行政院所屬各機關因公出國人員出國報告書

（出國類別：研習）

研習美國水資源經營與管理

出國人：

服務機關：經濟部水利處南區水資源局

職稱姓名：副工程司　　　　　莫評任

出國地點：美國

出國期間：89 年 6 月 28 日至

89 年 12 月 23 日

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行政院研考會編號欄
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目錄

圖目錄

摘要

台灣南部係本省之農業與工業重心，因人口與經濟活動的增加，都會區發展迅速，都市及工業用水之需求日益迫切。目前雖已有若干水庫營運使用中，但由於先天降雨時空分佈不均，水庫蓄水仍不敷所需，因此缺水事件頻傳，嚴重影響經濟發展；加上台灣河川特性坡陡流急，於雨季時常常造成水患，且由於農牧業的廢水排放造成河川水源汙染，影響飲用水質甚巨。凡此種種皆為台灣南部地區迫切解決之問題，近年來環保意識抬頭，大型水資源開發計畫（如美濃水庫）往往受阻於民意；供水單位為了供應質佳且量足之水源，常常捉襟見肘，窘態盡出，為了解決此一連串問題，實應由整個水資源管理的方向著手，諸如防洪、給水與水質改善，由河川上游至下游做一完整之經營與管理。為求規劃時之完整性及正確性，擬引進國外先進國家之技術及觀念，因此擬訂此次出國研究實習計畫，以期至先進之西方國家—美國，學習其對於整體水資源經營與管理

之技術與經驗，以為日後工作之借鏡。

水資源經營與管理研習行程表

研習期間：2000 年 6 月 28 至 12 月 23 日

日期	研習地點	研習內容
2000 年 6 月 28 日	台北一舊金山	啟程
2000 年 6 月 28 日至 12 月 23 日	加州大學戴維斯分校	水資源經營與管理
2000 年 9 月 18 日至 9 月 22 日	科羅拉多州丹佛市	參觀訪問美國墾務局及其所轄水利設施
2000 年 10 月 23 日至 10 月 27 日	美國陸軍工兵團水文中心（戴維斯市）	研習水文頻率專業課程
2000 年 11 月 27 日至 12 月 1 日	加州蒙特利市	參加 2000 年加州水資源年會
2000 年 12 月 4 日至 12 月 8 日	美國陸軍工兵團水文中心（戴維斯市）	研習 GIS 在水資源管理上的運用
2000 年 12 月 23 日	舊金山一台北	歸國

研習人員名冊

姓名	服務單位	職稱
莫評任	經濟部水利處南區水資源局	副工程司

壹、目的

台灣地區地狹人稠，天然資源缺乏，惟賴經濟發展以立足國際；南部地區為本省之農業與工業重心，係台灣經濟發展之重要命脈，近年來由於人口與經濟活動的增加，都市發展迅速，都市及工業用水之需求日益迫切，目前雖已有若干水庫營運使用中，但由於先天上降雨分佈不均，水庫蓄水仍不敷所需；因此，缺水事件頻傳，對於經濟成長及居民生活均有負面之影響；加上台灣河川特性坡陡流急，於雨季時常常造成下游水患，且由於農牧業的廢水排放造成河川水源污染，影響飲用水質甚鉅；凡此種種皆為台灣南部地區迫切解決之問題。

近年來環保意識抬頭，大型水資源開發案（如美濃水庫）往往受阻於民意，供水單位為了供應質佳且量足的水源常常捉襟見肘；經濟部水利處南區水資源局有鑑於此一連串問題應從整個水資源管理方向著手，諸如防洪、給水與水質改善，由河川上游至下游做一完整之經營與管理，故擬定此次出國研習計畫，由本人至西方先進國家－美國學習其對於整體水資源經營與管理之經驗與觀念，以為日後工作之借鏡。

