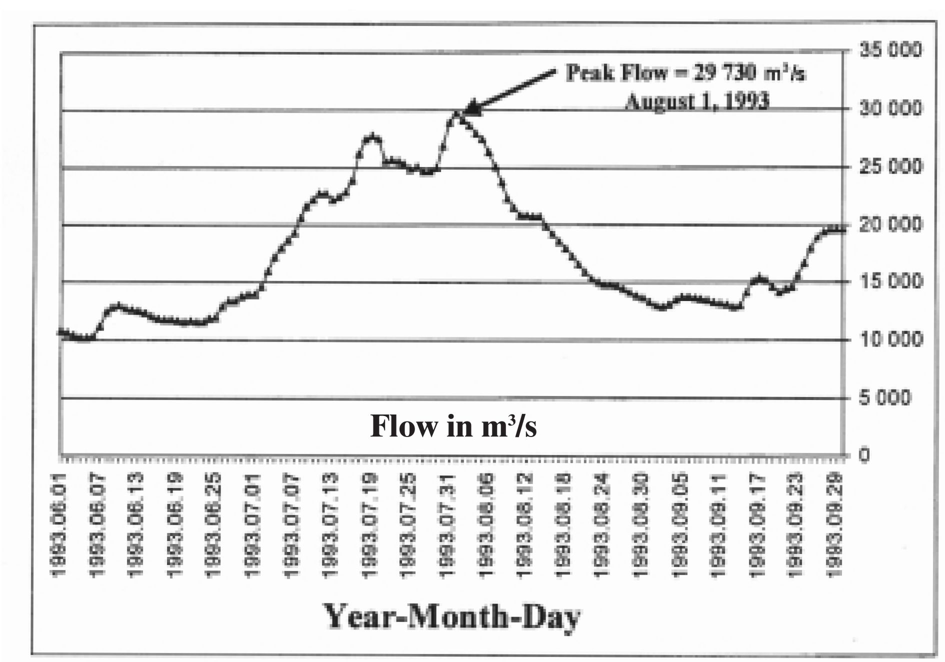


Fig. 3.14
Flood hydrograph at St. Louis, Missouri. June 1, 1993-September 29, 1993

3.4.3. Impacts on People and Property

After the storms abated and the floodwaters subsided, the National Weather Service estimated that 38 deaths and $ 15.6 billion in agricultural and property losses were caused by the heavy rainfall and the associated river flooding. In addition, portions of the extensive navigation system on the upper Mississippi and lower Missouri Rivers and the Illinois River were either closed or severely restricted for all or portions of the eight-month period from April through November. The navigation industry alone lost $ 300 million per month. In terms of non-quantifiable losses, the flood impacted thousands of historical and cultural resources, destroyed entire towns, forced the evacuation of more than 50 000 people, closed roads and railroads, shut down public facilities such as water and wastewater treatment plants and caused extreme environmental disruption. Without the heroic flood-fighting efforts of the citizens of the river valley and volunteers from all over, and without the flood damage reduction benefits produced by the flood control facilities built and operated by the U.S. Army Corps of Engineers, the U.S. Bureau of Reclamation, and local flood control (levee) districts, the death and damage toll would have been far greater (see section 5.2.6).

3.5. THE FLOODS OF 1995 IN THE NETHERLANDS [14]

3.5.1. Introduction

The 1995 flood period in the Rhine and Meuse in January and February caused considerable disturbance in the Netherlands and elsewhere in Europe. The Meuse overflowed its banks for a considerable length of time, causing substantial damage. The lengthy duration of the flood surge caused the water in the Meuse valley in Limburg to rise in some places even further than in 1993. Various villages and urban areas alongside the undiked section of the Meuse were evacuated, causing both material damage and distress.

The direct damage caused by the flooding was valued at around US $ 80 M, while the indirect damage (mainly due to the flood in the river Rhine) was estimated at between US $ 500 and 800 M.

Based on calculations regarding the flood embankment stability in some unstrengthened sections, the local Water Boards could no longer guarantee that the flood embankments would remain intact. As a precaution against flooding, therefore, local authorities decided to evacuate 200 000 people from the areas under threat. In the end, however, there were no breaches of primary water defences or inundation of polders.

Partly as a result of the experience with the flood period in 1993, the authorities were well prepared and the necessary precautionary measures, such as the construction of emergency dikes, were implemented without delay. Consequently, in many places such as the municipalities of Kampen, Arcen and Velden, inundation was prevented.

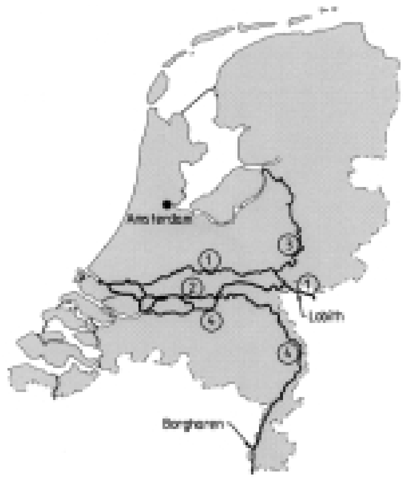


Fig. 3.15
Map of the Netherlands with main rivers

1) River Rhine
2) River Waal
3) River Ijssel
4) River Meuse

In Germany, January 1995 water levels in the Rhine broke many records. As a result of heavy rain, the Mosel, the Main, and the Neckar – all tributaries of the Rhine – conveyed large quantities of water. The long period of large discharges created an emergency situation in the Rhine between Koblenz and Cologne. On 31 January 1995, the water in Cologne reached a level of + 10.69 metres a.P. (equivalent to AOD + 45.66 metres) : with the water level of 1926 the highest ever recorded water level without ice jams (AOD = Amsterdam Ordnance Datum, i.e. the level of reference).

In the Netherlands, the water in the upper reaches of the Rhine, the Waal, and the IJssel remained just under record levels, the vast majority of which date in this area from the flood period of 1926. The highest known levels in the Meuse were recorded during the flood periods of 1926 and 1993.

In 1926, flooding occurred on a large scale, reducing the pressure on the flood embankments in the remaining areas. This was one reason for the 1995 decision to evacuate various parts of the areas around the rivers despite the fact that the predicted water levels in the Rhine were lower than those of 1926. The geotechnical stability of various unstrengthened sections of the flood embankments could not be guaranteed under the predicted circumstances. Fortunately, no breaches of the primary water defenses occurred during the 1995 event.

This was due to various reasons :

- In view of the lack of precise figures, the theoretical models of soil stability often include conservative assumptions with regard to soil parameters ;

- The water levels that occurred in the upper reaches of the rivers remained slightly below those forecast ;
- The extremely unfavourable wind conditions which were forecast (especially for Kampen) failed to occur ;
- Local Water Boards took large-scale measures to prevent or avert hazardous situation.

All around the world, people saw pictures of rivers overtopping summer dikes. Summer dikes are low, submersible dikes at the edge of flood plain and river channel. Overtopping of summer dikes is accepted as it brings the flood plains along the river into operation as storage and conveyance facilities (see also Fig. 3.16). Although the material damage and distress were very great, the situation did not technically amount to a disaster. Rivers overtopping summer dikes are a normal and controlled phenomenon. Apart from the un-diked parts of Limburg, no inhabited area of the Netherlands was inundated as a result of a dike breach or overtopping during the flood period of early 1995.

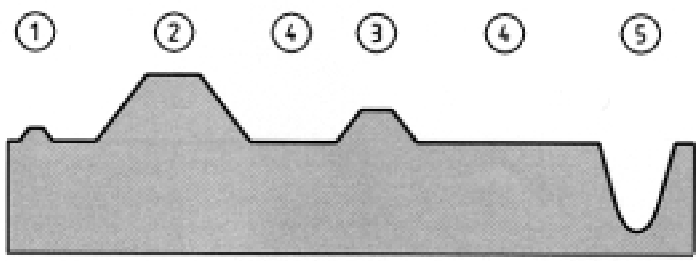


Fig. 3.16
System of main and summer dikes

1) Secondary flood embankment
2) Main flood embankment
3) Submersible flood embankment (so-called summer dike)
4) Flood plain
5) River

3.5.2. Weather Conditions

Precipitation in the upper reaches of the Rhine and Meuse during 18 to 30 January 1995 was, together with melt water, the main cause of the high discharges. The precipitation in the catchment area itself is brought in by prevailing westerly winds. The total average precipitation over that period was 165.9 mm. The heaviest precipitation was in the area around the sources of the Ourthe and the Lesse, on the slopes approaching the highest parts of the Ardennes. This is fact where precipitation is normally heaviest.

The events leading up to the flood period of 1995 were very important in dictating the ultimately prevailing water levels. A combination of melting snow and heavy rainfall severely limited the potential for retaining the precipitation.

Wind force and wind direction are important because the wind produces waves. Wave attack on dikes can cause damage to the inner slope (the river side). At extremely high water levels, there is also the threat of wave-over topping. Water flows over the top of the dike onto the land ward side. Depending on the quality of the outer slope (the landward side), this can lead to saturation and so threaten the stability of the dike.

During the 1995 flood period, southwesterly winds of up to force 5 to 6 on the Beaufort scale occurred along the Rhine, Waal and Meuse Rivers. Although this wind load is by no means negligible, the expected wind speeds were considerably worse. The weather forecasts predicted a northwesterly wind reaching as much as force 8 on the Beaufort scale. For the areas further downstream, like Dordrecht and in particular Kampen, this kind of wind speed (force 8) would have produced extremely threatening situations as a result of the set up of the mass of water in the North Sea and the IJsselmeer.

3.5.3. Water Levels and Discharges

The maximum discharge in the Rhine at Lobith during the flood period of January-February 1995 was around 12 060 m^3/s. This was only 5 % less than the record discharge of 1926. The flood of Christmas 1993 had a peak discharge of 11 100 m^3/s.

Comparison of the discharge hydrographs for Lobith in 1993 and 1995 shows that the 1995 flood period lasted longer than that of 1993. The flood of January-February 1995 lasted three days longer than the peak discharge of Christmas 1993.

The design discharge for embankment strengthening along the Rhine is 15 000 m^3/s. Comparison of the flood hydrographs for 1995 and 1993 with the design flood used in the guidelines for the design of river dikes (see Fig. 3.17) shows that neither of these floods periods was actually exceptionally long.

The maximum discharge of the Meuse at Borgharen during the flood period of January-February 1995 was 2861 m^3/s. During the flood period of Christmas 1993 it was 3120 m^3/s. Since the design discharge for the Meuse is 3650 m^3/s, the situation during the flood period of January-February 1995 was almost 800 m^3/s less than the design value.

Fig. 3.17 and 3.18 show two diagrams of the discharge hydrograph at Lobith (Rhine) and Borgharen (Meuse). For the last century, the Ministry of Transport, Public Works and Water Management has kept a reliable record of the water levels at Lobith on the Rhine and at Borgharen on the Meuse. These water levels were converted to discharge figures. On the basis of these discharge figures, estimates of the probability of an exceptionally high discharge and water level were then produced.

A comparison of the duration of the flood of 1993 with that of 1995 shows first, that the Meuse has a much less regular discharge hydrograph than the Rhine, and second, that the duration of the most recent flood period in January-February 1995 was very long (about 5 days) in comparison with the Christmas discharge of 1993 (about 1 to 2 days).

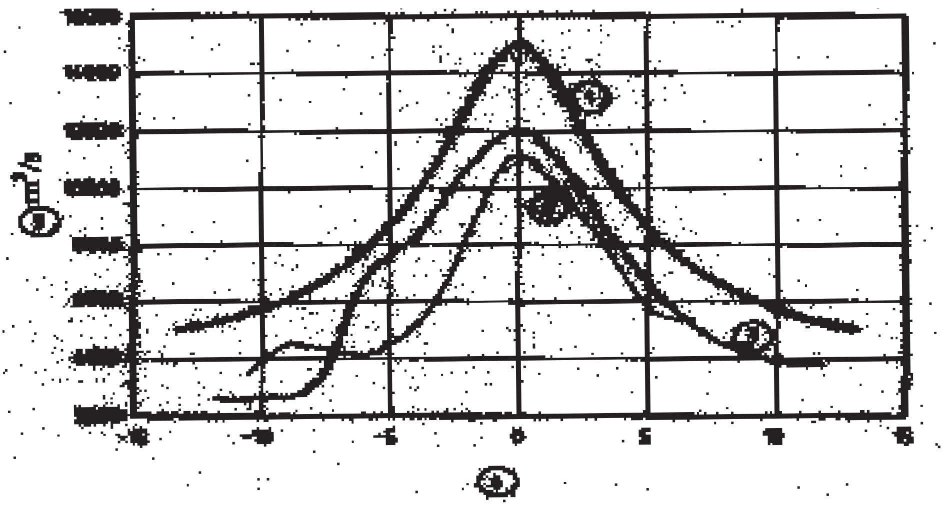


Fig. 3.17
Discharge-hydrographs at Lobith (River Rhine)

1) Design Flood hydrograph
2) Dec. 1993 Flood hydrograph
3) Jan-Feb 1995 Flood hydrograph
4) Duration (days)
5) Discharge(m^3/s)

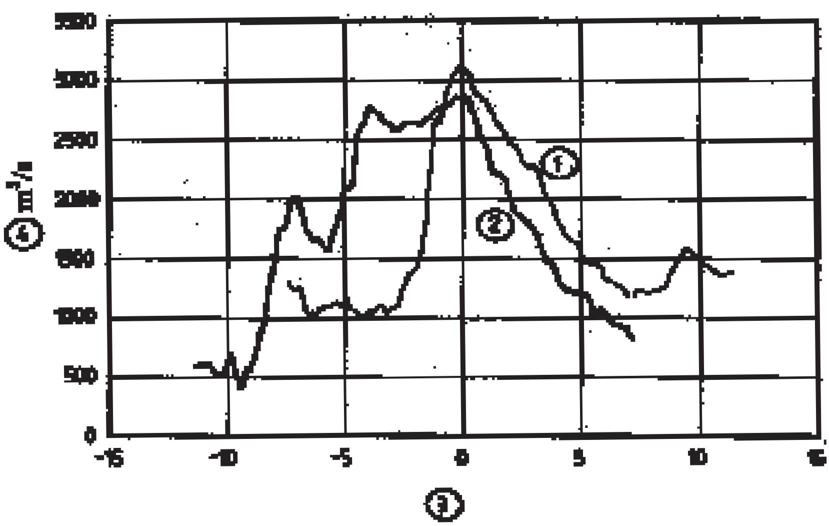


Fig. 3.18
Discharge hydrographs at Borgharen (River Meuse)

1) December 1993 hydrograph
2) Jan-Feb 1995 hydrograph
3) Duration (days)
4) Discharge (m^3/s)

Water levels in the Rhine

At Lobith, unlike in Cologne, no records were equalled. The maximum water level at Lobith was recorded on 2 February 1995 and reached AOD + 16.68 metres. The highest water level ever measured at Lobith was recorded in 1926 and reached AOD + 16.93 metres. However, since 1926 the river bed has been changed by human intervention. With discharges at 1926 levels, the maximum water level would now be AOD + 16.83 metres.

The design water level (NHW: the Normative High Water) for the Rhine flood embankment near Lobith is AOD + 17.65 metres. This water level has the prescribed exceedance rate of 1/1250 per year. The water level which occurred early in 1995 had, according to the current statistics for most places, an exceedance rate of around 1/80 per year.

Water levels in the Meuse

The maximum water level that occurred at Borgharen was AOD + 45.71 metres. This level was reached on 31 January 1995. Here too, the record water level is similar to that of 1926, when AOD + 46.05 metres was recorded. In the Meuse, as in the Rhine, the situation has changed since 1926, with the result that the 1926 discharge would now produce lower water levels. Taking these changes into account, the record water level is that of 1993 (AOD + 45.90 metres).

The design water level (the Normative High Water) for the Meuse flood embankments in Brabant and Gelderland is AOD + 46.30 metres at Borgharen.

3.5.4. River Dike Protection

In the Netherlands, the Rhine and its tributaries are lined by embankments called river dikes, which are interspersed here and there by higher ground, such as the Wageningse Berg. From the Dutch border, the Meuse flows through an area by higher ground. When the rivers are flooded, this part of the Meuse basin becomes much broader that of the Rhine because no flood embankments have been constructed in this area.

The economic and other importance of areas (flood plains) outside the flood embankments has increased over recent years. At the same time, public acceptance of flooding and the damage it causes has declined. These factors have triggered a process whereby in a number of cases, summer dikes are being strengthened and increased in height. This process is increasingly strengthening the summer dikes so that the probability of a dike overflowing or breaching in built-up areas relates more to the primary flood defenses than to the summer dikes. This means that the area between the summer and winter dikes is seldom or never flooded. As a result, public pressure is increasing to protect these areas as if they were situated behind a primary flood defense.

In some cases, measures taken in one area may cause flooding in another. Authorities have to agree between themselves what consequences are acceptable for which area.

3.6. THE JULY 1996 FLOOD ON IMJIN RIVER IN KOREA

3.6.1. Introduction

Flood and typhoon are two major causes of natural disasters that affect Korea. On the average, the Korean peninsula experiences a major flood almost every year, which causes loss of human lives and property damage. The social impacts of natural hazards are tremendous such that the average number of victims per year from any one natural disaster may reach up to 256 persons, with average annual damages of 504 M$ per year. The Han River flood that occurred in 1990, had the greatest effect on Korea. Maximum rainfall was recorded as 330.8 mm/day at Daekwan-Ryung, and the resulting flood caused 726 M$ of damages and took 179 victims.

3.6.2. 1996 Flood on Imjin River

Between July 26 and 28, 1996, two series of heavy storms that concentrated mainly on the Imjin River basin caused a historical flood in the river. The flood ended with 89 fatalities and more than 660M$ worth of property loss. The number of those who were temporarily evacuated from their inundated houses exceeded 35 000.

The Imjin River basin is located at the center of the Korean peninsula and the river flows from North Korea to South Korea across the Demilitarized Zone (DMZ) which divides the Korean peninsula into two since 1953. The basin of the Imjin River is 8118 km^2, which occupies one third of the Han River basin, and 225 km in length. The largest tributary of the river is the Hantan River with its basin area of 2435 km^2, about 30 % of the total river basin. Annual precipitation of the river is about 1270 mm. The Imjin River Basin is presented in Fig. 3.19.

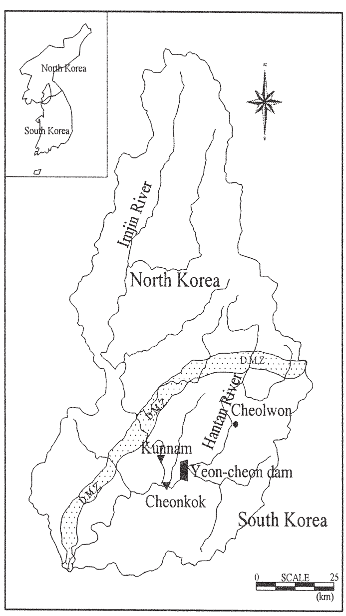


Fig. 3.19
Imjin River basin

The precipitation only lasted for 2 days between the early morning of July 26 and the morning of July 28, 1996. Table 3.10 shows the daily precipitation levels in major cities within the river basin. Total amounts vary from 370 to 690 mm. The precipitation level during the flood approached nearly 40 % of the annual average. According to the data, return period of the 2-day storm at Yeoncheon far exceeds 200-year and that at Cheolwon exceeds 100-year.

Table 3.10
Point precipitation during the flood

Station	July 26	July 27	July 28	Total	Annual average
Cheolwon	225	268	34	527	1257
Dongducheon	288	244	46	578	–
Yeoncheon	162	448	77	687	1250
Munsan	206	170	67	443	1253
Cheoksung	263	329	46	638	–

The two river stage data collected from Kunnam gauging station on the Imjin River reach and Cheonkok gauging station on the Hantan tributary reach were analyzed. Fig. 3.20 shows hydrographs from the two gauging stations during the flood. As shown in the Fig., the two graphs have similar shapes and peak times. This implies that two main floods from the main river and the tributary almost merged at the same time causing a greater flood on the downstream reach. The second peak flow at Kunnam exceeded the 100-year design flood stage of 31.8 m by 1.4 m, while the second peak flow at Cheonkok exceeded the 80-year design flood stage of 31.7 m by 3.2 m. These flows surpassing the design floods caused the overtopping of the levee and ruptures at many places along the river, and resulted in the failure of a small hydropower dam, the Yeoncheon dam, located at the Hantan River, as shown in Fig. 3.21.

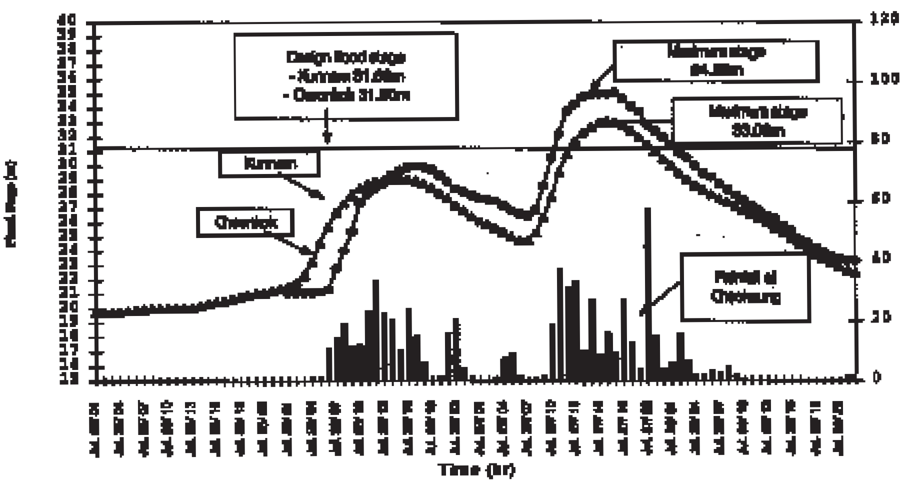


Fig. 3.20
Flood stage hydrograph during the flood

Fig. 3.21
The failure of Yeon-cheon dam

3.6.3. Flood Impacts

The floods caused by the 2-day storms were concentrated mainly in the Imjin River and partly in the eastern vicinity of the river basin. A total of 89 fatalities including many military personnel were recorded. Furthermore, a total of 35 631 inhabitants were evacuated. About 75 % of these losses occurred around the Imjin River basin alone. In addition to the loss of human lives, other major damages due to the flood included levee failures along numerous tributaries of the river and subsequent inundation of houses, roads, railways, and croplands, and shutdown of water purification plants and telecommunication lines. The flood also caused mass landslides in the mountainous areas located at the upstream of the Hantan River, which led to most military fatalities.

The amount of the precipitation for the 1996 flood was the greatest during the 40-year record. It exceeded the 200-year return period and was estimated up to 500-year at some places in the basin. The intensity of the flood losses was tremendous despite the fact that the areas swept by the flood were sparsely populated and less developed compared to other parts of Korea. The present disaster, therefore, may have been "inescapable", which means that the intensity of the 1996 flood in Korea far exceeded the economically and socially acceptable upper borderlines. Nevertheless, this flood gave us some valuable lessons provided below.

1) Extension of Flood Warning System
   The Imjin River has no modern flood warning system because no hydrologic data is available from North Korea. In order to establish the flood warning system, installation of the radar rainfall gauging system that can cover the North Korea basin is needed.

2) Improvement of Design Standard of Dam Spillway
   PMF concept for hydrologic design of dam should be considered even for a

small-scale dam because of the potential extent of losses incurred by the dam failure.

3) Reinforcement of Flood Countermeasure for Military Camps
Storm loss prevention tactics near military zones are to be considered and carefully thought out when military camps and facilities are constructed on the steep mountains and valleys.

3.7. FLOODS IN HIGH LATITUDE COUNTRIES [15]

Flow regimes in countries at high latitudes and in mountainous areas in the temperate zone where snow cover is built up during the winter, are characterized by strong seasonal variations. The flow is usually low in the winter, with a peak during the snowmelt period which may occur early in the spring in the lowlands, in the early summer in the high mountains, and in midsummer in glacierized basins. The rainfall increases through the summer and reaches a peak in the autumn. The snowmelt period is typically characterized by a snowmelt flood, which can be of considerable magnitude if the melting is combined with heavy precipitation. The late summer and autumn are characterized by heavier precipitation, which can lead to local flooding [16, 17].

The predominant flood-generating factors are snowmelt, rainfall and initial soil conditions [18, 19]. The snowmelt is dependent on the available amount of snow, the air temperatures, humidity, and wind velocity during the intensive melting. The albedo and water content of the snow pack are also of importance. Because the air temperature increases towards the summer, a prolonged cold spell in the spring will conserve the snow pack to a time of the year when the energy supply available for melting will be significantly higher and the melting is therefore more intensive. The precipitation will also normally increase through the spring. Therefore a late snowmelt carries a much higher risk of turning into a large flood. These factors, a combination of delayed snowmelt and heavy precipitation, caused a large flood in Norway in late May-June 1995 [20]. The snow storage was high in the higher parts of the basin, but not extreme. The rainfall lasted for several days, but the intensities were neither extreme. The largest floods occur typically when several unfavourable factors combine, and are not necessarily extreme.

Large basins, at least in Norway, are also usually characterized by a fairly large difference in altitudes. During an early snowmelt, only the lower part of the basin contributes much to the discharge, while the snow in the higher part of the basin may melt much later, when the lowland flood is over. When the snowmelt is delayed, the mountain flood and the lowland flood may combine to cause a severe flood. The basins are rather flat in northernmost Norway as well as in Finland. When the snowmelt starts there, the entire basin will contribute to the flood at the same time, causing occasional large floods, as in the river Neiden in the early summer 1996. This is also the case in other subarctic countries.

The risk of convective precipitation increases considerably through the summer and into the autumn. Convective storms lasting several days can cause large floods, especially in smaller basins. The maximum intensities are, however, significantly lower than in Southern Europe. The maximum observed rainfall in one day in

Norway is 230 mm with estimated PMP (probable maximum precipitation) values around 250-350 mm. Since the mid-1980s many events have been recorded with maximum daily rainfall around 400 mm in Switzerland, Italy, Southern France and Germany, causing locally severe flood damages and loss of lives.

The soil conditions are also of importance, especially for rainfall floods. A significant soil moisture deficit will reduce the flood magnitude. Provided that the soil moisture content is high, even moderately intensive rainfall will cause flooding. Flooding can also be enhanced by frozen ground in cold countries. When the ground freezes, water infiltration will stop. This is a particular problem in coastal areas during winter rainfall, especially when combined with a snowmelt after a long period of cold weather. If the soil freezes up deeply, and a large flood occurs after the thaw, landslides may occur, aggravating the flood damages. This was the case for the largest flood on record in Norway, (Storofsen), which occurred 20-23 July 1789, in eastern Norway. This flood was primarily caused by extremely heavy rainfall over an unusually large area, but the initial soil moisture content was probably high after the spring flood. Snowmelt in high mountain areas was probably also a contributing factor.

Another cause for flood inundation is ice in the rivers, raising the water level. Winter ice may also form barrages across rivers, which can cause considerable damage when the ice breaks up. This was even a problem in the Rhine during the Little Ice Age. Many of the largest floods there were associated with ice dams.

A particular type of flood is the jökull-hlaup – a flood caused by glacier - dammed lakes. When the water level rises above a certain level, the water pressure may lift the ice and the lake will empty itself into the glacier river, causing damaging floods. This type of flooding was a problem at a few locations in Norway, but construction of drainage tunnels as well as a decrease in the thickness of the glaciers have now virtually eliminated the problem. This problem may reappear if the thickness of the glaciers should increase again. In Iceland, this type of jökull-hlaup occurs quite frequently. A far more devastating type of jökull-hlaup is caused by subglacial volcanoes, as the one that occurred under Vatnajökull Glacier in the autumn 1996. The maximum discharge, estimated at 45 000-55 000 m^3/s of the resulting flood when the meltwater collected in Grimsvötn (a volume of more than 3 km^3), burst out at the glacier front.

3.7.1. Flood Regimes in Norway

The Scandinavian Peninsula is characterized by a mountain range running parallel to the coast of Norway with lowlands to the east. The western side of this mountain range is exposed to extra-tropical weather systems from the Atlantic. The precipitation is high, with a maximum in the late autumn and early winter. There is a maximum precipitation zone approximately 50 km from the coast with annual observed precipitation in the range of 2000-4000 mm. Discharge measurements indicate that the precipitation may reach almost 6000 mm a year near the most maritime glacier. East of the mountain range there are mountain valleys in the rain shadow with annual precipitation less than 300 mm a year. A lasting snow cover forms regularly in the mountains and the inland, with a more variable snow cover in areas close to the coast [21].

The largest observed floods in western Norway have occurred in connection with heavy frontal rainfall, typically in the late autumn. This is also the case for floods in the southernmost tip of Norway. The basins in eastern Norway are generally larger and cover both lowland and mountain areas. The annual precipitation is lower, but convective precipitation is more dominant in the late summer and early autumn than elsewhere in Norway. The largest annual flood can be both a snowmelt flood and a rainfall flood. Floods in the mountains usually occur in late spring/early summer, but a few large floods have been caused by heavy convective rainfall, even in the mountains. The snowmelt floods are characterized by large volumes. They will usually be contained in reservoirs, which are low after the winter drawdown. In basins with large reservoirs, the downstream flood magnitude has generally been reduced compared to the period without reservoirs. Rainfall floods are usually sharper, but the volume is less. These floods may still be more of a problem in regulated watercourses because they often occur when the previous snowmelt flood has filled up the reservoirs.

The variability over time in Nordic runoff series has been studied by Hisdal *et al.* [17]. A number of floods have occurred since the mid-1980s, with one in 1995 causing the largest damages. Between 1924 and 1940 a similar concentration of floods is apparent in the data series. The few long-term series going back into the 19th century indicate a humid period in the 1860s. The occurrence or lack of floods is linked to the large-scale atmospheric circulation. A moderate change in the predominant path of storms from the Atlantic can have strong implications for the runoff in Norway, because of the mountain ranges. The largest floods in Southeast Norway seem all to be caused by precipitation from a southeasterly direction.

3.8. FLOODS IN CHINA

Several types of disasters occur in China, such as floods, droughts and waterlogging, cyclonic storms, hail and low temperatures, earthquakes and slides, and debris flows.

Estimates from the Institute of Engineering Geologic Research and Ministry of Civil Administration indicate that about 40-47 million hectares of farmland have been damaged annually since the People's Republic of China was founded. Every year, victims number from several to ten thousand. Economic losses have increased throughout the decades, and, by 1990 prices, was 47.6 billion yuans per year in the 1950s, 56.4 billion yuans in the 1960s, 63.5 billion yuans in the 1970s, 76 billion yuans in the 1980s, and about 100 billion yuans in the 1990s ($ 1 = 7.5 yuans) [22].

By these estimates, the economic losses for different type of disasters in China are as follows:

Flood and waterlogging disasters mainly appear in eastern and southern China. From 206 BC to 1949, 1029 floods occurred, i.e. about once in two years. From 1950 to 1990, areas affected by disasters encompassed 8.425 million hectares per year. For instance, about 13 million hectares of land was damaged in 1963, and 17 million hectares of land was damaged in 1975 with an economic loss of about 10 billion yuans [23, 24].

Table 3.11
Annual economic loss from disasters in China

Disaster	Economic Loss (billion yuans per year)
Flood and waterlogging	15-20
Drought	15-20
Cyclonic storm	5-6
Hail and low temperature	2-3
Earthquakes	1-2
Slide & debris flow	2-3

Several serious and extreme floods occurred in China during the 20th century. These floods and relevant information are provided in Table 3.12.

Table 3.12
20th century floods in China

Year	Location	Flooded/Damaged/ Submerged Areas	Number of Victims	Affected Persons (Millions)	Economic Loss (Yuans in billions)
1915, July	Pearl River	Cities of Wuzhou and Guangzhou, and 432 000 ha farmland	Tens of thousands		10
1933	Yellow River	11 000 sq. km	18 000		0.23
1954	Changjiang	27.84 million hectares	33 000	19	10
1957	Huaihe	12 000 sq. km		11	10
1963, August	Huaihe	4.86 million hectares of farmland	5640	22	6
1975, August	upstream Huaihe (Henan Province)	1.3 million hectares of farmland	30 000		6
1985, August	Liaohe	60 cities and towns, and 21 million hectares of farmland	230		0.47
1991	different parts of 20 provinces	26.4 million hectares of submerged farmland (3.2 million hectares fully damaged)	2295	220	69
1995, June	Hunan Province	57 cities and towns, and 133 000 hectares were submerged	387		14
1996	20 provinces	over 24 million hectares submerged	3048	200	100
1998 [25]	12 provinces	21 million hectares submerged	> 4100	240	150

4. ANALYSIS OF EXTREME FLOODS

4.1. INTRODUCTION

Data from studies of extreme floods at a world level date back to 1984, when the International Association of Hydrological Sciences (IAHS), published the "World catalogue of maximum observed floods" [1]. These data have been continuously referenced for the analysis of the extreme floods, both at a world level and at a regional level [2-5]. The maximum floods indexed have an envelope curve adapted to the equation given by Francou-Rodier [6, 7] :

$$\frac{Q}{Q_0} = \left[\frac{A}{A_0}\right]^{1-K/10}$$

where :

Q = Maximum flood discharge, m^3/s

A = Catchment area, km^2

$Q_0 = 10^6$ m^3/s

$A_0 = 10^8$ km^2

K = Coefficient of Francou-Rodier

Therefore, for a determined extreme flood, the coefficient of Francou-Rodier is :

$$K = 10\left[1 - \frac{\log Q - 6}{\log A - 8}\right]$$

where :

Q = Flood peak discharge, m^3/s

A = Catchment area, km^2.

IAHS (1984) presents data of 41 maximum observed floods, that were obtained discriminating the results of an ample survey that met the following requirements : where $K > 5.75$ for $A > 100$ km^2, and $K > 5.1$ for $A < 100$ km^2. The 41 floods meeting these requirements had K values ranging from a minimum of 5.19 for the San Rafael station in California, to a maximum of 6.76 for the 1953 River Amazon flood in Obidos, Brazil. From these 41 extreme floods, 14 floods exist that surpass the value $K > 6$, and the envelope curve of these maximum floods adapt well to the Francou-Rodier equation, with a value of K of about 6, which reached the maximum value of 6.4 for the upper envelope, with the exception of the River Amazon ($K = 6.76$).

One of the problems which stood out from the IAHS (1984) studies, was the poor precision of the maximum flood data (peak flow), because of the difficulty in obtaining good measurements from river gauging stations during extreme floods conditions. An analysis of the flood data presented in IAHS (1984) estimated an accuracy of ± 15 %.

More than 20 years have passed since the publication of the IAHS (1984) studies. In 1994, the ICOLD Committee of Dams and Floods was created to study the interrelationships between dams and floods around the world, with the goal to revise previous extreme flood estimates and update global extreme flood data to the present, with an emphasis of those floods that occur in large dams. To achieve this goal, the Committee of Dams and Floods conducted a survey to determine the maximum floods observed in large dams during the last two decades. Data for this analysis were selected from various countries depending on the number and importance of large dams, and the incidence of floods. Methodologies to evaluate the peak flow of extreme floods from at large dams are more precise, thereby reducing measurement error and increasing data accuracy.

4.2. EXTREME FLOOD DATABASE

From the maximum flood data survey, 340 new datasets concerning extreme floods (mainly from large dams) were obtained, which correspond to 25 countries that are the most important in the field of the dams, with nearly 90 % of the world large dams. Additionally, some data from IAHS (1984) were revised, and new peak flow and K values were calculated for the 1979 flood in the River Machhu II in India, with a peak flow of 16 308 m^3/s, and principally, for the 1953 River Amazon (Obidos Station) flood in Brazil, which has permitted, in the light of the 1989 flood, the better estimation of the 1953 flood assessing it in some 320 000 m^3/s. From these data, 73 new datasets of extreme floods were obtained that had Francou-Rodier coefficient K values greater than 5. Using the same requirements used in the IAHS (1984) study ($K > 5.75$ for $A > 100\ km^2$; $K > 5$ for $A > 30\,000\ km^2$; and $K > 5.1$ for $A < 100\ km^2$), these datasets have provided 31 new datasets of maximum extreme floods in the world, which includes the revised 1979 Machhu and 1953 Amazon flood data (Table 4.1) for analysis.

Table 4.1
Number of extreme floods considered

K Francou-Rodier coefficient	IAHS (1984)	New data	Total
$K > 6$	14	7 (+ 2)*	21
$5.75 < K < 6$	18	7	25
$K > 5$ (for $A > 30\,000\ km^2$)	4	11	15
$5.75 > K > 5.0$ (for $A < 100\ km^2$)	5	4	9
TOTAL	41	29 (+ 2)*	70
* Corrected flood data values for Machhu II, 1979, and River Amazon, 1953			

Table 4.2 gives the values of the 21 maximum observed floods in the world which have a coefficient K > 6. Table 4.2 includes extreme values from 14 floods in IAHS (1984), and 7 new values, together with 2 corrections, from the work conducted by ICOLD. Among the new data, the 1975 extreme flood in the River Ruhe, China (Shimantan and Banquiao Stations), which gave rise to the catastrophic failures of the Banquiao and Shimantan dams [9, 10], stands out. Also are encountered the large floods of the river Narmada in India, a new great flood of the river Amazon in the year 1989, and the floods of the river Chungju in Korea and of the river Yate in New Caledonia.

Table 4.2
Maximum floods in the world

Country	Station	Source	Basin area	Peak flow	Year	K (FR)
Taiwan	Hualien	IAHS-84	1500	11 900	1973	6.01
India	River Narmada	ICOLD	88 000	61 229	1973	6.03
Korea	Chungju	ICOLD	6648	21 899	1990	6.03
Korea	Han Koan	IAHS-84	23 880	37 000	1925	6.05
Korea	Toedong Gang	IAHS-84	12 175	29 000	1967	6.07
China	Shimantan	ICOLD	230	6280	1975	6.09
USA	Pecos	IAHS-84	9100	26 800	1954	6.11
Japan	Nyodo Ino	IAHS-84	1560	13 510	1963	6.11
New Caledonia	River Yate	ICOLD	436	8680	1988	6.15
Mexico	Cithuatlan	IAHS-84	1370	13 500	1959	6.16
USA	W. Nueces	IAHS-84	1800	15 600	1959	6.19
Taiwan	Tam Shui	IAHS-84	2110	16 700	1963	6.20
India	River Narmada	IAHS-84	88 000	69 400	1970	6.21
India	River Machhu	IAHS-84/ ICOLD	1930	16 308	1979	6.21
Taiwan	Cho Shui	IAHS-84	259	7780	1979	6.22
Brazil	Obidos	ICOLD	4 620 000	313 000	1989	6.22
India	River Narmada	ICOLD	88 000	71 000	1994	6.24
Japan	Shingu Oga	IAHS-84	2350	19 025	1959	6.28
Brazil	Obidos	IAHS-1984/ ICOLD	4 620 000	320 000	1953	6.29
China	Banquiao	ICOLD	762	13 000	1975	6.31
New Caledonia	Ouaième	IAHS-84	330	10 400	1981	6.38

4.3. ENVELOPE CURVES. MAXIMUM FLOOD PEAKS

Fig. 4.1 shows all peak flow data available for extreme floods (from IAHS 1984 and ICOLD) by the catchment area, together with the world envelope curve of the Francou-Rodier equation, for the maximum value of 6.38, or approximately 6.4 (established in 1984). Fig. 4.2 shows the peak flow values of the 70 maximum observed floods selected according to the criteria in Table 4.1.

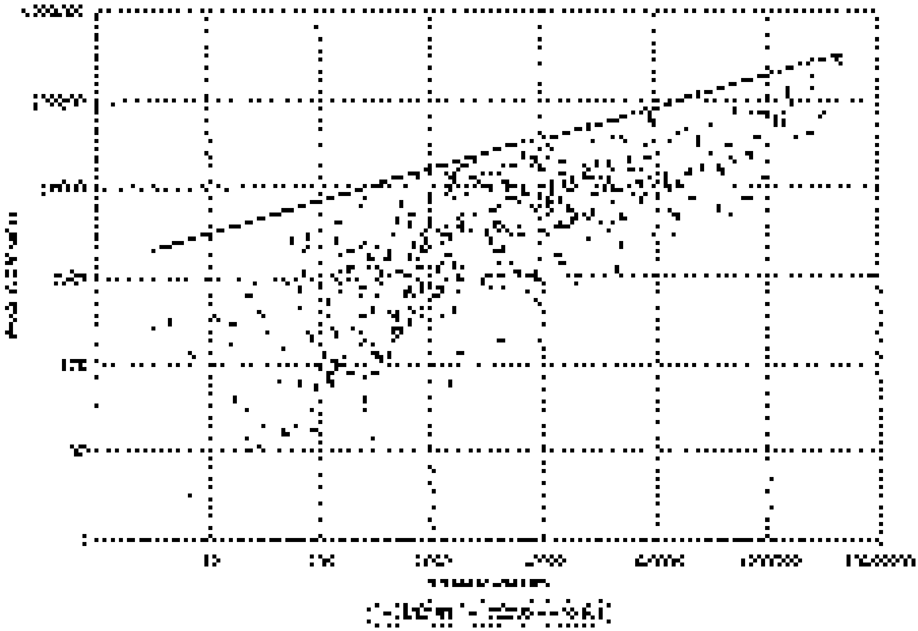


Fig. 4.1
Maximum floods in the world

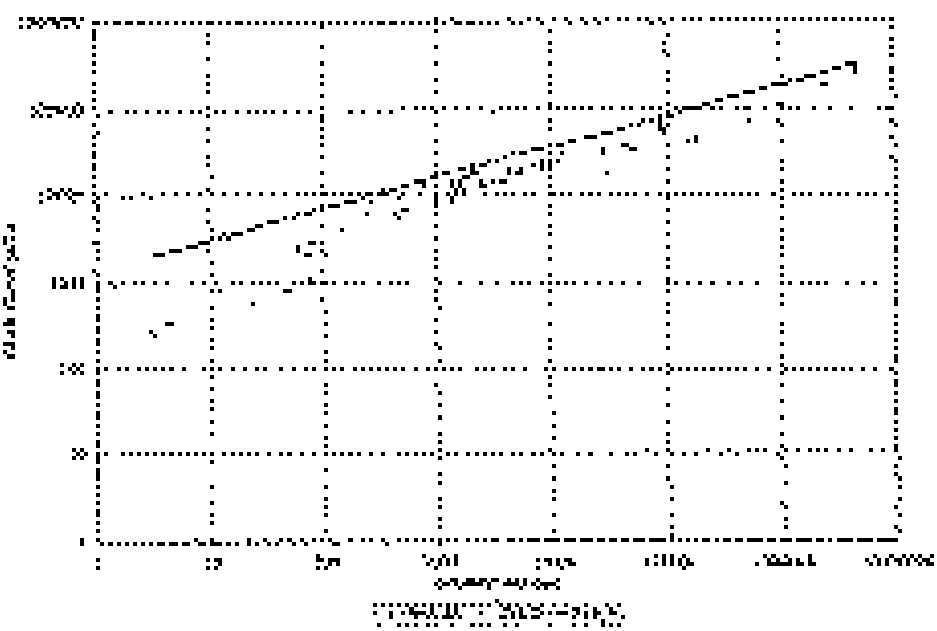


Fig. 4.2
Extreme floods in the world

Fig. 4.3 shows the values of the specific flows as a function of the basin catchment area, together with their envelope at a world level for K = 6.4. Fig. 4.3 also shows that even for small basins, no specific flow data surpasses 100 $m^3/s/km^2$. So then, the envelope curve at a world level corresponds to a value K of Francou-Rodier of 6.4. A variance analysis of the coefficient K during the last decades (Fig. 4.4), also shows that the maximum K value of 6.4 (established in 1984), has not been surpassed. In conclusion, no increase of extreme floods is observed to surpass the Francou-Rodier envelope curve of 6.4 at a world level.

From the extreme flood and envelope curve analyses, it is observed that there is a change in the general behaviour for the basins with an area less than 300 km^2 changes. These areas correspond with transition zones, where rainfall intensity affects the formation of the peak flow, in addition to the area of the basin. For this reason, data from 70 new extreme floods in small basins were analyzed. Data were selected from the following criteria: where $5 < K < 5.75$, and $K > 5.75$, for basins with an area of less than 300 km^2. Table 4.3 shows the total number of datasets meeting this criteria (n = 19).

Table 4.3

Number of extreme floods considered in basin areas of < 300 km^2.

K Francou-Rodier coefficient	IAHS (1984)	New data	Total
K > 5.75	2	1	3
5 < K < 5.75	7	9	16
TOTAL	9	10	19

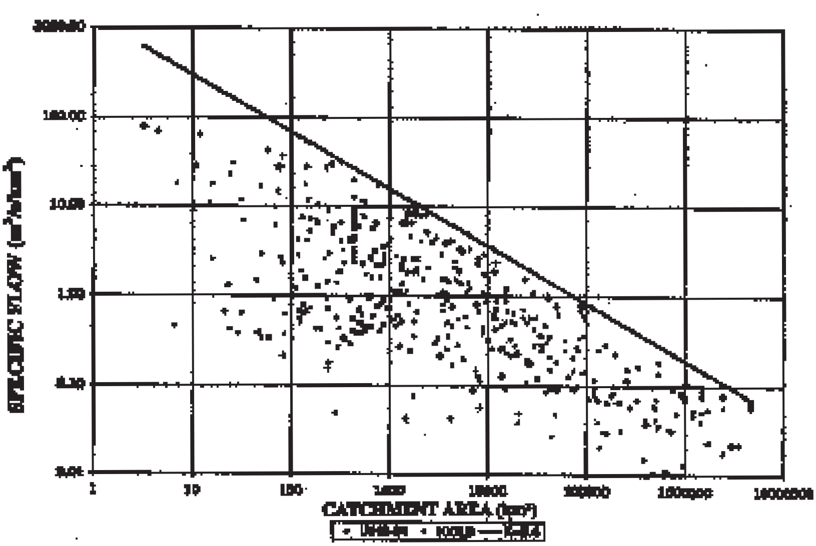


Fig. 4.3
Extreme floods. Specific flow

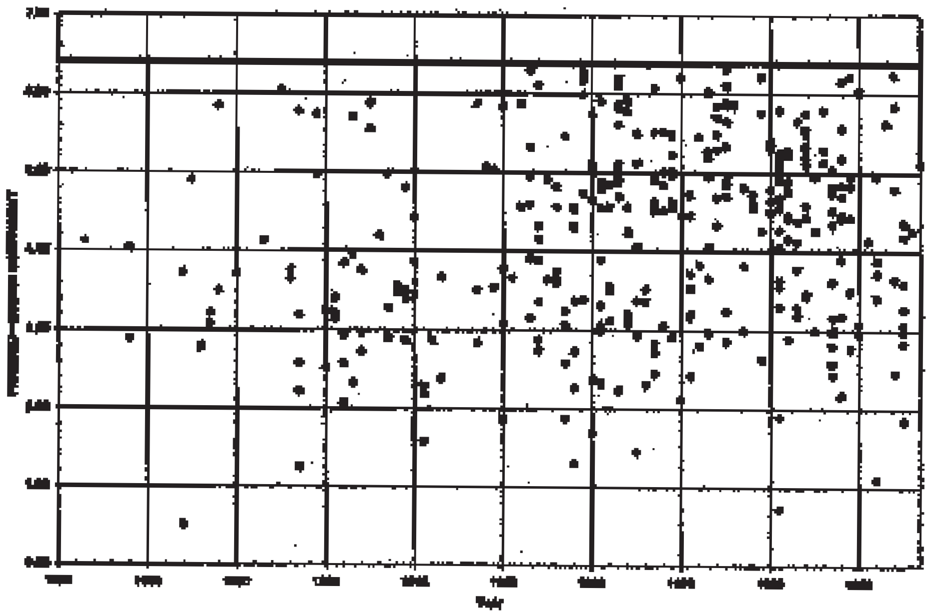


Fig. 4.4
Temporal evolution of extreme floods in the world

From this data analysis, it is observed that the general envelope curve does not adapt well for the extreme floods in basins less than 300 km^2. A new envelope curve is proposed for these smaller basins, to which the extreme floods are better adapted :

S < 300 km^2.

$$\frac{Q}{Q_1} = \left[\frac{A}{A_1}\right]^{1-R/10}$$

with :

Q = Peak flow in m^3/s

Q_1 = 10 000 m^3/s

A = Catchment area in km^2

A_1 = 300 km^2

R = Coefficient of the envelope curve

The envelope curve of the maximum observed floods has a value of R = 2.1. Fig. 4.5 shows the values of the extreme floods selected according to the criteria given in Table 4.3. The Fig. also shows that basins of less than 300 km^2 adapt well to the new envelope curve as do specific flows of extreme floods (Fig. 4.6).

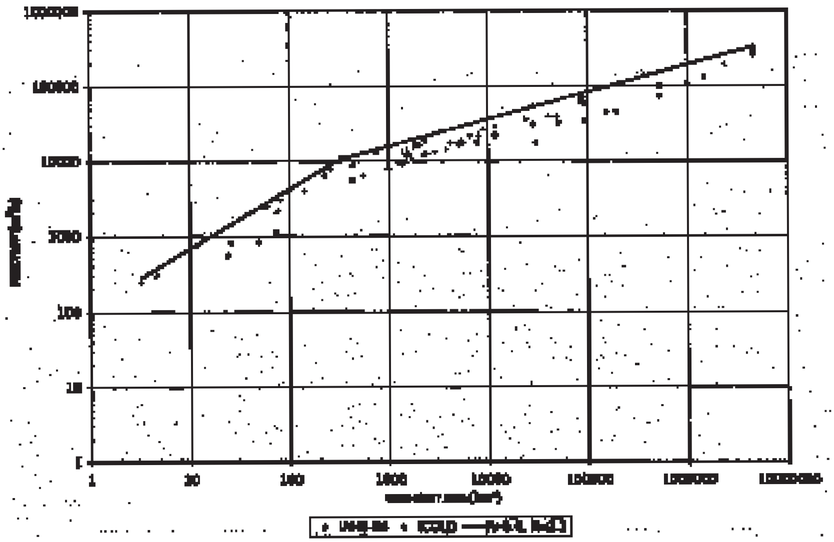


Fig. 4.5
Extreme floods in the world

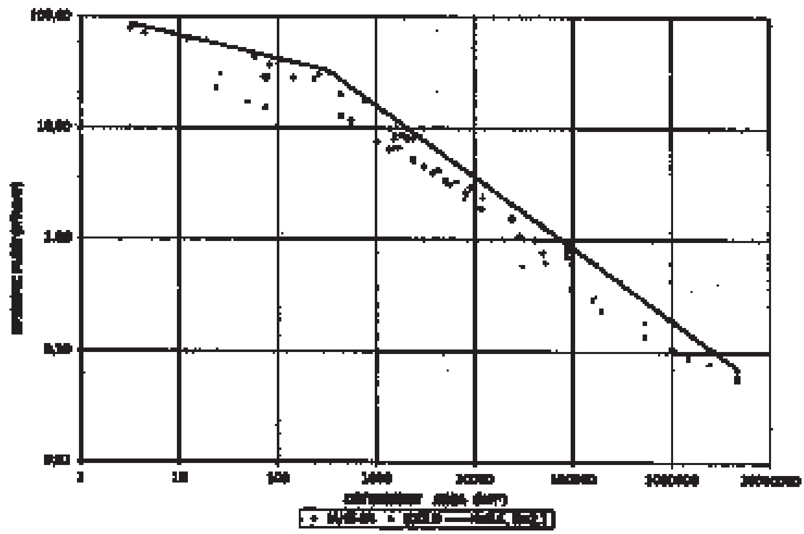


Fig. 4.6
Extreme floods. Specific flow

In conclusion, the data from extreme floods in the world confirm that good envelope curves for the peak flows exist, as a function of the basin area. For basins with A > 300 km^2 an envelope curve coefficient of K = 6.4 is verified. For basins with A < 300 km^2 a new world envelope curve is proposed, which is defined with a coefficient R = 2.1.

5. THE ROLE OF DAMS IN FLOOD CONTROL AND DAMAGE REDUCTION

5.1. INTRODUCTION

Analyzing the natural history of a flood, the measures to foresee and reduce the damages that it produces, can be classified in the following manner [1,2] :

A. STRUCTURAL. Measures that interfere in the flood formation and routing phenomena :

   A-1. Soil conservation and correction of basins

   A-2. Dams. Flood control and regulating reservoirs

   A-3. Hydraulic works in the rivers (Levees and dikes, diversions, channel improvements)

B. NON-STRUCTURAL. Measures that mitigate or reduce damages produced by floods :

   B-1. Risk maps

   B-2. Flood plain zoning and land-use patterns

   B-3. System of Insurance

   B-4. Legal regulations, Building regulations (Coding)

C. NON-STRUCTURAL. Actions that predict floods and forewarn to reduce the damages produced by floods :

   C-1. Flood forecasting and flood warning systems.

   C-2. Emergency action plans.

Among the measures used to reduce flood damages, reservoirs constitute a very efficient structural solution. Reservoirs are the only measure that can store water in a very significant manner, and hence, modify the hydrograph to significantly reduce peak flow. Dams in the world have efficiently reduced damages produced by the floods in a great number of cases [3, 4, 5].

The effects of a dam should be studied in the whole of the basin. Additionally, to obtain a greater effect, it is also necessary to study the possible combination and implementation of structural and non-structural solutions, since it is necessary in many cases to develop zoning and land-use patterns downstream of the dam. So then, in each case it is necessary to analyze the various possible measures outlined above and plan with regards to the basin.

Nevertheless, it is necessary to point out that, in general, the effects of the regulating floods with dams and reservoirs are more pronounced with floods having low and medium return periods. For these types of floods, the reduction in the peak flow could be considerable, and, in turn, significantly mitigate damages downstream. In most cases, provided that storage space is available, flood volumes of such "small" floods can be stored in the reservoir.

Regulating extreme floods with a dam and reservoir almost always have positive effects, but the effects on extreme floods are often less pronounced than those floods with low and medium return periods. The exceptions are dams constructed with the principal or single objective to control floods (Flood Control Dams), in which logically, reservoir flood routing of extreme floods are considered in the design.

Usually, reservoirs that have the main or single purpose of flood attenuation are referred to as flood control reservoirs, a name which suggests that flood control reservoirs are capable of controlling all floods and therefore, will prevent any damage to the inhabitants and townships downstream. This capability is obviously not possible, and less so in view of the existing uncertainty regarding the subject of floods, for which an absolute zero risk level cannot be attained with actual physical and technical knowledge. For this reason, it would be better to refer to flood control dams and reservoirs as Flood Mitigation Dams and Reservoirs, thus indicating the capability of these structural measures to mitigate or reduce flood damages [1], rather than completely preventing downstream flood damages.

It is not always possible to completely store the flood volume of a larger, more extreme flood. This undesirable situation can only be improved upon by combining flood storage with three other actions :

- controlled and timely release of waters from the reservoir in terms of timing and volume ;
- application of rule curves aiming at an efficient joint use of the reservoir live space for various purposes (e.g., hydropower, irrigation, flood control) ;
- increased discharge capacity of flood routes (e.g. rivers) downstream of the reservoir.

Usually, floods volumes of long duration cannot be completely stored in the reservoir but can be mitigated to a large extent by the combination of “storage in reservoir-increased downstream discharge capacity”. However, flash floods (i.e. floods having a high peak and short duration) should be mitigated mainly by storage and less by discharge capacity. This combined mitigation of floods can be studied by using the concept of hydraulic effectiveness of flood control measures.

Hydraulic effectiveness compares a flood situation *with* a flood control measure with a flood situation *without* the measure and, for a range of historical floods, calculates the extent to which the flooding would have been prevented by the flood control measure concerned. For this calculation, suitable flood parameters must be determined to characterize the flooding in the area concerned. Suitable flood parameters include volume of bank overspill, water level, and depth of flooding.

Hydraulic effectiveness of a flood control measure could be determined by the following formula :

$$\underset{i=1}{\overset{i=n}{\varepsilon h}} = 100\ \left(\sum Xi - \sum Yi\right) / \left(\sum Xi - n \bullet EWL\right)\ [\%]$$

where :

Xi = maximum annual water level without protection

Yi = maximum annual water level with protection

EWL = emergency water level (m)

n = historical floods (only those exceeding emergency water level)

εh = Hydraulic Effectiveness

The upper graph in Fig. 5.1 illustrates an example of the calculation of hydraulic effectiveness using the selected flood parameter "water level" over and above the 5.50 m emergency water level mark at Corrientes, Argentina (River Parana) [19].

The lower graph in Fig. 5.1 illustrates an approximation hydraulic effectiveness which can be determined by the formula :

$$\varepsilon' h = 100 \times (\text{area ABED / area ABCA}) \, [\%].$$

where :

$\varepsilon' h$ = Hydraulic Effectiveness by approximation

(3) = Exceedance curve for situation without project

(4) = Exceedance curve for situation with project

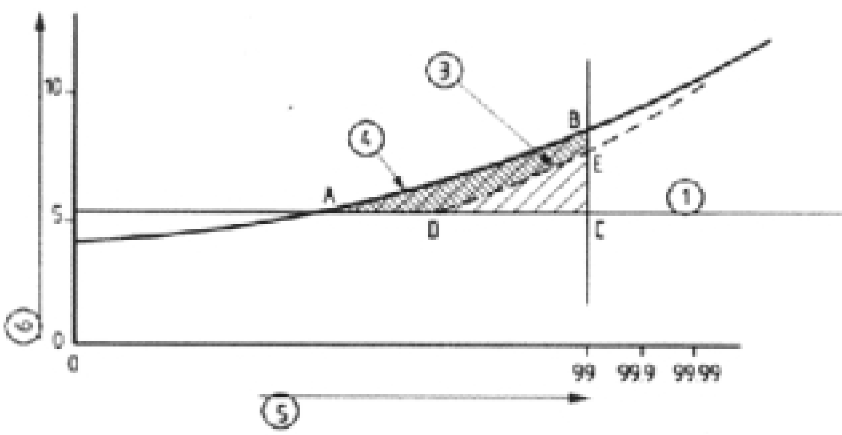


Fig. 5.1
Definition of hydraulic effectiveness of a flood control measure
using water level as a selected flood parameter

Table 5.1 shows that hydraulic effectiveness rapidly increases if flood storage in a large or a small reservoir is combined with an increased discharge capacity of the river system downstream of the reservoir.

The same effect occurs with the exceedance curves for bank overspill after the introduction of flood control measures (Fig. 5.2). The overspill volume (i.e. the volume of flooding) decreases for a flood having a certain return period if storage in a reservoir is combined with a higher discharge capacity downstream [20].

Table 5.1
Hydraulic effectiveness of different protection measure combinations [20]

RESERVOIR SIZE	LIVE STORAGE	DISCHARGE CAPACITY WHOLE SYSTEM D/S OF RESERVOIRS (m^3/s)					
	(hm^3)	AT PRESENT	2500	3000	3500	4000	5000
LARGE	800	80	89	95			
	1600	73-90	85 - 97	96			
	2100	87-94	94 - 97				
SMALL	260	36	54	70	80	87	
	160	31					

For instance, a flood having a return period of 50 years will cause bank overspill in the order of :

- 1550 hm^3 without any flood control whatsoever
- 1000 hm^3 for a discharge capacity of 3000 m^3/s and no storage
- 440 hm^3 for a combined live storage of 1700 hm^3 and discharge capacity of 2200 m^3/s

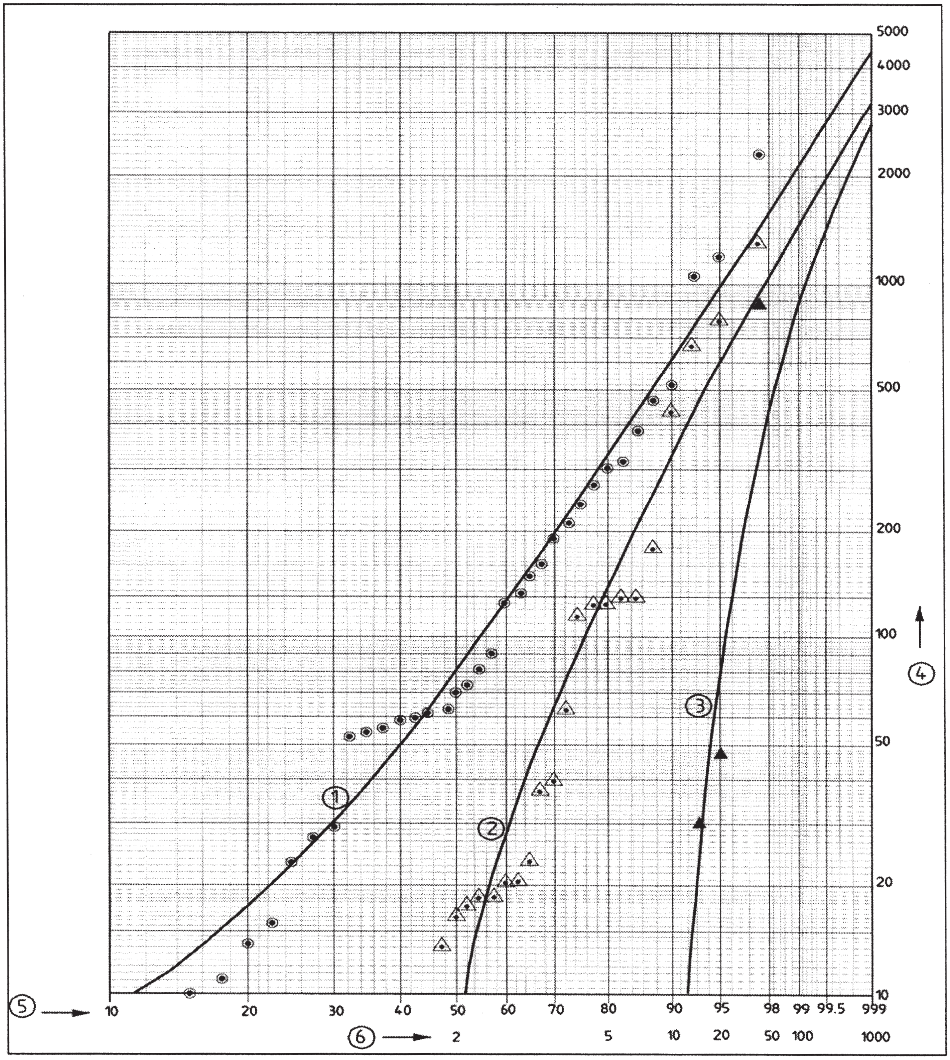
All dams, provided they are well designed and are operated correctly, present some flood mitigation benefits. The largest benefits, however, are obtained from Flood Mitigation Dams, which are principally built for flood routing effects and downstream damage reduction.

Quantifying these effects depends on the interrelation between the inflow design flood, the volume available in the reservoir for flood mitigation and the outflow design flood, which in turn, depend on the conditioning factors downstream.

Three hydrological criteria should be followed when designing Flood Control Dams :

1. *Dam safety, or hydrological safety of the dam.* The inflow design flood should be equal to the safety design flood, as a condition of safety, and for high hazard dams the PMF (Probable Maximum Flood) or high return periods of 5000 to 10 000 years are applied.
2. *Reduction of flood damages.* Usually, Flood Control Dams should provide protection against floods having return periods of 50 to 200 years, or, in the cases of important townships downstream, up to return periods of 500 to 1000 years, if it is possible.
3. *The overall occurrence of floods in the basin.* Take into account the effects of the flood peak reduction and lag times caused by the flood routing, and its downstream impacts, together with the occurrence of floods in tributaries or in other reservoirs. In any case, and specifically for the operation of these reservoirs, the viability and effectiveness of introducing flood forecasting systems should be analyzed.

In flood mitigation dam studies arises on numerous occasions the alternative of, either constructing a larger dam on the main river close to the area to be protected, or various small dams located on the headwaters or middle reaches of the rivers in the basin and on the tributaries of the river. In general, the smaller dams scattered



(1) without flood control
(2) at 3000 m³/s discharge capacity
(3) at 2200 m³/s discharge capacity and storage of 177 hm³
(4) bank overspill volumes in hm³
(5) probability (%)
(6) return period (years)

Fig. 5.2
Exceedance curves for (remaining) bank overspill after introduction of flood control measures

over the basin, although numerous, give less protection than one single large dam situated immediately upstream of the zone to be protected. For instance, the Miami Conservatory District in the valley of Miami showed that five large retention reservoirs gave much greater protection with less cost than the construction of numerous small dams on the tributaries. In another example, the U.S. Army Corps of Engineers showed that the construction of 13 small dams in the Merrimak River basin was only 52 % effective in comparison with two large dams located on the main river [6].

On the Onyar River in Girona, Spain, it has also been seen that the effectiveness in flood mitigation decreases significantly as soon as the dams were moved away from the zone to be protected, or were situated on the tributaries [1]. It can be concluded, therefore, that in general, greater protection is obtained with reservoirs situated upstream of the area where the flood damages have to be mitigated. However, in numerous occasions the economic, social and environmental aspects present a problem for the construction of dams immediately upstream of the townships to be protected.

On some occasions, principally in the case of large dams on major rivers, reservoir routing is designed to be operative only in a seasonal manner (i.e. during the flood season) and in combination with other multi-purpose uses (generally, irrigation, water supply or hydropower). In these cases, the operating rules for freeboard and the processes of reservoir emptying and filling should be precise before and after the flood season. It is also advisable to adapt the reservoir operating rules to the flood history, the changes and evolution in the basin, and in the downstream developments. In general, the downstream protection works are designed for controlling floods not having a return period of more than 50 years or, at the maximum, 100 years. In this case it is convenient to establish flood forecasting systems and other non-structural operations in order to consolidate the effective protection in the downstream townships.

It must also be taken into account that in many countries of the world, large populations and important townships have been established over the years along the rivers which form the backbone of the country, where the application of some of the non-structural measures already mentioned is not feasible (*i.e.,* resettlement, land-use patterns). The only possible action in these circumstances is then to reduce the frequency of the constant and repeated floods, which is a role that can only be carried out by the Flood Mitigation Dams. Although the Flood Mitigation Dams do not provide total protection (25 to 100 years), they reduce the grave impacts of the almost “annual” floods significantly.

On several occasions dams have been blamed for increasing flood damages and risk, with arguments such as the following [7] :

“Although flood control is one of the justifications for large dam projects, the risk of flooding and actual flood damage cost often increases as a result of dam construction. Over-zealous dam proponents create the impression that dams alone will eliminate the risk of all flood damages downstream, thereby encouraging development of the flood plain. But only the frequency and severity of flooding can be reduced.”

Previously, it was stated that no solution exists which will totally eliminate the risk of floods, but the flood mitigation dams constitute a very effective measure in the reduction of damages, without eliminating all the risks. As with any other solution, the perception of total security should be avoided, even though safety will be much greater than without the flood mitigation dam. The future development of the flood plains, the introduction of non-structural measures such as zoning, and land-use control and flood forecasting and flood warning systems must be actively planned and implemented in combination with the planning and construction of the flood mitigation dam.

It should also be mentioned that efficient flood forecasting systems have been developed during the last decades, which permit real-time knowledge of the meteorological and hydrological condition in the basin, the state of the reservoirs in terms of inflow and outflow, and the condition and hydraulics in the rivers downstream of the reservoirs. Thanks to these data, the effects of the flood mitigation dams are increased with regard their principal objective, *i.e.* the reduction of the damages produced by the floods.

So, by using flood forecasting systems, which become more integrated and reliable every day, the following flood management operations can be carried out :

1. Anticipation of reservoir discharge with the objective of obtaining an optimum reservoir routing for the flood which will arrive at the reservoir. To conduct this operation it is necessary to have available adequate hydrological and hydraulic models of flood run-off and flood routing coupled to the forecasting systems.
2. Coordination of the operations of all the reservoirs of the basin during a flood situation, with the objective of minimizing downstream damages.
3. Establishment of alarm systems, in order to be able to develop the necessary operations in the case of danger to people and properties.

It is not possible to globally quantify the beneficial effects of dams for flood control. However, it is possible to discuss outstanding cases that do have quantitative values, and which show the value of dams and their reservoirs in the reduction and, in some cases, the total elimination, of the damages produced by the floods.

So, for this chapter, significant cases were selected to show the important role of dams in flood control and damage reduction. These cases are assembled into three groups:

1. Real cases of dams in operation.
2. Dams under construction, in which flood mitigation plays an important part.
3. Large future projects in flood mitigation dams.

5.2. REAL CASES OF DAMS IN OPERATION

The performance of dams and reservoirs for flood mitigation is presented in the following cases:

- Tone River, Japan
- Chikugo River, Japan

- 1982 Ebro River Basin Floods, Spain
- January 1990 flood, Tunisia
- September 1990 flood, Han River, Korea
- 1993 Great Midwest flood, USA.
- 1995 spring floods, Glomma River, Norway.
- Flood control by operation of hydroelectric reservoirs, Parana River, Brazil.
- Miami Conservancy District. USA.

In these cases, in general, the flood control plan refers to a group of dams in a river basin, which combine the effects of the reservoirs, the canalization of the rivers, and the operation in real time of the flood forecasting systems.

The return periods of the protection flood vary between 35 years and 200 years and are greater in the cases of rivers where the flood plains are significantly occupied, as is the case of the Tone River in Japan. In general, reduction of flood effects are significant, varying from 25 to 82 % in the reduction of the peak flow, and from 10 to 73 % in the reduction of the flood volume, with the higher values in the cases in which the flood control capacity of the reservoir is greater. In some cases, the reservoirs have stored the total volume of the flood, as is the case of the floods of January 1990 in Tunisia. It is also necessary to point out that hydropower reservoirs, which were designed with a single purpose in many cases, can contribute significantly to the flood reduction damage by modifying the flood operation rules seasonally, and with the help of the flood forecasting systems. This scenario is demonstrated at the Glomma River in Norway, and the joint operations of the hydropower reservoirs in the Parana River in Brazil.

5.2.1. Tone River, Japan

5.2.1.1. History

The Tone River ran southward from Kurihashi before the 16th Century and poured directly into Tokyo Bay. However, a diversion channel was constructed in 1621, the beginning of the Edo Era, so as to reduce the flood damage in Tokyo, where the Tokugawa Shogunate was located, as well as to ensure navigation. Since that time, most floodwater in the Tone River flows eastward to Choshi City and pours into the Pacific Ocean as shown in Fig. 5.3.

In spite of the construction of the diversion channel, serious flooding occurred continuously and hit Tokyo several times. Serious flood damage in the Tone River Basin after the commencement of modern Japan in 1868 were experienced in 1910, 1935, 1937, 1941, and 1947. Out of these floods, the 1947 flood caused by Typhoon Catherine produced the most damage. This flood caused a embankment breaches at several points in the upper and middle reaches of the Tone River. Among the breaches, the flood flow coming from Kurihashi Point reached the Tokyo Metropolitan Area and brought about tremendous flood damage.

The Flood Control Master Plan for the Tone River was first formulated in 1900. The Master Plan was significantly revised four times, in 1911, 1939, 1949, and 1980. The design discharge at the Yattajima Point was first established in 1900 as

3750 m³/s, then increased to 5570 m³/s in 1911, 10 000 m³/s in 1939, and 14 000 m³/s in 1949 after the large floods mentioned above.

The 1949 Master Plan revision proposed construction of multipurpose dams in the Tone River System aiming at drastic reduction of flood discharge as well as at water supply and power generation. Located in the Upper Tone River, these dams were planned to reduce the project flood discharge (under condition of without dams) of 17 000 m³/s at the Yattajima point by 3000 m³/s. The first of these proposed dams, the Fujiwara Dam, was completed in 1958. Table 5.2 provides dates of completion for the Fujiwara and other multipurpose dams in addition to catchment area, dam height, reservoir capacity, and flood control capacity data.

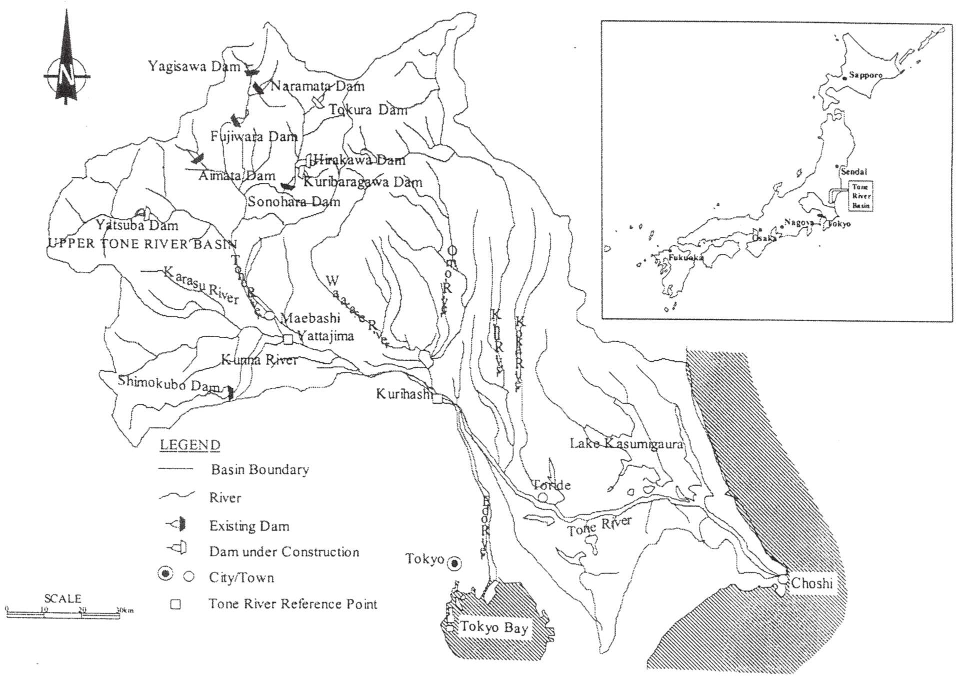


Fig. 5.3
Location of Tone River basin and dam site

5.2.1.2. Present Master Plan for Tone River System

The present Flood Control Master Plan for the Tone River was formulated in 1980, and took into consideration the economic and social development of the basin. The project flood discharge at Yattajima point was increased to 22 000 m³/s for a 200-year return period. Ten flood control dams located in the upper reaches (Fig. 5.3) were planned to regulate 6000 m³/s flood discharge and the river channel will confine the remaining 16 000 m³/s, which corresponds to 50-year return period (Fig. 5.4).

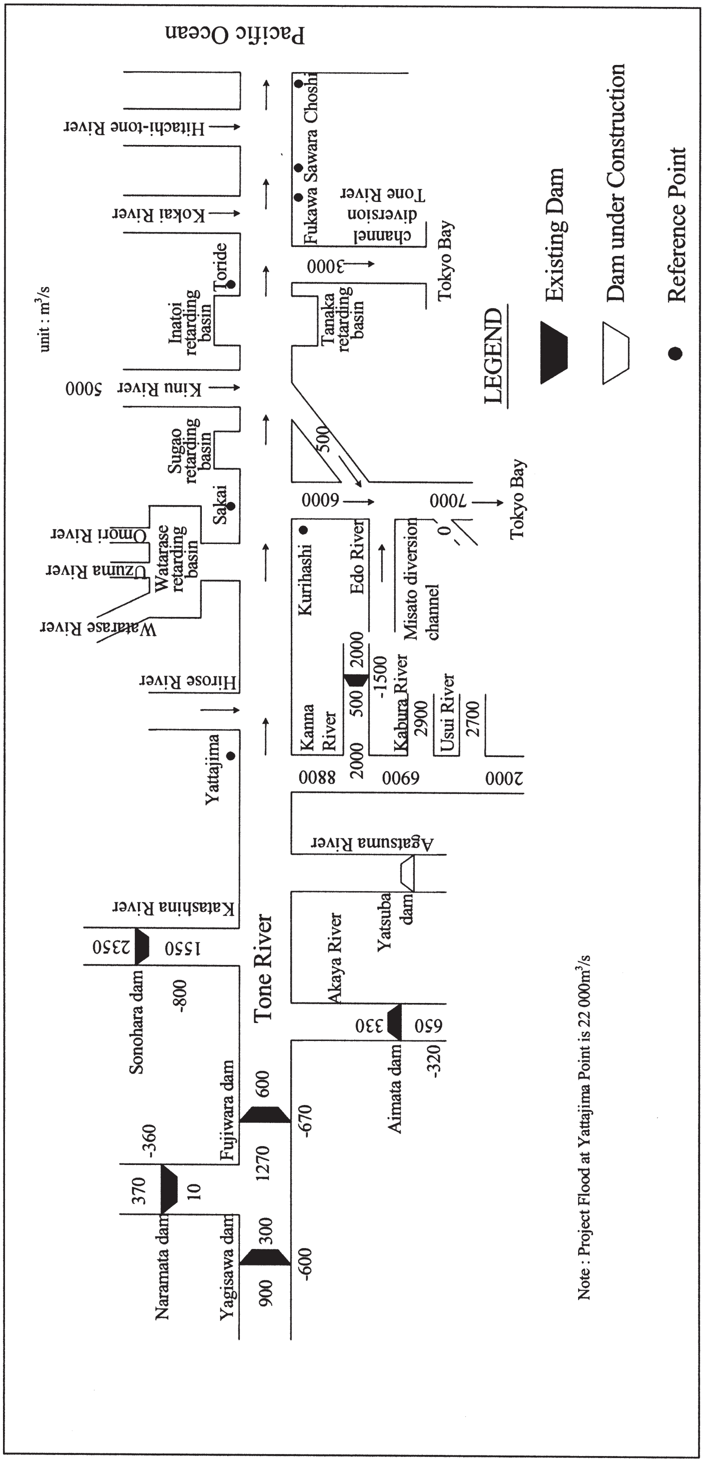


Fig. 5.4
Design discharge for Master Plan of Tone River System

Out of ten dams planned for construction in the upper reaches from the Yattajima Point under the Master Plan, six dams (Table 5.2) with the total flood control capacity of 115 million m^3 have been completed and the remaining four dams are under construction (Table 5.3).

Table 5.2
Existing dams in Upper Tone River

Name of Dam	Completion Year	Catchment Area (km^2)	Dam Height (m)	Reservoir Capacity (million m^3)	Flood Control Capacity (million m^3)
Fujiwara	1958	401.0	95.0	52.49	21.20
Aimata	1959	110.8	67.0	25.00	9.40
Sonohara	1965	493.9	76.5	20.31	14.14
Yagisawa	1967	167.4	131.0	204.30	22.10
Shimokubo	1969	322.9	129.0	130.00	35.00
Naramata	1991	360.1	158.0	90.00	13.00

Table 5.3
Dams under construction in Upper Tone River

Name of Dam	Completion Year	Catchment Area (km^2)	Dam Height (m)	Reservoir Capacity (million m^3)	Flood Control Capacity (million m^3)
Yamba	–	707.9	131.0	107.50	65.00
Tokura	–	72.5	158.0	92.00	23.00
Kawafuru	–	34.4	160.0	76.00	10.00
Hirakawa	–	62.1	146.0	50.00	16.00

5.2.1.3. Performance of Flood Control

Serious floods in the Tone River Basin have not been experienced in recent years partly due to completion of the six dams mentioned above and the improvements to river channels. Table 5.4 shows the reduction in peak discharges measured at Yattajima during floods throughout the years. During floods caused by two typhoons in August and September 1982 (which brought about flood damage in many places of Japan), the five dams already completed in the Tone River were operated to reduce the flood damage (Fig. 5.5). Out of these completed dams, the Shimokubo Dam greatly contributed to the reduction of flood damage with a peak discharge reduction of 62 % for the August 1982 flood and 34 % for the September 1982 flood, respectively (Table 5.5 and Fig. 5.6). Table 5.6 lists the characteristics of the Shimokubo Dam/Reservoir.

Table 5.4
Major Floods in Upper Tone River

Date	Peak Discharge at Yattajima (m^3/s)	Basin Mean 3-Day Rainfall (mm)				Remarks
		Total Basin	Okutone	Azuma	Karasu & Kanna	
Sep. 15, 1947	(17 000)	318	273	271	388	Typhoon
Sep. 16, 1948		204	175	233	211	ditto
Sep. 1, 1949	9680	204	189	210	222	ditto
Aug. 5, 1950	6320	151	103	164	190	ditto
Sep. 18, 1958	9730	168	151	175	184	Tropical Depression
Sep. 27, 1958	5860	149	112	123	205	Typhoon
Aug. 14, 1959	9070	214	178	250	224	ditto
Sep. 27, 1959	5690	169	149	203	163	ditto
Jun. 27, 1961	2950	160	147	156	177	ditto
Jun. 26, 1966	5880	147	137	143	162	ditto
Sep. 24, 1966	6300	115	132	104	70	ditto
Sep. 16, 1972	4380	161	119	189	182	ditto
Sep. 1, 1974	5370	123	55	99	211	ditto
Aug. 23, 1981	7367	253	198	321	260	ditto
Aug. 2 1982	7529	223	155	286	256	ditto
Sep. 13, 1982	8006	209	181	199	244	ditto
Aug. 16, 1983	3229	199	102	237	278	ditto
Jul. 1, 1985	3968	90	76	91	87	ditto
Sep. 3, 1986	4715	131	93	156	74	ditto
Aug. 30, 1988	3046	81	52	120	77	ditto
Aug. 27, 1989	2905	86	45	105	95	Depression
Nov. 30, 1990	2841	125	59	107	168	Typhoon
Aug. 21, 1991	3340	181	129	150	175	ditto
Jul. 14, 1992	942	97	101	113	106	Depression
Sep. 10, 1993	1845	130	139	130	145	Typhoon

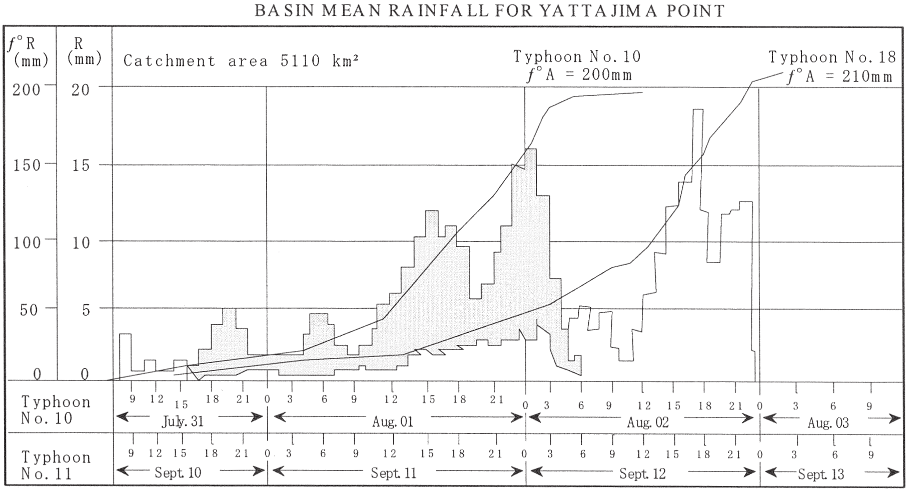
Note: Peak discharge was obtained by measurement except value in parentheses.

Table 5.5
Flood control effect of Shimokubo Dam operation

Date	Aug. 01, 1982	Sept. 11, 1982
Cause of Flood	Typhoon No. 10	Typhoon No. 18
Total Rainfall for 3 Days	319 mm	344 mm
Maximum Daily Rainfall	275 mm	191 mm
Maximum Hourly Rainfall	36 mm	32 mm
Peak Inflow Discharge	1309 m^3/s	865 m^3/s
Outflow Discharge at Peak Inflow	507 m^3/s	567 m^3/s
Total Inflow Discharge	37.3 million m^3	23.1 million m^3
Flood Control Effect Peak Discharge Reduction Total Regulated Discharge	802 m^3/s (62 %) Cut 12.7 million m^3 (34 %)	298 m^3/s (34 %) Cut 5.7 million m^3 (25 %)

Table 5.6
Features of Shimokubo Dam/Reservoir

Name of Dam	Shimokubo Dam
River	Tone River
Country	Japan
Date of Construction	1969 (Completion)
Type of Dam	Concrete Gravity
Height of Dam	129 m
Crest Length of Dam	598 m
Capacity of the Reservoir	130 million m^3
Flood Control Capacity	35 million m^3
Reservoir Area	3.27 km^2
Catchment Area	322 km^2
Average Annual Precipitation	1112 mm
Annual Mean Flow	6.67 m^3/s
Design Flood: Design Discharge for Downstream Areas Design Discharge of the Spillway	 2000 m^3/s 3000 m^3/s



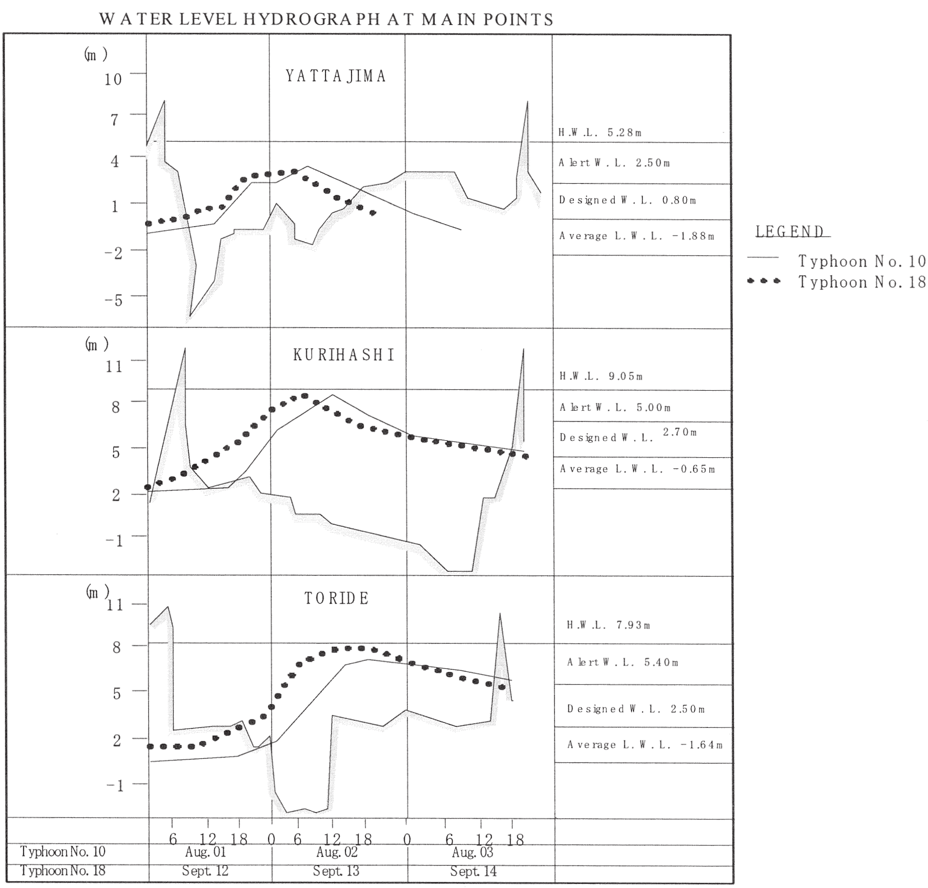


Fig. 5.5
Rainfall and water level in 1982 flood

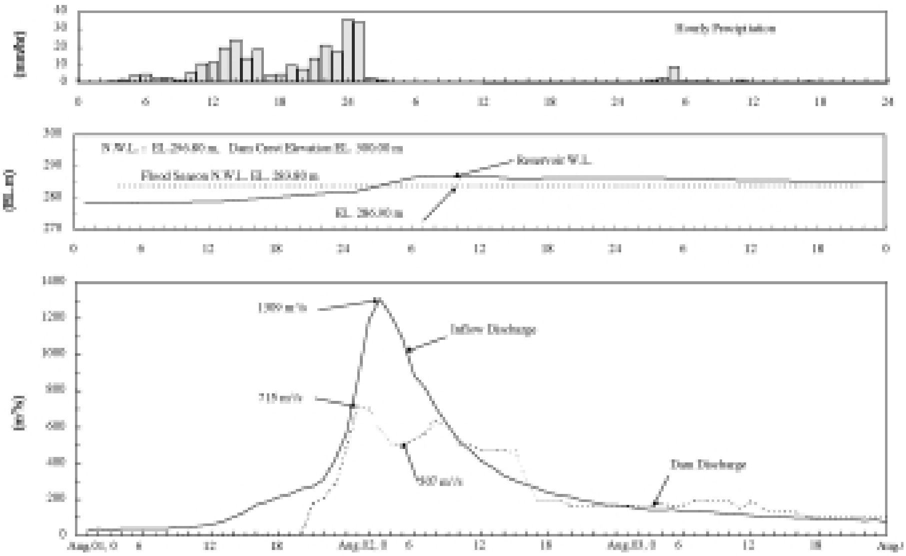


Fig. 5.6
Shimokubo dam. Hydrograph in 1982 flood

Integrated dam management is carried out by the Ministry of Construction to efficiently operate the upstream Tone River dams for flood control. The Integrated Dam Management Office located in Maebasi City receives hydrological information from telemetered rainfall and water level stations. In addition, a radar rainfall gauge has been used for rainfall observation and short-term flood forecasting since 1977.

Additionally, the Integrated Dam Management Office undertakes dam discharge warning operation through speakers and/or sirens in order to ensure safety of the public located in the downstream reaches during discharging from the dams. The warning stations are located every two kilometers along the river, and are controlled from the Office through radio. Moreover, the Office provides guidance for dam discharge operation to dam offices, including dams for power generation and local governments located along the river.

5.2.2. Chikugo River, Japan

5.2.2.1. History

The Chikugo River, which runs through the northern part of the Kyushu District in southwestern Japan, has a main river channel of 143 km and a catchment area of 2860 km^2 (Fig. 5.7). The basin mean rainfall amounts to about 2000 mm, and a large amount of rainfall occurs during the rainy season in early summer from June to July. Although the Chikugo River had been improved for flood prevention for old times, the deluge of June 1953 caused dike breach and overflow in many places and brought a great damage to the area (dead: 147, flooded house: about 74 000, inundation area: about 505 km^2). After the 1953 deluge, a flood control plan was formulated that utilized dams. For this plan, the Matsubara Dam and Shimouke Dam were constructed for the purpose of flood control and power generation. The characteristics of both dams are presented in Table 5.7.

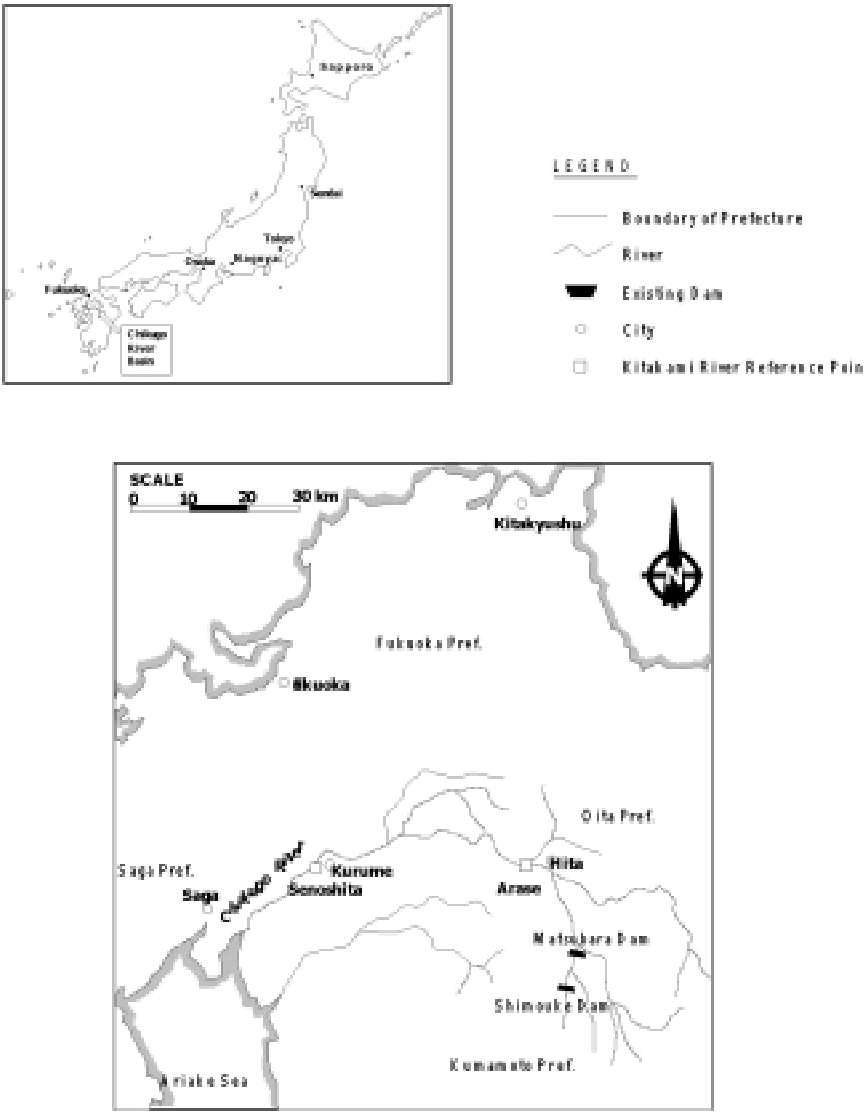


Fig. 5.7
Location of Chikugo River Basin and Dam site

Table 5.7
Matsubara and Shimouke Dam/Reservoir characteristics

Name of Dam	Matsubara Dam	Shimouke Dam
River	Chikugo River	Chikugo River
Country	Japan	Japan
Date of Construction	1973 (Completion)	1973 (Completion)
Type of Dam	Concrete Gravity	Arch
Height of Dam	83 m	98 m
Crest Length of Dam	192 m	248 m
Reservoir Capacity	54.6 million m^3	59.3 million m^3
Flood Control Capacity	45.8 million m^3	53.3 million m^3
Reservoir Area	1.9 km^2	2.0 km^2
Catchment Area	491 km^2	185 km^2
Average Annual Precipitation	2397 mm	2993 mm
Annual Mean Flow	32.90 m^3/s	14.58 m^3/s
Project Flood Discharge for Downstream Areas	2770 m^3/s	1700 m^3/s
Design Discharge of the Spillway	5000 m^3/s	2300 m^3/s

5.2.2.2. Flood Control Plan

The design discharge for Master Plan of the Chikugo River includes a safety degree of 150 year return period, and is indicated in Fig. 5.8. The project flood discharge at Arase Point is 10 000 m^3/s. Among the project flood discharges, flood control dams including the Matsubara and the Shimouke dams and other dams in planning, will regulate 3020 m^3/s at the dam site and the river channel will contain the remaining of 6000 m^3/s, corresponding to a 40-year return period.

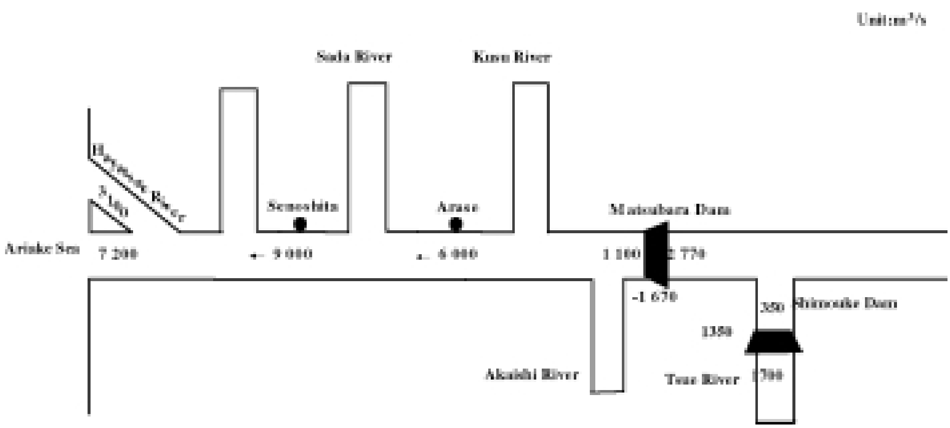


Fig. 5.8
Design discharge for Master Plan of Chikugo River system

5.2.2.3. Performance of Flood Control

Major floods of the Chikugo River recorded in recent years are summarized in Table 5.8. Out of these floods, the effects of flood control for operating the Matsubara and Shimouke dams in the 1982 flood is given in Table 5.9. Matsubara dam operation during the 1982 flood is also illustrate in Fig. 5.9. At Matsubara Dam, the peak discharge was reduced by 64 %, and 10 % of the flood volume was regulated. Flood control effects were even greater at the Shimouke Dam, where 82% of the peak discharge was reduced, and 39 % of the flood volume was regulated.

The integrated dam management is carried out by the Ministry of Construction. The Ministry of Construction office, located in Kurume City, receives hydrological information from upper Chikugo River basin, and has managed the Matsubara and the Shimouke dams since 1979.

Table 5.8
Major Chikugo River Floods

Date	**Peak Discharge at Arase (m^3/s)**	**Peak Discharge at Senoshita (m^3/s)**	**Arase Basin 2-day mean rainfall of (mm)**	**Cause of rainfall**
Jun. 25 to 29, 1953	8500	10 700	523	Seasonal rain front
Jun. 29 to Jul. 2, 1990	3740	5343	290	Seasonal rain front
Jun. 26 to Jul.1, 1979	4220	5150	350	Seasonal rain front
Jul. 23 to 24, 1982	4176	5127	244	Seasonal rain front
Aug. 28 to 31, 1980	3000	3800	274	Typhoon
Jun. 18 to 24, 1965	3600	3690	299	Seasonal rain front
Jul. 7 to 11, 1980	3050	3610	142	Seasonal rain front
Aug. 14 to 20, 1963	4580	3490	243	Typhoon

Table 5.9
Flood Control Effect of Matsubara and Shimouke Dams during 1982 Flood

Name of Dam	Matsubara	Shimouke
Date	July 24, 1982	July 24, 1982
Cause of Flood	Seasonal rain front	Seasonal rain front
Total Rainfall for One Event	1296 mm	1429 mm
Maximum Daily Rainfall	278 mm	301 mm
Maximum Hourly Rainfall	60 mm	51 mm
Peak Inflow Discharge	2911 m^3/s	1939 m^3/s
Outflow Discharge at Peak Inflow	1044 m^3/s	349 m^3/s
Total Inflow Discharge	121.1 million m^3	64.8 million m^3
Flood Control Effect • Peak Discharge Reduction • Total Regulated Discharge	 1867 m^3/s (64 %) 11.4 million m^3 (10 %)	 1590 m^3/s (82 %) 24.4 million m^3 (39 %)

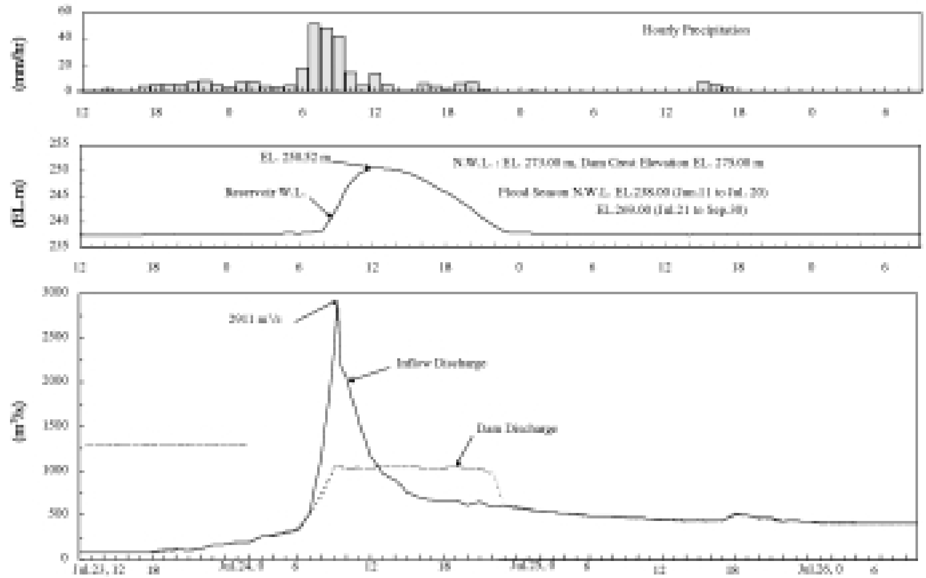


Fig. 5.9
Matsubara Dam Operation (Jul.23-Jul.26, 1982, Seasonal Rain Front)

5.2.3. Ebro River Basin. Floods in Spain in 1982

Floods in Spain represent a grave economic and social problem, and are the most significant natural disasters in the historic memory of its residents [8]. More than 2400 floods have occurred in the last five hundred years, which, on average, signifies approximately five major floods per year. The flooding of Vallés in Catalonia in September 1962 stands out as one of the most important floods. The Vallés flood took about 1000 victims, because of unprotected urban occupation of the flood plains on the banks of the Las Arenas and Rubi rivers. No structures or other flood protection measures existed.

The material damages produced by the floods are considerable, with strong temporary impacts, and continue with an increasing evolution throughout the years. In the 1980s, material damages from flooding were estimated at an annual mean of about 400 M$ per year. The flood damage estimate for the 1990s was about 500 M$. The Report of the National Hydrological Plan of 1993 estimated about 600 M$ per year for decade starting in the year 2000.

In Spain, numerous cases exist that illustrate the beneficial effects of reservoirs for reducing flood damages. One significant case was the 1982 flood that occurred during November 7-8 in the Ebro basin and in the internal basins of Catalonia. The 1982 flood affected the most eastern rivers of the left bank of the River Ebro (the Rivers Gallego, Cinca, Noguera Ribagorzana, Noguera Pallaresa and Segre), and the River Ebro itself from the Ribarroja Reservoir as far as its outlet to the Mediterranean Sea (Fig. 5.10). This flood produced 14 deaths and damages greater than 400 M$.

Fig. 5.10
Ebro River Basin

Studies were conducted on the effects of reservoirs on the flood routing that compared observed effects and measurements to the flood propagation based on the hypothesis that the dams did not exist [8]. The comparison showed that in general, the reservoirs were very efficient in reducing peak flows in their neighboring downstream zone by as much as 80 %. The routing effects of the reservoirs in the upper zones of the basin reduced peak flows in middle zones of the basin, which had ample unregulated areas, by almost 30 %. Additionally, the global effect of the reservoirs on the mouth of the River Ebro reduced overall peak flows by 57 %. For example, the peak flow discharged in the last Ribarroja dam was 3200 m^3/s, (which almost reached the capacity of the river). Peak flows estimated without the existence of the dams in the basin were 7400 m^3/s, more than twice the actual peak flows (Fig. 5.11). Additionally, the arrival of the flood was predicted prior to the actual flood, and it was possible to carry out a certain precautionary withdrawal (of about 1800 m^3/s) in the Ribarroja Dam which also contributed to the reduction of the flood peak.

After the flood of 1982, real-time flood forecasting systems were progressively installed in all the hydrographic basins of Spain. These flood forecasting systems constitute an essential element of flood management in Spain, and have significantly contributed to dam operations during floods. For example, in January 1997, a flood occurred again in the River Ebro Basin, coming this time from the River Ebro itself (near the city of Zaragoza) and affecting its middle and lower stretches. With the data and predictions calculated from the flood forecasting, flood managers were been able to maintain the peak flow discharged in the Mequinenza Dam at about 2700 m^3/s, without affecting cities located downstream of the Mequinenza Dam, such as Tortosa.

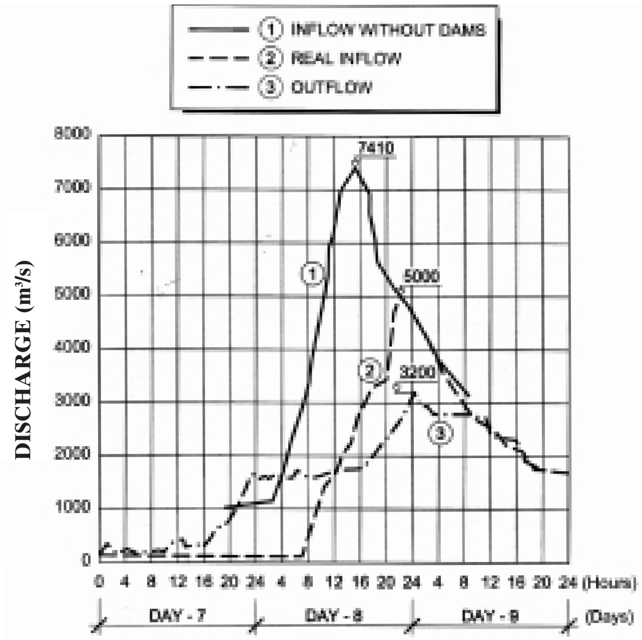


Fig. 5.11
Flood hydrograph. Ebro River. November 1982

5.2.4. The Tunisia Flood in January 1990

5.2.4.1. Introduction

Tunisia (Fig. 5.12) is a country with a generally dry climate. Rains are rare and scattered over most of the country. Rainfall intensity decreases from north to south, and is in the order of 300 mm around Kairouan, in the northern part of the country and drops to 100 mm in Tozeur, located in the central part of Tunisia and is close to the Sahara desert. Only rivers in the northern part of the country flow all year round. Rain falls as short but intense storms and the resulting runoff gathers in the wadis (rivers). Flash floods from the wadis frequently occur, and, destroy villages, communication networks, and cultivated fields and grazing lands. Such flooding disasters occur at regular intervals in Tunisia. The most recent floods occurred in September 1969, March 1973 and January 1990. These flash floods, in addition to destroying infrastructures and fields, maintain a climate of insecurity. This insecurity in turn, undermines the faith of investors, thus blocking any hope of middle- to long-term development of the region.

In January 1990, exceptional rains fell on the northern half of Tunisia causing catastrophic flooding. The final toll was heavy and included 30 deaths, 6000 homeless, and considerable material damages. The toll would have been much greater if the dams of Sidi Saad and El Haouareb, had not been built at the entry of the Kairouan plain. These dams made it possible to save this region from the consequences of the flooding. The Kairouan plain has always been one of the

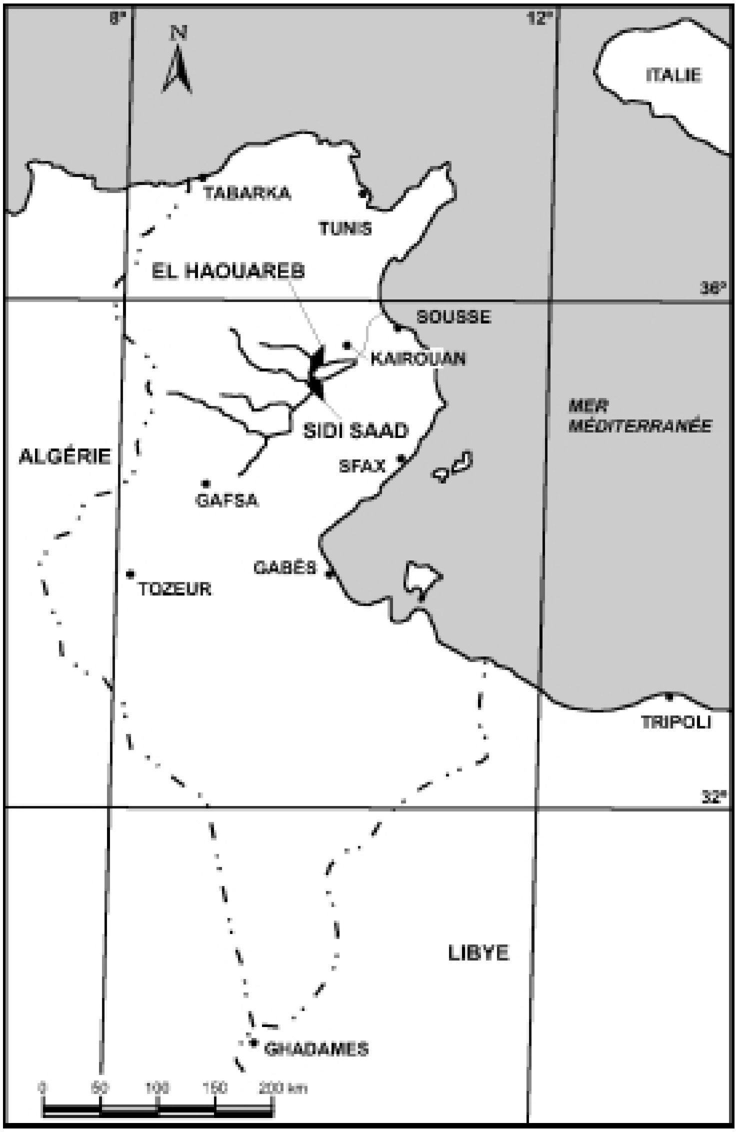


Fig. 5.12
Map of Tunisia

regions in Tunisia most affected by this type of disaster. However, in 1990, the floodwaters did not reach the area, because they were stored in the reservoirs of Sidi Saad and El Haouareb, which had been built shortly before for flood mitigation purposes. Moreover, these floodwaters were put to good use by irrigating nearby agricultural lands. These two dams were built between 1977 and 1990 within the scope of the Canada-Tunisia cooperation.

5.2.4.2. The Kairouan Plan

The 1969 flood was the worst in the 20th century and particularly severe for the Kairouan area (Fig. 5.13). The Tunisian government decided to take the necessary steps to avoid the repeat of such disasters. Canada offered both financial and technical help through the Canadian International Development Agency (CIDA). The protection plan, called “Kairouan Program”, consisted of the following tasks :

1. Building a dam at Sidi Saad to store the floods of the wadi Zroud
2. Building a dam at El Haouareb to store the floods of the wadi Merguellil
3. Developing irrigated perimeters downstream from the El Haouareb and Sidi Saad Dams
4. Drilling fresh water wells downstream from El Haouareb dam (this was not possible for the Sidi Saad dam, because of the high salt content of the water)
5. Reforesting the uncultivated lands upstream of the dams and improving agricultural methods to stabilize the soil and minimize erosion

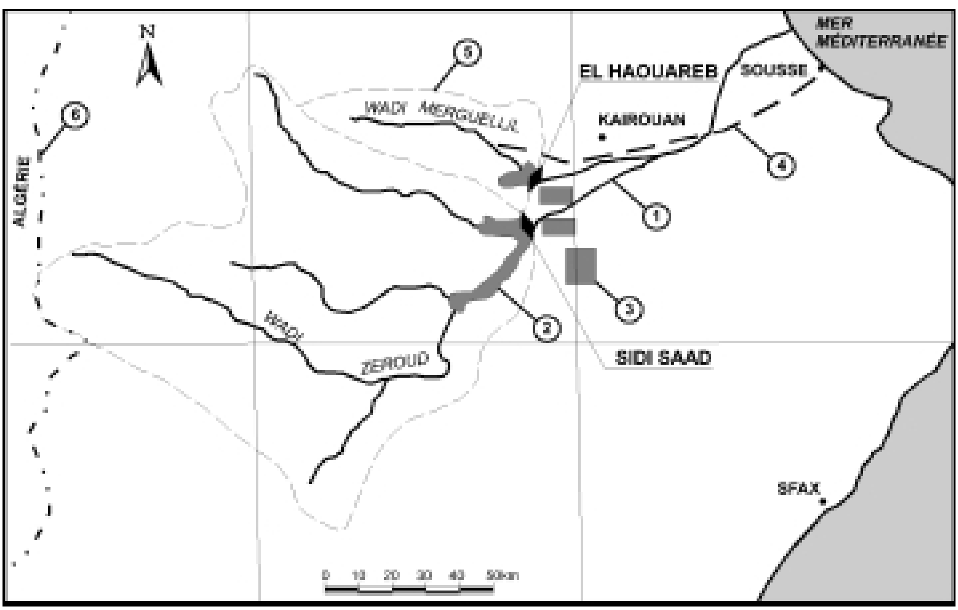


Fig. 5.13
Northern Tunisia - Sidi Saad and El Haouareb Projects

1. Wadi (Intermittent river)
2. Reservoir
3. Irrigated perimeter
4. Fresh water duct
5. Drainage basin
6. International boundary

Construction of tasks in the Kairouan Program was carried out as follows:

1978	Construction of Sidi Saad starts
1982	Sidi Saad commissioned
1985	Construction of El Haouareb starts
1986	Irrigated perimeters downstream from Sidi Saad commissioned
1990	El Haouareb commissioned
1991	Irrigated perimeters downstream from El Haouareb commissioned

Reforestation has been an ongoing activity since the mid Seventies. It is carried out by the local communities, and supported technically and financially by the Tunisian Government.

The Sidi Saad project (Fig. 5.14) consists of an earthfill dam with a maximum height of 82 m and a saddle dike. The total embankment volume is 7.5 million m^3. The ungated spillway has a total capacity of 6900 m^3/s. The reservoir volume is 2000 hm^3, and is divided as follows :

- 130 hm^3 for dead storage;
- 80 hm^3 for irrigation storage;
- 1800 hm^3 for flood control.

Irrigation water is supplied to a surface of 4000 ha.

The project cost was 175 million dollars.

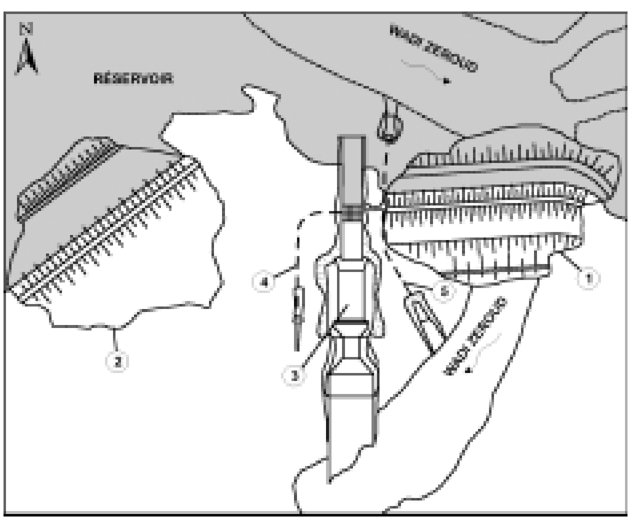


Fig. 5.14
Sidi Saad Project

1 - Main dam
2 - Saddle dyke
3 - Spillway
4 - Irrigation outlet
5 - Bottom outlet

The El Haouareb project (Fig. 5.15) consists of an earthfill dam of 2 km long and 32 m high, with a volume of 6 million m^3. The ungated spillway has a capacity of 2450 m^3/s and the reservoir volume is 215 hm^3:

- 95 hm^3 are for irrigating 2240 ha and supplying drinking water
- 120 hm^3 are reserved for flood control.

The project cost was about 100 million dollars.

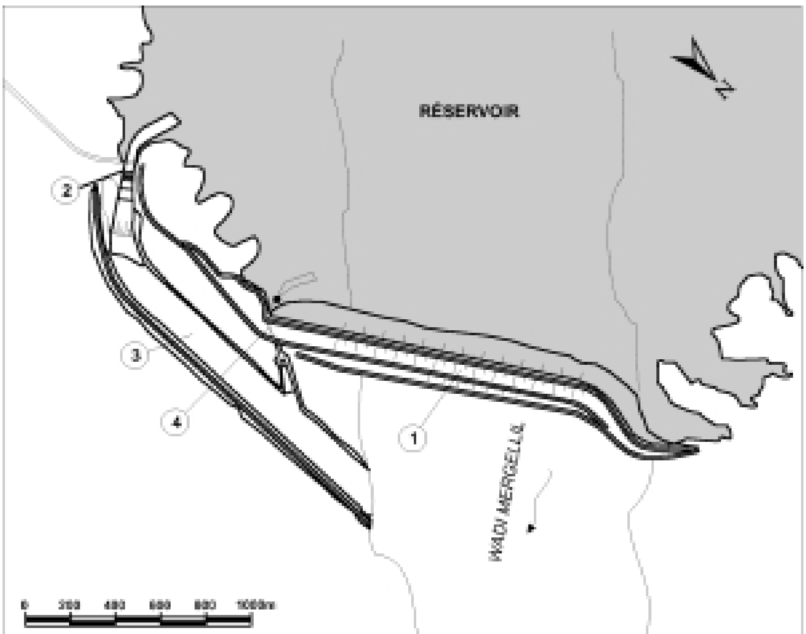


Fig. 5.15
El Haouareb Project

1. Main dam
2. Spillway control structure
3. Spillway outlet canal
4. Irrigation and bottom outlet

5.2.4.3. The January 1990 Flood

Indirect consequences of atmospheric disturbances that affected the north Atlantic and western Europe caused exceptional rain from January 22-25, 1990, over southern and central Tunisia in the quadrangle of Sfax-Gabs-Gafsa-Sousse, largely cutting into the Zroud and Merguellil watersheds (Fig. 5.16). The three-day rainfall exceeded the average annual rainfall for these regions. The resulting floods caused considerable damage. Bridges were washed out, roads were cut, homes were destroyed, and fields were submerged by sand and rendered sterile for several years. The human toll was very heavy, with 30 deaths and 6000 homeless.

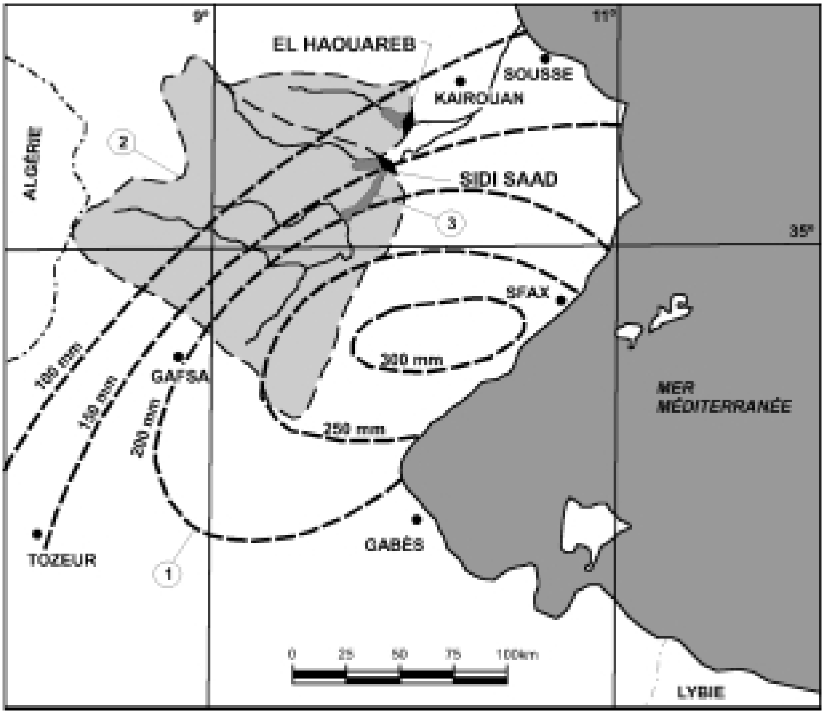


Fig. 5.16
The 21/25 January 1990 storm

1. Isohyet
2. Sidi Saad and El Haouareb drainage basins
3. Reservoir

Amidst this desolation, the Kairouan area – which had been the hardest hit during the disasters of 1969 and 1973 – was spared because flood waters from the Zroud and Merguellil watersheds were stored in the reservoirs of the Sidi Saad and El Haouareb projects provided for the purpose of flood mitigation.

On January 22nd, 1990, the Sidi Saad reservoir was almost empty. From January 22-25, 1990, a total volume of 170 hm^3 entered the reservoir. The peak inflow was reached on January 23rd, at 4 a.m. and was estimated to be 4000 m^3/s (Note that the average long-term flow at Sidi Saad is 3 m^3/s).

On January 23rd at 9 p.m., water started flowing through the spillway sluices. The maximum reservoir elevation was reached on January 24th at 1 a.m., when the outflow was 275 m^3/s.

By comparison, the flood at El Haouareb was relatively modest. The flood volume – 12 hm^3 – had a frequency of once every three years with a peak inflow was 350 m^3/s. The total flood volume was stored in the reservoir, even though construction was not completed at the time. This volume was thus made available for the irrigation of the downstream perimeters and for the supply of drinking water.

In conclusion, experience has shown that the dams built in northern Tunisia constitute essential elements of the region's infrastructure :

- They make it possible to irrigate over 6000 ha of agricultural land
- They guarantee a reliable supply of drinking water in this country bordered by the Sahara desert
- They protect the population and the infrastructures against the devastating effects of large floods.

5.2.5. The September 1990 Flood in the Han River, Korea

5.2.5.1. Flood Characteristics

The Han River Basin is divided into three sub-basins: North, South, and the lower basin that encompasses the Seoul area. Precipitation over the Han River began on September 9 and ended on September 12, 1990. Its significance lies in the fact that the rainfall began and ended simultaneously for the whole basin. The average rainfall for the whole basin was 370 mm, while 380 mm, 350 mm, and 400 mm were recorded for the North, South, and lower basins, respectively. From the record of the precipitation, the amount of a 24-hr precipitation was calculated as 230 mm in the whole basin, 250 mm in the North basin, 200 mm in South basin, and 300 mm in the lower basin. The 24-hr precipitation numbers slightly exceeded those of the 200-year return period for each basin.

The peak inflow at the multi-objective Chungju Dam located in South Basin, was calculated as 22 000 m^3/s. The area of the South Basin is shown in Fig. 5.17. The peak inflow of 22 000 m^3/s is less than the PMF of 26 680 m^3/s, which is estimated from the PMP (Probable Maximum Precipitation). However, the peak inflow of 22 000 m^3/s recorded in 1990 is much larger than the maximum inflow of 14 000 m^3/s recorded in 1972. Considering that the flood with a 10 000 year return period was estimated as 26 500 m^3/s for the design of the dam, the 1990 recorded peak inflow would at least exceed the value of 1000 year return period. On the other hand, at Soyang dam, another multi-objective dam situated at North basin, the peak inflow was around 10 600 m^3/s. This number corresponds to 10 500 m^3/s of 200-year return period which was estimated for the design of the dam, and the amount of precipitation during the flood in the Han River also corresponds to the 200-year return period in the Soyang Dam basin. From a precipitation analysis, it is concluded that a 24 hour-precipitation has 200 year return period in case of the North and South Han River Basins. However, for the overall Han River Basin, the return period of precipitation slightly exceeds 200 years.

According to the flood analysis, the Soyang Dam inflow reached its peak 24 hours after the precipitation started. Therefore, it is reasonable to conclude that the peak inflow of Soyang Dam has a 200-year return period. However, in the case of Chungju dam, the watershed area is much larger than that of Soyang dam, and it took 36 hours for the inflow of Chungju dam to peak. Thus, in order to compare the return period of the inflow with that of precipitation, the 36-hour duration of precipitation must be taken into consideration. But the return period of 24-hour precipitation being larger than that of 36 hour, it may be reasonable to estimate the peak inflow to having about 200-year return period in the case of Soyang Dam.

Fig. 5.17
The Chungju dam Basin

Tables 5.10 and 5.11 show the precipitation and the inflow (at the Chungju Dam) recorded during the 1990 Han River flood.

Table 5.10
Precipitation (September 9, 03:00 – September 12, 04:00)

Basin	Watershed Area (km^2)	Mean Precipitation (mm)	24 hour duration	
			Precipitation (mm)	Return Period (year)
Whole	26 018	370	228	200
South Han river	10 835	346	198	200
Chungju dam	6648	332	207	200

Table 5.11
Flood at Chungju Dam

Type	Flood (m^3/s)	Return Period (year)
Design Flood	16 200	= 200
Past max. Flood	14 000	100(1970)
1990 Flood	22 000	1000

Chungju dam must be operated below 138 m (restricted stage) during the flood season (June – September). When the precipitation began on September 8, 1990, at 16:00, the water level at Chungju dam was maintained at 135 m. According to the weather forecast, the precipitation was estimated to be 150-250 mm. However, the actual rainfall reached 332 mm. The discharge of Chungju dam was continuously increased from 700 m^3/s at the start of the flood. The maximum water level of the dam reached 146.03 m on September 12, 12:00. This stage was 1.03 m higher than the flood stage (145 m) for the design of the dam. Therefore, the downstream region of Chungju dam was damaged by this sharp increase in discharge.

5.2.5.2. Dam Operation

Chungju dam was constructed in 1985, and five major floods occurred there from 1987 to 1990. These five floods and their characteristics are listed in Table 5.12.

Table 5.12
Chungju Dam Major Flood Characteristics

Year	Peak Time (Inflow)			Max. Inflow (m^3/s)	Max. Stage (El, m)
1987	July	22	22:00	11 761	140.75
1987	August	4	23:00	4558	141.49
1987	August	31	10:00	6294	142.47
1990	June	26	07:00	4418	136.60
1990	September	12	00:00	22 164	146.04

The 1990 flood in September was the most severe among those floods listed. The overview of dam operation for this maximum flood is shown in Fig. 5.18 as variation curves of the reservoir inflow, outflow, and stage.

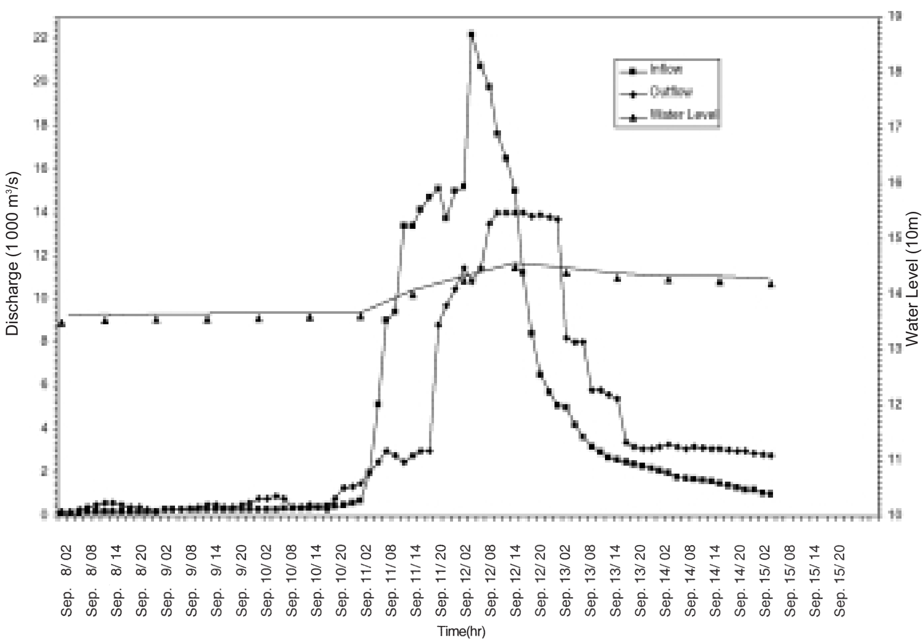


Fig. 5.18
Variation curve of reservoir inflow, outflow, and stage (Chungju Dam)

Two major floods, which occurred in Soyang dam (constructed in 1972), are listed in the Table 5.13.

Table 5.13
Soyang Dam Major Flood Characteristics

Year	Peak Time (Inflow)	Max. Inflow (m^3/s)	Max. Stage (El. m)	Max. Outflow (m^3/s)
1990	September 1 23:00	2818	192.98	1013 (September 3, 20:00)
1990	September 11 15:00	10 653	197.99	5575 (9.12. 01:00)

The overview of Soyang Dam operation for the September 11 flood is shown as variation curves in Fig. 5.19.

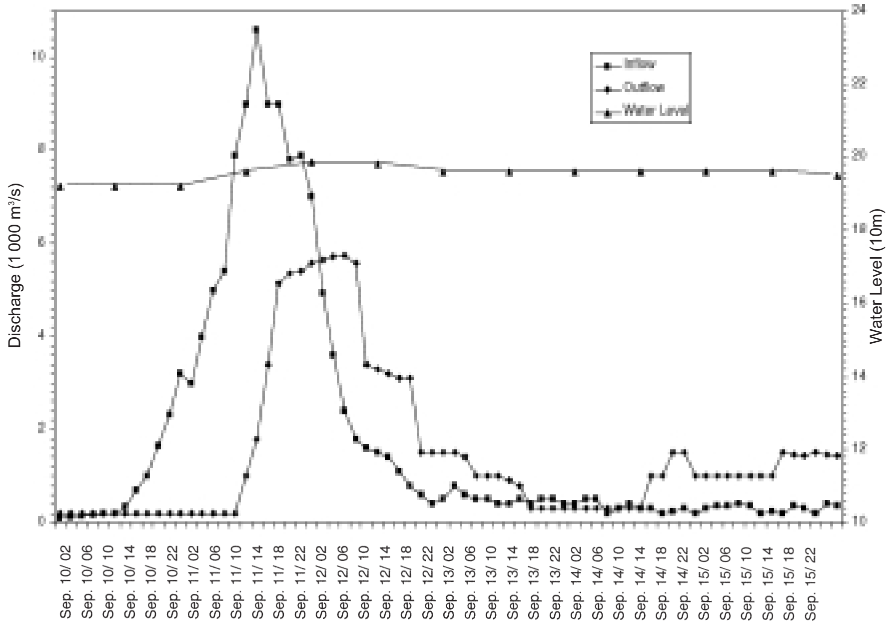


Fig. 5.19
Variation curve of reservoir inflow, outflow, and stage (Soyang dam)

The initial stage of Soyang dam had been maintained as El. 191.6 m. When rainfall started, the preliminary discharge was not made in order to secure water resources for the next year's supply. Therefore, the stage of dam reached El. 197.99 m. This stage corresponds to the Soyang Dam design flood stage (El. 198 m).

5.2.6. The Great Midwest Flood of 1993, USA

5.2.6.1. Introduction

The Great Flood of 1993 in the Midwest was a hydrometeorological event without historic precedent in the United States. The Midwest 1993 flood surpassed all previous floods in terms of precipitation amounts, record river stages, flood duration, areal extent of flooding, crop and property damage, and economic impact. Record and near-record summer rains fell on soil saturated from previous seasonal precipitation and spring snowmelt, and produced flooding along major river systems and their tributaries over a region that encompassed all or most of nine states in the upper Mississippi River basin (see Section 3.4 for a more detailed description of the flood and its impacts).

5.2.6.2. Flood Control Dams and Reservoirs

The U.S. Army Corps of Engineers (USACE) has built 76 reservoirs in the upper Mississippi River basin for the purpose of flood damage reduction. The capacity of these projects to store flood runoff is vast; the aggregate volume is more than 49 billion cubic meters and the controlled drainage area totals 956 076 sq. km. Most reservoirs are operated to provide protection to communities and agriculture along tributaries, and some are operated for the main stem rivers. The majority [49] of the reservoirs are located in the Missouri River basin where the USACE also operates 22 U.S. Bureau of Reclamation (USBR) reservoirs for flood control. The USACE reservoirs in the Missouri River basin contain almost 35 billion cubic meters of flood space and those of the USBR provide and additional 6.7 billion cubic meters. Combined flood storage space in the 24 USACE reservoirs on tributaries other than the Missouri River, that drain into the Mississippi River above St. Louis, amounts to 5.6 billion cubic meters. The USACE has also constructed three reservoirs on tributaries to the Mississippi River below St. Louis with a combined flood storage volume of 1.6 billion cubic meters.

5.2.6.3. Reservoir Regulation for Flood Control

Most of the reservoirs were regulated throughout the 1993 flood to store floodwater and reduce the level of flooding in the rivers on which they are located. Hydraulic and hydrologic engineers in USACE Division and District Reservoir Control Centers in the flood-affected region executed water control management. USACE and USBR flood control reservoirs on the main stem and tributaries in the Missouri River basin reduced peak discharges on the Missouri River by storing almost 21 billion cubic meters of flood water between June and August. In addition, USACE reservoirs in other parts of the upper Mississippi River basin stored about 4.3 billion cubic meters thereby making the total combined peak storage volume equal to 25.3 billion cubic meters. To put this in perspective, that combined amount of stored flood water is greater, by more than 6 billion cubic meters, than the water volume carried by the Mississippi River past St. Louis in the average month of June.

In addition to the reservoirs, the USACE has constructed or improved over 3500 kilometers of levees for the protection of communities and agriculture in the upper Mississippi River basin. Though records on the Federal levees are kept by the USACE, there is no known inventory of the estimated 9300 kilometers of non-Federal levees that have been constructed in the basin.

The USACE estimates that flood damage reduction facilities (reservoirs, floodwalls and levees) in place during the 1993 flood prevented $ 19.1 billion in damages in the Mississippi and Missouri River basins. Of that total, $ 11.5 billion in damages were prevented along the Missouri River while $ 8 billion in damages were prevented along the Mississippi River. Damages prevented by utilization of the flood storage reservoirs amounted to $ 7.4 billion in the Missouri River basin alone; $ 4.0 billion by the storage of flood water in the six main stem Missouri River reservoirs, and $ 3.4 billion by the reservoirs on Missouri River tributaries such as the Kansas River. The other $ 4.1 billion in damages prevented along the Missouri River is attributed to levee projects. Upstream reservoirs and levees prevented damages of about $ 5.6 billion at Kansas City, Missouri and Kansas City Kansas, on the Missouri River. In the St. Louis metropolitan area along the Mississippi River, approximately $ 3 billion of damages were prevented with the combination of upstream reservoirs, levees, and floodwalls. It has been estimated by the USACE that storage in upstream reservoirs in the Mississippi River basin reduced flood peak stages at St. Louis by nearly two meters. Inasmuch as the flood crested within one meter of the top of the floodwall in downtown St. Louis (Fig. 5.20), the value of these reservoirs as flood damage reduction measures is obvious.

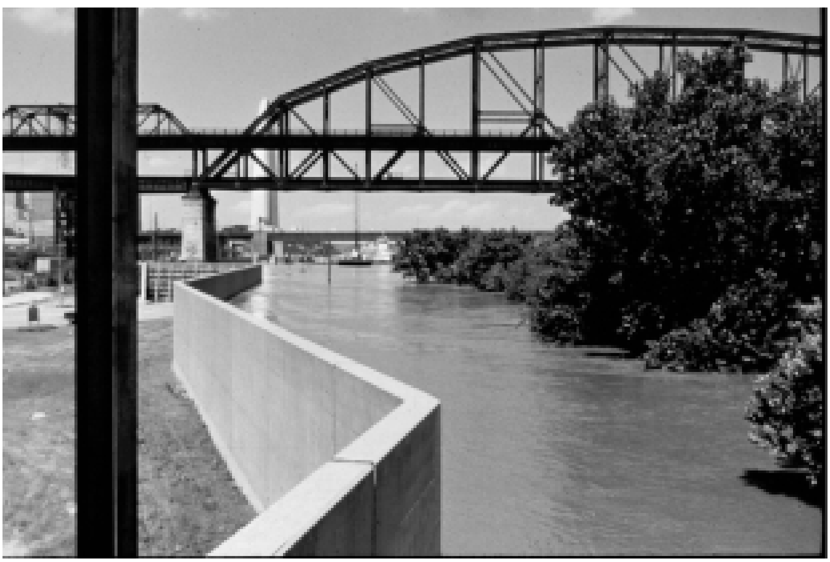


Fig. 5.20
Floodwall at St. Louis, Missouri. Mississippi River looking upstream at St. Louis
(Photo courtesy of U.S. Army Corps of Engineers)

5.2.7. The Spring Flood of 1995, Glomma River, Norway [9,10]

During the spring flood of 1995 in the 42 000 km^2 Glomma River basin in Norway, hydropower reservoirs had a significant flood-reducing effect. The 26 hydropower reservoirs in the catchment have a storage capacity of 3500 million m^3, which amounts to only 16 % of the average annual runoff, *i.e.* 22 000 million m^3. Normally, reservoir operation in Norway is almost exclusively aimed at hydropower production. However, provisions are made for flood protection in the relevant legal regulations. During the 1995 Glomma flood, reservoirs reduced the flood peaks at strategic locations by as much as 15-20 %.

Snow surveys during late winter 1994 showed that the water equivalent of the snow was well above average in large parts of the basin, and that a major snowmelt flood in the spring could result. Permission was granted to deviate from licensing conditions that regulated the timing and amount of water releases, in order to make room for the expected snowmelt inflow. When the air temperature rose rapidly in late May, heavy and long-lasting rains accompanied it. The combined effect of intense snowmelt of the heavy snowcover and high rain amounts, created a flood of 100-200 years return period, which has been surpassed only once in recent centuries in Norway, in 1789.

The reservoir operational strategy during the flood was to continue forced releasing until the actual arrival of the peak, then immediately reducing to a minimum, thereby cutting the peak discharge. The risk was that a premature reduction in release may have resulted in available storage capacity loss before the peak occurred. In contrast, a delay in release reduction would have increased the peak. Fig. 5.21 shows that regulation of the upstream reservoirs dampened the natural flood culmination of approximately 4200 m^3/s by some 800 m^3/s at the town of Elverum.

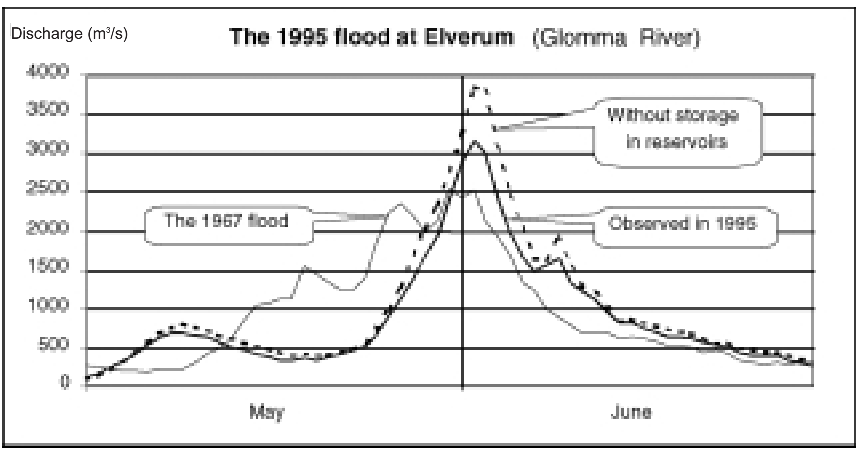


Fig. 5.21
Hydrographs based on daily data of the spring floods at Elverum, Glomma River, Norway, in 1967 and 1995

The largest lake in Norway, lake Mjøsa (368 km^2, regulated height 3.61 m) is situated in the largest tributary to the Glomma River, and is located immediately upstream of the confluence. It has several important cities along its shores, including Lillehammer, the site of the 1994 Winter Olympic Games. Some 40 km downstream of the confluence, Glomma enters Lake Øyeren, which is also occupied with important population centres. During the 1995 spring, the flood peak in the main river approached the confluence before the forecasted culmination of Lake Mjøsa and the tributary. The release from Lake Mjøsa was reduced by 300 m^3/s for a period of 24 hours, and was timed to the moment of flood peak passage in the main river at the confluence. The temporary increase in water storage and lakewater level in Lake Mjøsa, was compensated by faster release during the following few days, and in time before the local flood peak arrived. Thus, it was possible through reliable forecasts, and well performed manoeuvring, to reduce flood peak levels in both lakes.

5.2.8. Flood Control in the Paraná River Basin, Brazil. Using Hydroelectric Reservoirs

5.2.8.1. Introduction

Most (96 %) of electricity generation in Brazil is from hydropower. Gross annual energy generation was 296 000 GWh in 1995, including 74 000 GWh produced by the bi-national Itaipu power plant. Total installed capacity is 55 500 MW, including 50 % of Itaipu 12 600 MW capacity. Thermal power plants add up to less than 5000 MW. About 75 % of the energy is generated in the southern/southeastern systems, mostly in the Paraná River basin.

Because of the lack of thermal generation, energy regulation is based on the operation of large storage reservoirs. Flood control becomes a natural by-product. Although not contemplated in the economic feasibility studies for the reservoirs, flow regulation from the reservoirs added the benefit of reduced flood frequencies downstream and induced economic activities in the flood plains. Gradually, with the development in the valley, the economic impact of floods increased, in addition to the need arose for specific consideration of potential flood damages in the reservoir operation rules.

The consequences of large floods in the upper Paraná River basin, including the overtopping and breaching of Euclides da Cunha and Limoeiro Dams in 1977, compelled Eletrobrás, the Brazilian Official Agency for Electric Energy, to establish an operation program for the main hydroelectric systems, which would specifically take flood control into account. Empty volumes were to be established in the reservoirs at the beginning of the flood season to limit the outflows above restriction levels to a given frequency that was economically justified.

An increase in flood safety level means a reduction in the probability of filling the reservoir at the end of the flood season. In contrast, filling the reservoir at the end of the flood season is the target for maximum reliability of energy production. In multiple purpose projects this conflict of interest is taken in account in the planning stages. Allocation of storage for multiple uses is part of the project and tied to the appropriation of construction funds. The Brazilian experience has been one of adapting the existing hydroelectric system, which was conceived and implemented for power production, to flood control purposes. Judgment as to the level of protection given to the flood plains downstream became a socio-political problem handled by Eletrobrás as the representative of the Federal Government.

The control release rate to prevent downstream flood damages is defined for each project. Empty reservoir volumes, at the beginning and throughout the flood season, are defined for a range of risks of occurrence. Simulation of the hydroelectric system is used to evaluate the risk of thermal generation and energy shortage as influenced by the flood control volumes. Economic and socio-political consequences of floods as well as those of power shortage are assessed if not accurately estimated. The volume for flood storage, which is variable along the flood season, is recommended for each reservoir to provide the selected degree of safety.

5.2.8.2. Flood Operating Rules of the Dams

The methodology of the flood control process and the results and benefits of the operation are illustrated for the Paraná River basin above Jupiá reservoir, which encompasses an area of 470 000 km^2. Twenty-six power plants are installed in the basin with a total capacity of about 24 000 MW (see Table 5.14).

The most common operational constraint is that the flow release is bound to produce flood damages downstream from each project. Frequent limitations include bridge crossings and roads at relatively low levels (Estreito, Jaguara, Porto Colombia, Marimbondo, Emborcação, São Simão). Towns, villages, and farm housing along the valley, as well as productive agriculture land can be flooded (Camargos – Itutinga, Volta Grande, Itumbiara, Barra Bonita, Jupiá). In a few cases, flooding of the project powerhouse itself is the main outflow restriction, as for the Peixoto project. Temporary limitations may exist that will lessen river handling risks in a dam under construction downstream (Nova Ponte).

The control release rates for each project are indicated in Table 5.14, together with the frequency of the respective natural discharge. In many cases the control discharge has a low return period, between 2 and 10 yrs. The potential benefit of reservoir operation is apparent.

The volume × duration curve method is used to estimate the flood control space. A curve of runoff volume versus duration is obtained for different probabilities of occurrence. The Log Pearson III distribution was adopted for frequency analyses of flood volumes. The runoff volume curve is compared to the volume to be released at the constant control release rate. The maximum difference between the two volume distributions indicates the empty volume needed at the beginning of the wet season.

For a long flood season, 4.5 months for the Paraná basin (end of November to mid April), the risk of not re-filling the reservoirs is high. To reduce this risk, the flood control space is computed at successive time intervals, from each time to the end of the wet period. For the same protection level, the control storage needed is gradually reduced. In practice, 100 % of the empty volume is required in the two first months (December, January), reduced to 60 to 70 % of the initial volume by the end of February and then reduced to about 15 % at the end of March.

This method is initially applied to each project in relation to its own control release rate. Run of river projects cannot have volumes allocated for protection of the stream immediately downstream. That is the case of Estreito, Jaguara, Volta Grande, Porto Colômbia and particularly Jupiá. For the latter, about 12 km^3 of storage are needed for the risk of one in 35 years while the net reservoir volume is less than 1 km^3.

Storage space has to be created in the upstream reservoirs and operation planned to respect the critical downstream restrictions. Except for Camargos and Barra Bonita, all reservoirs are used for that purpose. The volume is distributed proportionally to the average of the maximum monthly historical flows of the wet season (December-April), so as to allocate larger volumes at reservoirs closer to Jupiá. The resulting volumes in each reservoir are, in general, greater than those needed for the immediate downstream control. Furnas reservoir provides the

Table 5.14
Paraná River basin system – Flood control volumes

PROJECT		INSTALLED CAPACITY (MW)	RIVER	BASIN (km^2)	NET RESERVOIR VOLUME NRV (km^3)	CONTROL RELEASE RATE (m^3/s)	RETURN PERIOD OF CONTROL FLOOD (yrs)	EMPTY RESERVOIR VOLUME 1:35 FREQUENCY – LOCAL (km^3)	FOR THE SYSTEM (km^3)	FOR THE SYSTEM % NRV
Camargos	1960	43	Grande	6280	0.672	400	< 2	0.273	–	41
Itutinga	1955	54	Grande	6280	0.007	400	< 2		–	
Furnas	1963	1312	Grande	50 460	17.217	4000	6	1.016	1.425	8
Peixoto	1956	478	Grande	59 600	2.500	4400	9	0.893	0.271	11
Estreito	1969	1104	Grande	61 940	0.178	–				
Jaguara	1971	616	Grande	62 700	0.090	4500	3	0.987	–	
V. Grande	1974	380	Grande	68 100	0.268	5000	3	1.069	–	
P. Colômbia	1973	328	Grande	73 400	0.233	7000	–	0.259	–	
Caconde	1966	80	Pardo	25 070	0.504	10 (min)			–	
E. Cunha	1960/79	108	Pardo	43 070	0.005	–			–	
Limoeiro	1958/79	32	Pardo	44 050	0.016	25 (min)			–	
Marimbondo	1975	1488	Grande	116 700	5.260	8000	11	0.520	1.590	30
A. Vermelha	1979	1380	Grande	139 900	5.169	–			1.528	30
Emborcação	1982	1192	Paranaíba	29 300	12.521	5000	67	0.000	0.426	3
Nova Ponte	1993	510	Araguari	15 300	10.380	2500	20		0.963	9
Itumbiara	1980	2280	Paranaíba	95 000	12.454	7000	5	0.810	1.972	16
C. Dourada		638	Paranaíba	98 970	0.150	–				
São Simão	1978	1680	Paranaíba	171 000	5.540	16 000	250	0.000	1.784	32
Barra Bonita	1963	140	Tietê	32 330	2.566	1800	4	0.832	–	32
Bariri	1965	144	Tietê	35 430	0.060	–			–	
Ibitinga	1969	132	Tietê	43 500	0.056	–			–	
Promissão	1975	264	Tietê	57 610	2.128	–			0.450	21
N. Avanhandava	1982	303	Tietê	62 300	0.380	–			–	
Ilha Solteira	1973		Paraná							
Três Irmãos	1990	3888	Tietê	445 360	8.956	–			4.285	48
Jupiá	1968	1414	Paraná	470 000	0.903	16 000	2	11.990	–	
TOTAL					88.213				14.694	17

protection for the series of river plants in the Grande River. The last two columns of Table 5.14 show the typical results for the selected level of protection with an annual probability of 1:35. Volumes at Peixoto, Itumbiara and Nova Ponte were arbitrarily increased to further reduce the probability that the local control release rates would be exceeded.

The actual computations are not without difficulties. The frequency analyses of flood volumes are marred by the effects of sample variations. Occasionally, the runoff-duration curve is not concave as expected. Analytical procedures have to be developed to smoothen out the differences and adjust consistent equations. Conceptually, the selection of the probability distribution function is an open question. No physical principles justify any one flood probability distribution. Best fitting to the samples is not a valid indicator. Fortunately, the study deals with low return period events, between 10 to 50 yrs, mostly within the duration of the historical series, and within the influence of the distribution function is not significant. Empty volumes, which should decrease gradually towards the end of the flood season, may follow irregular trends because of sample variation. Sophisticated analytical procedures, utilizing a stochastic hydrology approach as the critical trajectories method, have been developed to improve the consistency of the analysis. Practical results have been achieved in studies of empty reservoirs for individual projects. The methodology is yet to be improved for extension to a system of reservoirs.

The large reservoirs effectively control floods with minimum burden to the electric supply system. In the Paraná River basin, total net storage (NRV) attains 88 km^3, which is about 55 % of the mean annual runoff at Jupiá. The NRV provides flow regulation for energy production purposes which enables the system to span periods of drought extending for several years. Energy supply planning is based on simulation of the hydroelectric system and evaluation of energy shortage risks. Present risk level is about 1:20. Traditionally, the reservoir system was designed to cope with the historical critical dry period extending for 5 yrs, from 1951 to 1956, for a risk of about 1:50. Allocation of about 17 % of the NRV to flood control, early in the wet season, has not measurably reduced the reliability of energy supply, basically because of the ample storage available, the close control of actual operation, and the well-defined flood season.

Flood operating rules are developed for each dam in accordance with the respective project elements and downstream and upstream constraints of discharge and levels. Rule curves are set up for operation during both normal and emergency conditions, and the latter occurs because of the risk of filling up the flood storage volume or in the case of loss of communication with the flood operation center.

The experience on flood control by operation of hydroelectric reservoirs in Brazil has been positive. The annual risk of damage-producing floods has been significantly reduced in many systems. Natural flood return periods of 2 to 10 yrs have been reduced to the 15 to 100 yrs range. In theory, the return period of the critical discharge constraint of 16 000 m^3/s in Jupiá has been improved from 2 to 35 yrs. Actually, in the last 14 years, the control release rate was broken only once by an exceptional outflow of 24 700 m^3/s in 1991, and slightly surpassed (16 700 m^3/s) in 1992. The improved flood control conditions over the 2-yr return period are being confirmed.

An assessment of results attained so far indicates that further benefits can result from institutional measures related to land use and occupation in the flood plains. Integration of power project owners, government and local communities, for a common planning and management of potential flood areas, are at present the main issue.

5.2.9. Miami Conservancy District - USA

5.2.9.1. Introduction

The purpose of this section is to discuss, in general, the contributions to flood damage reduction that have been made by non-Federal entities in the United States and in particular, present an overview of one unique entity, the Miami Conservancy District. In the U.S., flood damage reduction is the high priority use for 1649 of the 6356 existing dams that are 15 m or more in height [11]. Of these dams, over 1300 have been constructed by non-Federal interests, including state and local governments, separately established flood control districts, public utilities, and private interests (National Inventory of Dams, U.S. Army Corps of Engineers). Even though non-Federal entities have constructed many flood control dams as evidenced above, few of these dams have been single purpose or part of a comprehensive basin plan. Rather, they have been built for protection of local business and residential areas to permit further development or to address very specific flooding problems. The Miami Conservancy District, one of the oldest separately established flood control and conservation districts in the U.S., is an exception. Because of the way the Miami Conservancy District was established and the time at which it was established, it holds a special place, both politically and technically, in the historical development of flood damage reduction technology in the U.S. The following discussion of the Conservancy District is based primarily on a book by Arthur E. Morgan [12]. Mr. Morgan was the Chief Engineer of the Conservancy District during development and implementation of the comprehensive flood damage reduction plan for the Miami River Valley.

5.2.9.2. The Need for Flood Control in the Miami Valley

The Miami River rises in the northwestern part of the state of Ohio and generally flows in a southerly direction to its confluence with the Ohio River, draining an area of 10 360 sq. kilometers. The drainage basin of the Miami River is shown on Fig. 5.22. The drainage basin is composed primarily of gently rolling hills. Stream slopes average about 0.6 m/km along the main stem with tributaries being somewhat steeper ranging from 2.0 to about 20 m/km. The steep tributary slopes thus promote rapid runoff from the watershed during periods of high rainfall, which results in severe flooding along the main stem of the Miami. The average annual precipitation for the Miami River drainage area is approximately 960 mm.

The Miami Valley was settled around 1800. From these earliest days flooding was a major problem for the residents. Large floods occurred along the Miami River in 1805, 1814, 1828, 1832, 1847, 1866, 1883, 1884, 1897 and 1898. Levees were first constructed following the 1805 flood to provide some measure of flood protection to the residents in the developing urban areas. After each successive flood, the levee system was repaired and minor improvements were made to increase the amount of protection. In spite of these efforts it was becoming more and more apparent that the battle against flooding was being lost. The Great Miami River Flood of 1913 erased any doubt about the amount of flood protection required.

In March 1913, heavy rain fell on the Miami River watershed for five days. Between 200 and 250 mm of precipitation fell on frozen ground, resulting in unprecedented amounts of runoff. On March 25 and 26, the Miami River was out of its banks for nearly its entire length. Levees were overtopped and breached, farmlands were destroyed, roads, bridges and other infrastructure were washed away, and cities were devastated. Flooding in the urban areas of Pique, Troy,

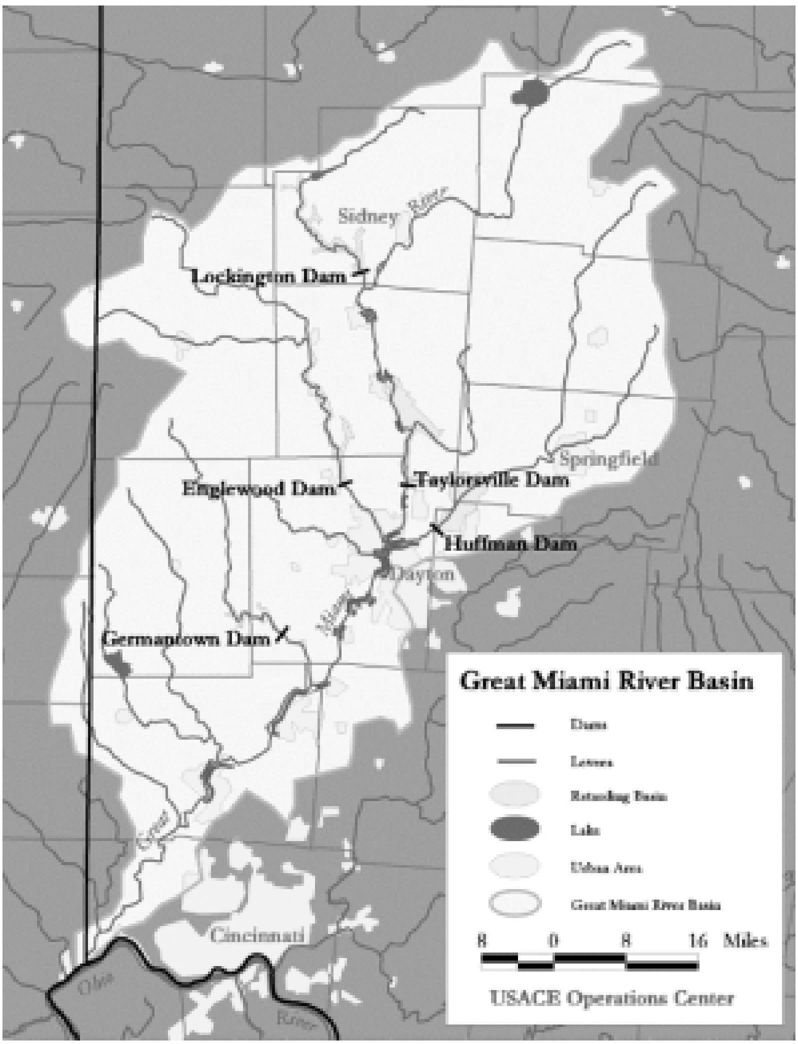


Fig. 5.22
Miami River Basin

Dayton and Hamilton reached depths greater than three meters with attendant high velocities. By the time the rain stopped and the flooding receded, more than 360 people had lost their lives and property damage exceeded $ 100 M. In Dayton, the peak discharge of the flood was estimated to be 7100 cubic meters per second, nearly three times the discharge of any previously known flood.

It is ironic that when the 1913 flood occurred, plans to increase the capacity of the levee system through Dayton to accommodate a peak discharge of about 2500 cubic meters per second were being debated. It was now clear that additional flood protection measures were needed that could provide substantially higher levels of protection to the urban areas. Further, concerns were being raised as to whether levees alone were capable of providing this needed protection and many prominent engineers were actively advocating the use of detention dams (not an accepted practice in the U.S. at that time) as a complimentary flood damage reduction measure.

5.2.9.3. Commitment to Developing a Comprehensive Plan

In the wake of the 1913 catastrophe, the people of the Miami Valley developed a stronger resolve to address their flooding problems in a more comprehensive manner. Citizen groups were formed to push for complete flood protection in the valley. The residents of the Miami Valley were determined to have comprehensive flood protection and to obtain it using their own resources. This commitment was demonstrated when the city of Dayton appropriated $ 2 M to begin planning for the new flood protection system.

With no legal or engineering precedent for an effort of this magnitude by a non-Federal entity, the task would prove to be extremely difficult. A consortium of diverse interest groups with a common goal would need to be formed and substantial new financial resources would need to be identified. Most importantly, new state legislation permitting the establishment of a conservancy district, with the power to tax citizens to raise the required funding for design, construction and operation of the flood protection system, would be required. Opposition to passage of such legislation was considerable. This opposition by citizens in the Miami Valley came primarily from the rural areas, upstream of the major urban areas, where there was a reluctance to provide the lands that would be needed to construct the flood detention facilities that were being discussed. These and other objections (primarily related to safety of embankment dams upstream of urban areas) to the proposed new legislation were pursued by opponents of the plan. Supporters, after a difficult fight prevailed and the Ohio State Legislature passed, and the governor signed into law, on 7 February 1914, the Ohio Conservancy Act.

The Miami Conservancy District could now be formed under the law, with the approval of a Conservancy Court that would be established under provisions set forth in the Act. However, bitter opposition to the creation of the District continued and many specific issues were appealed all the way to the Ohio Supreme Court. After four years of legal arguments, the District was finally organized by the Conservancy Court, with three directors appointed by the Court, and was ready to move forward with the development of an official comprehensive plan, as required by the Act, for flood damage reduction in the Miami Valley.

5.2.9.4. Preparation of the Plan

Immediately following the 1913 flood, while a preliminary plan was being prepared, several interim measures were put into place. First, an early warning system was established. As part of this system, nine river gages were installed to monitor river conditions. In addition, river channels were cleared of sediment and debris left by the 1913 flood to provide increased conveyance, and levees that had been breached or weakened were rebuilt. The main purpose of these efforts was to reduce the hazard to the populace if another flood should occur prior to implementation of a comprehensive plan.

The planning process included the study of several types and combinations of flood damage reduction measures. Among the plans studied were channel improvements, levees and detention reservoirs, and numerous combinations thereof. As a result of these studies a conceptual plan was developed that included detention dams in rural areas to prevent floodwaters from reaching damage centers, and levee and channel improvements to increase conveyance through urban areas. Operating in concert, these measures could be expected to provide a high level of flood protection to the valley residents. Further, the combination of both detention dams and channel improvements resulted in the elimination of the need for many costly bridge relocations that would be required for a plan composed solely of channel improvements.

In spite of the considerable effort expended by the District in developing the plan, opposition to it continued. It was alleged that ongoing flood control studies in the Ohio River Basin by the Federal government would bring into question the viability of using detention dams as flood control measures in the Miami Basin, primarily due to safety considerations. When results of the Federal study were reviewed by the Conservancy Court and an independent group of engineering consultants, the District plan was praised for its completeness and the quality of its technical analyses. No safety issues were raised as a result of the review.

5.2.9.5. Implementation of the Plan

The official Plan as required by the Conservancy Act, and which was ultimately developed and implemented, consisted of five detention dams and levee and river channel improvements in nine urban areas. The five earthen dams ranged in height from about 20 to 37 m. Release facilities consisted of uncontrolled low level outlets and uncontrolled ogee crest emergency spillways, thus eliminating the need for human intervention during flood operations. Release facilities were sized to limit outflows to non-damaging discharges at damage centers downstream, while giving full consideration to the increase in capacity resulting from the levee and channel improvements.

In order to develop an appropriate design flood that would assure a high level of flood protection, extensive studies of eastern U.S. rainfall and flooding were undertaken. In spite of the fact that at that time only short periods of record, often of uncertain quality, were available, 160 storms in the eastern U.S. were studied in great detail. Also, in an effort to establish an understanding of the relationship between the occurrence of large floods in short periods of record and what might be expected over longer periods of time, long-term streamflow records from Europe on the Danube at Vienna (900 years), the Tiber at Rome (2300 years) and the Seine at Paris (300 years), were also investigated. Based on these studies, it was postulated that a flood having a magnitude 20 % greater than the highest flood recorded was reasonable. To provide a greater factor of safety, a flood 40 % greater than the 1913 flood was selected by the District for design purposes.

In order to establish a basis for the tax assessment on residents of the valley, a detailed economic evaluation was performed to estimate the benefits expected from the Plan. Over 77 000 individual tracts of land and 195 km of river were assessed to determine both benefits and costs of the Plan. Two objectives were set for the economic evaluation: the benefits had to exceed the costs by a ratio of at least 3 to 1, and there had to be an equitable distribution of the costs among the landowners. When the appraisal study was completed in 1917, and the objectives were met, a levy of nearly $ 28 M was made on the valley residents to execute the Plan, and implementation proceeded. With financing assured, construction began in 1918.

5.2.9.6. Performance of the System

Construction of all elements of the plan was completed in 1929. Since completion, the detention dams have stored floodwater more than 1000 times. In 1959, when rainfall neared the amount estimated to have fallen in 1913, only 32 % of the total storage available was utilized for detention storage. Over the years, the system has provided substantial benefits to the valley although annual and aggregate benefits are not determined and published by the Conservancy District. Continued funding for the operation and maintenance of the facilities is derived from assessments of both private and public property owners. These assessments are based on depth of flooding that occurred during the 1913 flood and current property values [13].

5.2.9.7. Another Attempt: The Muskingum Watershed Conservancy District

The Muskingum Watershed Conservancy District was formed in June, 1933, also under the Ohio Conservancy Act, to develop a comprehensive flood control and water conservation plan for the Muskingum River Basin. The Muskingum River, a tributary of the Ohio River, drains an area of about 20 800 sq. kilometers, slightly more than twice the size of the Miami River Basin, in southeastern Ohio. The plan developed by the District, centered on the construction of 14 dams to provide flood protection to urban areas throughout the Basin. One important difference between the Miami River and Muskingum River plans was that rather than being locally funded, the Muskingum plan was to be a shared responsibility between the District and the Federal government. Construction began in 1934 and was completed in July of 1938, with the Conservancy District assuming all responsibility for operation and maintenance at that time. In August, 1939, the U.S. Congress passed legislation incorporating the 14 Muskingum dams into the larger Ohio River Basin flood control plan and directing the U.S. Army Corps of Engineers to assume all operation and maintenance responsibilities. Those responsibilities remain with the Corps of Engineers today.

5.2.9.8. Conclusions

The creation and continued operation of the Miami Conservancy District is one of the great engineering stories of our time. Established in the early part of the 20th Century, at a time when engineering data was scarce and technology was in its infancy, it is a tribute to the innovative abilities of the District staff. This staff was composed of many of the best engineering minds of the time. In developing and implementing the Plan for the Miami Basin, many new engineering analysis techniques were developed from the considerable R&D effort undertaken as part of the design process, including new innovative ways to gather and analyze engineering data. When construction of the plan was completed and the staff disbursed, these individuals provided the engineering talent for numerous Federal (e.g., the Tennessee Valley Authority) and non-Federal water resources developments throughout the U.S., based on their experience gained in the Miami Valley studies. It has been estimated by the District that the current replacement cost of the system would be in excess of $ 600 M. An undertaking of this magnitude by a non-Federal entity will probably never be attempted again in the U.S.

5.3. DAMS UNDER CONSTRUCTION, IN WHICH FLOOD MITIGATION HAS AN IMPORTANT ROLE

5.3.1. Three Gorges Dam, China

5.3.1.1. Introduction

Near the downstream end of the Upper Yangtze, in China, lie three reaches of scenic gorges with torrential flow. These are known as the Three Gorges. A gigantic water-control project will be built in the last reach of gorge and has been named the Three Gorges Project (TGP). The general location of this project is shown in Fig. 5.23.

TGP is a multi-purpose project. Its main purpose, however, is flood control, although with an installed capacity of 18 200 MW and a reservoir about 600 km long, the benefits in power generation and shipping are also tremendous. In fact, these side benefits are so impressive that many people, particularly those outside the circles of hydraulic engineers, have erroneously taken TGP for a power generation project (Table 5.15).

5.3.1.2. Flood Control

The Yangtze River where the TGP is located is known for frequent occurrences of large floods. According to Chinese archives kept for 2000 years before 1911, the Yangtze River gave rise to 214 major floods or about 1 major flood every 10 years. In the 450 years between 1499 and 1949, the Middle Yangtze, which flowed in a plain immediately downstream of the Three Gorges, breached the main levees every 2 to 3 years. Table 5.16 gives the discharge of the major historical floods passing through Yichang, at the exit of the Three Gorges. During the 82 years from 1788 to 1870, three large floods (shown in Table 5.16) were either close to or exceeded the

1 % flood of 86 300 m^3/s occurred. The 1860 and 1870 floods not only breached the levees and inundated up to 30 000 km^2, but they also changed the local topography permanently by carving out two new rivers in the right bank, each of a length on the order of 100 km and discharging into Lake Dongting.

Table 5.15
Main characteristics of the Three Gorges Project

DAM	
Type	Gravity
Height	175 meters
Elevation Top of Dam	185 m
Length	2309 m
Spillway Capacity	116 000 m^3/s
Installed Capacity	18 200 MW
Annual Output	84.7 TWh
Units	26 of 700 MW
Principal Shiplock	5 Levels/Two Way
Shiplift	1
Excavations (Rock and Earth)	147×10^6 m^3
Concrete	28.4×10^6 m^3
Reinforcement	354 300 t
Metallic Structures	281 000 t
Period of Construction	17 years
RESERVOIR	
Normal Operation Level	175 m
Storage Capacity	39 300 hm^3
Flood Control Level	145 m
Flood Volume Control	22 150 hm^3
Reservoir Surface	1084 km^2
Cultivated Areas	27 820 ha
Population to be Displaced	1 million persons
Basin Area	1×10^6 km^2
Mean Inflow	451 000 hm^3 (14 320m^3/s)
Mean Transported Sediments	530×10^6 t

Table 5.16
Leading historical floods at Yichang

Year	**Discharge at Yichang, m^3/s**
1153	92 800
1227	96 300
1560	93 600
1613	81 000
1788	86 000
1796	82 200
1860	92 500
1870	105 000

Stage records on the Yangtze River date back to 1865 and discharge has been measured since 1922. Table 5.17 gives the major floods of this century observed at Yichang and Table 5.18 gives the losses inflicted by 4 of these floods in the riparian plains of the Middle Yangtze.

Table 5.17
Leading floods of the 20th century at Yichang

Year	Discharge at Yichang, m^3/s
1931	64 600
1935	56 900
1949	58 100
1954	66 800
1981	70 800
1983	53 500
1998	63 600

Table 5.18
Flood losses in the Middle Yangtze

Year	1931	1935	1949	1954	1998
Farm flooded (km^2)	33 000	15 000	18 000	32 000	4000
People affected $\times 10^6$	28.9	10	8.1	18.9	14
Death toll $\times 10^3$	145	142	5.7	30	3

Table 5.18 shows that large areas and people by tens of millions were affected. Many people were killed directly by the flood. Many more were lost to disease and epidemics.

Since 1949, the levee system in the Yangtze valley has been considerably strengthened. Still, the capacity of the Middle Yangtze to pass floods is less than 60 000 m^3/s, even when discharge is bypassed to Lake Dongting by rivers in the right bank of the Yangtze. When the flood entering into the Middle Yangtze has a discharge between 60 000 and 80 000 m^3/s, then, in addition to bypassing, diversion areas are also used to reduce the peak of the flood to a magnitude that will pass through the Middle Yangtze without overtopping its levees. The diversion areas are, however, densely populated. To use these areas to store a portion of a flood would require rapid evacuation of a large number of people with short notice. For instance, Jingjiang Diversion Area is located near the upper end of the Middle Yangtze and is the only one equipped with a gated spillway for flood diversion. If this diversion area were committed for flood discharge, then 290 000 people would have to be evacuated in 1 day with about 2 days notice. Obviously this would be a difficult to accomplish. Even with the help of the Jingjiang Diversion Area, however, if the flood entering the Middle Yangtze should exceed 80 000 m^3/s, the reduced flood would still be too large to pass through the Middle Yangtze.

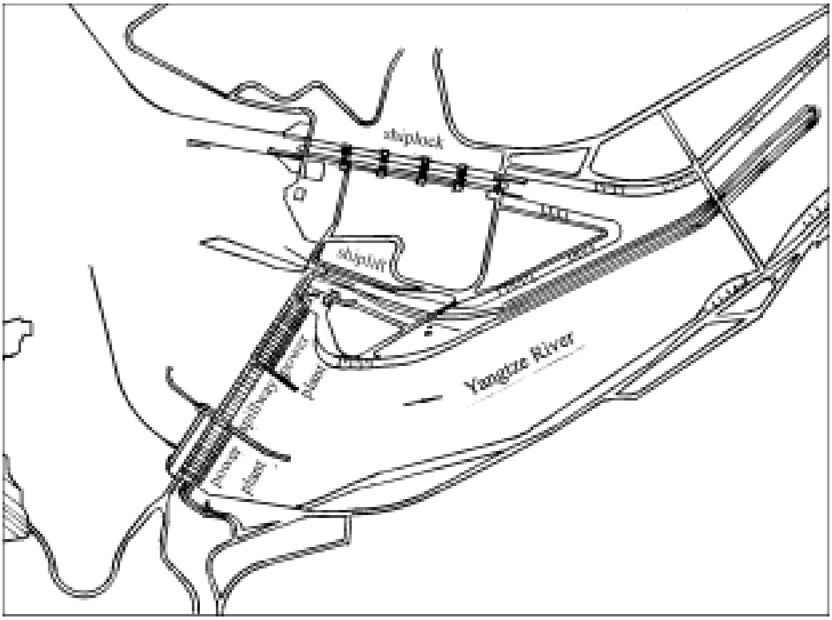


Fig. 5.23
General layout of Three Gorges Project (TGP)

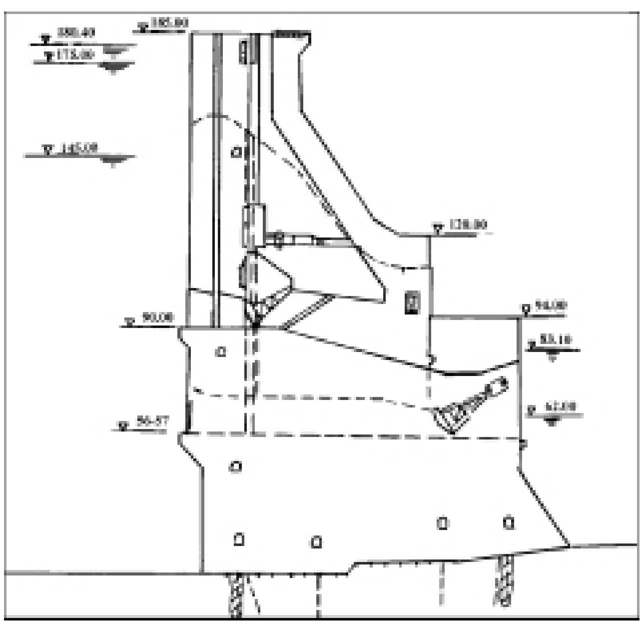


Fig. 5.24a
Cross section of spillway for TGP

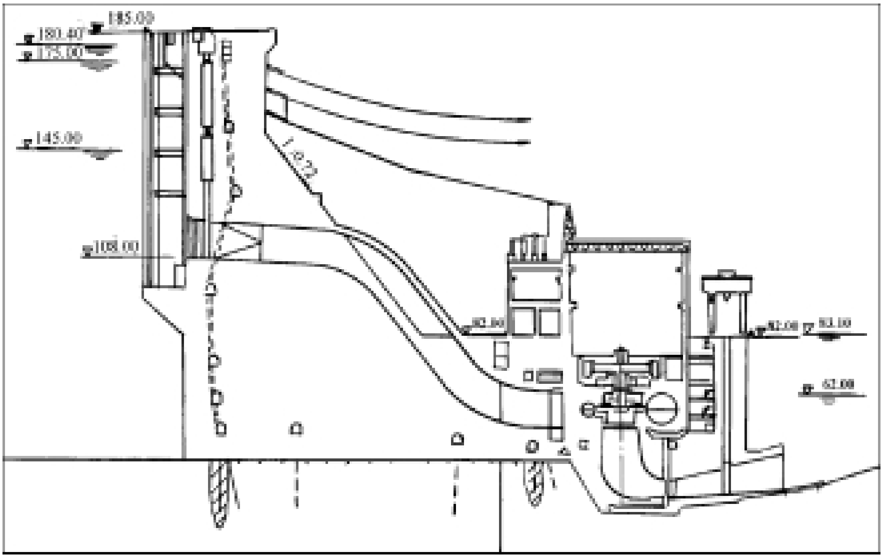


Fig. 5.24b
Power intake cross section

An upstream reach of the Middle Yangtze, about 320 km long and known as the Jingjiang River, is a bottleneck during flooding. Most of the Jingjiang River is lined with levees 10 to 16 m high. The maximum flood stages at some places may be as much as 10 to 14 m higher than the ground outside the levees. About 15 million people, 1.5 million ha of farmland, and many industrial cities are protected by the levees. If the levees breach, the water will inundate and devastate an extensive area. To prevent this catastrophe from happening, one needs not only good levees but also reservoir storage to regulate the flood flow. TGP has been planned to provide this vital storage needed.

TGP is to be located in the Xiling Gorge with dam site at Sandouping, which is 42 km upstream of the city of Yichang. The crest of the dam will be 2309 m long and will be set at elevation 185 m corresponding to a maximum dam height of 175 m. A reservoir will be constructed that will be 600 km long (Fig. 5.24a and b). Under the normal pool of 175 m, the total storage is 39.3 billion m^3 of which the storage for flood control is 22.1 billion m^3. This storage is sufficient to regulate a 1 % flood of 86 300 m^3 /s at Yichang, so that the reduced flow released from TGP reservoir may be safely conducted through the Jingjiang reach without enlisting any flood diversion area. Even if a 0.1 % flood of 105 000 m^3 /s at Yichang should occur, regulation with the TGP reservoir plus the storage by Flood Diversion Area could still reduce the flood discharge to a magnitude that may be safely passed through the Jingjiang reach.

The scheme of operation of the reservoir is based on lowering the water level in late May and early June to the level 145 m, and maintaining this level during the monsoon period until the end of September. In this way, an additional volume is available during the flood period to control floods of 22 150 hm^3, or 56.3 % of the reservoir capacity. During this period, retention will only occur when the intake flow is superior to the safety flow downstream. After the passage of the maximum flood flow, the level of the reservoir will drop again to the level of 145 m. With this operation, the level of protection at the Jingjiang reach, which as mentioned, presented greater risk, will pass down 10 years to the 100 year flood without the need to carry out controlled flood operations, and increase to the flood of 1000 years with the flood operations at Dongting Lake. Therefore, the construction of the TGP is essential for the protection of tens of millions of people downstream, particularly those living along the Jingjiang River. In fact, it is the flood control for people downstream that dictate the site of TGP in the neighbourhood of Yichang, for only a dam built here across the Yangtze may control the flood from the entire watershed of the Upper Yangtze.

5.3.2. Seven Oaks Dam Flood Control Project, California, USA [14]

5.3.2.1. Introduction

Seven Oaks Dam is a single purpose flood control project currently under construction by the U.S. Army Corps of Engineers, Los Angeles District. The project is located in a steep walled canyon in the headwaters of the Santa Ana River, and is part of a comprehensive flood damage reduction plan for the entire Santa Ana River Basin.

The Santa Ana River Basin drains approximately 6345 sq. kilometers, excluding an area of 83 sq. kilometers tributary to Baldwin Lake and 26 sq. kilometers tributary to Perris Reservoir. Of the total basin, 5850 sq. kilometers of the drainage area is upstream from Prado Dam, a major existing flood control structure on the Santa Ana River, located about sixty-five kilometers downstream of the Seven Oaks Dam site. Approximately 23 percent of the basin is within the San Gabriel and San Bernardino Mountains ; about 9 percent is in the San Jacinto Mountains; and 5 percent is within the Santa Ana Mountains. Most of the remaining area is in the valleys formed by the broad alluvial fan along the base of these mountains. The San Bernardino Mountains are the source of the Santa Ana River. The Santa Ana River has an average gradient of about 46 m/km in the mountains. A map of the basin is shown on Fig. 5.25.

The upper Santa Ana River drainage area above Seven Oaks Dam is nearly 460 sq. kilometers, excluding the 83 sq. kilometers tributary to Baldwin Lake, and has its headwaters in the San Bernardino Mountains. Elevations vary from 3 250 m at Anderson Peak, 3 505 m at San Gorgonio Peak, to 628 m at the dam site, which is located approximately 1.6 km upstream from the canyon mouth. The river upstream from the dam site has an average gradient of 57 m/km, however, smaller tributaries originating in the high mountains have gradients that exceed 360 m/km.

Urban development exists downstream from the canyon mouth, with the cities of East Highlands, Mentone and Red lands located less than 13 km away. Development within the canyon is limited to a hydropower system owned by the Southern California Edison Company, which was constructed at the turn of the

Fig. 5.25
Santa Ana River Basin

century, and to irrigation and water facilities primarily operated by the San Bernardino Valley Municipal Water District. A few scattered residential developments exist in the upper canyon area.

5.3.2.2. The Flood Problem

The Santa Ana River is currently uncontrolled for much of its length in San Bernardino and Riverside Counties down to Prado Dam. Prado Dam is the only flood control reservoir along the 105 km length of the Santa Ana River. The area upstream from Prado Dam is threatened by major storms. In Riverside County, floods in 1969 seriously damaged two sewage treatment plants, ruptured numerous sewage lines, washed out several bridges and bridge approaches, flooded large areas of agricultural land, and caused heavy bank erosion along most of the river. In San Bernardino County, widespread overflow occurred in much of the 100-year floodplain, levees were eroded, and county roads and utilities were damaged. Damages were also significant on several major tributaries to the Santa Ana River. Floods on Temescal Wash and San Timateo Creek covered residential, commercial, and agricultural property with mud and sand.

The flood problem on the Santa Ana River mainstem is a basin-wide problem. It is necessary to protect heavily developed communities downstream from Prado Dam and to provide protection for the rapidly growing upstream communities.

Studies concluded that the only acceptable flood control improvement plan to protect against the largest flooding considered reasonable to the area would involve :

a) Enlarging the existing channel downstream from Prado Dam.

b) Enlarging Prado Reservoir to the extent that resulting socioeconomic impacts are held to acceptable levels.

c) Providing additional flood storage upstream from Prado Dam through the construction of Seven Oaks Dam.

The comprehensive project includes parts of San Bernardino, Riverside and Orange Counties, and will provide additional flood protection for two million residents of southern California.

5.3.2.3. Seven Oaks Dam

Three alternatives were evaluated to select the dam type. The three alternatives evaluated for the dam were : (1) A Roller Compacted Concrete (RCC) dam, (2) an earth and rock fill dam, and (3) a concrete faced rock fill dam. Preliminary cost studies indicated that the three methods of construction would be competitive at this site with potential cost savings being attributed to the RCC dam.

Due to the site geology and the proximity of the dam to the San Andreas fault and other smaller faults and trace faults, a determination was made that the dam would need to be designed to accommodate 4 ft of displacement of the foundation rock at any location and in any direction. The choice between the RCC and an earth and rock fill dam was based on the degree of assurance of satisfactory performance during a major seismic event with a high reservoir pool. Given the displacement criteria, unprecedented defensive measures were incorporated in the proposed design of the RCC alternative. Further, the effectiveness of the defensive measures against foundation movements is untried and largely judgmental. In addition, there is no feasible measure known, which can be incorporated in the design to monitor the occurrence of random cracking within the structure. Hence, there is a lack of confidence in the performance of the RCC dam with a full reservoir and random cracking of the structure. Based on consideration of the postulated displacements, the unconventional defensive measures required for the RCC design, the lack of effective defensive measures for random cracking within the RCC dam, the earth and rock fill dam was selected to provide the flexibility and self-healing capability to safely accommodate the seismic motions and differential movements anticipated at the site. Whereas, the rigidity of the RCC or concrete-faced alternative could lead to cracking due to the postulated differential movement in the foundation, at the abutments, and within the concrete structure.

The embankment will be an earth and rock fill structure with a height of 167.6 m above the existing stream bed, crest width of 12 m, crest length of 802 m and crest elevation of 795.5 m. The upstream slope will be 1 vertical on 2.2 horizontal and the downstream slope will be 1 vertical on 1.8 horizontal. The zoned embankment will contain an impervious core, graded interior pervious layers and an exterior rock shell.

5.3.2.4. Hydrologic Design of the Spillway and Dam

Because of the limited number of potential locations for dam sites and the significant damage potential in the lower valley, a high degree of storage capacity was sought for this project. As such, the selected Reservoir Design Flood (RDF - the flood that fills the flood pool to the spillway crest elevation) was based primarily on economic criteria, i.e. downstream damage reduction. The following data compare the RDF and PMF peak inflow, volume and maximum pool reached during the routing with the selected operational plan. All reservoir routings were performed for a 4-day runoff hydrograph under expected future conditions. The starting water surface elevation was assumed at elevation 701.0 m (top of the future condition debris pool), which is 30.5 m higher than current conditions and accounts for sediment accumulation.

Flood Event	Peak Discharge	Inflow Volume	Max. Pool Elevation
RDF	2395 m^3/s	1.41×10^8 m^3	786.4 m
PMF	5100 m^3/s	4.39×10^8 m^3	794.0 m

Should the RDF occur, the estimated time to empty the reservoir would be about 15 days, assuming no additional storm inflow.

The spillway crest width and top of dam elevation were established as follows. A spillway width versus embankment height analysis was developed for the Seven Oaks Dam by evaluating three different types of rock cut emergency spillways: a broadcrested weir with (1) sloping upstream and downstream channels, (2) entirely horizontal and (3) horizontal with an ogee crest at the downstream end. Based on an economic analysis, the broadcrested weir with sloping upstream and downstream channels was selected. Further optimization analysis focusing only on the broadcrested spillway revealed that the most economical configuration would consist of a width of 152.4 m and have cut side slopes of 1H on 2V that included benches for geotechnical considerations. The overall slope averaged nearly 1V on 1H.

In determining the final top of dam elevation, the Probable Maximum Flood (PMF) was selected for design. Based on the governing engineering regulations, the 72-hour storm basin-average rainfall total was about 1200 mm. Runoff was estimated to be nearly 935 mm of rainfall. This runoff translates into a flood volume of almost 4.40×10^8 m^3 from the drainage area of 458 sq kilometers above the dam site. Based on these calculations, the PMF runoff volume is over 3 times the RDF volume.

Reservoir routings were performed through the facility under different scenarios. Three freeboard scenarios were evaluated :

1. Routing the PMF with the starting pool level at spillway crest (786.4 m) plus 0.92 m of freeboard.
2. Routing the PMF with the starting pool level at spillway crest (786.4 m) plus wave runup plus wind setup
3. Routing the PMF with the starting pool level at the elevation that utilizes 50 percent of the flood control storage plus 1.5 m of freeboard.

At Seven Oaks Dam, Case 3 governed, which resulted in the top of dam elevation established at 795.5 m.

When completed and placed in operation, Seven Oaks Dam will be operated in conjunction with the downstream Prado Dam. During the early part of the winter flood season, runoff will be stored behind the Seven Oaks Dam to create a debris pool to protect the outlet works. Small releases will be made on a continual basis in order to meet downstream minimum release requirements. During a flood, Seven Oaks Dam will store water as long the Prado reservoir pool is rising. When the downstream flood threat has passed, Seven Oaks Dam will be operated to release stored water at a rate that does not exceed downstream channel capacity. At the end of each flood season, the reservoir at Seven Oaks will gradually be drained and the Santa Ana River will flow through the project unregulated.

The project is currently under construction. The outlet works and diversion tunnel are complete. The embankment and spillway are nearly complete. Construction will take approximately four and one-half years with completion of the dam and major features scheduled in 1999. A plan of the dam is shown in Fig. 5.26.

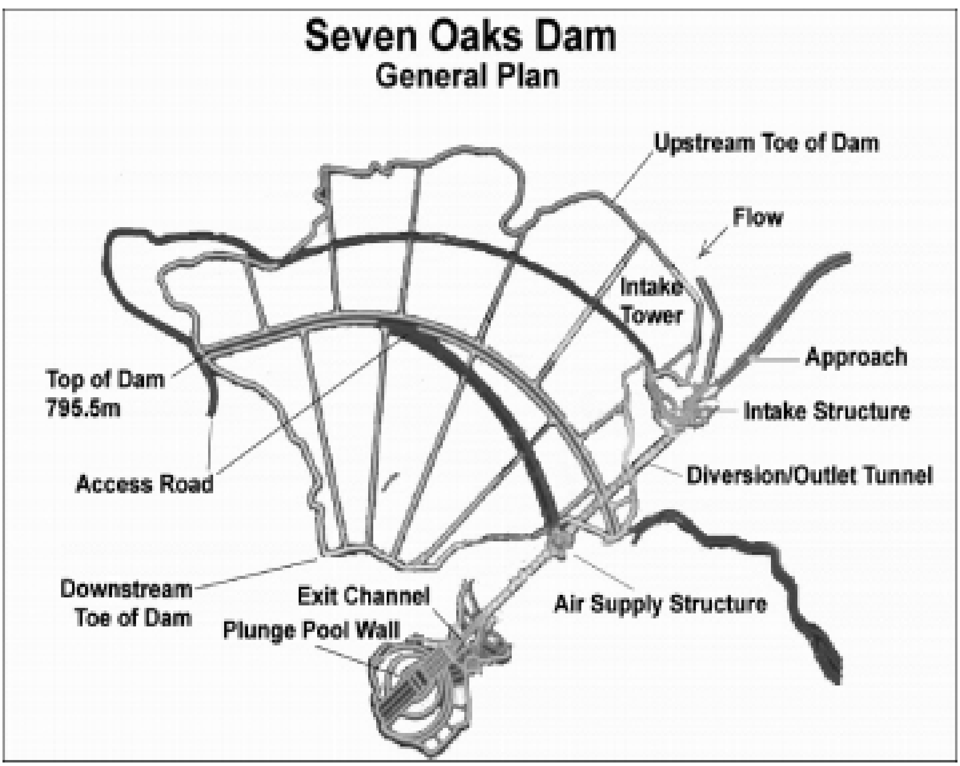


Fig. 5.26
Seven Oaks Dam

5.3.3. Portugues and Bucana Rivers, Flood Control Project, Puerto Rico [15]

5.3.3.1. Introduction

The Portugues and Bucana (P&B) Rivers originate along the southern slopes of the Cordillera Central Mountains and flow to the Caribbean Sea. The Portugues River drainage area is 58.5 sq. kilometers while the Bucana River drainage area is 81.3 square kilometers. Ponce, the second largest city in Puerto Rico, is located in the lower coastal area of the two basins. A map of the project area is shown in Fig. 5.27.

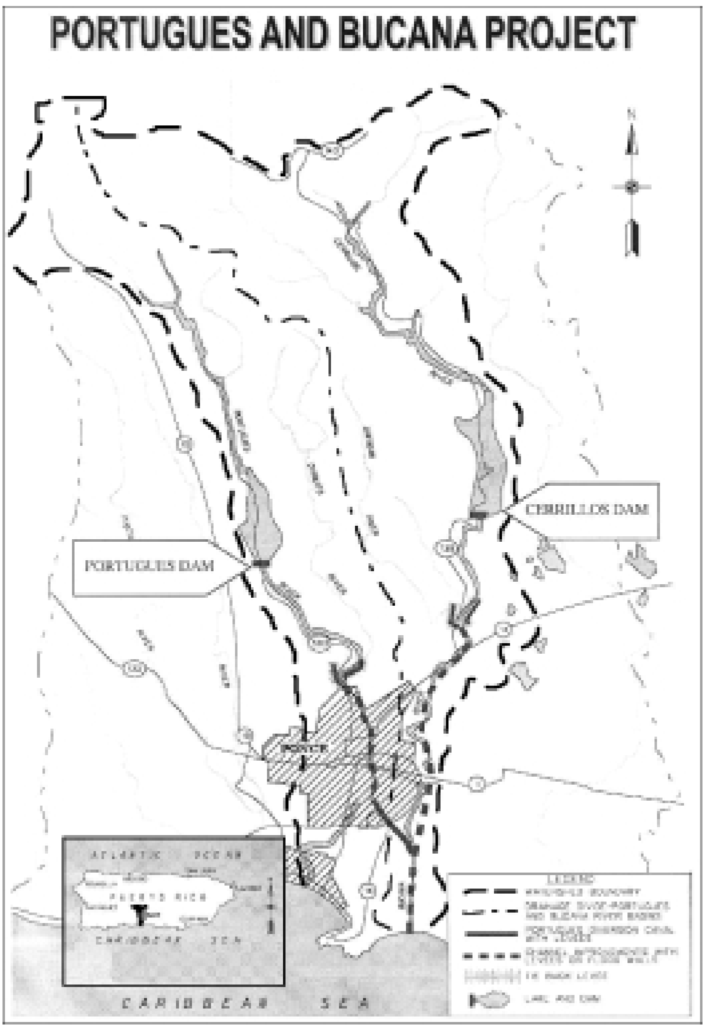


Fig. 5.27
Portugues and Bucana project area

The P&B River Basins are among the steepest on the island and are mountainous to within 6.5 km of the coast. The highest point in Puerto Rico, Cerro de Punta (elevation 1340 m) is located on the central ridge in the upper Bucana River Basin, only 22.5 km from the coast. The P&B Rivers are very steep in the upper part of the basins, with slopes in excess of 140 m/km at their sources and average about 55 m/km. Because of the steep slopes, few of the upper watershed areas have been cleared and are covered mostly with tropical vegetation. Some lands along the streams have been cleared for agricultural uses. The coastal area appears flat when contrasted with the mountainous background, but it has slopes of about 4.8 m/km. Much of the low coastal area is now being developed for industry and residential homes. Large tracts of land that once were low are now being raised by landfill for protection against flooding. During large floods, the P&B Rivers form a common inundated area in the lower 5 km of the coastal plane.

Rainfall records for Ponce began in 1899. The mean annual rainfall for the area averages about 840 mm and ranges from 600 to 1800 mm. About 52 days per year (usually non-consecutive) have an average precipitation of 0.25 mm or more at Ponce. Maximum months of rain are May through November or December. Peak monthly values of 400 to 425 mm have occurred during these months.

The steep slopes of the mountains and channel gradients of the upper reaches of the P&B Rivers are conducive to very flashy storm runoff. In the upper reaches, runoff is very rapid with measured velocities of 3.3 m/s. Channel velocities, downstream, in the Ponce area have been measured as high as 2.4 m/s.

5.3.3.2. The Flood Control Project

The P&B Rivers flow across densely developed sections of the city of Ponce. Because of their inadequate channel capacity, the area is subject to severe flooding in the low regions causing considerable damage to property with risk of loss of lives. The channelization of these rivers along with the Portugues and Cerrillos multi-purpose reservoirs will provide significant protection to the city from floods.

Prior to the initiation of project construction, flooding occurred almost annually. Major floods occurred in 1954, 1961, 1970, 1975, 1985, and 1992. The value of property subject to flooding exceeds $ 0.6 billion. The total project involves construction of 15.3 km of channel improvements and two multi-purpose dams with uncontrolled emergency spillways. The project will provide flood protection, a dependable water supply for the Ponce area, and recreational facilities on the lakes and channels.

5.3.3.3. Cerrillos and Portugues Dams

The Cerrillos Dam, controlling runoff from an area of 45 square kilometers, is located on the Cerrillos (Upper Bucana) River, 14.6 km above its mouth. The Cerrillos Dam is 98 m high. Its reservoir will provide 59.1×10^6 cubic meters of flood control and water supply storage. The Portugues Dam, controlling runoff from an area of 27 sq. kilometers, will be located on the Portugues River, 13.4 km above its mouth. The Portugues Dam will be 83 m high. Construction will be staged in two phases. Phase one will be for flood control only with water supply to be added later.

When completed, the reservoir will provide 29.9 × 10^6 cubic meters of flood control and water supply storage.

The Cerrillos Project (Fig. 5.28) consists of a rockfill dam with outlet works consisting of an inclined intake structure, a regulatory outlet tunnel, and a stilling basin located in the left abutment, and a spillway excavated in the ridge adjacent to the right abutment. The main embankment is a 98 meter high zoned rockfill embankment consisting of a central clay core flanked by a two-stage filter, which is supported by zoned rock shells. The embankment's axis is approximately 475 meters long and arched upstream. The crest elevation of the dam is 195 meters, and it is 10 meters wide along its entire length. The upstream and downstream sideslopes are 1V on 2.2H and 1V on 2.0H, respectively. A 32-meter high cofferdam constructed within the upstream slope of the main dam to protect downstream areas during construction of the main dam, serves as an integral part of the main dam. Riprap is provided on the upstream slope for protection against wave action. A triple line grout curtain is provided beneath the embankment along the centerline axis.

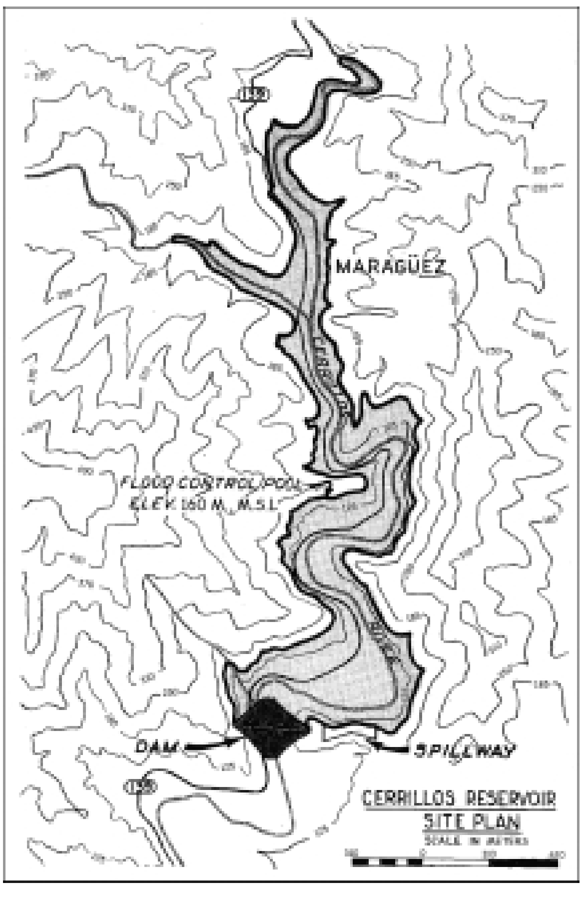


Fig. 5.28
Cerrillos Dam

Hydrologic analysis of the Cerrillos Dam contributing watershed was conducted using long-term local rain gage records and synthetic unit hydrograph methods. The reservoir design flood produced an inflow peak of 1342 cubic meters/second and a volume of 29.1×10^6 cubic meters. The uncontrolled emergency spillway was designed, along with reservoir storage, to discharge 430 cubic meters/second during this event. The spillway design storm has a peak inflow discharge of 2573 cubic meters/second and a volume of 57.9×10^6 cubic meters. Emergency spillway discharge is 2132 cubic meters/second during this event and was routed through the reservoir with a starting water surface elevation equal to the emergency spillway crest. From a total reservoir storage of 57.7×10^6 cubic meters, flood control storage represents 34 percent, water supply storage is 54 percent, and environmental and sediment storage is 12 percent.

The Portugues Project (Fig. 5.29) will consist of a double curvature concrete arch dam, a regulating outlet works through the dam and an uncontrolled emergency spillway in the dam. The dam will be 83 meters high from base to crest. The spillway crest will be 46 meters in length at an elevation of 174 meters.

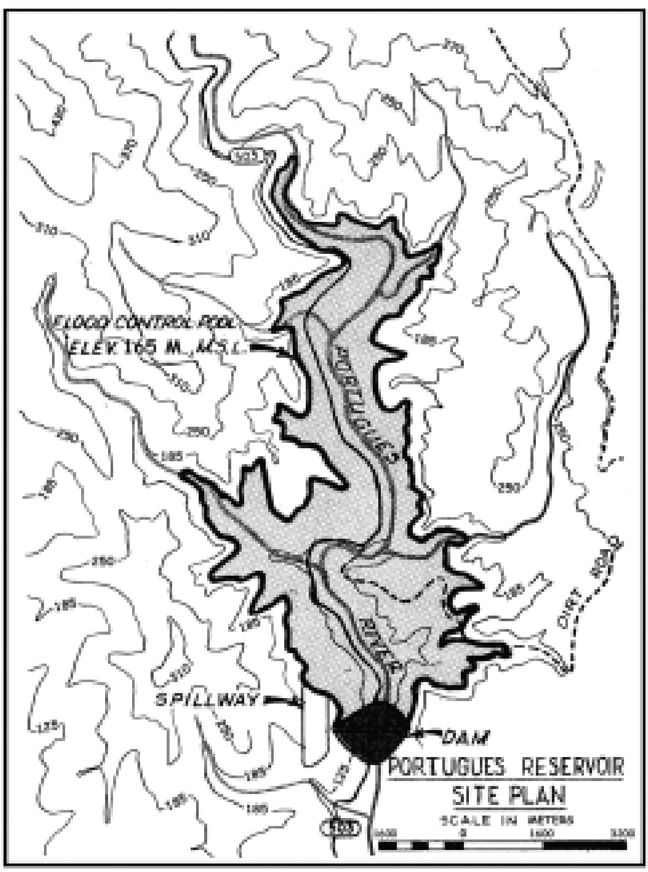


Fig. 5.29
Portugues Dam

The Portugues Dam's hydrologic analysis was conducted using long-term local rain gage records and synthetic unit hydrograph methods. The reservoir design flood produced an inflow peak of 750 cubic meters/second and a volume of 17.1×10^6 cubic meters. The uncontrolled emergency spillway was designed, along with reservoir storage, to discharge 442 cubic meters/second during this event. The spillway design storm has a peak inflow discharge of 1441 cubic meters/second and a volume of 34.3×10^6 cubic meters. Emergency spillway discharge is 1371 cubic meters/second during this event and was routed through the reservoir with a starting water surface elevation equal to the emergency spillway crest. From a total reservoir storage of 31.1×10^6 cubic meters, flood control storage represents 33 percent, water supply storage is 56 percent, and environmental and sediment storage is 11 percent. The U.S. Army Corps of Engineers authorized the P&B Project for construction in December 1970, at a cost of $ 559 600 000. Construction on some of the channel portions of the project began in 1975. Work on the Bucana Channel and most of the Portugues Channel has been completed. The Cerrillos Dam was topped out 30 August 1990 and the dedication ceremony held 16 April 1991. Filling began in August 1992. The Portugues Dam construction agreement with the Commonwealth of Puerto Rico was executed in August 1993. Abutment and foundation preparation for Portugues Dam are presently underway.

The current plan of operation for the Cerrillos Reservoir provides for maintaining a normal pool elevation of 175 meters. Water stored above elevation 175 meters, encroaches upon the flood control pool. Therefore, any water above elevation 175 will be released as soon as practicable. Flood control storage space is reserved between elevations 175 (top of conservation pool) and 186 meters, (top of flood control pool) with surcharge storage provided above the crest of the uncontrolled emergency spillway (elevation 186 meters). Conservation storage between elevations 137 and 175 meters is reserved for water supply and low-flow water quality releases. The project design for water supply allows a continuous maximum withdrawal of about 1.0 cm/s with a 95 percent dependable yield level. The overall plan for water control will require amendments in the event a hydropower plant is added in the future.

The Water Control Manual dated July 1997 calls for flood control releases to be made when the Cerrillos pool elevation is above 175 meters. According to the Water Control Manual, "Post flood evacuation is to be accomplished as soon as possible by releasing flows that produce non-damaging stages in the downstream reaches of the river. Because of the distance from the dam to Ponce and the amount of uncontrolled drainage area between Ponce and the dam, releases from the Cerrillos Dam will sometimes be terminated at the beginning of a storm to prevent discharges from contributing to uncontrolled floodwaters downstream."

5.3.3.4. Hurricane Hortense, 1996

On 10 September 1996, Hurricane Hortense battered Puerto Rico with torrential rains and strong winds, resulting in major flooding. During this hurricane, nearly 430 mm of rainfall was recorded at one location above the Cerrillos Dam. While only 107 mm of rainfall was recorded at the damsite, contributing flows upstream raised the stage behind the dam approximately 4 meters in a 24-hour period. Without the partially completed P&B Project with Cerrillos Dam and the channel improvements in place, it is expected that catastrophic flooding would have occurred in the Ponce region. Damages prevented by these works are estimated to be $ 132 000 000. This damage prevention is attributed to the essentially complete Cerrillos Dam and constructed channel works.

5.3.3.5. Hurricane Georges, September 1998

Background of Hurricane Georges - Puerto Rico & Virgin Islands

Hurricane Georges moved across Puerto Rico during 21-22 September 1998. According to the NWS' San Juan "Hurricane Georges Preliminary Storm Report", Hurricane Georges was the most destructive hurricane to strike the entire island of Puerto Rico since Hurricane San Ciprian in 1932. Georges tracked across the U.S. Virgin Islands, Vieques, and Culebra, but unlike its predecessors, Hugo and Marilyn, its impact in these smaller islands was not as severe as in Puerto Rico. Georges tracked across the entire length of Puerto Rico from east to west following the spine of the Central Mountain Range. Before reaching Puerto Rico, Hurricane Georges had been reclassified from a Category 4 storm to a Category 3 to a Category 2 and then back to a Category 3 storm. Hurricane Georges made landfall at around 7 pm AST on Monday September 21st in the vicinity of Yabucoa-Humacao in southeast Puerto Rico. During its track across Puerto Rico, Hurricane Georges indiscriminately lashed furiously at the island's major population centers and rural areas with estimated maximum sustained winds of up to 185 kph and gusts of 240 kph. With an eye of 40 to 50 kilometers in diameter, no part of the island remained unscathed by the fury of Hurricane Georges. Hurricane Georges moved generally westward across Puerto Rico at a speed of 25 kph. By early morning, Tuesday, September 22nd, the center of the eye of Hurricane Georges had passed over the island of Puerto Rico and was headed for the U.S. mainland.

Antecedent rainfall during the prior weeks resulted in highly saturated soil conditions prior to peak rainfall intensity. The two-day rainfall totals of 560 mm and 460 mm were recorded between 21 and 22 September, respectively, at the USGS recording precipitation gages at Cerrillos Dam and in the upper Portugues basin. A short-term rainfall-recording gage at Tibes recorded the two-day rainfall total at 470 mm, of which 465 mm fell in a 24-hour period. The National Weather Service Report stated that their cooperative network observers reported a 2-day total rainfall of 350 mm at Ponce, 440 mm at Juana Diaz, and 770 mm at Jayuya. The NWS reported that Ponce Airport recorded a peak sustained wind of 120 kph, with gusts up to 155 kph.

Based on a 27-year record at the nearest upper basin gage at Corral Viejo, the 24-hour rainfall return period at Tibes was approximately a 70-year. The intermediate 6-hour and 12-hour durations remained consistent at about a 50-year return period.

Statistics for P&B Project

The hydrology for Cerrillos Dam was revised in Feature Design Memorandum 17, Cerrillos Dam and Spillway, dated 1983. The reservoir elevation for the 10-year flood was estimated as 179 meters; the 50-year lake elevation as 182 meters; and the 100-

year lake elevation as 183 meters. The Cerrillos Reservoir contains 20×10^6 m^3 of allocated flood control storage, which is equivalent to 440 mm of runoff over the watershed. The crest of emergency spillway is set at 175 meters. It is estimated that about a 200-year flood would produce flow over the emergency spillway. The reservoir design routings assumed inflows would occur with a starting lake elevation of 175 meters and that the regulating outlets would be inoperative (*i.e.* closed) during the routing (reservoir design hydrograph was 24 hour duration). According to the Water Control Manual, evacuation of floodwater stored in the flood control pool is to be accomplished as soon as possible by releasing flows that produce non-damaging stages downstream. The reservoir design inflow hydrograph volume was 29×10^6 m^3. The 100-year inflow hydrograph volume is 15×10^6 m^3. The Cerrillos Reservoir level was just below 175 meters when the Hurricane Georges flood occurred. The lake level peaked at 184 meters on 22 September 1998, which was the highest stage recorded to date. At that time, 78 percent of the flood control space was filled as can be seen in the table below. Water Management has performed flood routings and has estimated that the reservoir stage would have risen to about 185 meters if the regulating outlets had not been operated on 22 September. The USGS reported a peak flow of 460 cm at the Cerrillos gage upstream of Cerrrillos Reservoir.

Cerrillos Reservoir : FY-98 Peak Stage due to Hurricane Georges

	Stage (meters)	Storage (m^3)	% Flood Control Space Filled
Top of Flood Control Pool	186	58×10^6	100
Peak Stage (22 Sept.)	184	53×10^6	78
Top of Conservation Pool	175	38×10^6	

Flood damages prevented by the P&B Project due to the effects of Hurricane Georges are estimated to be approximately $ 318 689 300. This number is significant considering that it is more than twice as large as the historical damages prevented by the project to date. No residual damage was recorded. In addition to flood damages prevented, loss of income that would have resulted to the downtown business district of Ponce in the absence of a project is estimated to be in excess of $ 28 000 000.

1. Portugues side. The peak discharge on the Portugues side is estimated to have been about a 1 in 50-year event. This was consistent with the hydraulic rating curve at the drop structure and the fact that the existing channels without the Portugues Dam are estimated to have about a 35-year capacity (without encroaching into project freeboard) ; damage occurred in the upper reach of the Portugues channel during Hurricane Georges. Therefore, the Portugues side of the P&B Project is estimated to have experienced a 1 in 50-year flood due to Hurricane Georges.

2. Cerrillos side. On the Cerrillos side, in the Upper Basin, rainfall was likely in excess of the 100-year storm although available data is limited. Based on flood storage in the Cerrillos Reservoir, it is estimated that a 170-year storm runoff was experienced. Therefore, the Cerrillos side of the P&B Project is estimated to have experienced a 170-year flood due to Hurricane Georges.

5.3.4. Kitakami River, Japan

5.3.4.1. Introduction

Running through Miyagi and Iwate prefectures in the Tohoku District, the Kitakami River has a main river channel of 249 km and a catchment area of 10 150 km^2 (Fig. 5.30). The major floods in recent years are shown in Table 5.19. Since Kitakami River has a bottleneck in Ichinoseki area, river improvement works have been made from old times aiming at flood prevention. The river development plan of Kitakami River was formulated in 1941 for the purpose of flood control and power generation, and five dams (Tase, Ishibuchi, Yuda, Shijushida, and Gosho Dams) have been completed. Moreover, the construction of Isawa Dam is ongoing in the downstream of the Ishibuchi Dam, aiming at flood control and water supply (Fig. 5.31).

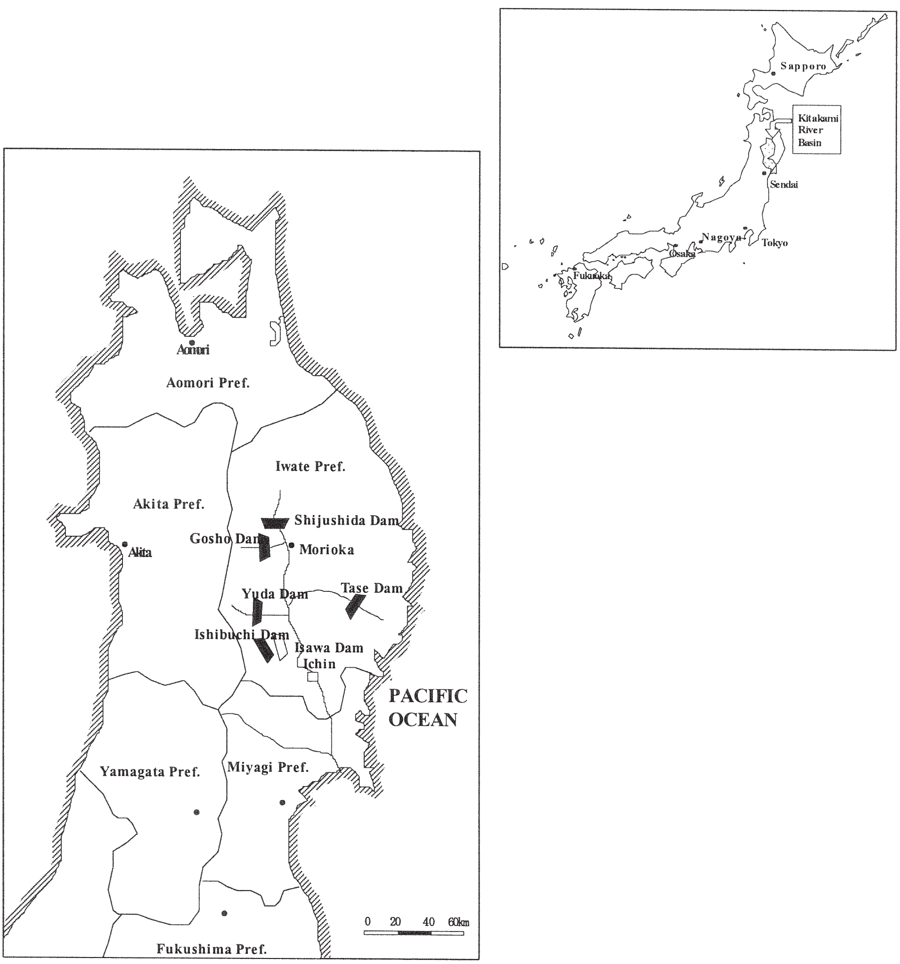


Fig. 5.30
Location of Kitakami River Basin and Dam Site

Table 5.19
Major Floods in Recent Years

Date	Peak discharge at Kozenji (Ichinoseki) (m^3/s)	2-day mean rainfall (mm)	Cause of rainfall
Sep. 1947	7900	183	Typhoon
Sep. 1948	5650	148	Typhoon
Jun. 1955	3908	105	Rain front
Sep. 1958	4079	132	Typhoon
Aug. 1979	4189	133	Rain front
Aug. 1981	5412	148	Typhoon
Aug. 1986	3043	131	Typhoon
Aug. 1987	4346	161	Rain front
Aug. 1988	3681	126	Rain front
Sep. 1990	4203	127	Typhoon

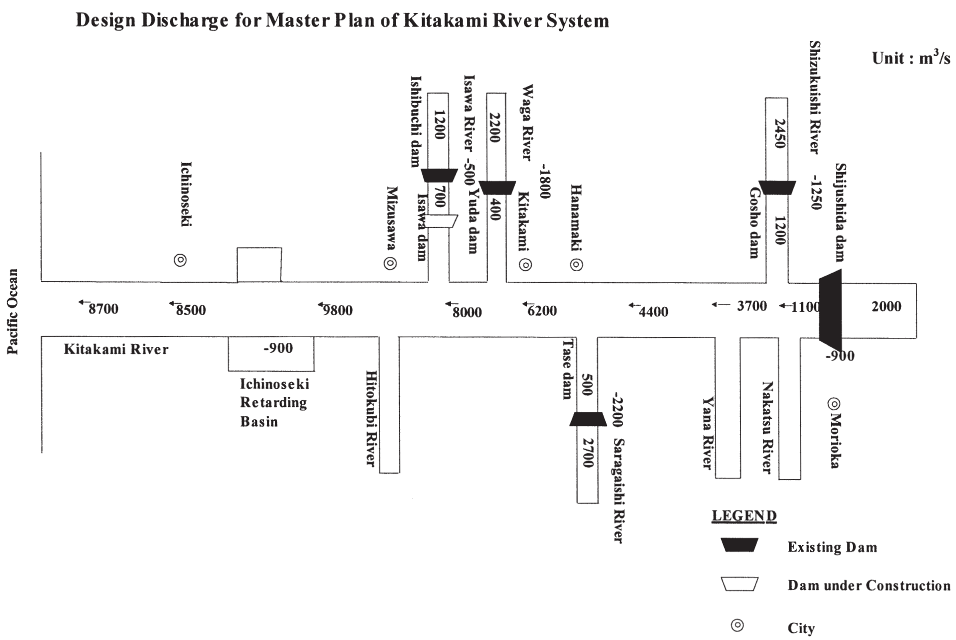


Fig. 5.31
Master Plan of Kitakami River

5.3.4.2. The Flood Control Plan

The design discharge for the Master Plan of the Kitakami River with the safety degree of 150-year return period is also indicated in Fig. 5.31. The project flood discharge at Ichinoseki Point is 13 000 m^3/s. Out of the project flood discharge, five dams will regulate the discharge of 4500 m^3/s and river channel will confine the remaining of 8500 m^3/s, which corresponds to 20-year return period. The total flood

Table 5.20
Features of Dams/Reservoirs in Upper Kitakami River

Name of Dam	Completion Year	Catchment Area (km^2)	Dam Height (m)	Reservoir Capacity (million m^3)	Flood Control Capacity (million m^3)
Shijushida	1968	1196	50.0	47.1	33.9
Gosho	1981	635	52.5	65.0	40.0
Tase	1954	740	81.5	146.5	84.5
Yuda	1964	583	89.5	114.2	77.8
Ishibuchi	1953	154	53.0	16.2	5.6
Isawa	Under Construction	185	132.0	143.0	51.0

Table 5.21
Flood Control Effect of Dam Operation

Name of Dam	Shijushida	Gosho	Tase	Yuda	Ishibuchi
Date	Aug. 23, 1981	ditto	ditto	ditto	ditto
Cause of Flood	Typhoon No. 15	ditto	ditto	ditto	ditto
Total Rainfall for One Event	200 mm	347 mm	119 mm	158 mm	265 mm
Maximum Daily Rainfall	118 mm	303 mm	69 mm	113 mm	191 mm
Maximum Hourly Rainfall	17 mm	29 mm	16 mm	23 mm	47 mm
Peak Inflow Discharge	729 m^3/s	1686 m^3/s	1587 m^3/s	1003 m^3/s	812 m^3/s
Outflow Discharge at Peak Inflow	449 m^3/s	950 m^3/s	333 m^3/s	394 m^3/s	542 m^3/s
Total Inflow Discharge	62.5 million m^3	96.5 million m^3	89.3 million m^3	33.8 million m^3	27.1 million m^3
Flood Control Effect Peak Discharge Reduction	280 m^3/s (38 %) Cut	736 m^3/s (44 %) Cut	1254 m^3/s (79 %) Cut	609 m^3/s (61%) Cut	270 m^3/s (33%) Cut
Total Regulated Discharge	6.2 million m^3 (10 %)	13.8 million m^3 (14 %)	32.5 million m^3 (36%)	24.6 million m^3 (73 %)	1.5 million m^3 (5 %)

control capacity of the five dams is 242 million m^3 at present and will be increased by 51 million m^3 after completion of Isawa Dam under construction as tabulated in Table 5.20.

5.3.4.3. Performance of Flood Control

The 1981 flood was caused by a typhoon, and is one of the biggest floods experienced in the Kitakami River in recent years. During this flood, 5 % to 73 % of total inflow discharge was regulated by the respective dams as indicated in Table 5.21. The Tase Dam, which has the largest reservoir capacity as well as the greatest flood control capacity among the existing dams (Table 5.22), greatly contributed to flood control of the Kitakami River. The dam operation of the Tase Dam during the 1981 flood is shown in Fig. 5.32. The peak inflow discharge of 1587 m^3/s was cut as low as 333 m^3/s (79 % cut) and total regulated discharge reached 36 % of the total inflow discharge of 32.5 million m^3.

Integrated dam management is carried out by The Ministry of Construction. The office, located in Morioka City, receives hydrological information from upper Kitakami River basin and has effectively controlled floods with the five dams since 1975.

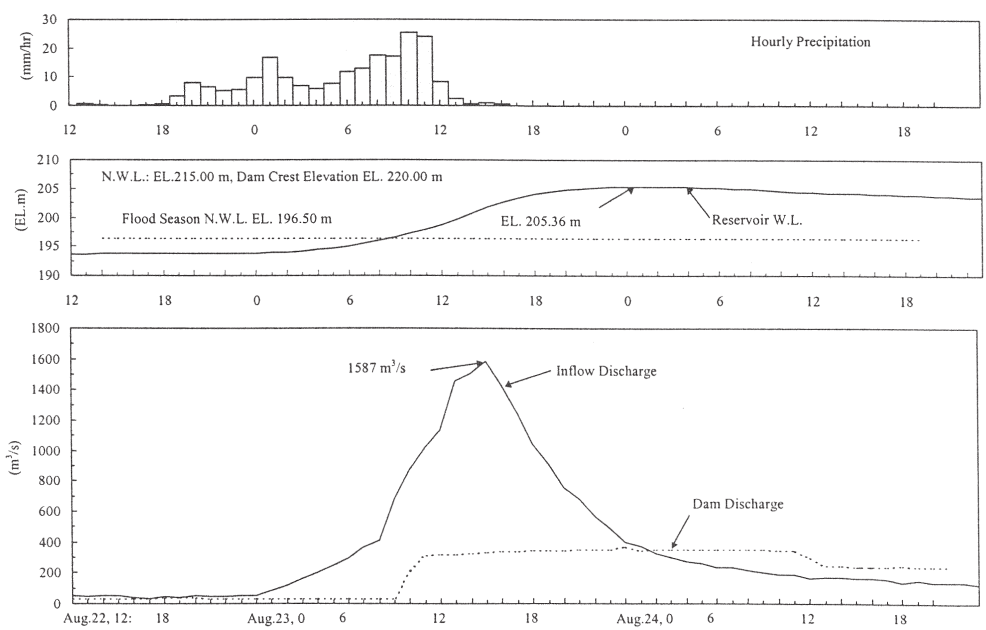


Fig. 5.32
Tase Dam Operation (Aug.22-Aug.24 1981, Typhoon No.15)

Table 5.22
Features of the Tase Dam/Reservoir

Name of Dam	Tase Dam
River	Kitakami River
Country	Japan
Date of Construction	1954 (Completion)
Type of Dam	Concrete Gravity
Height of Dam	81.5 m
Crest Length of Dam	320 m
Capacity of the Reservoir	146.5 million m^3
Flood Control Capacity	84.5 million m^3
Reservoir Area	6.0 km^2
Catchment Area	740 km^2
Average Annual Precipitation	1249 mm
Annual Mean Flow	20.88 m^3/s
Project Flood Discharge For Downstream Areas	2700 m^3/s
Design Discharge of the Spillway	3000 m^3/s

5.3.5. Segura and Jucar Basins, Spain [8, 16]

5.3.5.1. Introduction

After catastrophic floods occurred in 1982 and 1983, the National Commission of Civil Protection carried out a series of studies and planning in all the basins to determine preventive and corrective measures that would diminish the substantial damages produced from the floods. The results of these studies demonstrated that from a total of 1037 potential risk zones, the consideration of risk mitigation reservoirs was necessary in 228 zones, or in about 22 % of the cases. Those results gave rise to the recommendation of carrying out more detailed studies of 323 possible flood mitigation dams, which included 22 cases of raising the dam height (Table 5.23).

Floods during the 1980s produced substantial impacts and damages, and a National Defense Plan against flooding was initiated (1987 – 1992) in which more than 1000 measures were contemplated in high risk zones and an investment of some 1.2 M$. As a result of this Plan, flood mitigation has been one of the principal objectives of the majority of dams constructed during the last decade, either as an important priority in the multi-purpose dams, or as the only objective. Within this “National Flood Protection Plan” there are two specific plans for Basins of the Mediterranean Area : “The Flood protection Plan of the River Segura” and “The Flood Protection Plan of the River Jucar”.

Table 5.23
Number of flood mitigation dams studied. Risk zones

BASIN	MAXIMUM	INTERMEDIATE	MINIMUM	TOTAL
North	22	30 (3)*	30 (2)	82 (5)
Duero	--	8	10	18
Tajo	--	--	2	2
Guadiana	3	2	16 (1)	21 (1)
Guadalquivir	--	6	16	22
South	8	4 (1)	11	23 (1)
Segura	2 (1)	7	4 (2)	13 (3)
Jucar	2	2 (1)	4 (1)	8 (2)
Ebro	15 (1)	24	39 (5)	78 (6)
Eastern Pyrenees	6	21	29 (4)	56 (4)
TOTAL	58 (2)	104 (5)	161 (15)	323 (22)
*( ) Dam heightening				

5.3.5.2. Segura River Flood Plan

The flood protection plan of the River Segura is being developed for the Basin of the Segura River, of about 19 000 km^2. The middle and lower valleys of the basin have ample zones dedicated to the cultivation of fruits and crops of high profitability, and which constitute the main base for the economic development in the area. The plan presents a global concept of the basin and has as its principal objective the protection against floods with a 50-year return period, with which the flow in the main river is reduced from 2000 m^3/s to 400 m^3/s, which is the maximum capacity Segura River in its lower stretch.

An essential part of this plan is the construction of 13 flood mitigation dams in almost all the important tributaries, in small area basins (between 100 and 300 km^2); in those which elevated peak flows due to the torrential “Mediterranean” rains, but in short times, generally not longer than two hours, and therefore with relatively reduced volumes, for which the floods mitigation can be carried out with dams of small capacity. Of the 13 dams, 10 are of concrete, and the remaining three are embankment dams. In eight cases, the dams are new, and five older dams have been heightened. The height of the dams varies between 26 and 80 m. The dam capacities range between 2.5 and 50 hm^3, with a total joint capacity (of the 13 reservoirs) of 178 hm^3. At the present time, 7 dams are in operation, and 6 are in construction, the completion of the dams referred to being foreseen in the year 2000.

The investment foreseen of the whole of the Plan (13 flood mitigation dams and 10 channel systems) is of 500 M$, and the benefits of the operational working life are about four times the investment.

5.3.5.3. Jucar River Flood Control Plan

The flood protection plan of the Jucar River is based on three plans already constructed on Tous, Escalona and Bellus. The Tous dam, located on the Jucar River, is a rockfill dam with a height of 135 m, and has a total capacity at the extraordinary maximum level of 792 hm^3 (for extreme flood situations). The normal operating level of the reservoir is 379 hm^3, which is at the level of the spillway crest. This normal operating level leaves a capacity of about 410 hm^3 for flood routing. Additionally, during the months of September, October and November, reservoir volumes are available in capacities 50 %, 80 % and 80 %, respectively.

The Escalona Dam, located on the river of same name, is a gravity dam of 80 meters. in height, with a reservoir capacity 142 hm^3 The major purpose for the Escalona Dam is for flood routing. The Bellus Dam, situated on the Albaida River, is also a gravity dam, of 46 m in height and forms a reservoir of about 128 hm^3, of which 69 hm^3 is the maximum normal reservoir level.

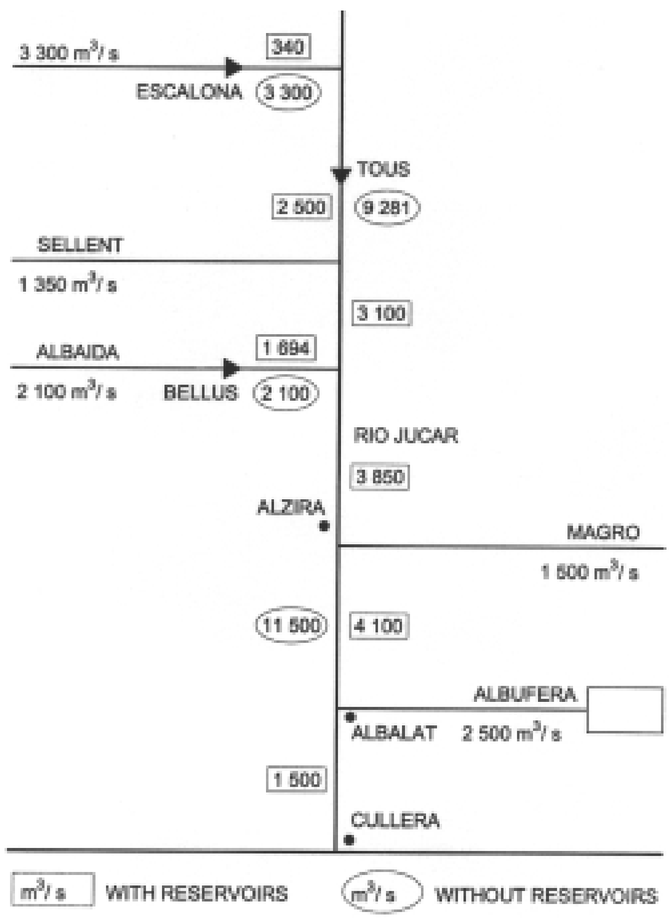


Fig. 5.33
Flood routing in Jucar river flood plan, for the flood of 500 years return period, with and without reservoirs

The principal objective of this plan is to avoid the inundation of townships for the 500-year flood and to limit the volumes of the floods circulating by the Jucar River. Because of these dams, peak volumes for the 500-year flood are significantly reduced. This reduction in volumes is shown in Fig. 5.33, which compares the discharge estimates for a 500-year flood with and without the dams and reservoirs. Numbers enclosed in circles in Fig. 5.33 represent discharge estimates without the dams and reservoirs, and numbers enclosed in rectangles represent the discharge estimates with the dams and reservoirs from the plan. These estimates show that a flood similar to the 1982 flood would be reduced by about two thirds, which means the townships would not be flooded. The total investment of the Plan, up to the present time, has been about 500 M$.

5.4. FUTURE PROJECTS IN FLOOD MITIGATION DAMS

Of the 25 410 large dams in operation, 18 000 (70 %) are single purpose dams, and the remaining 7400 (30 %) are multipurpose dams (according to the World Register of Dams of ICOLD, 1998 [17]. Large Dams are defined as dams higher than 15 m or dams between 5 and 15 m that impound more than 3 hm^3. Among the single purpose dams, 8 % are Flood Control Dams. About 40 % of the multipurpose dams, have flood mitigation as an objective. This signifies that nearly 20 % of the existing large dams (more than 4500), play an important role in flood control or have it as their only objective.

The demographic increase in the world and the concentration of the population in urban areas vulnerable to the natural hazards gives rise to an increase in the flood plain occupation, where damages produced by the floods have increased in an alarming manner. Flood control dams constitute one of the very effective actions for damage mitigation in towns and consolidated urban nuclei, and there exists an increasing social demand for these type of solutions. Nonetheless, flood control dams must be considered along with an all-embracing view of the basin as a whole, and combined with other types of structural and non-structural actions, such as that of avoiding a progressive increase of the occupation of the flood plain areas and the establishment of flood forecasting systems. For all this, a considerable increase is foreseeable in the construction of new flood mitigation dams.

This section presents actual conditions and future provisions at a national level in Japan, China and Spain, as significant examples, in addition to an interesting situation in the American River in USA.

Flood Control Dams in Japan play an important role in the reduction of damages produced by the floods. The Japanese count on some 500 large dams for flood control, which protect the population in a range between the 50 years and the 200 years of return period, while providing a total flood control capacity of some 3700 hm^3. In the future, construction of some 400 new large dams is predicted, which will contribute an additional 2400 hm^3 to the flood control reservoir capacity.

China has also increased construction of large dams which will play an important part in flood control. Tables 5.24 and 5.25 show the flood control dams under construction and planed in the next 10 to 20 years, in the Yellow River and Yangtze River.

Table 5.24
Dams with the flood control on the Yellow River

Name of Project	Height of dam (m)	Type of dam	Control area (*1000 km^2)	Total storage (*10^9 m^3)	Storage for flood control (*10^9 m^3)	Maximum discharge flow (m^3/s)	Total installation capacity (MW)	On the main stem	Construction period
Laxiwa	250.0	Arch			1.0	6000	3720	Upstream	Future
Gongboxia	133.0	Rockfill			0.55		1500	Upstream	Future
Xiaoguanyin	143.0				7.02		1400	Upstream	Future
Jikou	143.5	CFRD	431.09	12.57	2.8		1800	Middle reach	Future
Guxian	186.0	Rockfill	489.95	16.0	4.65		2560	Middle reach	Future
Wanjiazhai	105.0	Gravity	395.0	0.896	0.3		1080	Middle reach	1994-2001
Xiaolangdi	154.0	Rockfill	694.16	12.65	4.05	14 000	1800	Middle reach	1994-2001

CFRD: concrete faced rockfill dam

Table 5.25
Dams with the flood control on the Yangtze River

Name of Project	Height of dam (m)	Type of dam	Control area (*1000 km^2)	Total storage (*10^9 m^3)	Storage for flood control (*10^9 m^3)	Maximum discharge flow (m^3/s)	Total installation capacity (MW)	On the main stem	On the main tributaries	Construction period
Xiluodu	273.0	Arch	454.4	12.23	3.62	49 605	12000	Upstream		Future
Xiangjiaba	161.0	Gravity	458.8	5.185	1.15	48 726	6000	Upstream		Future
Pubugou	188.0	Rockfill		5.55			3300		Daduhe	Future
Hongjiadu	178.0	CFRD		4.59			540		Wujiang	Future
Goupitan	225.0	Arch		5.69			2000		Wujiang	Future
Silin	122.0	Gravity		1.20			840		Wujiang	Future
Pengshui	115.0	Gravity		1.16			1200		Wujiang	Future
Shuibuya	233.0	CFRD	10.86	4.312	2.383	18 808	1600		Qingjiang	future
Three Gorges	175.0	Gravity	1000.0	39.3	22.15	102 500	18 200	Upper reach		1994-2009
Jiangya	131.0	RCC	3.711	1.741	0.74		300		Lishui	1995-2000

Through the struggle with flood control over the past 40 years, only ordinary floods of the major rivers in China occurring with a return period of 10 to 20 years can be put under control, whereas the extraordinary floods cannot be controlled. Flood control standards for medium and small rivers in China are even lower. With the increases in economic development and population growth, flood control has become a necessary requirement. Additionally, because of sediment deposition, the flood passage capacity of rivers and flood storage capacity of lakes and reservoirs are gradually degrading, so the threat of floods, especially in the cases of the Yellow and the Yangtze Rivers, will be more serious if no corresponding preventive measures are taken. The difficulty in harnessing the Yellow River lies in insufficient water, excessive sediment, and in non-coincident water and sediment sources. Two large regulating reservoirs on the Yellow River, Longyangxia and Liujiaxia, have brought enormous economic benefits after their completion, but at the same time they have also changed the water and sediment conditions in the middle and lower reaches of the river, causing new situations and new problems. Therefore, a new great project the Xiaolangdi reservoir is now under construction in the middle reach of the Yellow River.

For the Yangtze River, the situation is even more urgent due to the gradual degradation of flood storage capacity of the Dongting Lake. The TGP functions as the backbone of the flood control system to protect the areas in the middle and lower reaches of the Yangtze River. Its favorable geographical location will make it possible to effectively control huge floods from upstream of the Yangtze River.

With 22.2 billion m^3 of flood control storage capacity of the reservoir, the Jingjiang river section, a most critical section in flood control, will improve flood control capacity from the present 10-year frequency flood to the 100-year flood. Even if a 1000-year frequency or greater flood occurs, the vast plains on both sides of this river section, with the existence of the TGP, and with the assistance of appropriate operation of the flood diversion and retention works, would be protected. The flood damages and losses in the middle and lower river reaches and threat of flooding to Wuhan Municipality would be mitigated. And so will the TGP create favourable conditions to thoroughly harness and improve of Dongting Lake area.

In Spain, 30 flood mitigation dams exist, which represent about 3 % of the existing dams, and in the future some 40 new reservoirs for flood mitigation are planned for construction.

An interesting and illustrative case is that of the American River Basin in California, USA, which has a grave flooding problem in the city of Sacramento. In 1991, the US Corps of Engineers carried out a Feasibility Report and Environmental Impact Statement, which proposed a Detention Dam. The issue has been problematic because of environmental pressures and is still under discussion. Therefore, solutions contemplated in the Plan have not yet been implemented. In similar cases, it has not been until after catastrophic floods, when flood control alternatives were carried out.

5.4.1. Flood Control Practices of Major Rivers in Japan

5.4.1.1. Introduction

Japan is a country vulnerable to flooding and once flooding occurs, flood damage tends to be tremendous due to climatic, geographical, and social conditions.

Thus, flood control measures have been one of the basic concerns of Japanese rulers and people since time immemorial. The present flood control practice and further development for flood damage mitigation in major rivers of Japan are described below.

The rivers in Japan are characterized by (1) large fluctuations of river regime with a maximum to minimum discharge ratio of 200 to 400; (2) large quantities of sediment; (3) steep riverbed gradients and high flow velocity; (4) short duration of rainfall and floods of 1 to 3 days; (5) large fluctuations in rainfall amount throughout the year; and (6) almost constant occurrence of flood season.

Most of Japan is located in a temperate monsoon climate zone. The annual rainfall is about 1800 mm, which is twice as much as the world's average of 900 mm. Annual rainfall varies from approximately 1000 to 4000 mm in various regions of the country. Fig. 5.34 indicates the monthly precipitation in the different regions of Japan. The seasonal rain front brings heavy rain from May to June except for the northern part of Japan and also typhoons bring heavy rain in September to November. Niigata and the areas facing the Japan Sea are subjected to heavy snowfall from December to February.

5.4.1.2 *Flood Control Measures*

Flood control measures in Japan are based on the future basin conditions predicted with runoff models. The project flood discharge, which is river channel discharge without a storage facility such as a dam/reservoir, is first calculated from the design rainfall based on past rainfall records. Then, the design discharge is obtained by distributing the project flood discharge to the flood control works such as river channels and the storage facilities.

The safety degree of the flood control works in Japan is set within a range of the 200-year return period to the 50-year return period depending on the importance of the river basin. The design flood is controlled to flow safely down to the sea by the flood control facilities such as river improvements, dam, retarding basins, and diversions.

The scale of each flood control facility is set to minimize the total construction cost of each facility. In Japan, the major cities are concentrated on alluvial plains along rivers mainly because of rapid urbanization in the post-war period of particularly high economic growth. Therefore, river channel improvements, diversions and retarding basins along downstream reaches of rivers are more difficult to construct and thus, the construction of dams in the upstream reaches has become more important.

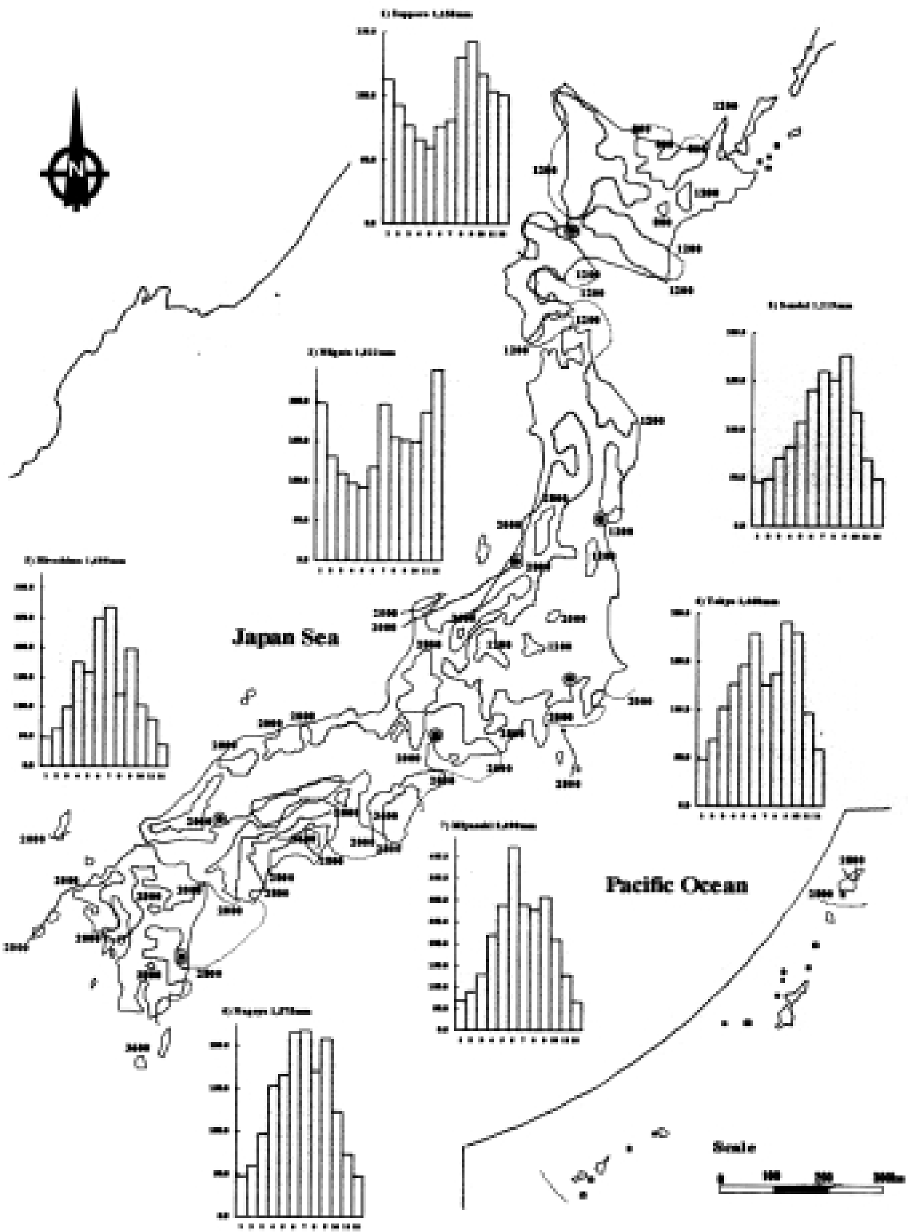


Fig. 5.34
Precipitation in Japan

In Japan, the regulation of floodwaters by dams was considered in the late 1920s. However, the construction of the first group of dams with flood control capacity such as the Tase Dam on the Kitakami River and the Ikari Dam on the Kinugawa River began in the 1940s and was completed in the early 1950s because construction was suspended during the Second World War.

5.4.1.3. Operation Practices of Flood Control Dams

Many dams in Japan have been planned as multipurpose dams to cope with the increasing demands for flood control as well as irrigation, industrial and domestic water supply, and power generation. By 1995, approximately 500 dams for flood control including dams having single purpose of the flood control had been completed in Japan, providing a total flood control capacity of approximately 3.7 billion m^3 (Fig. 5.35).

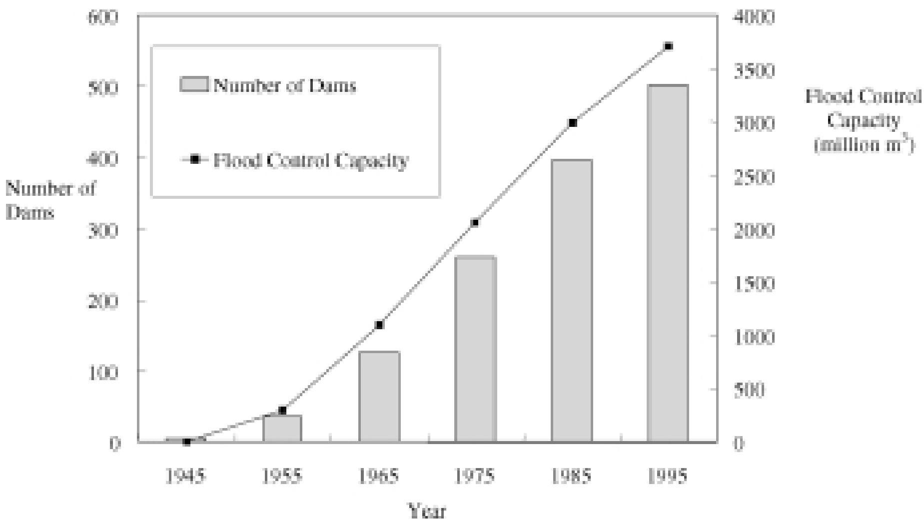


Fig. 5.35
Construction of Flood Control Dams in Japan

Most of these dams were constructed as part of comprehensive river development projects managed by Japan's Ministry of Construction. In first class river systems, which include Japan's major rivers, construction works were executed and managed as government projects. On other classes of rivers, construction works were executed and managed as Prefectural projects with the support of government subsidies.

Additionally, the Water Resources Development Public Corporation has been created to handle projects on behalf of the national government, particularly, along the Tone, Kiso, Yodo, Yoshino, and Chikugo Rivers, to conduct an integrated development of water resources that supply the major cities in these river basins where the country's population and assets are concentrated.

Table 5.26 presents an outline of flood control provided by dams on the major Japanese river systems shown in Fig. 5.36. These dams are characterized by a relatively small average effective storage capacity ranging between 30 and 90 million m^3, including the capacity for water utilization purposes. The average flood control capacity is also small, ranging from 18 to 57 million m^3 per dam. This capacity is equivalent to 56 to 162 mm of rainfall for the dam catchment area and 32 to 79 mm for the whole catchment area. Therefore, the effectiveness of the dams for flood control is limited, when considering Japan's major floods where two-day rainfall amount reaches 300 mm or more.

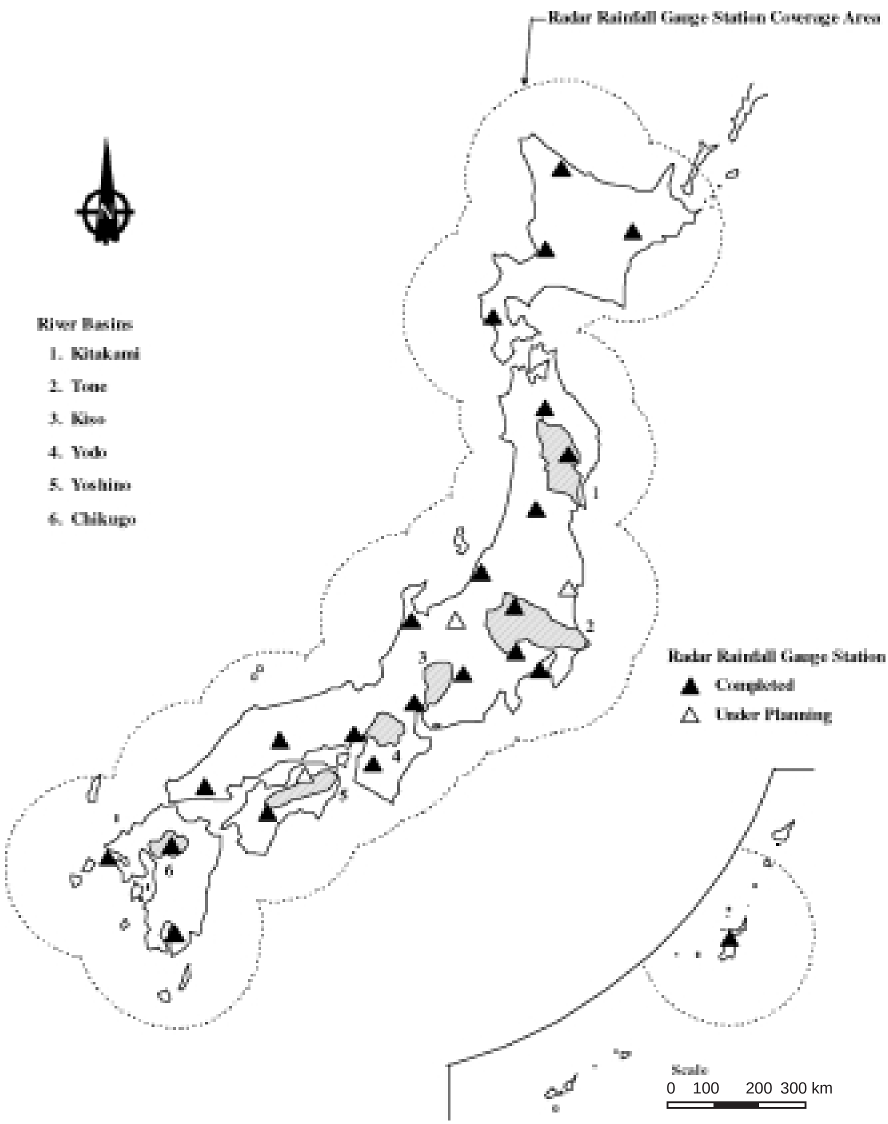


Fig. 5.36
Main River Basins in Japan and Radar Rainfall Gauge Stations

Fig. 5.37 indicates the typical flood control operations by dams and reservoirs in Japan. Peak cut operation methods such as the constant rate control and/or the constant volume discharge are generally applied, because it is extremely difficult both topographically and socially to construct a dam with a large capacity that is sufficient to store the total volume of floods. Consequently, floodwaters are generally controlled by integrated operation of a number of flood control dams in the upper reaches to reduce the floodwater volume in the downstream reaches.

Table 5.26
Characteristics of flood control by dams in Japan's Major Rivers

Characteristics	**River/Reference Point**					
	Kitakami/ Kojenzi	**Tone/ Yattajima**	**Kiso/ Inuyama**	**Yodo*/ Hirakata**	**Yoshino/ Iwazu**	**Chikugo/ Arase**
Reference point catchment area (km^2)	7060	5114	4684	3433	2810	1440
Project flood discharge (m^3/s)	13 000	22 000	16 000	17 000	24 000	10 000
Specific discharge of project flood discharge (m^3/s/ km^2)	1.84	4.30	3.42	4.95	8.54	6.94
Design discharge (m^3/s)	8500	16 000	12 500	12 000	18 000	6000
Volume to be stored in dams (%)	4500	6000	3500	5000	6000	4000
Rate to be stored in dams (%)	35	27	22	20	25	40
Number of dams with flood control functions	5	11	4	9	5	4
Catchment area covered by dams (km^2)	3345	1923	2674	1387	1904	702
Rates of catchment area covered by dams (%)	47	37	57	40	69	49
Average effective storage capacity ($10^6 \times m^3$)	81	72	89	29	76	38
Total flood control capacity ($10^6 \times m^3$)	287	209	150	160	120	114
Average flood control capacity ($10^6 \times m^3$)	57	19	38	18	24	29
Equivalent rainfall at dam site (mm)	86	109	56	115	63	162
Equivalent rainfall at reference point (mm)	41	41	32	47	43	79
* Lake Biwa basin (3848 km^2) is excluded from the Yodo River						

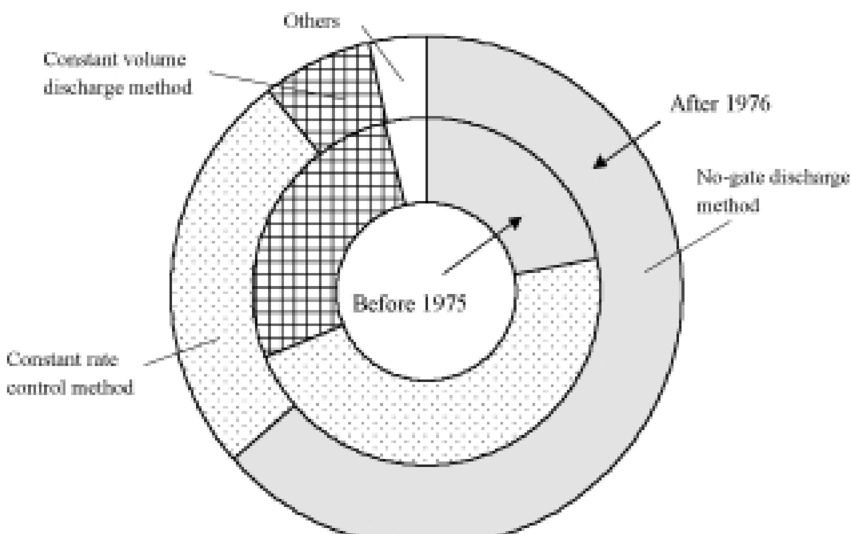


Fig. 5.37
Flood Control Operations by Dams and Reservoirs in Japan

Selection of the particular peak cut operation method is made based on the characteristics of each watershed, and the floodwater control capacity. Fig. 5.38 indicates the ratio of the flood control operation methods in the existing dams. In recent years, the no-gated discharge method, which does not require complicated gate operations during a flood time, is employed with highest ratio in the completed dams.

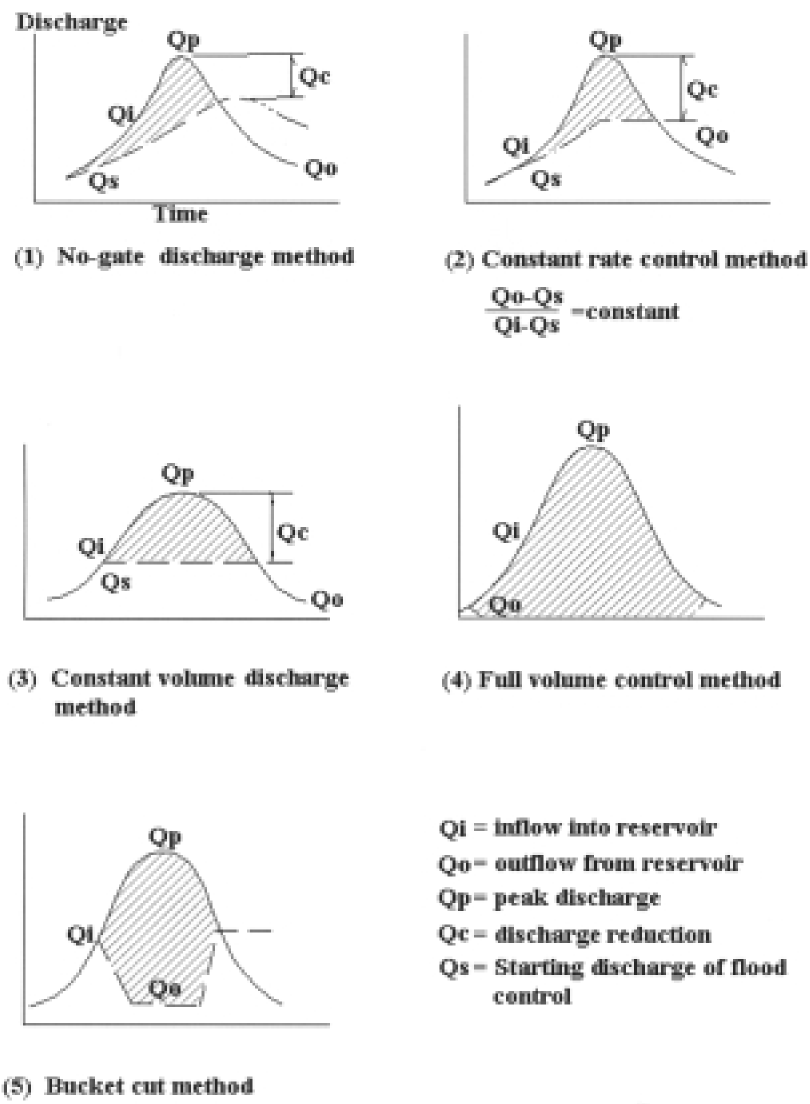


Fig. 5.38
Flood Control Method

Another characteristic of Japan's dams operation is the use of two different normal water levels for the flood and non-flood seasons, because the flood seasons of Japan's rivers are fairly consistent. In the flood season, the normal water level is lowered to provide the flood control reservoir capacity to regulate the flood water, while during the non-flood season in winter, the reservoir water level is raised to maximize water utilization.

5.4.1.4. Flood Forecasting Systems

It is extremely important to forecast floods to achieve effective flood control operation with limited storage capacity. Qualitative forecasting including typhoon tracking can be made with rather high accuracy at present, although it is still difficult to accurately forecast the rainfall amount.

In order to obtain accurate rainfall information, telemeter rain gauges have been installed at one location in every 50 km^2 of each dam catchment area to provide real-time information together with water level data on major rivers. In addition, radar rain (snow) gauges have been installed at a total of 23 locations (the installation of 3 new radar gauges is now being planned) to cover the entire nation. These radar rain gauges perform observations of rainfall intensity in 3 km grids in a circular region with a radius of 120 km around each radar site (Fig. 5.36).

Moreover, Doppler radar that can track the movement of rainfall zones is in now in use. At present, it is possible to forecast rainfall within a range of a few hours by synthesizing this information, but systems that can satisfactorily forecast rainfall further in advance have yet to be developed and further technological development must be carried out in this field.

5.4.1.5. Integrated Dam Management

A number of dams have now been constructed and are in operation in major river systems. To maximize their functions, it is necessary to integrate the information to control the flowing waters with the operations of the dams. Therefore, the Ministry of Construction is implementing integrated dam management within major watersheds, to collect and control hydrological data of rainfall, discharge and water levels as well as dam operation data including reservoir water level, inflow and outflow discharge (shown in Fig. 5.39). This information is also provided to local governments in each river basin. Fig. 5.40 is a picture of the Control room in the Tone River Integrated Dam Management Office.

5.4.1.6. Further Development

Japan is a country of steep mountains and beautiful scenery that was blessed with an abundance of water. In the past, this water was used mainly for agriculture. In recent years, however, modern social and economic growth has rapidly increased the domestic and industrial water demand, and is accompanied by a steady increase

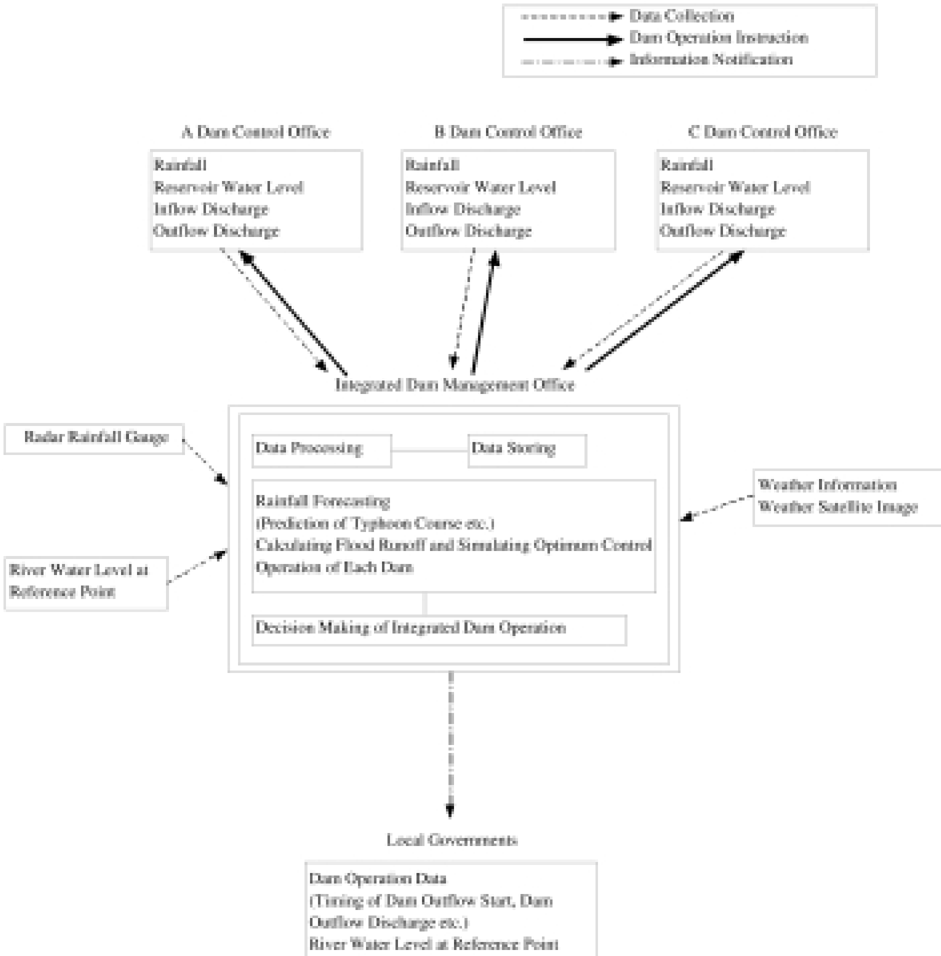


Fig. 5.39
Schematic Diagram of Integrated Dam Operation

in the water demand for agricultural use. As a result, chronic water shortages have occurred simultaneously, and cities in various parts of Japan are faced with a water crisis every year.

In addition, Japan has had serious functional problems against flood disasters due to long rainy seasons and heavy typhoons. Approximately 50 % of the population has concentrated in flood inundation areas, which accounts for only 10 % of the total land area of the nation. About 75 % of the monetary assets of the nation are concentrated used towards flood damages. In 1995, the aggregate asset damage accrued from floods was more than 150 billion yen (approx. 1.25 billion US dollars).

From the viewpoint of both flood control and water utilization, the storage capacity of the existing dams is still insufficient to control the flow volumes of rivers in Japan. To mitigate the burden caused by flood disasters and to provide stable

Fig. 5.40
Control room in the Tone River integrated dam office

water supply service, the Government of Japan has been making continuous efforts with enormous investments on dam construction projects. Although 500 flood control dams with the total storage capacity of 3.7 billion m^3 exist as of 1995, more than 400 flood control dams, which will provide approximately 2.4 billion m^3 of new flood control reservoir capacity, will be planned and constructed to further solve the occurrence of flood disasters. After the completion of the additional dams, the total available flood control reservoir capacity will be 6.1 billion m^3.

5.4.2. American River Basin, California [18]

5.4.2.1. Introduction

The city of Sacramento is located at the confluence of the Sacramento and American Rivers in central California. Throughout history, the city has been particularly vulnerable to floods occurring along the American River. The topography of the American River, which drains an area of about 5440 square kilometers, varies from flat valley areas, to rolling foothills, to steep mountainous terrain.

5.4.2.2. The Existing Flood Control System

Sacramento is protected by a complex system of dams, diversions and levees. Folsom Dam, located on the American River about 47 kilometers upstream from Sacramento, is a key feature in the flood control system protecting the city. Folsom

Reservoir has a capacity of 1200 × 10^6 m^3, which includes a minimum of 490 × 10^6 m^3 of storage seasonably dedicated to flood control. Currently, Folsom Dam is operated to provide additional flood space in years with high forecasted runoff under an agreement between the U.S. Bureau of Reclamation (USBR), which owns and operates Folsom, and the Sacramento Area Flood Control Agency (SAFCA). Fig. 5.41 shows the existing reservoir and levee flood control system in the Sacramento area.

Fig. 5.41
American River basin project

Releases from Folsom Reservoir flow through a system of levees in Sacramento. The "objective release", or flow that can be safely conveyed by the leveed downstream channel, is 3256 cubic meters per second. Studies have shown that the levees along the American River downstream from Folsom are likely to fail at several locations when sustained flows are between 3680 and 4530 cubic meters per second. The risk of levee failure during the occurrence of a 100-year flood is about 60 percent with the present operation of Folsom Dam, as determined using risk-based analysis.

Levee failure along the American River could result in flooding of more than 40 500 hectares, and potentially affecting more than 400 000 residents in the flood plain. Damages would range from $7 billion from flooding from a 100-year flood to more than $16 billion for a 400-year flood. Such flooding would result in the loss of many lives due to drowning from rapid inundation of the flood plain, and other impacts on public health and safety after the floodwaters recede. Damages from toxic and hazardous waste contamination would be extensive, and environmental resources in the flood plain would be lost. Disruptions to commercial activities and transportation would be catastrophic.

5.4.2.3. The Flood Problem

In February 1986, the "flood of record" in the American River basin severely tested the flood control system. Releases from Folsom Dam reached about 3795 cubic meters per second for a few days, placing the entire flood control system in jeopardy. Significant flood damage occurred in unprotected areas and it is estimated that if the high releases had continued, major levee failures would have resulted with significant loss of life and billions of dollars in damages.

As a result of the 1986 flood, a flood plain study was conducted in 1988 for the Federal Emergency Management Agency (FEMA). This study concluded that much of the Sacramento urban area was within the 100-year flood plain. The existing flood control system was estimated to be capable of providing only a 63-year level of protection, well below the 100-year level required under the National Flood Insurance Program administered by FEMA.

5.4.2.4. Studies and Evaluations

As a result of the flood threat, in July of 1986 the State of California and the Corps of Engineers initiated a feasibility study of the American River Watershed. In December 1991, the Corps published a Feasibility Report and Environmental Impact Statement describing the results of this study. It identified a selected plan to resolve the problems. Public comments on a draft of the report and the selected plan were incorporated into a final report that was submitted to Congress for construction authorization. The report, entitled "The American River Watershed Investigation Feasibility Report", contained a recommendation to construct levee and related improvements in the Natomas area of Sacramento and a flood detention dam on the North Fork American River, near Auburn, CA. In 1992, Congress authorized construction of the Natomas portion of the recommended plan, which has since been constructed by SAFCA. Congress also requested additional information on the flood detention dam and other feasible flood protection measures for the mainstem of the American River because of environmental concerns with the detention dam portion of the plan.

In response to the 1992 Legislation, the Corps prepared a new report as a supplement to the feasibility report. This report reassessed the risk to the Sacramento area from flooding by the American River and evaluated a range of flood protection measures to reduce the risk. It described several additional alternatives, including combinations of the individual measures. This report has a main report, which focuses on the flood protection alternatives, and a final supplemental environmental impact statement. Alternatives were formulated to substantially increase Sacramento's flood protection. Increasing the seasonal flood space and surcharge storage in Folsom Reservoir together with lowering the spillway and enlarging the regulating outlets could increase flood protection to nearly the 200-year level. These changes plus levee work downstream to accommodate larger flood releases from Folsom could increase protection to about a 300-year level. Higher levels of protection were possible only with additional flood storage upstream from Folsom Reservoir. The alternatives were presented in the November 1994 Alternatives Report. Again, the Detention Dam Plan was recommended. The report reflects information and comments from extensive review by both the public and governmental agencies.

5.4.2.5 Detention Dam Plan (Auburn Dam)

The Detention Dam Plan, shown in Fig. 5.42, would reduce the probability of flooding to less than 1 chance in 500 in any year. The major features of this plan are : (1) constructing a 155 m high flood detention dam on the North Fork American River in the Auburn area, to create a flood storage capacity of 1100 $\times$ 10^6 m^3 (2) constructing slurry walls in about 39 km of existing levees along the lower American River (3) strengthening and raising about 20 km of levees on the east side of the Sacramento River between the Natomas Cross Canal and the mouth of the American River and (4) changing the flood control operation of Folsom Reservoir back to the pre-1995 flood space of 490 $\times$ 10^6 m^3. Of the alternatives studied, this plan would provide the highest level of flood protection to the Sacramento area. It would also have a beneficial effect on water supplies and hydropower generation by restoring the flood control operation of Folsom Reservoir to 490 $\times$ 10^6 m^3 of seasonal storage space. The objective release from Folsom Dam would be maintained at 3256 cubic meters per second, to minimize the extent of required levee and related improvements downstream.

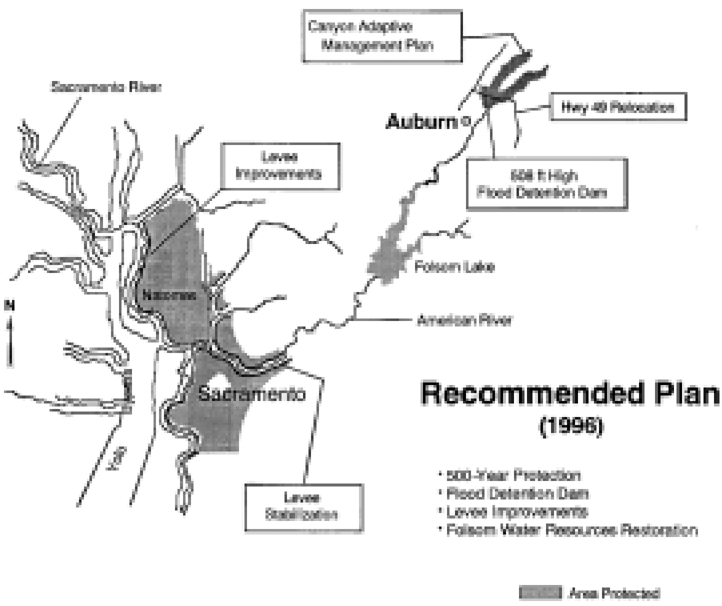


Fig. 5.42
The Detention Dam Plan

Of the three plans, the Detention Dam Plan has the highest construction cost, estimated to be about \$950 M. However, it also produces the greatest net economic benefits. The detention dam proposal has received support from interests committed to a very high level of flood protection for Sacramento while it is opposed by others concerned with protection of the environmental resources of the American River.

In February 1996, the Corps of Engineers completed a Supplemental Information Report (SIR) and Supplemental Environmental Impact Statement, further augmenting the information in the December 1991 Feasibility Report. In the SIR, three plans identified in the 1991 Alternatives Report were further evaluated. These included a Folsom Modification Plan (modifying Folsom Dam and increasing the dedicated flood space), a Folsom Stepped Release Plan (modifying Folsom Dam and the downstream levee system to allow increased objective releases), and the Detention Dam Plan (new flood control storage in a detention dam upstream of Folsom Dam). All three plans also included a slurry wall to strengthen the lower American River levees, levee and berm raising along the Sacramento River, telemetered gages above Folsom Dam to improve flood forecasting, and flood warning system improvements for the lower American River.

The three plans were reviewed at a series of public hearings on the draft SIR in October and November 1995. After the hearings, the State of California, the non-Federal project sponsor, chose the Detention Dam Plan, but with a 500-year level of flood protection, as the preferred flood protection plan. The SAFCA Board of Directors, representing local interests, voted to support this plan in November 1995.

In the SIR, the Corps of Engineers' Sacramento District recommended the Detention Dam Plan, which was preferred by state and local constituents. However, the Chief of Engineers, in his June 27, 1996, Chiefs Report, deferred a decision on a comprehensive flood control plan due to environmental pressures. Instead he recommended that the features common to all three plans along with continued reoperation of Folsom Dam be authorized as the first component of a comprehensive plan. The common features, shown in Fig. 5.43, were included in the Water Resources Development Act (WRDA) of 1996, as a means of immediately providing some additional flood protection to Sacramento. Also, in WRDA 1996, the USBR was directed to work with SAFCA to continue the interim reoperation of Folsom Dam until a comprehensive flood control plan is implemented. The implementation of these common features and continued Folsom reoperation will provide the Sacramento area with a 100-year level of flood protection, thus removing current FEMA restrictions on development.

As explained in the Chiefs Report, the common features are not intended to be, nor should they be, considered a complete plan. They should be the first component of a larger comprehensive plan that must be pursued if Sacramento is to have adequate flood protection. A significant flood threat still remains, not only to existing residents and development, but also to many thousands of new residents who could move into the flood plain with FEMA restrictions on development lifted. Based on growth projections, implementation of the common features could cause greater loss of life than taking no action, because it would allow population growth to occur in the floodplain. In addition, reoperation reduces the ability of Folsom to provide intended water supply and power benefits and recreation in the reservoir and along the lower American River.

5.4.2.6. Residual Risk Considerations

All of the alternatives that were evaluated in the SIR other than the detention dam have considerable residual risk associated with them, both in terms of potential damages and hazard to human life. When a flood damage reduction project is constructed, great care must be taken to incorporate features into the final design

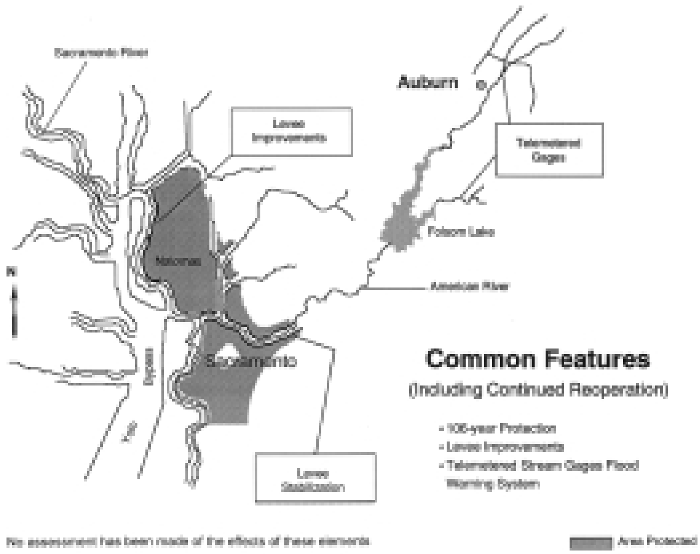


Fig. 5.43
The common features

that will minimize the adverse impacts of a design exceedence. The question is NOT WHETHER the project design will be exceeded BUT what are the consequences WHEN the design is exceeded ? Typically levee projects are designed so that initial overtopping occurs at the least hazardous location along the line of protection, generally at the downstream end, and filling of the protected area takes place at a gradual rate. This provides maximum time to institute emergency evacuation measures, thus greatly reducing the potential for loss of life. The main areas of protection in Sacramento are configured such that this type of design is not possible. If the American River levees are overtopped, the protected areas will fill rapidly to depths in excess of three meters. Egress will be severely limited and catastrophic loss of life will likely result, unless a means to manage the design exceedence is included in the project. Folsom Dam does not have sufficient capacity nor operational flexibility under any of the non-detention dam plans studied to allow for management of a design exceedence. The only way to provide enough time for evacuation as levee overtopping conditions are approached is to construct the dry dam and develop the water control plan for this condition in conjunction with Folsom. The detention dam project provides the best combination of a high level of flood damage reduction and residual risk management for the city of Sacramento.

5.4.2.7. Conclusion

In January 1997, the American River basin was once again hit with near record flooding, bringing into question once again the level of flood protection provided to the city by the existing flood control facilities and what additional measures should be pursued. Both the state of California and SAFCA have committed to exploring alternatives to provide more protection. Presently the Corps, the state, and SAFCA are restudying the Folsom Modification and Stepped Release Plans along with several other non-dam alternatives. The dam is not being considered at this time due to substantial environmental pressure. At the same time, the Corps of Engineers is moving forward with the construction of the common features as directed by Congress. Ironically, this interim measure may do more harm than good in the long term. If 100-year protection is attained, there is the likelihood that developmental restrictions in the floodplain will be lifted, and the city of Sacramento may never be provided with adequate flood protection. Experience has shown that when lands on which development is restricted are removed from the 100-year floodplain, development proceeds, residual risk increases, and the impetus for additional flood protection is often lost.

5.4.3. Flood Control in China

5.4.3.1. A Prospect on the Development Trend of Flood Control

Through great efforts of river flood control work from 1949 to 1989, comparatively complete systems of flood control measures have been formed for the major rivers in China and preliminary control has also been achieved for medium and minor rivers. The frequency of flood disasters over large areas has been remarkably reduced. Flood control project facilities play the role of protection barriers in ensuring the safety of cities and industrial and mining enterprises, and in guaranteeing the continual increase of agricultural production. Thus, the flood control facilities have produced huge economic benefits. However, under the conditions of continual population growth and rapid economic development, the flood disaster is still one of the natural calamities that bring the heaviest economic losses, largest casualties of people, and greatest social impacts. In looking forward to the new millennium, China will still face many major flood control problems, which are described below.

1. The natural evolution of rivers, lakes and seas and the social economic activities of mankind both have adverse effects on flood control. The rivers, lakes and seas have their own characteristics of natural evolution. From the long-term viewpoint, the expansion of alluvial plains, the extension of coastlines and the development of estuarine deltas, as well as the shrinkage of lakes due to sedimentation are all development trends that are independent of man's will. In China, the impacts of human activities are aggravating soil erosion in many mountainous and hilly areas, increasing artificial obstacles due to land acquisition on the flood passage way of the river courses, and expanding land reclamation in the flood plain and delta areas of rivers and lakes. All these factors lead to an obvious trend of decreasing river discharge capacity and reducing the role of lakes and low-lying land in flood detention and storage.
2. Flood plain areas in the middle and lower reaches of the large rivers can only resist in general 5-year to 20-year floods which are rather low in standards for flood control. In case of floods greater than the 5- to 20-year floods, measures of using flood diversion and storage basins have to be adopted for protecting the majority of the area at the sacrifice the localized area, in order to restrict flood disasters within the planned flood diversion and storage

basins. However, the use of flood diversion and storage basins presents great conflicts, which sometimes cannot guarantee the timely operation of flood diversion. Accordingly, major tasks in future flood control will need to determine how to effectively control the over-standard floods to decrease losses, and in particular, large numbers of casualties.

3. Many medium and minor rivers are still not effectively controlling ordinary floods, and flood disasters are quite common and on the increase, especially in the mountainous and hilly areas. Although the affected areas are scattered and do not influence the overall situation, the total losses are still very great. Since the numbers of medium and minor rivers are many, the flood control work in the mountainous and hilly areas will continue to be arduous tasks for a long period.

4. The deterioration of flood control facilities without timely repair significantly increases the amount of strengthening and rehabilitation work.

5. Non-engineering flood control measures have not yet received general attention. In view of the above-mentioned existing problems, the situation of flood control in China is still very serious at this time. Therefore, considerations must be made that the construction of flood control projects should be included in the national overall long-term planning for prevention of major natural calamities, protection of ecological environment and management of territorial land. Flood control project construction should also be taken as an important component part of the long-term economic development. The concrete countermeasures are described below.

1. Flood control facilities will be further strengthened all around by first, continual construction of controlling reservoirs on the various rivers. Apart from the rehabilitation and strengthening of the existing reservoirs, new reservoir projects with multiple-purpose development of water resources will be built for enhancing flood regulation and operation capacity. These reservoir projects include the Three Gorges Project on the Yangtze River, Xiaolangdi Project on the Yellow River, Buxi Project on the Nenjiang River, and the Longtan and Datengxia Projects on the Pearl River. Moreover, construction of safety facilities and supplementary projects in the flood diversion and storage basins will be accelerated to enable flood diversion and storage in proper time and proper quantity to the greatest possible extent, to reduce the damages of fixed assets in the flood diversion and storage basins, to decrease the losses, and to lighten the burden of rehabilitation and relief work after the disasters. The second method to strengthen flood control facilities is the construction of river training works and flood protection levees, which are the basic control measures and the sustained tasks for a long period. With the social economic development, the riparian land along the large rivers and the coastal zones are and will be concentrated with cities, industrial and mining enterprises, and land and water communication junctions. The development and protection of the riparian land coastal zones must be put under overall planning and unified arrangement for opportunely carrying out the control work and for coordinating the contradiction between economic development and flood control. The third way to strengthen flood control facilities is to forge ahead

with water and soil conservation in the vast mountainous and hilly areas for controlling water and soil losses and small gully development, and for reducing the natural disasters of mountainous torrents, landslides and rock avalanches. In the above work of flood control, various measures will be adopted to turn floodwater into utilizable water resources.

2. Management of flood control facilities will be strengthened to keep them in effective and good condition. Since the occurrences of flood disasters are random in nature, the various flood control projects and facilities are not in frequent operation. For example, the flood storage of a reservoir may not be used for several years or only a part of it is used in most years; the flood plains and deltas of the rivers and lakes may also not be used as natural floodways or storages for many years; the flood diversion and detention basins may be used only once in several years or in several decades; or the telecommunication facilities for flood control are used only for a short period during the flood season. These features show that the flood control projects and facilities necessitate special methods of management. In addition to establishing and improving management regulations, systems, and institutions; employing necessary personnel; executing strict management under the laws; and strengthening maintenance and repair, it is necessary to study new approaches for strengthening the management and for rational multiple-purpose use of flood control facilities such as flood passage and detention land, and telecommunication facilities. The regulation and rule systems for flood control should be gradually completed; the rivers are to be classified into different levels, and separate management and responsibility systems at different levels will be established and improved for normalizing the flood control project management and the emergency flood prevention work.

3. Special emphasis should be placed on adaptation to the specific local conditions when considering engineering measures in close association with non-engineering measures. Under the present conditions in China, the conventional projects can only help in many areas to attain the target of resisting ordinary floods. Non-engineering flood measures should be relied on in case of large and extraordinary floods for protecting the safety of life and property and for decreasing the economic losses.

   Strengthening and improving non-engineering flood control measures are important tasks in future flood control work. The whole society should foster the idea of disaster prevention for a long period. In areas where flooding is possible, systematic propaganda about flood disasters and flood measures should be distributed among the residents to help them fully understand the serious consequences of the flood damages and the countermeasures to be adopted during flood occurrence; Plans for all sorts of construction should consider adapting to the requirements for flood prevention; and flood insurance should be gradually implemented.

   Chinese government at all levels and the water administration departments should establish regulations, rules, and guiding documents for all sorts of construction, and should provide guidance and restrictions to production and construction activities in these flood-prone areas. Strict management and control of population growth in the plain and delta areas of rivers and lakes and in the flood diversion and storage basins should be maintained, by limiting random land acquisition and construction of man-made obstacles. All existing obstacles on the floodways should be cleared away in a timely manner to maintain the flood discharge capacity and the detention and storage capacity.

5.4.4. Flood Control Dams in Spain

Because of the grave impacts of floods in the 1980s, Spain has considerably increased the number of reservoirs dedicated exclusively to or with the principal purpose of flood routing. So, in the 1991 Inventory of Spanish Dams, only 7 of the 939 reservoirs in use that year were identified with flood control purposes, representing only 0.7 % of the existing reservoirs. Of these 7 reservoirs, 3 were dedicated exclusively to flood mitigation, 3 were for routing and irrigation, and 1 was for routing and supply. Nevertheless, the 1991 inventory of dams referenced that of 69 dams under construction, 12 had reservoirs for flood control purposes, representing 17 % of the dams in construction. Of these, 9 were dedicated exclusively to flood routing, 2 were for routing and irrigation, and 1 (the Tous Dam) for routing, irrigation, and supply.

Table 5.27
Structural Flood Mitigation Measures

BASIN	NUMBER OF FLOOD MITIGATION PLANS		
	WITH RIVER WORKS	WITH FLOOD MITIGATION DAMS	TOTAL
North	12	-	12
Duero	2	-	2
Tajo	11	-	11
Guadiana	6	3	9
Guadalquivir	14	1	15
South	12	1	13
Segura	1	3	4
Jucar	18	1	19
Ebro	25	2	27
Galicia Coast	1	-	1
Internal Basins of Catalonia	3	1	4
TOTAL	105	12	117

At the beginning of January of 1996, 28 of the 1038 reservoirs in operation in Spain existed for flood control, which represents 2.7 % of the total. Of these 28 reservoirs, 17 were for flood routing, 9 were for routing and irrigation, 1 was for routing and supply, and 1 was for routing, irrigation and supplies. However, in all these reservoirs, the objective of flood damage mitigation was the principal purpose.

Furthermore, two flood control dams, the Puentes II and Los Charcos Dams, were in construction in the River Segura basin. The greater part of flood control reservoirs (19 out of 30) in Spain are located in the Segura Basin.

Faced with such a flood situation, it is necessary for Spain to continue to construct dams and create reservoirs for flood control. These ideas will be contemplated in the future considerations developed in the National Hydrological Plan. This Plan, which considers structural actions for flood control, contemplates 117 actions of which 12 are construction of flood control reservoirs. These plans correspond to the execution of some 40 new flood control dams (Table 5.27).

6. GUIDELINES FOR THE HYDROLOGIC DESIGN OF FLOOD MITIGATION DAMS

6.1. INTRODUCTION

Flood mitigation, or flood damage reduction dams constitute an important element in comprehensive planning for the reduction of flood damages. In any case implementation of such dams must be contemplated jointly with other structural measures (*i.e.* dikes, levees, channel improvements, river diversions, interbasin diversions) and non-structural actions (*i.e.* zoning, systems of insurance, flood forecasting systems), to reduce flood damages more effectively.

Flood mitigation dams may be classified into three general categories : 1) Single-purpose dams which are built solely for flood damage reduction. These dams could be combined with recreation activities by utilizing small amounts of storage, 2) Multi-purpose dams, in which flood mitigation is the primary objective, combined with secondary objectives such as water supply, irrigation or hydropower, and 3) Dams in which flood mitigation is an important objective, but secondary to purposes associated with water storage.

In some cases, the objective of reducing damages is related to a specific flood season. In these cases, the dam is operated for primary or secondary flood mitigation, during the flood season and operated for other purposes during other periods of the year. Such a dam would be named a flood season mitigation dam.

Because of the large increase in flood damages over the last few decades, many existing dams are now being re-evaluated to determine their capabilities for flood mitigation. When feasible, operating rules for many existing dams whose main purpose might be water supply, irrigation or hydropower, are being modified to also provide flood peak reduction. In these cases, the capabilities of the dams are generally limited to reducing flood peak discharges for low and medium return period floods (20- to 50-year recurrence intervals), unless there is a major reallocation of storage. Such projects are often operated in concert with other types of structural and non-structural measures for mitigation of larger floods.

In general, single-purpose flood mitigation dams are subjected to intermittent design loads. Therefore, in some cases, it may be possible to permit certain decreases in design requirements. For example, the impermeability of the dam or its foundation may be designed using less stringent criteria than would be the case for a dam with a permanent pool, but only with the assurance that selection of lower design criteria does not affect safety.

Studies for the design of flood mitigation dams should generally be carried out at a basin level, and with analyses of all technically feasible alternatives. In general, large dams should be situated close to the area to be protected in order to maximize the benefits. These dams give greater protection than would be obtained with many small dams dispersed throughout the basin, on the principal river farther upstream, or on its tributaries. Nevertheless, on numerous occasions economic, social and environmental considerations may present a problem in constructing a flood mitigation dam in the reach of the river immediately upstream of the area to be protected.

The construction of flood mitigation dams is often carried out in many countries at a national or federal level, as they constitute major actions of public interest. It is usual however for construction to be carried out in cooperation with state, regional, provincial and local governments and the costs to be shared by the beneficiaries.

This chapter presents some important concepts, makes certain recommendations and proposes guidelines that might assist the design engineer in developing the hydrology for single purpose or multi-purpose flood mitigation dams, operating individually or as an integral part of a reservoir system.

6.2. GENERAL PRINCIPLES

6.2.1. Dams as Flood Reduction Measures

The most effective structural remedy for flooding is to control streamflows and river levels by storing runoff. Reservoirs can achieve this desired result and can be operated to reduce flood inundation damage by retaining or temporarily storing basin runoff. Depending on the severity of the runoff and the available storage capacity, releases can be limited and more controlled. Reservoirs are especially well-suited for flood damage reduction when damageable property is spread over a large geographic area throughout the floodplain, and relatively few intervening flows occur downstream of the dam.

Planning reservoirs that will meet flood damage reduction objectives involves a series of hydrologic studies. These studies should include reviews of historical and hypothetical floods, alternatives in location of dams and storage capacities, downstream flood control objectives, likely coincident flows from uncontrolled drainage areas below the project, channel capacities, flood damage surveys, benefit-cost determinations, and a general plan of regulation to meet the flood reduction requirements. Study results are then used to determine and optimize the size, location and degree of protection provided, based upon the project objectives. Project elements are sized based on the overall evaluation of river basin development considering economic, environmental and social values. Later, during the detailed technical design phase, the project studies are refined, the hydraulic features are designed, and the water control plan for the project is developed.

6.2.2. Objectives for Reservoir Control of Floods

Generally, reservoirs are not designed to provide complete protection against extremely large floods but are usually capable of storing the significant runoff from minor or moderate flood events. However, for large floods, the storage capacity is usually sufficient to reduce downstream flood levels thereby avoiding a major flood disaster. The water control plan defines the basic goal of reservoir regulation, relative to control of minor and major flood events. In special circumstances when reservoir inflows can be forecast several days or weeks in advance (for example, when the runoff occurs from snowmelt), the degree of control for a particular flood may be determined on the basis of current forecasts to best utilize the storage space. Also, the amount of flood reduction storage space may be varied seasonally, if the reservoirs are used for multiple purposes.

One important aspect of the reservoir water control management plan deals with the evacuation of the stored water. Release of the stored water may result in a long duration of high river levels that are at or near bankfull stage at downstream control points. The water control management plan must account for these releases, which may entail a compromise between rapid evacuation of stored water to assure control of subsequent floods, and slow evacuation to allow downstream river levels to recede as quickly as possible.

6.2.3. Reservoir Systems

A multi-reservoir system is generally regulated for flood damage reduction to provide flood protection both in intervening tributary areas and at major downstream damage areas along the main river. The extent of reservoir regulation required for protecting these areas depends upon local conditions of flood damage, the spatial and temporal distribution of rainfall, storms movement, antecedent ground conditions, uncontrolled tributary drainage, reservoir storage capacities, and the volume and time distribution of reservoir inflows. Either the upstream or downstream requirements may at times govern the reservoir regulation, and usually the optimum regulation is based on the combination of the two. Reservoir releases are based on the overall objective of limiting discharges at critical control points to predetermined non-damaging stages. The regulation plan must consider the travel times caused by storage effects in the river system and the local inflows between the reservoir and the control points. Since each flood is caused by a unique set of hydrometeorological conditions, the plan of flood control regulation for a reservoir system should be based on the specific details of that particular flood. This is most easily achieved by modelling the conditions of runoff and reservoir regulation through computerized simulation techniques. Thus, by determining the appropriate level of control at each project, the reservoir system can respond effectively and meet the water management objectives in achieving the desired downstream control.

System control can incorporate the concept of a balanced reservoir regulation, with regard to filling the reservoirs in proportion to the flood control capability of each, while also considering expected residual inflows and storage available. Evacuation of floodwater stored in a reservoir system must also be accomplished on a coordinated basis. Each reservoir in the system is drawn down as quickly as possible to provide space for controlling future floods. For multi-purpose projects, water control management plans containing regulation criteria in the form of variable elevation guide curves and regulation schedules for individual reservoirs may be used to define seasonal amounts of storage space as primary and secondary flood control storage. The objectives for withdrawal of water in the various zones of reservoir storage are determined to minimize the risk of encroaching into the flood control storage and to conserve water for future requirements.

6.2.4. Levees

Flood protection of land adjacent to rivers is often accomplished or supplemented by means of levees and floodwalls. Planning and design of these structures are typically based on a multi-discipline design approach that involves : a) hydrologic, hydraulic, environmental and economic studies of floods, local flood runoff, and damages ; b) construction materials, foundations, and alternative structural design studies ; c) Design of facilities to evacuate interior drainage from behind the levees and d) benefit and cost evaluations. The river control works may also include channel improvements and bank protection measures. The degree of protection provided by levees is based on planning and engineering considerations as well as the type of area, rural or urban, being protected. Levees, which protect urban areas, are usually designed with a higher degree of assurance in expected project performance due to the severe hazard and the potential for loss of life in the event of a levee failure. Consequences of a design exceedence are critical to project formulation and acceptability.

6.2.5. Combined Reservoir and Levee Systems

Flood protection is provided in many river basins through the combined use of reservoirs, levee/floodwall systems, channel improvements, diversion structures, off-channel storage facilities, and other improvements. The design of combined systems is based on planning and engineering considerations that encompass a complete analysis of the watershed and its potential future development. The degree of protection afforded by either levees or reservoirs is often limited by physical, economic or social considerations. The height to which levees can feasibly be built may be limited by foundation conditions, construction material availability, feasible rights-of-way, and overall cost-benefit analysis. Similarly, the protection that reservoirs can provide is limited by the availability of suitable construction sites, engineering and economic considerations, and social and environmental constraints in reservoir project development. Basin-wide studies should also investigate the hydrologic or hydraulic dependence of individual project elements located at various sites. For dependent facilities, a system design approach should be used to optimize the investment within the river basin. For those facilities determined to be independent, individual project element sizing can be performed. Management of combined reservoir and levee systems to achieve water control objectives involves the same principles as for flood protection by individual reservoirs and reservoir systems.

6.2.6. General Design Considerations

When discussing the hydrologic design of a flood damage reduction reservoir, it is useful to think in terms of two separate design cases. The first case is related to dam safety and is used to establish the overall size of the dam and release facilities. This analysis establishes the top of dam elevation necessary to fully accommodate the selected inflow design flood without overtopping the dam and with appropriate vertical clearance (freeboard) above the maximum water surface elevation to account for wind and wave action. This case is the basis for determining the overall storage and release requirements for the dam.

In the second design case, the project is formulated with the appropriate amount of storage to adequately control downstream flooding. The amount of dedicated storage required in a flood damage reduction reservoir is based primarily on the most economical reduction of downstream damages, although residual risk considerations, in terms of both remaining economic damages and hazard to human life, social and environmental considerations may result in some adjustments. In a traditional analysis, the amount of protection provided is usually expressed as the return period of the largest flood for which inflows can be completely controlled to non-damaging levels downstream by regulation of reservoir releases. In the past, uncertainties in hydrologic, hydraulic and economic parameters were not directly accounted for in the formulation of flood damage reduction projects. Today, risk-based analysis methods, which directly incorporate uncertainty, may be used to determine the optimal storage allocation. When risk based-analysis is used, aggregate project performance over a range of possible floods is used to describe the amount of protection provided.

Hydraulic, hydrologic, geotechnical and structural design follow, in general, the same engineering requirements as for any other dam. Some lessening of structural and/or geotechnical requirements may be permitted if the reservoir under design is a single purpose flood control facility and will not store water on a permanent basis as would a hydropower or water supply project. As a general philosophy, to reduce cost, dams are usually designed to allow for some damage resulting from an extreme flood occurrence. However, the dam must be designed such that an extreme flood would not result in a breach due to overtopping and sudden loss of stored water. No damage to the structures should be permitted during the occurrence of the flood that just utilizes the full amount of controlled reservoir storage space.

In some countries flood damage reduction projects are developed on a cost-shared basis with state, regional or local government sponsors. Following construction, the local sponsor is responsible for operation, maintenance, repair, rehabilitation and replacement of the project. In this instance, the local sponsor must agree to institute sound floodplain management practices in the protected area, including mitigating downstream problems, implementing appropriate zoning measures to reduce hazard potential, and assuring that the flood insurance requirements are met.

Reservoir regulation strategies are project-specific and may be either based on single project operation or on coordinated operation of groups of projects on a system-wide basis. These strategies are initially developed to meet the objectives established during project planning studies and are continually reviewed and adjusted to reflect changing conditions and priorities.

6.3. HYDROLOGIC DESIGN

6.3.1. Formulation and Design Objectives

Flood damage reduction projects are formulated to provide safe, efficient and effective protection to lives and properties in flood prone areas. Projects are formulated by analyzing flood plain damage potential, damage prevention performance, and cost over a full range of project sizes and configurations. The final plan selected is based on maximizing net economic and social benefits consistent with acceptable risk and functional performance [1].

The engineering challenge for designing flood damage reduction projects is to balance risk of design exceedence with flood damage prevented, uncertainty of flood levels with design accommodations, and provide for safe and predictable performance. The task is made difficult because economics dictate that less than complete protection be accepted ; risk of capacity exceedence is real and must be planned for because it may occur within the life of the project ; and uncertainty in flood levels exists because of imperfect knowledge.

The hydrologic criteria for the design of flood mitigation dams is based on two design conditions : 1) An Inflow Design Flood (IDF) is established and must be accommodated to assure hydrologic dam safety and 2) The protection design flood and the optimal amount of controlled storage for downstream flood reduction must be established.

1. Inflow Design Flood for Hydrologic Safety

The criteria for the selection of the IDF for dam safety have typically been based on a formal Dam Hazard Classification System, with greater hydrologic safety required for dams that posed a greater risk. A wide variety of formulations for dam classification systems have been used, based on size, on a hypothetical dam factor (e.g. H x V, height x volume of reservoir), on characteristics and size of power plants, on the irrigated area, on the use of the regulated water, or in some cases on general criteria applicable to all dams due to the fact that all dams present a hazard. In many cases the downstream hazard has been used as the basis for establishing the fundamental criteria. A complete discussion of hydrologic safety is presented in Chapter 7.

The majority of world cases are usually within the values described in Chapter 7. Recommendations for the selection of the design flood are shown in Table 6.1.

In the final analysis, each country must determine what is an acceptable level of risk according to its own resources and technical, economic, social, cultural and political criteria, while taking into account the necessity for measures of maximum safety for dams which present a high downstream hazard.

Table 6.1
Selection of design floods

DAM HAZARD CATEGORY	LOSS OF LIFE	ECONOMIC, SOCIAL, ENVIRONMENTAL, POLITICAL IMPACTS	DESIGN FLOOD	SAFETY CHECK FLOOD OR INFLOW DESIGN FLOOD
High	N	Excessive	% PMF or 1000 - 5000	PMF or 5000 – 10 000
Significant	0-N	Significant	% PMF or 500 -1000 or ERA. (Economic Risk Analysis)	% PMF or 1000 - 5000 or ERA
Low	0	Minimal	100	100 - 150

2. The Protection Design Flood and Determination of Flood Storage Requirements

The amount of flood protection provided by a flood mitigation dam is often expressed in probabilistic terms or as a given return period, for which the dam is designed to reduce downstream flows to non-damaging levels. The analysis must take into account the interrelationship among a range of inflows, capacities of the reservoir, and acceptable volumes of flow downstream, in relation to the downstream flood plains. The amount of storage provided for flood reduction is site specific. However based on worldwide experience, the following values to describe the amount of protection provided could be considered typical:

- Rural areas. Flood protection in agricultural areas has typically been provided for return periods between 20 and 50 years.
- Urban areas. Flood protection in developed areas has typically been provided for return periods between 50 and 200 years. However if economic, social and environmental considerations are favourable, and in the cases of important cities downstream, return periods of 500 years or even 1000 years may be justified.

6.3.2. Uncertainty in the Analysis

Planning flood damage reduction projects requires information on discharge/frequency, stage/discharge, and stage/damage relationships at points along the stream where protection is to be provided. Such information is obtained from observed and measured data, or is estimated by various synthetic procedures and modelling techniques. The information is frequently based on short records and small sample sizes, and subject to measurement errors and inherent limitations and assumptions associated with the analytical techniques employed. These estimated values are, to various degrees, imprecise or inaccurate and thus induce some uncertainty in key variables and decision-making parameters.

6.3.3. Traditional Formulation Approach

The traditional method of analysis is to first develop discharge/frequency data at representative stream locations for the area under study [2]. When gauged data are available, standard statistical methods are employed. If no data are available, regional analysis or watershed modelling is appropriate. Uncertainty in both the data and the analysis is often considered by performing sensitivity analyses and evaluating the results for reasonable adjustments to model variables. The outcome of sensitivity analyses may result in adoption of model coefficients to ensure that results are conservative.

Based on the discharge/frequency information, several project sizes are selected for analysis. The next step is to perform water surface profile computations along the study reaches for the selected levels of protection to develop stage/discharge data. When flow is complex or circumstances are unusual, unsteady flow and/or two-dimensional model computations are needed. Models are calibrated and if possible, verified with observed high-water-marks, available rating curves at stream gage locations, and published guidelines. Uncertainty is considered by performing sensitivity analyses and evaluating the results of reasonable adjustments of model variables. The outcome of sensitivity analyses generally results in adoption of model coefficients to ensure that computed water surface profiles are conservative.

A stage/damage curve is developed to provide an estimate of damages as a function of river stage. Damages are highly sensitive to a variety of factors, such as mapping accuracy, first floor elevations, types of structures and contents. These factors are important in describing the variation in damage but are rarely empirically verified. Uncertainty is sometimes considered by performing sensitivity analyses.

The discharge/frequency, stage/discharge and stage/damage data are then combined to develop the damage/frequency curve, shown in Fig. 6.1, which is used to determine the flood damage reduction benefits for each size project selected for evaluation [9]. The selected project is the one that reasonably maximizes net flood damage reduction benefits. Only projects with acceptable performance are considered in the evaluation.

Performance is usually characterized as a unique level-of-protection for a given project configuration. With the traditional approach, uncertainty is generally considered by application of professional judgement and conducting sensitivity analyses. For example, in the formulation of a levee or floodwall project in an urban area, uncertainty is accommodated by including an appropriate freeboard in the final design to ensure performance for the design flood. For a reservoir project, controlled storage may be increased or operation modified to assure that flood reduction objectives will be met.

Project performance considers level of protection as the primary performance indicator. Level of protection is defined as the exceedence probability of the event that corresponds to the capacity of the project. The importance of this single indicator is often overemphasized, while ignoring other performance information needed to ensure proper project comparisons in selecting the alternative to be recommended for implementation. Final project selection and recommendations are based on maximizing net economic, environmental and social benefits.

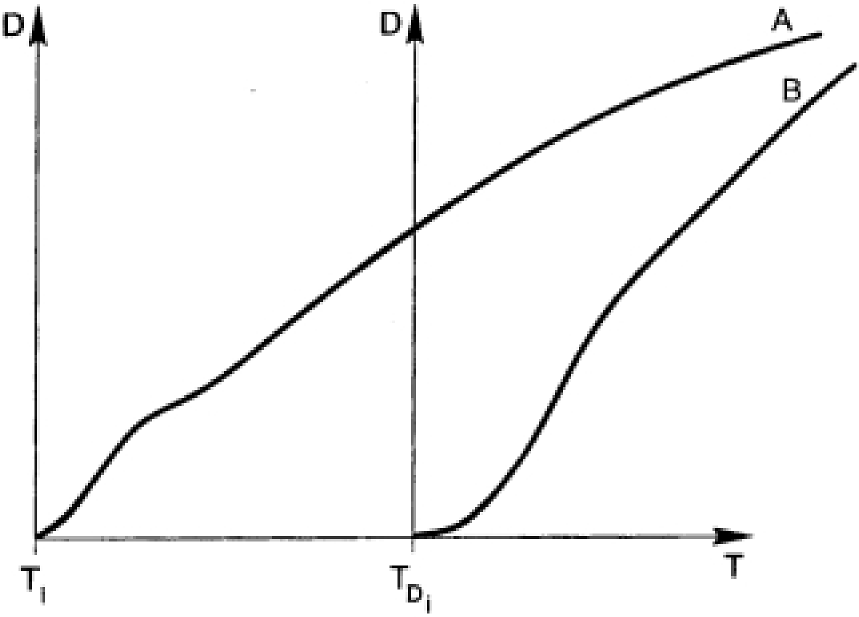


Fig. 6.1
Damage – Frequency Curve (D-T)

A - WITHOUT FLOOD MITIGATION DAM
T_{D_i} - RETURN PERIOD OF THE PROTECTION DESIGN FLOOD
B - WITH FLOOD MITIGATION DAM

6.3.4. Risk-Based Analysis Approach

6.3.4.1. Introduction

Risk-based analysis (RBA) is a method of performing studies in which uncertainty in technical data is explicitly taken into account [1, 3, 5]. With such analyses, trade-offs between alternatives, risk, and consequences are made highly visible and quantified. The overall effect of risk and uncertainty on project design and economic viability can be examined and conscious decisions made reflecting an explicit tradeoff between risk, consequences, and costs.

The RBA approach has many similarities with traditional practice in that the basic data are the same. Best estimates are made of discharge/frequency curves, water surface profiles, and stage/damage relationships at critical locations (index points). The difference between the traditional approach and the risk-based approach is that uncertainty in technical data is quantified and explicitly included in evaluating project performance and benefits. Using RBA expected project performance can be stated in terms of the probability of achieving stated goals.

Also, adjustments or additions of design features to specifically accommodate uncertainty are not necessary.

Risk-based analysis quantifies the uncertainty in discharge/frequency, elevation/discharge, and elevation/damage relationships and explicitly incorporates this information into economic and performance analyses of alternatives. The process requires a statistical sampling method to compute the expected value of damage and damage reduced, while explicitly accounting for uncertainty.

6.3.4.2. Methodology

The method used to develop the discharge/frequency relationships at critical damage locations depends on data availability. For gauged locations, an analytical fit is appropriate [2]. Uncertainties for discrete probabilities may be represented by the non-central t distribution, for example. For ungauged locations, the discharge/frequency function may be adopted from applying a variety of approaches [4]. When justified, curve-fit statistics for the adopted functions are computed. An equivalent record length is assigned based on the analysis and judgements about the quality of information used in adopting the function. Regulated (e.g. by a flood control dam) discharge/frequency, elevation/frequency and other non-analytical probability functions require different methods [9]. Non-parametric statistical estimation methods are applied to develop the probability function and associated uncertainty for these situations.

Elevation/discharge functions are developed for index locations from measured data at gages or from computed water surface profiles. For gauged data, uncertainty is calculated from the deviations of observations from the best-fit rating function. Computed profiles are required for ungauged locations and for proposed project conditions that are modified from that of historic observations. Where sufficient historic data exists, water surface profile uncertainty is estimated based on the quality of the computation model calibration to the historic data. Where data are scant, or the hydraulics of flow complex, such as for high velocity flow, debris and ice jams, and flow bulked by entrained sediments, special analysis methods are needed. One approach is to perform sensitivity analysis of reasonable upper and lower bound profiles and use the results to estimate the standard deviation of the uncertainty in stage. Unless data indicate otherwise, the uncertainty distribution for flow-stage functions is generally taken to be Gaussian.

Elevation/damage functions are derived from inventory information about structures and other damageable property located in the flood plain. The functions are constructed at damage reach index locations where discharge/frequency and elevation/discharge functions are also derived. Presently, separate uncertainty distributions for structure elevation, structure value, and content values are specified and used in a Monte Carlo analysis to develop the aggregated structure elevation-damage function and associated uncertainty. The uncertainty is represented as a standard deviation of error at each elevation used for defining the aggregated function at the index location.

The basic steps to carry out the RBA are :

a) Develop best estimates of discharge/frequency curves, water surface profiles (stage/discharge ratings), and stage/damage relationships for the existing (without project), existing with project, future without project, and future with project conditions.

b) Develop statistical descriptions of uncertainty for each of the above relationships.

c) Nominate alternative project capacities ; compute costs and flood damage prevented ; array results and select a plan according to appropriate criteria ; and

d) Make appropriate refinements to ensure project performance and function, such as providing design features to control overtopping location and management of subsequent flooding for levee projects, and operational accommodations required for reservoir storage, channel, and diversion projects.

The above steps are repeated as needed for each alternate measure evaluation, or combinations of measures, to enable comparison of major project alternatives. Step c brings together all the elements to determine the selected project capacity. To correctly incorporate uncertainty in the several elements, they must be allowed to interact with one another. For example, the possibility of error for higher flows (or lower flows) must be allowed to couple with the full range of possible stage and damage errors. Because of the nature and complexity of the error distributions, the interaction cannot be uniquely accomplished analytically. An alternative approach is to use Monte Carlo simulation. In this approach, the basic relationships and error distributions are sampled by exhaustive trial to allow the interaction to take place. For a given size project, various combinations of the parameters are evaluated (approximately 5000 samples) and for each interaction, success or failure is established. Other project sizes are evaluated, and a matrix describing economic outputs and performance for each is produced. The matrix forms the basis for selection of project size.

The results of the analyses are probability distributions of the various parameters (flow, stage, and residual damage) as a function of project capacity. The expected cost and benefit can then be computed and the project capacity selected according to the appropriate criteria. Tabulations of the likelihood of project capacity exceedence for flood events will enable characterizing risk-exceedence and performance. The risk-based analysis framework will quantify the reliability and performance of project design. This reliability and performance will be reported as the protection for a target flood with a specified annual non-exceedence probability. For example, the proposed project is expected to provide protection against the one-half percent (0.5 %) chance exceedence flood, should it occur, with a ninety percent (90 %) chance of non-exceedence. This performance may also be described in terms of the percent chance of controlling a specific historic flood to non-damaging levels.

6.3.4.3. Summary of Risk-Based Analysis

Imperfect knowledge of the "true" nature of the hydrology and hydraulics in an area creates uncertainty in project designs and in the estimate of their reliability. Additionally, uncertainties in expected damage with and without the project, influence the selection of an alternative plan for design. Risk-based analysis procedures provide an approach to explicitly quantify the uncertainties associated with discharge/frequency, stage/discharge and stage/damage relationships that are required in the formulation of flood damage reduction projects. The method uses the same basic data as that used in traditional practice, but has the distinct advantage of providing considerable information regarding expected project performance for a broad range of hydrologic conditions. Goals and objectives of project studies are enhanced with RBA due to the ability to consider a much wider range of project alternatives.

6.3.4.4. Risk-Based Analysis and the Design Process

RBA is one component of a much larger process in development of a flood damage reduction project. While RBA provides the engineer with a wealth of information that is not available with traditional approaches, it is not intended to be a substitute for good engineering practice. The RBA, as discussed in the preceding section, is used to formulate the type and size of the optimal structural plan that will meet the study objectives. This plan maximizes the net benefits of all the alternatives evaluated. It may or may not be the recommended plan based on additional considerations. The RBA identifies the optimum plan and provides a starting point for the remainder of the design process.

An analysis of the residual risk for the formulated project must be performed to determine the consequences of a design exceedence. For a flood damage reduction project, the question is not IF the capacity will be exceeded, but what are the impacts WHEN that capacity is exceeded, in terms of both economics and the threat to human life! If the project induced and/or residual risk is unacceptable, and a design to reduce the risk cannot be developed, other alternatives must be further analyzed. Either a larger project, that will assure sufficient time for evacuation, or a different type of project, with less residual risk, should be selected to reduce the threat to life and property. To attain confidence that the outputs envisioned in the formulation of the selected project will be realized, specific design requirements are developed. For a levee, increments of height are calculated to provide for settlement and consolidation, allow for construction tolerances, and permit the building of a road along the crown for maintenance and access during flood fights. For a channel project, superelevation, if required to contain the design water surface profile, is determined. For a reservoir, allowances to accommodate the Inflow Design Flood without endangering the structure and providing vertical clearance for wind and wave action are estimated. These specific requirements must be included in the design.

When the type and size of the project have been selected, the detailed design can be completed. The design must include measures to minimize the adverse impacts of a design exceedence. In this type of discussion, it is useful to discuss special design considerations for a range of possible measures. For levees, the final grade is set so that initial overtopping will occur at the least hazardous location along the line of protection. This location is usually at the downstream end of the levee, so the protected area will fill in a gradual manner. This same approach is taken in the final design of channel projects. For reservoirs, the water control plan is developed such that the point of design exceedence is approached as a gradual increase of outflow from the project to provide time to initiate appropriate emergency measures downstream. Upstream diversions are also configured to allow a gradual increase in flow during a design exceedence. Notwithstanding these design efforts, it is normal practice to include a flood warning system in the final plan as a last measure for risk reduction.

Design of a flood damage reduction project places a special responsibility on the design engineer because of the potentially catastrophic consequences of a capacity exceedence. Of the types of structural projects usually considered in a flood damage reduction study, a levee is by far the most dangerous due to the severe consequences that may result from overtopping. If a levee cannot be designed to assure gradual filling of the protected area when the design is exceeded, then it simply should not be built. Reservoirs, channels and upstream diversions are generally better structural choices than levees. They provide some measure of protection even after their design is exceeded. These types of structural solutions are better suited to minimize the adverse impacts of a design exceedence since they can be designed and/or operated to effect a gradual increase in flows and inundation in the protected areas.

6.4. DEVELOPMENT OF DAM SYSTEMS

6.4.1. General

In order to develop and implement a comprehensive flood damage reduction plan for a river basin, a variety of structural and non-structural measures need to be considered. When implemented, the plan must assure a high degree of predictability with respect to individual and system-wide project performance during a major flood. Such a comprehensive plan should be founded on several fundamental principles. First, a high degree of protection must be provided to major urban areas, although a uniform standard need not necessarily be adopted. Concern for economics and residual risk needs to be fully considered. Secondly, protection for agricultural and other lower hazard areas should be evaluated on a reach specific-basis to establish appropriate protection requirements. Full consideration of how these elements will interact and compliment other system components needs to be addressed and understood. Thirdly, environmental protection and enhancement should be an integral part of the plan, or implementation will be difficult. Finally, the considerations above must be carefully integrated with non-structural elements, e.g. fair, universal and actuarially-sound flood insurance programs, buyouts and reclaiming the natural floodplain.

6.4.2. System-wide Objectives

The basic objective of planning for river basin development is selecting the system of reservoirs, which, in conjunction with other works and measures, will best serve the purpose of controlling the flood runoff. In developing such a system, consideration must be given to reservoir projects of large, intermediate and small size, other complimentary measures and to combinations thereof.

In the design and construction of dams, the application of sound engineering principles to actual field conditions will govern. However, as smaller dams are investigated, the types of embankments and structures that can be applied to an economic engineering solution become limited, and repeated application of a specific design may be acceptable. This is true provided the requirements at each individual site are satisfied.

6.4.3. Incidental Benefits

A system of dams controlling runoff for flood damage reduction in major urban areas may be able to provide flood protection for agricultural lands and smaller communities that could not be justified individually, particularly from floods of frequent occurrence. Such secondary considerations may afford an opportunity to provide flood protection to areas that cannot be economically protected by other means and should be considered in developing the system-wide plan.

6.4.4. System-wide Study

The principles involved in determining the functional capabilities and economic justification of a group of dams, either acting alone or in combination with other improvements for flood protection, are basically the same as those applicable in individual reservoir planning. However, in view of the many possible combinations of dams, as well as possible combinations with other types of flood protection measures, it is necessary to utilize simplified methods and criteria to screen multiple sites and combinations thereof. In conducting system-wide studies, the following steps using either traditional or risk based analysis approaches to set storage requirements are followed:

a) Investigate the historical nature and extent of flooding in each damage area over the full range of potential flood magnitudes. Estimate the damages that may be expected to result under anticipated future conditions. Divide the river basin into appropriate sub-areas, river reaches and damage centers.

b) Review historical flood characteristics and frequencies affecting each damage reach or center, and establish the relationships between floods of various magnitudes (frequencies). Estimate the damages to be expected from future occurrences. The flood analysis should include a determination of flood volumes and the hydrologic characteristics of runoff from all upstream areas.

c) From the flood analysis (hydrologic modelling), potential dam sites at various locations upstream from the damage centers or reaches under consideration should be identified, and trial plans developed considering the sources of runoff, routing of flood flows and the objective of achieving maximum net benefits. This plan might consist of a single large reservoir, a limited number of reservoirs of intermediate size, or a large number of smaller reservoirs, depending upon the problem or problems to be solved.

d) With appropriate flood routing studies, determine the reductions in river stage and discharge at each damage reach or center by the alternative reservoir plans. Estimate project benefits.

e) Using preliminary estimates of project costs and potential benefit estimates determine the benefit-to-cost ratio as determined from annualized costs and benefits. Determine the economic justification for flood damage reduction under study. By progressive trial adjustments in the number, size and grouping of reservoirs, determine the most economical and effective system plan to provide flood protection to the areas under consideration.

Prior to the selection of the final plan, a field reconnaissance investigation should be performed at each proposed project site by a qualified multi-discipline design team to reduce uncertainty and validate preliminary studies. While at the site, the following activities should be carried out:

a) Make a superficial examination of soils and rock and other physical characteristics of each proposed site. Determine suitability of location.

b) Assure that the selected dam sites minimize the taking of valuable lands and improvements, and do not require extensive relocation of upstream communities, transportation routes, communication facilities and underground utilities.

c) Identify tentative locations for spillways and outlets, and record indeterminate conditions that might increase construction costs.

d) Prepare a schematic layout showing all pertinent features of each proposed dam site and a tentative plan for each dam.

e) Inspect the areas and river reaches to be benefited by flood flow regulation and confirm the potential damage prevention and other benefits attributable to the dam.

f) Based on the previously formulated hydrologic requirements for flood storage, evaluate the approximate size of each proposed dam, the requirements for spillways and outlets, and record any site characteristics or physical limiting parameters that might affect the hydraulic functioning of the project.

6.4.5. Design Considerations

A preliminary concern during design is to minimize the potential adverse consequences of failure from the proposed facility. The basis for design criteria selection is essentially the risk that will be accepted by construction of the dam, first to downstream loss of life and property damage, and secondly, to loss of use of the facility with consequent loss of the investment in the project itself. There are ample records where failures of dams with even small heights and capacities have resulted in large hazards to life and heavy property damage. Therefore, it is essential that design criteria be developed to first consider the dimensions of the consequences of dam failure, and secondly to reflect flood damage reduction goals.

A large high hazard dam can be cited as the upper end of the scale insofar as avoidance of risk is concerned. For such a structure, design criteria should be selected on the basis that catastrophic loss of life is intolerable, that potential damages could approach disaster proportions, and therefore, failure must be minimized. At the other end of the scale would be a small dam built for protection of a low hazard area where failure would not jeopardize human life nor create damages beyond the capabilities of local ownership to recover. For such a structure, design criteria can be established on a less conservative basis. Between these two extremes there might appear to be a wide range of intermediate types of dams with corresponding criteria to govern their design, but such is not the case. In fact, only a small number of small, low risk dams exist. All others fall into a category where reasonable conservatism in design is essential. This is especially true in basins that are experiencing development and urban growth. Downstream populations will not remain static but will change over time during the life of the project. Therefore, periodic review of the downstream hazard potential is warranted as the requirement remains that a dam failure must not present a serious hazard to human life.

6.4.6. Design Criteria for Dams

The information developed in the formulation study and gathered during the field reconnaissance is used in preparation of final plans and cost estimates for each dam. Sound engineering principles governed by local site conditions and hydraulic requirements will result in the minimum cost for each dam.

The amount of controlled storage capacity for each dam should be determined by analysis of runoff characteristics and the requirements for degree of protection at each damage center selected in the formulation study. The storage capacity, downstream channel capacity and the design of the regulating outlet are very closely related, and one should not be adopted without considering the others. Inactive storage capacity is provided as an allowance for sediment accumulation over the life of the project. Flood control capacity must be adequate to regulate the reservoir design flood (flood based on meeting downstream stage reduction requirements) assuming appropriate antecedent conditions, and assuming the flood regulating outlets are operative.

Outlet capacity is selected based on consideration of the bankfull capacities of streams below the various individual and groups of dams in the system. Bankfull capacity is taken as that discharge at which appreciable flood damage begins. The location for the determination of bankfull capacity is within the reach of the stream below an individual dam in which the release from the dam would be relatively large. The final sizing of the outlet is chosen by also considering other factors such as storage evacuation requirements, care and diversion of water during construction and timing aspects of the stream network.

The capacity of the spillway for each dam is related directly to the magnitude of property damage and loss of human life that would result from failure of the dam during a severe flood. In all cases, risks likely to be associated with failure of the dam during floods should be minimized insofar as practicable by design to avoid sudden breaching. Spillway dimensions and structural designs should be conservative when the dam is located immediately upstream from an area of human habitation, there is any doubt as to the ability for evacuation during an emergency, or where potential heavy damage to property is evident. For the spillways design, the hydrological criteria and the inflow design flood are described in Section 6.3. The spillway site should be selected to minimize erosion from flow in the approach and discharge channels and to avoid impingement on the embankment. Uncontrolled saddle spillways removed from the embankment are preferred.

The type of dam will be established by determining the most economical structure that will perform the project functions. In general, the physical site conditions and local availability of construction materials will dictate the type of dam selected.

6.4.7. Gated versus Ungated Spillways

The determination of whether a spillway should be gated or ungated is site specific and must be made by the design engineer based on evaluation of many factors. Experience has shown that ungated spillways are generally preferable for single purpose flood damage reduction reservoirs because of their essentially fail-safe operation. However, it is not always possible to utilize an ungated spillway and still attain the desired degree of protection due to site specific considerations such as storage availability.

Specific factors to be considered in the selection of spillway type include but are not limited to : 1) water control management objectives, 2) dam safety, 3) economics, 4) topography or other site limitations, 5) rate of rise of the hydrograph during occurrence of major floods, 6) storage evacuation requirements, and 7) operation and maintenance requirements. General guidance on the selection of gated or ungated spillways is contained in references [6, 7, 8, 9, 10]. These references recognize that specific project considerations dictate whether selection of a gated or ungated spillway is most appropriate.

In the selection of spillway type, the following general observations should be considered:

- Ungated spillways need greater reservoir capacity to provide the same degree of protection.
- Ungated spillways cannot influence the reduction of peak flows nor flood wave travel times for large floods through modified operation.
- Ungated spillways provide a safer operation by not having mechanical operating elements. Therefore, under certain conditions - such as dams located in remote areas, dams that are difficult to maintain or dams in areas subject to flash floods - ungated spillways are preferable.
- Gated spillways require close inspection to assure safe operation of the gates during floods.
- Gated spillways allow flexibility in regulating large floods based on downstream flows and changing runoff conditions. Operational efficiency can be enhanced if flood-forecasting systems are installed in the basin, allowing,

for example, pre-flood release of stored water before peak inflow to the reservoir occurs.

6.5. PRINCIPLES OF RESERVOIR OPERATION

6.5.1. Operating Rules

In the design and operation of flood mitigation dams, well-defined operating rules that govern the functioning of the dam during the occurrence of major floods must be developed. Reservoir operation can be grouped according to the general methods shown in Fig 6.2.:

A. Free spillway (Ungated spillway)

B. Constant rate control method

C. Constant volume discharge method

D. Constant discharge method

E. Full control method

F. Bucket cut method

G. Advance discharge method

In the case of ungated spillways (Method A), it is not possible to control discharge volumes nor retention times after spillway flow begins. Gated spillways are required to control flow and operate according to Methods B, C, and D. In general, it is not possible to obtain total retention of the reservoir design flood, due to economic, social or environmental considerations. Thus, the full control method (E), or the bucket cut method (F), can only provide flood protection for floods having low return periods. In cases where flood forecasting systems are installed and provide reliable and timely information, it is possible to use methods with different schemes of advanced discharge (G).

The selection of an operating rule is site specific, and depends on many diverse factors such as : 1) Characteristics of the basin and typology of the floods, 2) Form and characteristics of the hydrograph for design levels of protection, 3) System operation of the various dams in the basin, 4) Existence of reliable real time flood forecasting systems, 5) Flood wave travel times, and 6) Warning times. In general, the operating objective is to pass as much of the flood as possible prior to storing water in order to retain flood mitigation storage volume. All operating rules must be developed considering downstream emergency action plans, including warning systems and evacuation.

In developing operating plans, the initial reservoir level for a single purpose flood mitigation dam is generally assumed to be very low. In the case of seasonal flood mitigation dams, the usual operation is to prescribe a specific amount of storage in the reservoir for control of floods during the months of the year in which floods normally occur. In both cases, after the passage of a flood the reservoir must returned to its original level as quickly as possible to provide the storage for the next

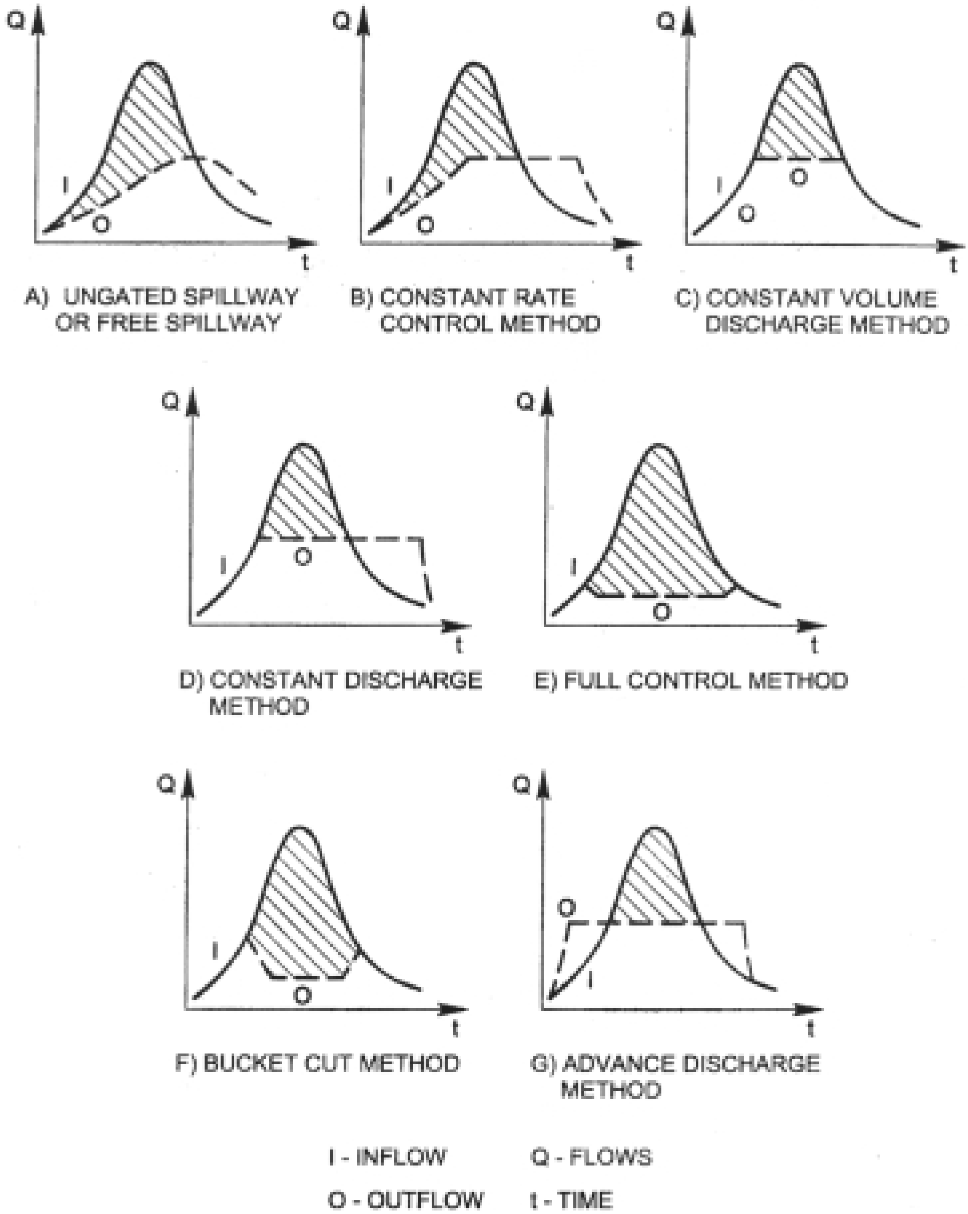


Fig. 6.2
Operation rules in flood mitigation dams

flood. This is accomplished by making maximum releases from the reservoir, considering flow conditions downstream.

6.5.2. Flood Control Measures

Control of floods by structural remedies, such as reservoirs, levees, diversions, channel improvements, etc., has long been pursued. In recent years non-structural means, such as flood plain zoning, flood proofing or flood insurance, limited buyouts, or floodplain reclamation have been incorporated into overall flood damage reduction plans to augment structural control. In the operational phase of all of these measures, the overall objective is to minimize flood damage in a given area. Most structural alternative measures to control floods and alleviate flood damage require specific water control plans based on current hydrometeorological conditions, flood control objectives, and the capabilities of appropriate flood control facilities. Unusually large floods may require flood fight activities at damage centers or other special measures to meet flood damage reduction objectives. Streamflow forecasting is therefore an important element in the management of water during floods, and timely use of flood forecasts provides a means for reducing damage by evacuating people and moveable goods from unprotected flood areas. The management of river systems must integrate the information on all aspects of flood damage reduction in order to best manage the control works.

While most levee projects do not require day-to-day water management, it is important that water control managers be properly apprised of the status of levee projects. This is particularly critical during times of major floods when the water managers should be alerted to any signs of weakness in any reach of the levee system. The water managers should disseminate to emergency personnel overall evaluation of flood hazard information in conjunction with the regulation of reservoirs or diversion structures. Also, flood-fighting activities, which involve special precautions to insure the safety and integrity of levees, require the coordinated efforts of many operational elements. The latest forecasts of river levels and anticipated potential for future flooding should be assessed and disseminated to field personnel. In some cases, special requirements are incorporated into the design of levees for placing closure structures (temporary bulkheads at street or highway crossings or sandbags at vulnerable locations) that insure the continuity of protection. Inasmuch as this must be accomplished ahead of the flood event, it is essential that actions taken to relay and utilize forecasts of anticipated river levels and flood potentials be coordinated with all responsible officials.

7. HYDROLOGICAL SAFETY OF EXISTING DAMS

7. 1. INTRODUCTION

Historical experience shows that extreme floods pose an important hazard for the safety of dams, and that this hydrological-hydraulic problem merits special attention both in the design of the dams, as well as in their operation during flood situations.

Numerous studies and statistical data exist on failures and incidents that occurred at large dams. The International Commission on Large Dams (ICOLD) has made a great effort to disseminate this information so that the international dam community can make the maximum possible use of the knowledge of the causes of large dam failures. ICOLD has done this with the aim to promote safer conditions for large dams [1-9].

The first global statistics by ICOLD of failures of dams were developed in 1974 [1]. In order to extend and bring up to date these studies of dam failures, and examine the validity of the analyses made in 1974, ICOLD formed a Committee on Dam Safety in 1982. This Committee in turn established a Subcommittee with a main objective to study the causes of dam failures. In 1988, in order to continue with this work, an “Ad Hoc” Committee was established to carry out the statistical interpretation of dam failures (ICOLD 'Ad Hoc’ Committee on Statistical Interpretation of Failures of Dams). This Committee published Bulletin No 99 in 1995 on the statistical analysis of the failure of dams [8]. The studies reported upon in this Bulletin make it possible to draw the following principal conclusions [10]:

1. The failures of large dams have significantly reduced during the last several decades. Whereas previously, until 1950, the percentage of failures was 2.3 %, the percent failure of dams constructed after 1950 has dropped to 0.5 %. This is a clear indication of the achievements attained in the field of dam safety.
2. As far as the type of the dams is concerned, almost 80 % of the failures refer to embankment dams, and the remaining 20 % correspond to concrete and masonry dams. Thus, embankment dams present a greater number of failures, the value of the ratio between number of failures and total number of dams being 70 % greater in the case of embankment dams than for concrete dams. Also, most of the failures (70 %) have occurred in dams having a height of less than 30 m. So, in absolute terms, the greater proportions of failures occur at relatively small dams, which represent the greater proportion of the dams constructed. It should, however, be mentioned that the ratio between the number of failures of dams of height H and the number of dams constructed of height H varies very little with the height.
3. The greater part of failures occur in “young” dams, with 70 % of failures occurring in the first 10 years of operation, of which 45 % correspond

to the first year. This indicates the high risk of failure and importance of the phase of first filling of the reservoir.

4. The most frequent cause of failure is "overtopping" which constitutes 36 % of the failures ; 87 % of these failures caused by overtopping have occurred in embankment dams.
5. In concrete or masonry dams the percentage of failures due to overtopping is 24 %, and in embankment dams this figure reaches 50 %.
6. The greater part of failures (73 %) has occurred in embankment dams having a height of less than 30 m, and due to overtopping. This represents 31 % of the total number of dam failures.

From the previous analysis of the studies and statistical data of failures the following general conclusions can be deduced:

1. Hydrological safety is an essential part of the safety of the dams, for which criteria of minimum risk must be adopted in the evaluation of the Design Flood.
2. Embankment dams are extremely vulnerable to overtopping during their operational life including the period of construction and first filling. Generally speaking, it is therefore necessary that embankment dams have greater hydrological safety than others, with the selection of a larger design flood, and with the adoption during their construction of suitable measures to prevent overtopping of the part of the dam already constructed.

During the last decades these conclusions have resulted in significant changes in the regulations and guidelines on dam safety of numerous countries. These regulations and guidelines now have more demanding conditions for hydrological safety and design flood assessing by using more conservative criteria than in the past.

Hydrological safety is a factor to be considered during all the phases of the life of the dam:

- In the design, with an appropriate selection of the Design Flood, and by using methods of calculation and hydrological simulation techniques adequate for the characteristics of the basin and for the analysis of extreme values ;
- During the construction period with a sufficient safety factor to determine the floods which will probably occur during that period, and with a corresponding rhythm of construction, and,
- During the operation period, with hydrological safety standards in the face of extreme floods, in which flood forecasting systems and emergency action plans can play a very important part.

In this Bulletin we are going to refer principally to the themes related to the hydrological safety of existing dams, developing the present situation of the basic criteria for its determination, and the new trends in the assessment of the Design Flood, which are basically the new concepts arising from the application of Risk Analysis.

7.2. PRESENT SITUATION AND NEW TRENDS IN THE ASSESSMENT OF THE DESIGN FLOOD

In 1992, ICOLD published Bulletin 82, the "Selection of Design Flood- Current Methods" [11], which still remains a basic reference. Therefore in this Section only the development of the criteria for design assessment which occurred during the last decade is discussed, together with an updating of the actual situation, and a description of the new criteria and methods which could be developed in future.

The criteria for the selection of the design flood employed prior to the 1980's are summarized in two enquiries published by ICOLD in 1969 and 1979 [12, 13]. The first showed that probabilistic methods were used in the majority of countries, while in the second enquiry it was observed that some countries were already using deterministic criteria and methods, with the calculation of the "Probable Maximum Flood" (PMF). Also, in a few countries the magnitude of the design flood was dependent on the type of the dam to be built, with a larger design flood for embankment dams.

Analyses of data from these two enquiries show the wide variability in the criteria employed in different countries. For example, very different methods of calculation were used, with return periods ranging from 200 to 10 000 years. In most cases, however, the design flood was the flood having a return period of 1000 years or the PMF. No doubt, the most outstanding feature was that the first generation criteria for the selection of the design flood were based on empirical and general considerations, and applicable to any dam and in any situation, without taking into account size or type of the dam, volume of reservoir, nor downstream hazards.

Obviously, dam safety is not only a general concept but it is also specific to each dam. Consequently, the fundamental philosophy of hydrological safety must be the relationship between the design flood and the downstream hazards, with particular attention to the safety of high hazard dams. This is why, in general, present criteria for the selection of the design flood (the second generation criteria) are based principally on the classification of dams according to the hazards that a potential failure would cause downstream (loss of life, economic losses, services affected, and social and environmental impacts). The first practical applications of these concepts were carried out from the year 1979 onwards with the publication by the US Army Corps of Engineers of their "Recommended Guidelines for the safety inspection of dams" [14]. Dams were classified according to their size into small, intermediate and large, and according to the downstream potential hazard, (loss of life and economic loss), into low, significant and high hazard. Once a dam has been classified according to these two factors (size and downstream hazard) a design flood is selected for that particular dam. For high hazard and large or intermediate dams the design flood is PMF.

In the last decades a great variety of criteria for the classification of dams have been derived from these concepts. Classification of dams has been based on their size, on the "dam factor" H x V (= height of dam times volume of reservoir), on the characteristics of the power plants, on the destination of the regulated water, or, in some cases, on general criteria applicable to all dams which present a high hazard. Nevertheless, in most cases the tendency is that it is the downstream hazard that provides the basic and fundamental safety criterion.

The current hazard Potential classification system in use by the U.S. Army Corps of Engineers (COE) [15] is a modification of the system developed under the National Dam Safety Inspection Act in the 1970's. Hazard classification systems currently in use by other Federal and state agencies in the U.S.A. are similarly structured. The COE system is shown in Table 7.1, and incorporates four categories as the basis for classification : 1) direct loss of life, 2) lifeline losses, 3) property losses, and 4) environmental losses. Dams are placed in one of three classifications indicating whether they represent a low, significant or high hazard to downstream residents. Note that all hazard classification systems reflect neither the hydrologic nor the structural adequacy of the dam itself. Rather, they are a measure of the potential for downstream catastrophic consequences should a dam failure occur.

Table 7.1
Hazard potential classification for civil works projects

CATEGORY [1]	**LOW**	**SIGNIFICANT**	**HIGH**
Direct Loss of Life[2]	None expected (due to rural location with no permanent structures for human habitation)	Uncertain (rural location with few residences and only transient or industrial development)	Certain (one or more extensive residential, commercial or industrial development)
Lifeline Losses[3]	No disruption of services – repairs are cosmetic or rapidly repairable damage	Disruption of essential facilities and access	Disruption of critical facilities and access
Property Losses[4]	Private agricultural lands, equipment and isolated buildings	Major public and private facilities	Extensive public and private facilities
Environmental Losses	Minimal incremental damage	Major mitigation required	Extensive mitigation cost or impossible to mitigate

Notes :

1. Categories are based upon project performance and do not apply to individual structures within a project.
2. Loss of life potential based upon inundation mapping of area downstream of the project. Analyses of loss of life potential should take into account the extent of development and associated population at risk, time of flood wave travel and warning time.
3. Indirect threats to life caused by the interruption of lifeline services due to project failure, or operation, *i.e.,* direct loss of (or access to) critical medical facilities or loss of water or power supply, communications, power supply, etc.
4. Direct economic impact of value of property damages to project facilities and down stream property and indirect economic impact due to loss of project.

In the U.S.A., State and Federal agencies have generally adopted a deterministic approach to establish hydrologic safety criteria. Federal guidelines [16], developed by an Federal Interagency Committee on Dam Safety (ICODS) task group and published by the Federal Emergency Management Agency (FEMA), for determining hydrologic safety requirements are deterministic but acknowledge the use of risk-based approach.

For high and significant hazard dams, the Probable Maximum Flood (PMF) is considered the upper limit in determining the Inflow Design Flood (IDF) which a given project must safely pass. The PMF is defined as the flood that may be expected from most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the drainage basin under study. For low hazard dams, the upper limit in determining the IDF may be established as a flood less than the PMF and is usually based strictly on economics. The approach taken to determine the appropriate IDF is referred to in the FEMA guidelines as an incremental hazard evaluation. The evaluation considers the incremental loss of life (LOL) that could be expected to occur downstream of the dam based on a comparison of with and without failure conditions. The selected IDF is that flood for which there is no increase in LOL between failure and non-failure conditions. The IDF is used to define the extent of remedial action, if any, that is needed to meet the hydrologic safety criteria. The flow charts in Fig. 7.1 and 7.2 [16] depict the steps in the incremental hazard evaluation.

Low hazard dams are often designed using a smaller IDF than would be required for significant or high hazard dams. However, with the possibility of development in the downstream areas occurring over time, low hazard dams should be periodically reviewed to determine if that classification should be changed. If changed conditions exist, the classification of the dam may need to be upgraded and the dam reevaluated using the more stringent criteria used to develop the IDF for significant and high hazard dams.

On the other hand, in the regulations of various countries [11, 17], the possibility exists of considering two floods in the design and operation of dams : The design flood and the extreme flood or safety check flood. The design flood is the flood that has to be taken into account in the hydraulic design of spillways and energy dissipating structures, with a safety margin provided by the freeboard. The safety check flood represents the most extreme flood conditions to which the dam could be subjected without failure, but also with a low safety margin (scenario limit). In this case, a limited overtopping may be permitted for concrete dams, but not for embankment dams.

From the above it follows that the standards and regulations for determining the design flood used in different countries lead to very different situations, which show that the selection of the design flood is not only governed by technical factors, but is influenced extensively by geographical, historical, cultural, social and political considerations. Thus, in some countries, for example France and Switzerland [18], there are no formal rules and, instead, they work with accepted practices that can be adapted to and modified for special situations. In other cases, recommendations and guidelines have been drawn up by scientific or technical organizations, or by governmental agencies, which, without being legally recognized, are generally accepted by dam engineers. This is, among others, the case in the USA [19], Great Britain [20], Australia [21, 22], Canada [23], India [24] and South Africa [25]. Finally, there are countries, which have legal standards (laws and regulations) for the selection of the design flood, such as Japan [26], Portugal [27] and Spain [28].

The criteria used from the 1980s (Second Generation Criteria) imposed a very important and positive change in the field of hydrological safety. However, these criteria have also been subjected to various criticisms among which the following must be mentioned [17]:

1. The dam hazard classifications are qualitative in general, without well-defined limits between the different classes, and without a clear and precise determination of the potential downstream hazards.

FLOWCHART 1--PROCEDURES FOR DETERMINING THE APPROPRIATE INFLOW DESIGN FLOOD (IDF) AND THE NEED FOR REMEDIAL ACTION

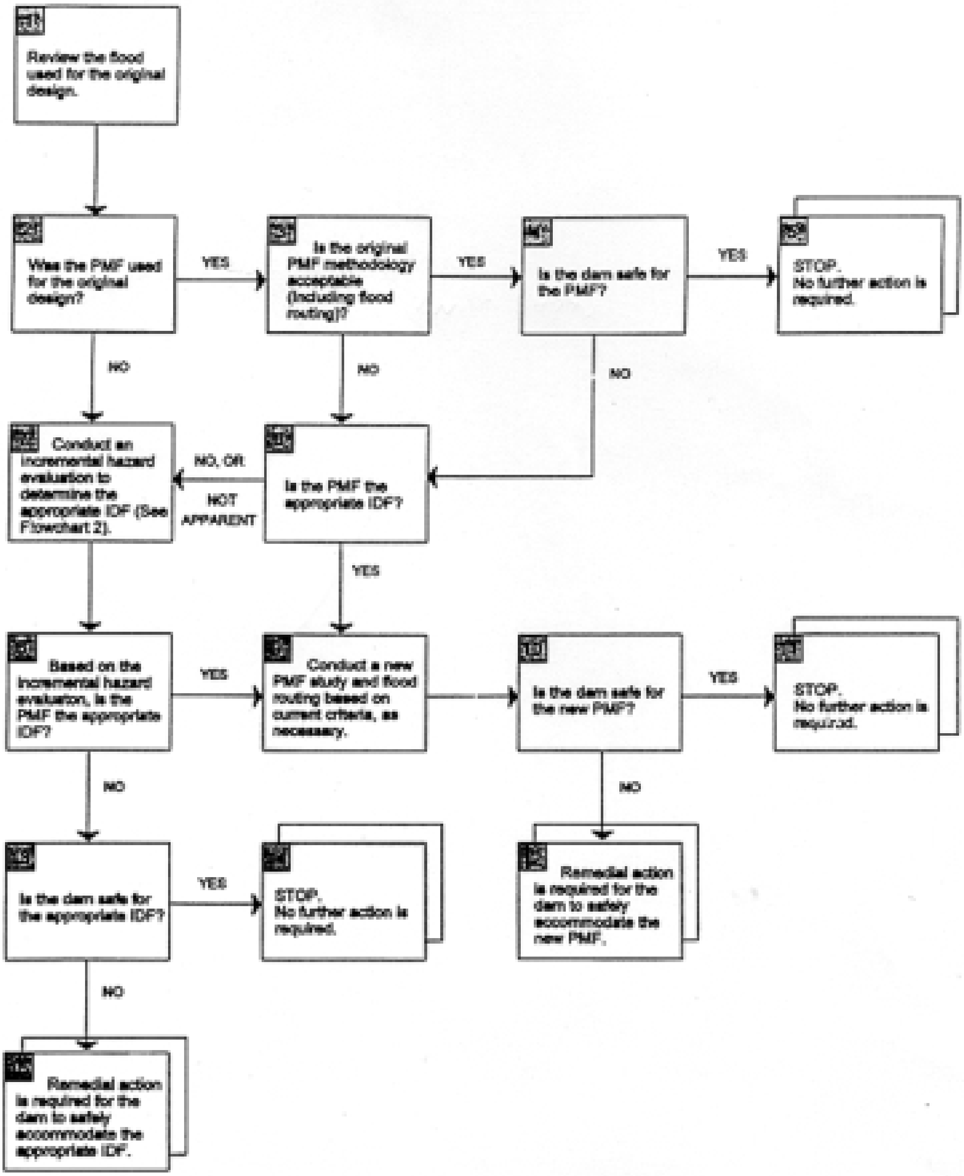


Fig. 7.1
Procedures for determining the appropiate inflow design flood (IDF) and the need for remedial action

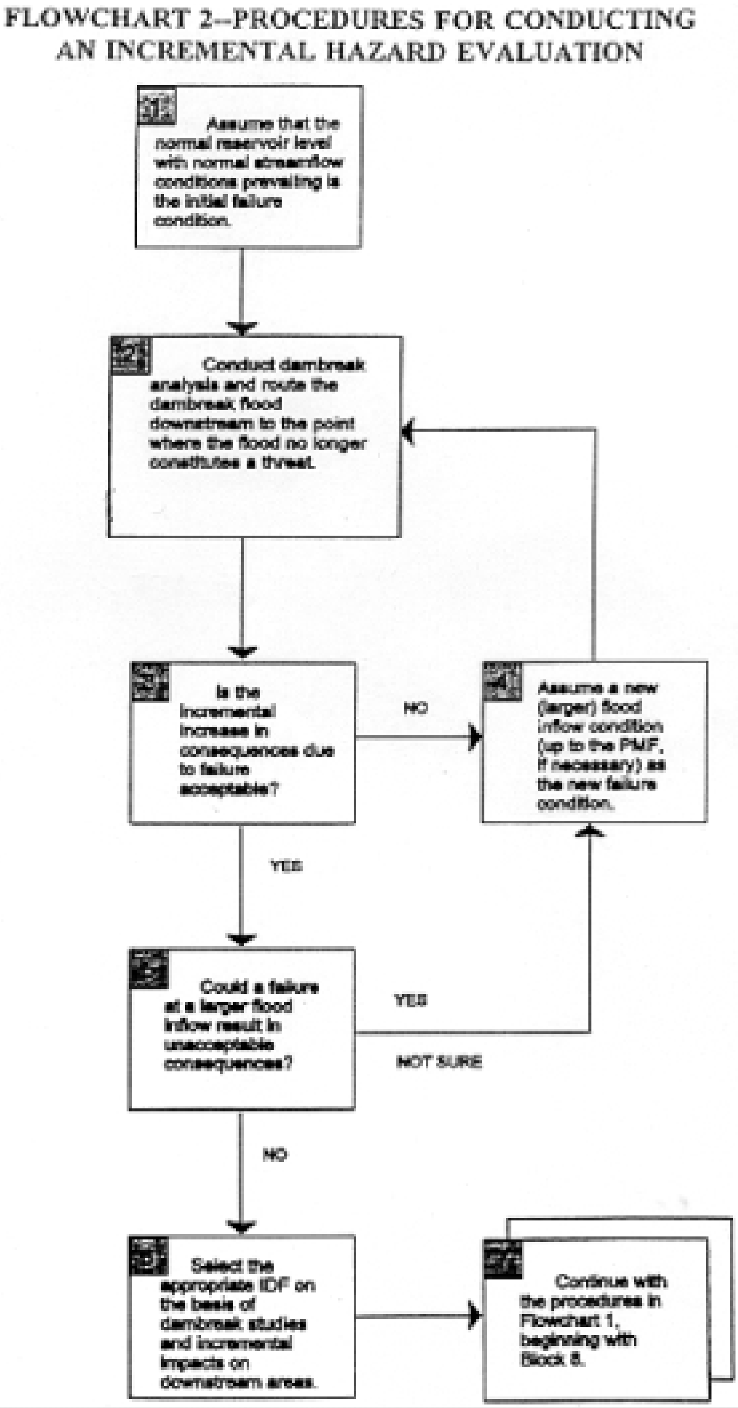


Fig. 7.2
Procedures for conducting an incremental hazard evaluation

2. The classifications contain very subjective components when, on the other hand, a large variety of criteria presently exist which have given rise to the formulation of a great number of classifications with a great variation in the selection of the design flood. This situation requires for recommendations with more homogeneous formulations.

3. Among the possible potential problems downstream, like loss of life, material damages and economic, social and environmental consequences, maximum attention should be paid to the loss of life, which has to be the determinant factor in the establishing of the different categories of dams.

4. PMF criteria used for high hazard dams are very conservative and represent a high cost, but provide a high degree of safety.

5. Classifying dams with the objective of selecting the design flood, has to be carried out with an evaluation of the incremental damages downstream produced by the additional flooding caused by dam failure, and with a comparison of these damages to those caused by the design flood without dam failure.

All these factors have given rise to a progressive development of criteria for the selection of the design flood. This development can be summarized as follows:

1. The general criteria, valid for any type of dam, are disappearing and are being replaced by formulations in which the principal criterion for assessing the design flood is the classification of a dam according to its potential downstream hazard (Dam Hazard Classification).

2. In general, the classification of dams is established in three categories (High, Significant and Low hazard dams), the potential loss of lives being the determining factors for the classification. The boundary between High hazard dams and Significant hazard dams is established quantitatively according to the loss of lives being equal to or higher than N, the value of N varying normally between 1 and 10 lives.

3. Maximum safety measures and standards are adopted for High hazard dams. In this case, the design flood is the PMF or a flood having a very long return period (10 000 years). In general (for historical, technical and professional reasons) two world trends have developed:

    1) USA, United Kingdom, Canada, Australia and countries under their economic and technological influence employ deterministic criteria and methods (PMF).

    2) Most European countries use probabilistic methods with return periods ranging from 1000 to 10 000 years for High hazard dams.

4. The selection of the design flood for Significant and Low hazard dams presents ample variations in the existing recommendations and guidelines, with various percentages of the PMF and different return periods. In the cases of significant hazard dams Economic Risk Analysis (ERA) could be used.

5. The possibility of considering two floods on the dams, the design flood and the extreme flood or safety check flood, is being recognized.

Generally speaking, in the majority of the cases, the design flood is selected in accordance with the values presented in Chapter 6, Table 6.1.

The practical application of these criteria has given rise to numerous controversies in their application, principally in their application to existing dams and in their accommodation to the high standards. So, from the analysis of the actual criteria, a series of problems and considerations have arisen which are based on the following points [29, 30]:

1. The standard approach is very conservative in the selection of the PMF for High hazard dams. This includes the necessity of important investments to accommodation existing dams. But, because of financial constraints, it is necessary to develop a methodology to determine the priorities of the actuations.

2. As has already been indicated, the actual criteria are diverse, and it is therefore desirable to formulate more uniform guidelines.

3. The selection of the design flood based on the Dam Hazard Classification distinguishes, in general, three categories of dams, (High, Significant and Low hazard dams), as a function of the potential consequences downstream. Nevertheless, this relationship is stepped and discontinuous, when it should be more progressive and continuous. For example, for High hazard dams, for which N (number of loss of life) is supposed to be between 1 and 10 (and in many cases N is actually equal to 1) the selection of the design flood is the same for N = 10, 1000, 10 000, or 100 000 lives. This implies that such High hazard dams are dams with a wide range of potential consequences due to their failure and that there is little discrimination in the classifications [31]. Therefore, the relationship between the hazard dam classification and its consequences downstream should be more direct and continuous.

From these basic considerations, new criteria and methodologies are being formulated for the selection of the design flood based on Risk Analysis by using a more rational and objective approach. This results in a set of new criteria (THIRD GENERATION CRITERIA) for the determination of the design flood. In Section 7.3 of this Chapter, the general concepts and the methodology of the risk-based approach to dam hydrological safety are described. Nevertheless, it must be emphasized that these new criteria based on Risk Analysis are still in a stage of development and their use has been limited so far to some cases only without a full application. For example, in the USA, State and Federal agencies have generally adopted a deterministic approach to establish hydrological safety criteria, although the USBR is presently developing and using a risk-based approach.

Accordingly, the evaluations based on Risk Analysis should at present not be considered as an alternative to the traditional criteria based on Dam Hazard Potential Classification, but rather as a complement, having its principal application in the evaluation of hydrological safety at existing dams, as discussed in Section 7.4.3. of this Chapter.

7.3. RISK-BASED APPROACH TO THE HYDROLOGICAL SAFETY OF DAMS

7.3.1. Flood Risk

All human activities are accompanied by risks, though risks may vary considerably in magnitude and consequences. Dams represent a typical human activity, as they are essential for the needs of the community. But by building a dam a risk is imposed as well, and failure of the dam may result in catastrophic damages and loss of life. Such failure can be due to various causes like natural events (floods, earthquakes), technical inadequacies, aging of the dam or human errors in management and/or operation.

Records show that dams do fail and especially floods, often combined with malfunction or misoperation of gates, are a major cause of failure, in particular the overtopping of embankment dams. It is not feasible, and not even possible, to safeguard a dam against all possible failure scenarios, such as extreme floods. Dam engineers, however, have generally failed to emphasize this fact to dam owners and the general public : that engineering is not an exact science but a combination of analysis, experience and engineering judgement. Accordingly, communities have generally not appreciated that the possibility of dam failure exists and this, in turn, has resulted in an unrealistic idea of safety.

Though, during the last decades, considerable improvements have been made to dam safety (most to minimize in as far as possible the potential hazards downstream), one should bear in mind that spillways in principle are designed to protect the dam against floods up to a specific design flood event. And while dams can provide some protection to a community by attenuating floods, it must be explained to that community that this does not provide a complete protection against any flood that could feasibly occur.

The problem facing dam owners and designers is therefore to determine the degree of hydrological safety, or level of safety, to be provided against floods, bearing in mind that extreme floods are random events which may or may not occur during the operating life of a dam of say 100 to 200 years.

7.3.2. Development of Risk Management for Hydrological Safety of Dams

In this Bulletin a preliminary inventory is made of general trends being developed for the introduction of risk assessment into dam and hydrological safety procedures. It is the intention that the new ICOLD-Committee on Dam Safety will prepare definite guidelines on risk assessment by the year 2003. In this respect it is noted that the two-day workshop on Risk-based Dam Safety Evaluations, held in Trondheim in June 1997, initiated the first international discussions about this topic while it provided information on world trends.

Risk management has been used in industrial applications for many years, particularly in the hazardous industries such as the chemical and nuclear industries [46, 47] and the methodology that is applied incorporates risk to life criteria and consideration of economic risk costs. This methodology is now being adapted to assist in the review of the safety of existing dams and related remedial needs.

Dam owners have recognized the need for introduction of risk management procedures in order to arrive at a more defensible and rational basis for dam hydrological safety. This approach is in line with the current emphasis on asset management with limited financial resources. In this manner the risk-based approach provides a more systematic basis for decision making in conjunction with the present deterministic engineering-based approach : any dam, albeit new or existing, can then be assessed in terms of relative risks and costs for the specific situation in which it is placed.

It is worth mentioning that risk assessment is a methodology which, in combination with expert engineering judgement, will assist in the overall understanding of dam safety. Within this context the decision maker will have all information at his disposal to assess the institutional, political, and psychological issues of a dam failure. Obviously, this is a major change from the present engineering approach based on which is only based on technical standards, while noting that with the uncertainties in the estimation of probabilities and risk to life, the outcomes should be used for reference and not taken as absolute.

The proceedings of the Trondheim Workshop [39] give information of developments in various countries. Information of particular interest can be found in papers from Australia [32, 33, 34], Canada [35, 36], Netherlands [37, 38], Norway [39], South Africa [25, 40, 41], Sweden [42], USA [43, 44, 45].

It must be emphasized that a risk approach to flood scenarios does not necessarily provide a specific solution for hydrologic safety, but it does provide information on spillway alternatives, consequences, and cost and benefits to be considered within the total context of asset management. The main use of risk assessment is for portfolio studies of the dams operated by an Authority to identify the critical safety issues and allocate priority ranking of the dams and for detailed studies. The detailed studies, in conjunction with normal deterministic studies, then provide comprehensive data for decisions, including authority business management and responsibilities, and applicable political issues.

7.3.3. Terminology

The terminology used in risk management varies, as indicated for the following key terms :

Hazard is the “development”, for instance, a dam which has the potential for creating adverse consequences. Hazard and hazard categories are common terms used in dam practices. Some define “hazard” as the threat (that is the dam, or a flood event) separate from “consequences” (that is the outcome, the impacts of a dam failure). Others incorporate consequences and rate a hazard according to a scale of adverse consequences.

Risk is the likelihood or probability of adverse consequences. Note that in hazardous industries, and also by many dam authorities, “risk” is taken as the combination (or product) of probability and consequences ; while others consider risk only as the probability of failure.

Probability is expressed either in terms of an annual exceedence probability (AEP), or as a return period in years. However, return periods, such as the 1000-year flood, can give a misleading impression to the community as being a flood only occurring every 1000 years, rather than as a risk of 1:1000 of occurring, or being exceeded, in any year.

Risk assessment (RA) is the total process, making use of risk analysis; embracing consideration of all costs and benefits including the identification of risks and likelihood of occurrence, and comparing these with acceptable risk levels or criteria to assist in evaluating risk reduction strategies.

Factors to consider in this respect for the hydrological safety of dams are taken as the incremental impacts of the dam failure on top of the impacts from natural floods passing through the reservoir before failure. (Some authorities take the incremental effect above theoretical natural flooding without the dam to incorporate the benefits from the dam in flood attenuation).

Risk management is the systematic application of management policies, procedures and practices to the tasks of identifying, analyzing, assessing, treating and monitoring risks.

Socially acceptable risk is defined as that low level of risk “society” finds tolerable such that expenditure would not normally be directed toward its reduction. However, these levels tend to be set by risk practitioners from a study of risks, rather than as a consensus from “society”, noting that “society” is inconsistent in reaction to risks.

Event tree analysis is a technique which describes the possible range and sequence of outcomes which may arise from an initiating event.

Fault tree analysis is a systems engineering method for representing the logical combinations of all possible initiating events resulting in a defined fault or failure.

One may say that fault tree analysis starts at the definition of a failure and tries to list events that could cause that failure to happen, while in-event tree analysis one tries to find all possible outcomes (including failures) which could be the result of a certain event.

7.3.4. Risk Management Concepts

7.3.4.1. General

Risk assessment, in conjunction with good engineering practice, presents the various possible options as based on a good understanding of critical dam scenarios, information on incremental costs, and other effects. Such information then assists, in conjunction with the deterministic assessments, in the selection of an acceptable level of risk. Because of the uncertainties in estimating probabilities and potencial loss of life, the mathematical outputs from a risk study, which can be voluminous, should be subject to regular checks and tests.

Selecting an acceptable level of risk by using risk assessment techniques can be conducted in a more efficient and reliable manner than before when decisions were based solely on traditional extreme load analysis and/or economic considerations.

The process is iterative and can therefore be lengthy and costly. Consequently, the process should be accompanied at each stage by an assessment of the necessity and cost effectiveness of additional analysis of a certain aspect in greater depth. Such an aspect can concern a study of the size of the dam, consequences of failure, the suitability of a site for spillway upgrading and finally, the legal and financial liabilities related to dam failure.

Risk assessment can be characterized as a change in dam design philosophy, which is a departure from the traditional engineering approach aiming at conservative safety, based on a limited number of isolated design aspects, towards a more comprehensive ranking of risks of failure for a number of options. Still, it is realized that it is not possible to assess absolute risks and, more in particular, those of natural events like floods.

7.3.4.2. Criteria

Risk to life criteria, based on those used for reference in the chemical and nuclear industry [51], are being considered but there is no consensus as yet on the criteria or the methodology or the applicability as a reference or a prescreptive standard, or on the legal standing. Preliminary considerations used more as a reference only. There can be cases where the economic losses and environmental impacts downstream from a major dam failure are so severe as to require an even lower risk level than the risk to life criterion. On the other hand, there can be cases where the risk to life and the economic damages are not severe, but the loss of the water supply for irrigation, human consumption or power generation would be catastrophic, and it therefore requires a low risk level.

The owner must also consider financial criteria in regard to financial capability to meet potential damages and liability costs of a dam failure. In the final decision, all these factors must be taken into account.

7.3.4.3. Application

The application of risk assessment could vary from country to country, and even within countries, depending on the local socio-economic, political and legal issues, the availability and requirements for asset and third party insurance and, last but not least, regulatory/legislative provisions. The application will also vary with the perspective of the party concerned. The owner's main concern may be cost effectiveness of the dam asset, while public authorities will be concerned with protection of the community. The community in turn will expect continuous safety and supply.

The decision maker should consider the quantifiable factors, such as risk to life and risk costs, in conjunction with intangible effects, such as social and environmental disruption, as well as legal and political issues and any regulatory requirements. This also includes the potential devastating effect on the community of the loss of water supply, with probably many years required to reconstruct the dam and/or restore full supply.

Ideally, the information on alternative options, including relative risks and costs, and trade-offs for other community needs would be presented to the community in the same way that many countries require an Environmental Impact Statement to be prepared and presented.

7.3.5. Risk Assessment Process.

7.3.5.1. Procedure

While there are differences in procedure between authorities, four basic phases, as shown in Fig 7.3 and based on [43], can be considered for the application of risk assessment to dam safety decision making. Initially, management should set the context in regard to corporate aims and responsibilities. The procedure is also called a Failure Mode, Effect, and Criticality Analysis (FMECA). The assumptions and steps should be documented for record and review.

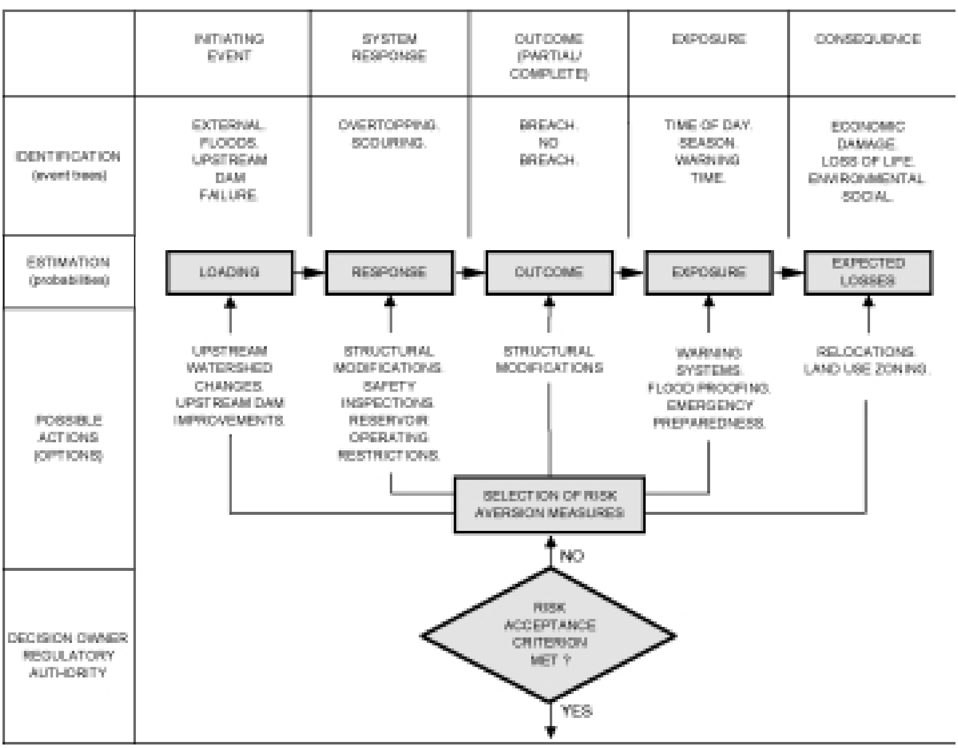


Fig. 7.3
Risk-Based Approach for Spillways (Adapted from [43])

The four basic phases concern the following:

- **Risk Identification.** An important initial phase to systematically identify flood failure modes and consequences. Event trees logically model and identify the critical failure modes.
- **Risk Estimation.** Allocation of probabilities to each branch of the event tree to assess the critical failure relative risk probabilities.
- **Risk Action.** Identification and evaluation of alternative remedial and rehabilitation actions for risk reduction, including structural and non-structural solutions, to assess the acceptability or non-acceptability of each alternative. Risk action is normally an iterative process by checking risk reduction alternatives, with associated changes to probabilities assigned to event tree branches, to assess optimum acceptable risk and cost effective solutions.
- **Decision.** Selection of the alternatives, which will effectively meet the relevant overall risk and consequence requirements in regard to all aspects.

A preliminary evaluation of load cases, degree of risk and consequences and possible alternatives can generally identify the critical factors for identification of potential inadequacies, and prioritization for detailed studies.

While flood probabilities are normally expressed as annual values, the risk over a longer period, such as the nominal life of a dam is the critical issue. Annual values can mask the recognition that even for a 20-year period, as a reasonable public recognition time, there is approximately a 20 % chance of a 1:100 AEP flood, and more than a 2 % chance of two such floods within 20 years.

An intermediate risk assessment can be made using the Base Safety Condition (BSC) procedure [48] also termed Zero Incremental Impact Flood (ZIF) to check whether the incremental effects of the dam failure become insignificant, or acceptable, at a smaller flood capacity than would be indicated by the risk criteria. This can then indicate a practical limit to upgrading.

Consideration can also be given to a conservative prescriptive fallback option, relative to hazard categories, following a preliminary assessment of consequences and order of costs of upgrading the spillway. This could be useful where owners have limited funds, the dam is small, or it is relatively easy to upgrade to pass extreme floods.

7.3.5.2. Event Trees

Event trees are a vital part of the risk assessment process and describe the possible range and sequence of outcomes from an initiating event. Fault trees are sometimes used and look at the combination of system states and causes that can contribute to a specified event. The basic aim of the tree analysis is the same as fault trees ; a tree analysis is a failure-driven approach that considers each dam as a unique entity to identify the principal failure scenarios over the whole range of flood loading.

Failure may, for instance, be due to inadequate spillway capacity with:

- Overtopping and breaching of an embankment dam, or unravelling of a rockfill dam.

- Stability failure of a concrete dam under the additional flood loading, or scour.

- Overtopping of spillway walls, erosion of the spillway stilling basin or toe of the dam causing undermining and failure of the dam.

The analysis, including all failure driven modes, may identify critical failure scenarios at lesser floods than the extreme flood; this can be due to factors such as gate misoperation or malfunction, or spillway blockage by debris, or severe scour downstream [49].

Trees can only provide relative probabilities, not a precise estimate of risk, but nevertheless are a powerful tool to be used in conjunction with engineering judgement, to assess the potential critical failure modes with a weighted assessment and relative probability of each mode. Accuracy is generally only required, and feasible, to within an order of magnitude to identify the relative risks for alternative failure modes. Accurate prediction of probabilities is not practicable due to the overall uncertainties in dam engineering and assignment of probabilities to floods. These trees can portray the relationship between events in time or logical sequence. Only those sequences, which could result in failure, need to be fully developed (Fig. 7.4).

A "think tank" approach using an experienced facilitator provides an effective basis for developing event trees [43]. A preliminary pathway analysis (critical path) will identify the steps and data required and timing. The assessment on a realistic basis of the conditional probabilities for the branches of the tree is important and requires expert input from the relevant areas of design, construction, operation and safety; including identification of the principal ways in which the specific dam could fail, and in selecting the key alternatives. A large number of alternatives may initially be identified, but only those sequences that could result in failure need be fully developed [50].

Consideration should be given to confidence limits of flood estimates for a sensitivity check due to the uncertainties in estimation. Salmon and Hartford [36] point out that variation in assigned probability, as well as detail of the tree, can make a significant difference of an order of magnitude of probability to the result.

The process for hydrological safety implies the development of risk models over the range of hydrological loading from start of spilling to the extreme flood, with estimated probabilities for each step. This includes a range of probabilities for different depths of overtopping, as distinct from the traditional engineering approach, such as for an embankment dam where it is usually conservatively assumed the dam will fail at the start of overtopping.

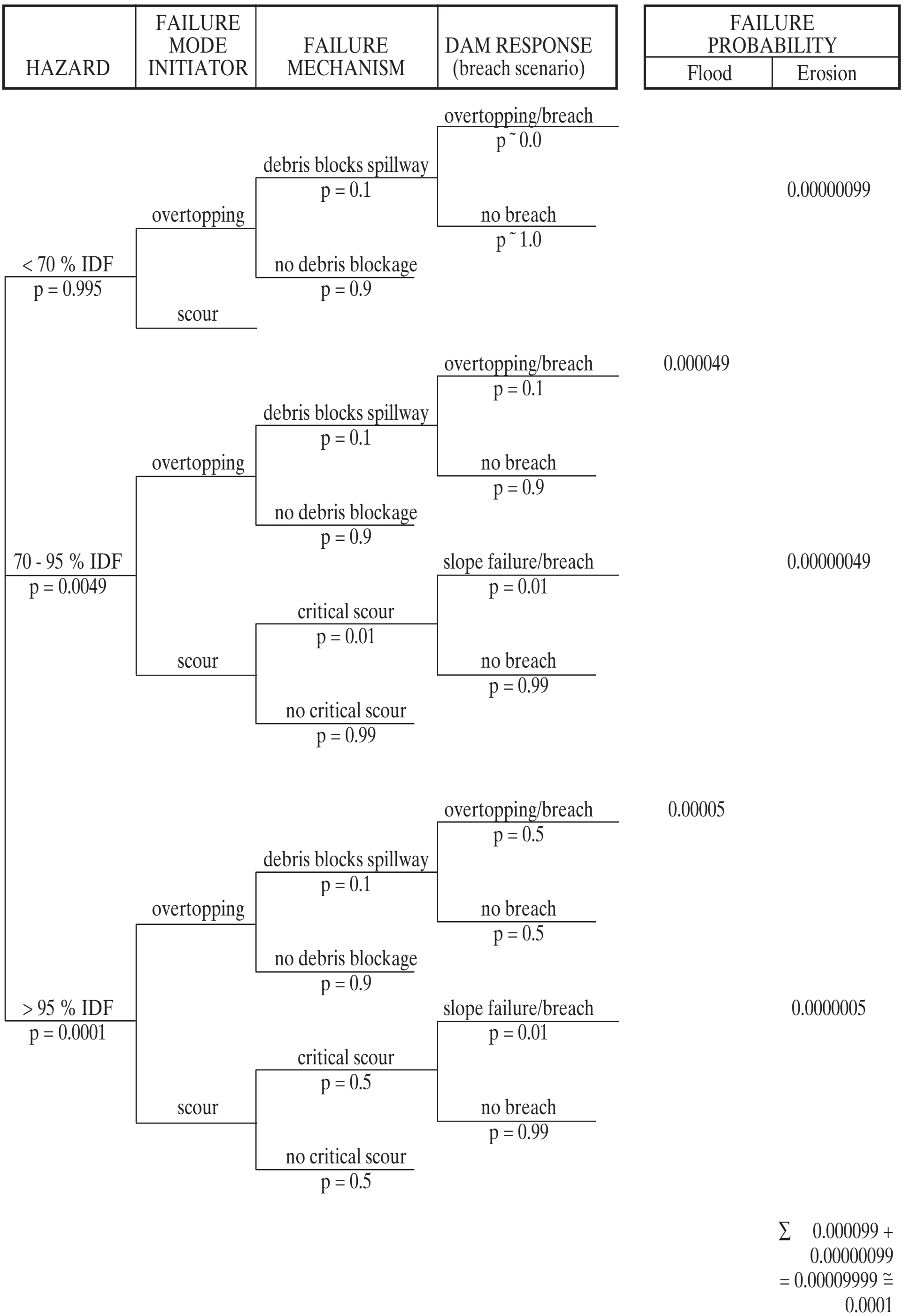


Fig. 7.4
Example of a hydrologic event tree for multiple failure modes (from [36])

7.3.6. Risk to Life Criteria.

7.3.6.1. General

The general approach in setting risk criteria is to relate to background levels of risks found in everyday life. Many statistics are available and Table 7.2 is indicative of a range of risk levels for accidents in the USA and a number of European countries [25, 51]. These risks are broadly categorized as :

- Everyday risks that are fundamentally unavoidable, such as lightning ;
- Risk theoretically avoidable but inherent in society, such as transportation ;
- Avoidable risk in that people exposes themselves by choice to gain a benefit, such as swimming.

Table 7.2
Indicative fatality statistics from western countries

Cause of death	Per million persons per year
All causes (including natural)	11 000
Accidents : All	460
Motor car	240
Falling	130
Fire	28
Drowning	24
Poisoning by other substances	15
Air transport	6.5
Water transport	5.3
Railway	4.3
Natural and environmental factors	3.8
Poisoning by medical drugs, etc.	3.5
Lightning	0.9

The overall basis used for assessing risk to life from dams normally recognizes two measures- individual and societal risk to life- with both measures to be satisfied. The societal risk criteria may be specific or within a range between an “Objective”(desirable level), and a “Limit” (maximum acceptable risk level), with the aim to meet the “Objective” criteria.

The acceptability of a dam with flood risk equal to or between the Limit and Objective can be assessed using the “as low as reasonably practicable” (ALARP) principle, developed for the UK Nuclear industry [51]. The adopted level should trend as far as practicable towards the Objective, to the extent where further risk reduction is considered impracticable, or the incremental cost for further risk reduction is grossly disproportionate to the improvement gained. There could also be cases where ALARP indicates the dam could be taken to even a lower risk than the Objective while still giving positive benefits.

The risk to life from a dam is an imposed risk, or involuntary risk ; the individual has no control over the hazard, or no perceived benefits. The community sees this as quite distinct from the many voluntary risks, some very high, that individuals take, such as driving a car, or hazardous sports, because they have perceived benefits. There are also natural risks such as from diseases and lightning.

It is generally accepted that imposed risks to life should be much lower than voluntary and natural risks such that the imposed risk does not add a significant order of risk to the normal background risks. The Netherlands has adopted 1 in 10 000 per annum (pa) as the background risk level, with an applied factor of 0.1 to 0.01 for imposed risk [38]. This is comparable with common industry criteria of 1 in 1 million annual risk as a limit for an individual.

The overall basis used for assessing risk to life from dams is related to individual risk to life criteria, and may include societal risk to life criteria, with in that case both measures to be satisfied.

7.3.6.2 Individual Risk to Life Criteria

Individual risk to life is defined as the total risk of death imposed by the dam on a particular person, an identifiable life. This is taken as the summation of risk from all failure modes, including floods, stability, seepage and piping, and earthquake. Criteria proposed vary between 1 in 1000 and 1 in 1 million per person per annum (pa).

Risks to life exceeding 1 in 1000 pa are generally considered unacceptable, and the Netherlands in consideration of imposed risks relative to the background risk level, have set individual risk criteria of 1 in 1 million pa for new plants, and 1 in 100 000 pa for existing plants [52]. The limit state design criteria for levees protecting Central Holland against natural hazards are 1 in 10 000 pa for overtopping and 1 in 100 000 pa for non-overtopping failure [38]. Australian Planning Regulations for hazardous industries generally indicate a limiting risk level of 1 in 1 million per person per annum for residential area exposure. The historical probability of loss of life from dam failures in USA due to all causes is said to be around 1 in 5000 pa [43].

The UK Health & Safety Executive [51] specify 1 in 10 000 pa as the maximum tolerable risk from any large industrial plant, “with the ALARP (as low as reasonably practicable) principle applying to ensure that the risk from most plants is lower or much lower”. The US Nuclear Regulatory Commission sets the maximum tolerable individual risk to life as 1 in 1 million pa.

Examples of individual risk to life criteria initially being proposed for dams are:

- Australia : ANCOLD, [32], dam failure from total causes:
    - 10^{-6} pa as an average risk to individuals over the population at risk
    - 10^{-5} pa as the maximum risk for the particular person most at risk with a proviso that a risk up to 10 times greater can be accepted for existing dams subject to ALARP.
- BC Hydro. Canada, has proposed risk criteria of 0.0001 pa, initially as an expected value, but then simply as risk, dropping the expected value concept.

- Netherlands'documents refer to general proposals for individual imposed risks from dams of 0.01 times background risk, *i.e.* 10^{-6} pa [43].
- South Africa SANCOLD [25] note indicative proposals for 1 in 1 million risk pa.
- Sweden: Graham [42] notes that literature indicates the potential for loss of life from a dam should not exceed 1 in 1 million pa, but notes the database on dam failures is small.
- USA: Bureau of Reclamation [45], are considering criteria for specific loads, with the justification for risk reduction diminishing as estimated risks are increasingly smaller than 0.001 average annual loss of life. However in regard to their portfolio of dams, they recommend each individual dam has a combined annual probability of failure of 1 in 10 000 to ensure an overall corporate safety level.

7.3.6.3. Societal Risk to Life Criteria.

Societal risk relates to the "public outrage" at multiple loss of life. It is only concerned with the total number of deaths N, regardless of whether they are identifiable or non-identifiable. The risk level is based on the cumulative annual probability as an excedence probability, F, of greater than or equal to N fatalities, plotted on F-N curves. The probability is the probability of the loading event/s combined with the probability of failure, given the event and probability of loss of life, given the failure. Indicative societal risk criteria for dams are shown in Fig 7.5.

There are different interpretations on the application of societal risk criteria, and plotting and interpretation of risk data. In Fig 7.5, the bases differ in that interim proposals from BC Hydro were to use expected value, which is the summation, for all load cases, of probability of dam failure multiplied by the estimated number of lives lost. Expected value gives an indication of the overall safety level, but not the critical cases. BC Hydro subsequently withdrew use of expected value.

Some countries interpret the societal risk for the total risk from all modes of failure, and for individual risk, with an overall consistent expected value risk of cumulative annual probability times loss of life at 0.001. Others interpret the criteria to apply to a specific mode of failure, such as floods, for a specific point of probability (f) versus loss of lives (N). Bowles [44] has used a plot of loading with cumulative probability, F, to give an overall picture of the critical loading situations.

Countries such as the Netherlands [52], and nuclear industry [46], specify a range between a "Limit", or maximum acceptable risk level, and an "Objective", or desirable risk level. The aim is to select an appropriate level within the range using the ALARP principle. Australia [34] is considering a F-N Limit curve from 1 life at 1 in 1000 cumulative annual probability, to 1000 lives at 1 in 1 million cumulative annual probability, with the Objective being an order of magnitude lower probability. South Africa Department of Water Affairs have developed societal risk relating to hours exposed and socio-economic losses to indicate relative risk levels of existing dams [40]. The setting and application of societal risk criteria requires further development and clarification.

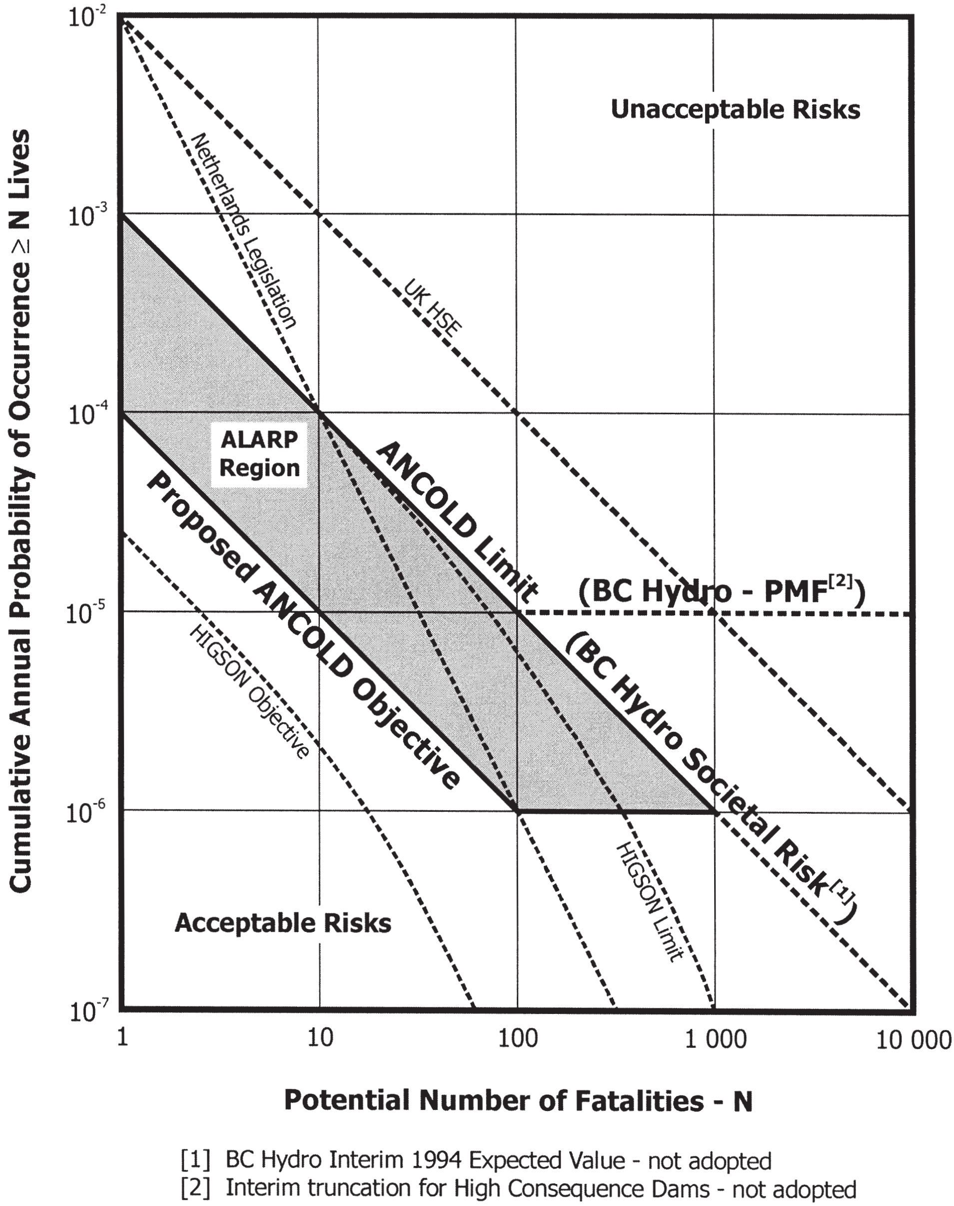


Fig. 7.5
Indicative Societal Risk Criteria

7.3.6.4. *Emergency Action Plans and Warning Systems*

Dambreak warning and evacuation plans should be developed as an extension of a general flood warning plan prepared with the emergency authority and the community. The warning system may include radio and TV bulletins ; telephone contact ; house-to house contact ; public alarms such as fixed sirens and mobile loud speakers ; and in-home alarms, although there appears to be a longer-term reaction developing against warning systems in the homes.

The Bureau of Reclamation, USA, and BC Hydro, Canada, have used Early Warning Systems (EWS) as an option for risk reduction and an alternative to upgrading spillways to full PMF. These were cases where it was considered EWS would provide a practical basis for evacuation of all people at risk, and it could reasonably be taken there would be no incremental loss of life. In some cases it was considered that the extent of flooding would require all people to be evacuated well before any dam break situation.

Proposals for Early Warning Systems should be coordinated with the regional/local emergency authorities to develop appropriate dam break warning and evacuation plans, with the public involved in the design and implementation of the system, and regular community education.

Australian Emergency Service Authorities have expressed concern on proposals for reliance by dam owners on such plans to reduce risk standards and expenditure based on the expectation that the plans will operate fully effectively, whereas in practice it is not feasible to guarantee effective performance. It can only be stated that an appropriate plan has been prepared, discussed with and accepted by the community and regularly reviewed.

7.3.7. Economic Factors

The identification and procedures for direct and indirect economic factors are well covered in [32, 43, 53]. Outputs can include:

- Benefit-cost ratios, net benefit and total economic cost ;
- Annual risk costs and annual probability of failure and fatality ;
- Incremental costs for damages, consequences and of reducing life loss ;
- Cost per life saved as an indicator of the cost per statistical life of providing safety, not a value of life, for comparison with other dams and industries [43]. These figures can be inconsistent between different threats, reflecting a community variability in valuing life according to factors such as dread, and impacts of the disaster.

Risk cost analyses are generally assessed on an normalized cost basis, which, combined with the relatively low probabilities for the extreme floods considered for dam safety, tend to indicate apparently low net direct cost benefits and very high costs to save a life. However a dam failure does not give a recurring very small annualized loss of life and damages, but a single catastrophic event, with once off total loss of life and damages. The relevancy of the annual cost basis to a single low probability, but high impact event, can be considered questionable [19]. The economic factors provide indicators for input to decision making, rather than absolute data. The uncertainties in estimation of floods and associated probabilities must also be recognized.

Corporate financial considerations may be a more critical factor than risk to life when consideration is given to the economic risk to the authority of asset loss and community damages costs. A financial risk criterion can be set to identify the safety level required to ensure financial risks are so low as to be negligible within the Authority's ability to meet such costs. If the authority has a portfolio of dams, the higher risk of any dam failing, and impact on corporate credibility should also be considered [45].

7.3.8. Environmental and Intangible Factors

Preservation of the environment is now recognized as a critical factor. The environmental impacts of a dam break should be related to the total environment comprising both the natural environment and ecosystems and the social environment relating to people. Reports on dam failures indicate likely severe and irreversible damages to the river valley environment. These impacts must be identified in the design and modification process, even though it is difficult to identify them in economic terms.

7.3.9. Legal Implications

The legal implications of adopting a risk based option which could represent a higher relative risk than traditional engineering standards will be dependent on the legal structure and the dam safety and regulatory provisions of each country. While there is no precedent to date, Courts in the USA and Australia tend to protect individuals, and have made judgements where there has been loss of life and that economic decisions have shown a callous disregard for life. [54, 55].

Court rulings quoted in [25] are representative and indicate on the one hand acceptance of financial limitations, "works ... should not be made impossible by prohibitively expensive protective measures", but on the other hand, "where rainfalls of great volume and severe intensity are common ... [it is] a duty.. to deal with flood water ... to provide a considerable margin of safety. An Act of God defense should not he upheld save on the clearest evidence."

Terms used such as negligence, peer practice, good faith, and reasonable practice are not strictly defined legally and are open to interpretation for specific cases. It appears that codes of practice and guidelines, even though qualified by statements such as "for guidance" and "not mandatory," will provide a basis for consideration by the Courts of reasonable practice. It would certainly appear that all decisions on a risk basis should be well documented with justifications relative to differences from codes of practice, and including data on financial restrictions and decisions on alternative uses and benefits to the community.

7.3.10. Summary

Risk assessment provides a logical, structured and comprehensive basis for assessment of dam safety. However it is still in an early stage of development and applications to date indicate the following areas requiring further research and development for improved and consistent outcomes:

- Allocation of probabilities to floods and failure modes ;
- Better and consistent definition of “socially acceptable risk” ; “reasonable” and “appropriate” levels of safety ; community expectations ;
- Estimation of possible loss of life ;
- Definition, presentation and interpretation of individual and societal risk to life criteria ;
- More in-depth analysis on the important impacts of human errors on design, construction, operation and management for inclusion in the risk process ;
- Ranking basis for social and environmental impacts ;
- Legal implications.

7.4. HYDROLOGICAL SAFETY OF EXISTING DAMS

7.4.1. General Overview

The adoption of new standards and regulations, and the progressive consideration of larger design floods can provide a greater safety for new dams. However, it poses a problem in its application to existing dams, and especially to the oldest dams which were designed using lower hydrological standards. For instance, during the application of the US Army Corps of Engineers National Dam Inspection Program in the 1980's , it was observed that of the 6800 dams in the USA, some 9000 were classified as high hazard dams, and of these one third were unsafe. Likewise, in the inspection of some 9000 non-federal dams, almost 25 % were designated as unsafe because of inadequate spillway capacity [56] On the other hand, the application of new and more demanding standards in dams can involve enormous costs, and although precise statistics from countries or other institutions do not exist, estimated costs are very high and funding does not exist to carry out the necessary modifications in order to adapt the dams to new requirements.

In view of this problem one may ask to what extent new demands concerning hydrological safety in the new regulations and guidelines should be applied to the existing dams. In general, the philosophy in the field of dams is that it is desirable to reach the same level of hydrological safety in existing dams as in new dams, which take into account the new criteria of design flood for safety evaluation. The problem is linked to the concept of risk management in which it is necessary to balance risk and global cost (economic and environmental). In some cases (due to the greater cost which the hydrological adaptations will require for the existing dams rather than for the new), various concepts of safety have been proposed, such as “extreme safe" for the new dams, and "very safe" for the existing dams. Nevertheless, because dams pose high potential risk and for the principle of social justice, the criterion that generally prevails is that it is desirable to demand the new hydrological standards for existing dams. For this reason in the majority of the laws and regulations of dams of different countries formal decisions have not yet been adopted for this problem, and existing dams are not distinguished from those new ones.

ICOLD Bulletin 59B, "Safety improvement of existing dams", provides a more detailed analysis of the criteria used in different countries for upgrading the safety of existing dams. Bulletin 59B indicates that "the criteria for determination of the design flood for an existing dam are intimately linked to safety philosophy", and, in general, the "Hydrologic deficiency of a dam requires re-analysis based on current standards" [57].

But the practical application of this philosophy and general criteria are fraught with real difficulties. Given this, the guidelines of some countries allow a certain "relaxation" of the criteria in cases where feasible alternatives do not exist for the accommodation of extreme floods due to technical, economic, social or environmental conditions [58].

The principles used to establish these "relaxations" are based on the following considerations:

1. The dam already exists, and therefore the people downstream already live with the preexisting hazard.
2. The abandonment of the dam could give rise to more unfavorable circumstances and in some cases, to greater risks due to the reduced flood protection to their increased frequency. Abandonment of the dam would also result in the loss of the benefits that the dam produces, which could be vital for the regional or national economy.
3. The economic costs for modifications are inaccessible.

Taking into account these considerations, and bearing in mind that (as indicated earlier) some countries accept relaxations in their guidelines, the most significant of these relaxations are briefly presented below.

In the USA, some agencies have explicit and different standards for existing dams, such as the Federal Energy Regulatory Agency, or some departments of the States of New York and Mississippi. Within these standards only 50 % of the PMF is demanded. The Federal Emergency Management Agency (FEMA) indicates in their "Federal Guidelines for Selecting and Accommodating Inflow Design Flood for Dams" that "the same flood selection guidelines for proposed dams should be applied to the maximum extent possible to existing dams." However, if this is not possible, the guidelines provide guidance values for the selection of the Inflow Design Flood (IDF). This, in reality, implies a relaxation with respect to the general criteria. For instance, for high hazard dams, the FEMA guidelines suggests exceptions in the cases of dams having reduced reservoir volumes compared to the volumes of flood, or in the cases where the PMF cannot be accommodated due to physical, economical or social conditions [59]. FEMA revised its Federal Guidelines in 1998 [16] to propose using the Incremental Hazard Evaluation for inflow design flood determination, which suggests that a flood less than PMF may be adopted as the inflow design flood in those situations where the consequences of dam failure, at flood flows larger than the selected IDF, are acceptable.

The Australian National Committee on Large Dams (ANCOLD) 1986 “Guidelines on Design Floods for Dams” [60] accepts that the hydrological safety of existing dams does not necessarily have to be the same as for proposed structures, since the option of not building the dam is no longer available. As a consequence thereof, exact compliance to the criteria for the Design Flood is not insisted upon in the evaluation of the safety of existing dams. Experience with these 1986 ANCOLD Flood Guidelines has shown the need for revision to integrate risk management procedures that will provide a more defensible and rational basis for selection of an acceptable dam flood capacity, while considering limited final resources (as is indicated in the Draft of the new Guidelines on Selection of Acceptable Flood Capacity for dams) [22].

In any case, one should be careful when applying these relaxations, and the relaxation(s) should only be accepted case by case, after conducting specific studies, and with the following conditions present where:

1. Possible alternatives do not exist and no funds are available for adaptation to the standards fixed for new dams.
2. The characteristics of the dam and of its areas upstream and downstream, do not permit technical solutions normally used for the construction of new dams.
3. The effects of abandonment of the dam have been considered in detail and such abandonment is not viable.

In conclusion, for application of the new criteria of hydrological safety to the existing dams one may consider the following points:

1. It is desirable to reach the same hydrological safety in existing dams as for new dams.
2. The general criterion adopted in the greater part of the regulations and guidelines of different countries does not formally distinguish between new dams and existing dams. This gives rise to problems in the practical application of the new regulations, when financing does not exist to make the necessary modifications.
3. The guidelines of some countries, in principle, permit relaxation of recommendations for evaluating the design flood. In certain cases only a percentage of the PMF is insisted upon, or more reduced return periods are accepted.
4. In cases where relaxations are admitted in high hazard dams, conservative criteria must be adopted and one should be very prudent and strict when accepting extensive relaxations. It is also necessary to carry out a site-specific study which takes the following factors into account:
    a) Performance of the dam in floods situations (real floods, levels, freeboards, operation of spillways, etc.).
    b) Dam classification. Review of the potential damages and consequences downstream, variations in the flood plain occupation, and detailed study and analysis of "incremental" damages for various floods.

c) Re-evaluation of the original designs and calculations, with comparisons to results using the new methodologies and recommendations for the evaluation of the design flood.

d) Analysis of the effects and adaptations for new floods in which safety is taken into account. Such effects could vary from a slight reduction of the freeboard demanded to an unacceptable overflow which would produce failure. Or, the analysis could concern intermediate effects such as overtopping (limited in depths and time) in concrete dams in scenario limits.

5. Carrying out a Risk assessment.

In practice and to prioritize the necessary actions to be carried out, two methodologies have been developed : (a) a ranking score has been established, and more recently (b) Risk Analyses have been applied. Both of which, as well as providing the basic data for the decision-making in the selection of the various alternatives for the adaptation of the existing dams, provide new criteria for the selection of the Design Flooda which generally requires a certain degree of relaxation from the existing standards.

7.4.2. Rankings Score. Application to the Evaluation of Existing Dams

To evaluate the relative safety of dams in a country, region or organization given the "normal" lack of economic resources, it is convenient to establish priorities based on dam safety rankings, that take dam characteristics and present conditions into account, in addition to the dam's maintenance and surveillance, and the potential downstream consequences of its failure.

The most general ranking is obtained by carrying out detailed studies for each case in order to arrive at a Risk Analysis (RA), but in many cases, the application of a rankings score, which implies the assignment of a global risk index to each dam, is sufficient. Among the existing rankings score one can refer to those proposed by Hagen [61] of the US Army Corps of Engineers, and that of the US Bureau of Reclamation in the Safety Evaluation of Existing Dams (SEED) program [62].

In 1982, Hagen proposed a simplified deterministic ranking system when considering a large number of dams that could be used to set priorities for remedial action. His proposed system was based on placing a numerical value on each of six separate elements that might be considered in a dam failure analysis. For each element, numerical rankings between 1 and 5 were set based on project studies, inspection records and other pertinent information. The six parameters he suggested are as follow:

1. Incremental difference between the number of residences downstream of the toe of the dam for the with and without failure conditions with the reservoir at maximum water surface elevation ;

2. Maximum flood expressed as a percentage of the PMF, that the project could safely pass ;

3. Structural resistance to overtopping-related failure ;

4. Number of residences at risk for a failure with the reservoir at normal pool elevation, the so-called “sunny day” failure ;

5. Degree of structural distress in the dam ; and

6. Susceptibility of the dam to seismic activity.

By then adding the numerical values for each of the six elements a composite ranking is established. This system can be used to set resource allocations by comparing the composite values of many projects.

In the USA, the Army Corps of Engineers has used for many years a modification of this ranking system, which only considers the hydrological portions. This particular system is relatively attractive because it is relatively simple to apply, it allows full consideration of engineering judgement and experience, it is fairly comprehensive in the elements it considers, and it gives reasonably uniform results. Each project is evaluated using the same straightforward criteria.

The application of the Rankings Score provides a semi-quantitative approximation of Dam Safety, and in particular of the hydrological safety, which although it has important subjective components based on engineering judgement, is, in general, very useful in practice and in determining priorities of a group of dams located in the same geographical area. Different organizations and countries have developed a number of Global Index Risk factors that permit the classification of the existing dams in relation to their relative safety and render priorities for the accommodations of certain existing dams to current safety criteria. These methods are, for instance, discussed in the Portugal’s codes of practice, the “Observation and Inspection of Dams” [63] and have been utilized in Switzerland [64] and Brazil [65], among other countries.

7.4.3. Application of Risk Analysis to Evaluate Hydrological Safety of Existing Dams

7.4.3.1. General

The aim of the risk-based approach is to provide a basis for the selection of an overall safe flood capacity relative to the consequences of dam failure during a flood. The process is iterative and interactive when identifying the critical flood failure scenarios by using event trees, and, subsequently, when selecting an appropriate probability flood with spillway options. The process includes assessment of the socially acceptable risk to life as a key factor, and overall consideration of the consequences of dam failure on the community and the owner.

The procedures are not exact and need to be applied with engineering judgement and experience, with consideration of cost effectiveness of alternative structural arrangements and non-structural solutions to meet the required risk to life criteria, as well as to minimize the economic, social and environmental issues.

A number of factors are relevant to the acceptable hydrological safety, including those related to the overall safety of the dam and protection of the community. Other factors relate to the hydraulic design of the spillway structures, the outlets, and the selection of freeboard for a range of operational and safety conditions.

7.4.3.2. Risk Procedures for Selecting the Design Flood

The risk-based assessment for consideration of the acceptable flood capacity can involve procedures ranging in degree of complexity, use of resources, and costs that depend on:

- Dam size, arrangement and site, with regard to practicality and incremental costs of providing or increasing flood capacity. These characteristics are site specific ;
- Extent of downstream developments and environmental significance, and the consequences of failure, including assessment of potential future developments ;
- Owner/community ability to cover costs of increased flood capacity on the one hand and damage costs on the other, community involvement of those threatened and those meeting the cost, and consideration of trade-offs with limited funds for other community needs ;
- Likely legal consequences on the owner, potential political, public and media reaction, and effect on owner credibility of a dam failure, and ;
- Availability and effectiveness of asset and third party insurance. Some major authorities are self-insured.

The re-evaluation of an existing dam flood capacity should be carried out periodically as part of the regular safety reviews, and should consider the latest design criteria and dam safety regulations, deterioration of the dam, and changed conditions in the valley, particularly downstream, which could change the consequences of failure. Reviews at 5 to 10 yearly intervals are generally desirable depending on the condition of the dam and the consequences of failure.

The procedures can be related to the risk assessment concepts defined in Section 7.3 and charted in Fig. 7.1. This can involve a number of steps given below, which may be iterative, and staged.

- Hazard/consequences assessment. Generally a detailed dam break analysis with inundation mapping will be required to assess the consequences and flood wave travel times for risk assessment, as well as for emergency planning ;
- Estimation of existing dam safe flood capacity and order of inadequacy to assess priority for investigations and remedial works related to regulatory, or fall back standards ;
- Event tree analyses to assess the most critical flood failure scenarios and relative probabilities to provide the bases for assessment of options and risk costs and risk to life ;
- Estimation of potential loss of life from assessment of emergency warning and evacuation plans and warning times, for comparison with risk-to-life criteria and assessment of risk reduction ;

- Assessment of a desirable safety flood, using the failure probabilities and possible loss of life estimates, to meet risk to life criteria, and consider options for providing this capacity ;
- Estimation of economic factors, risk costs etc, to consider incremental cost effectiveness ;
- Collation of all quantifiable risk data and non-quantifiable impacts, and cost effectiveness of alternative spillway and non-structural measures, to provide comprehensive information for making decisions that provide acceptable overall cost-effective safety levels. The final decision will also consider political issues and the question of authority perceived integrity and responsibility.

Each step can range from comprehensive, detailed procedures, to regulatory, or guideline standards as a fall back. This will depend on the impacts of failure (hazard), dam and site arrangements, incremental costs of increasing flood capacity, the range of alternatives and costs and trade-offs related to the owner's funding capabilities and community requirements, estimated resources and costs which would he required for the different phases of risk assessment studies, and the socio-economic conditions relevant to the State/Country. It would be desirable to have a public awareness program, with public involvement in considering risk, cost issues, trade-offs (alternative use of funds saved), and warning and evacuation plans.

At the present time, some countries, agencies or organizations are developing Guidelines based on using Risk Assessment for Design Floods and their application to existing dams. Among these countries Australia is cited, with its proposal of guidelines on Selection of Acceptable Flood Capacity [22], Norway, with its revision of the Regulations for Planning, Construction and Operation of Dams which allow for and encourage the use of systematic risk analyses [39], Sweden, with its new Dam Safety Guidelines and standards [42] and South Africa [40], with its DSA. Also, BC Hydro of Canada and the US Bureau of Reclamation are developing guidelines and different approaches based on Risk Assessment [35, 45, 53].

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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./