貳、過程

本計畫係依經濟部八十九年度人員出國研究實習案辦理，經本部甄試合格後通知本人辦理出國事宜，本次研習計畫係透過美國加州康郡防洪局資深工程師鄔寶琳博士安排至加州大學戴維斯分校（U.C　DAVIS）土木環工系從事水資源經營與管理之專題研究；經該校給予正式邀請函並同意本人以訪問學者之身分於該校跟隨 Devries 教授從事水資源方面之研究。Devries 教授為該校資深之水利專家，專長為水文數值模擬及水工模型試驗，並為該校水工試驗室之主持人；研習期間自八十九年六月二十八日至十二月二十三日，期間大部分時間於該校水工試驗室（圖一）接受 Devries 教授的指導，學習學理方面的知識，並於研習初期、中期及末期經該系及 Devries 教授之安排至相關單位參訪及參加課程訓練，以接觸實務之經驗。

由於 Devries 教授為加州水資源局之顧問，研習初期經其引見拜訪加州水資源局，並至其各部門參觀聽取簡報，其中包括防洪、發電、輸水及生態保育等部門，且參觀若干水利設施，如崔西（Tracy）抽水系統，美國第

一高土石壩—奧力佛壩（Oroville Dam）及佛森壩（Fosom Dam），以了解美國西部地區水資源之運用情形。

研習中期經該系安排於八十九年九月十八日至九月二十二日前往與水利處有技術合作關係之美國內政部墾務局（USBR）丹佛總部參觀訪問，於參訪期間經其安排參觀丹佛自來水公司，舒壯泉壩（Strotia Spring Reservoir）及墾務局工程中心，並聽取其對於水庫淤積問題之專題簡報；於研習末期經該校推薦至與其有建教合作關係之美國陸軍工兵團水文中心（US Army Corps of Engineers Hydraulic Engineering center）參與水文頻率分析及地理資訊系統在水資源上之運用二專業課程訓練，該中心為全美水文模擬系統發展之重鎮，從其訓練課程中受益良多；除此之外，本人亦在該校推薦下參加美國加州水資源年會，於五天的研討中深刻體會到整體水資源管理之重要。

以上參訪行程大致派有解說人員簡報並示範，以俾利深入了解其各項設施之功能及運轉情形，相關日程及參訪內容及感想分述如下：

一、加州大學戴維斯分校（UC DAVIS）研習期間

本次出國研習計畫主要研究地點為加州大學戴維斯分校土木及環工工程學系，期間再輔以專業課程及參觀訪問等實務性行程，研習期間從八十九年六月二十八日至十二月二十三日；加州大學為美國西部著名大學，其多所分校在學術上的地位都非常崇高，如洛杉機分校及柏克萊分校，戴維斯分校原為柏克萊分校之農學院，因學校日漸發展，終為一獨立之綜合大學，該校在美國人心中評價非常高，其農學院的科系在全美大學中均有不錯的排名；土木環工系培養出許多優秀人才亦有不少大師級的教授，該系水工試驗室歷史悠久，先後執行過許多重大水利工程計畫，Devries

教授更是該領域的專家，近年來他致力於水文系統的模擬，有很不錯的成績並有多篇論文發表，於研習期間承蒙 Devries 教授的指導對於水文模擬的技巧與觀念有更進一步的認識，且教授為美國加州水資源局之技術顧問，曾參與過加州許多水資源規劃工作，經其引導使本人更能進一步了解加州水資源的運作。

加州為美國第一大州，幅員廣闊人口眾多，農業與經濟活日益發達，對於水資源需求日殷，但因先天水文

條件之不良，造成多處地區水源不足；加州北部為傳統農業地區，雨量多集中於冬季，降雨量頗豐，於汛期時常常造成水患（圖二）；反之加州南部地區，屬於內陸沙漠型氣候，年降雨量非常稀少，加上全美第二大都會區洛杉磯位於此地，周遭近一千萬人口造成供水單位不少的壓力，如何解決此一問題為州政府迫切解決之重要議題；經拜訪加州水資源局及參觀相關水利設施後，對於整個加州地區的水資源運用有了更進一步的了解，原來多年前李前總統所提出的北水南引並非夢想，整個加州水資源運用模式即標準之北水南引；對整個加州水資源經營與管理實際上的操作，由防洪、輸水，生態維護，能源運用等項目分述如下：

（一）防洪：

由於區域水文特性，大部份的雨量集中於北加州地區（舊金山灣區以北）及喜愛拉內華達山區（sierra nevada），雨量最豐處達年雨量 3500 公厘以上，每逢雨季來臨時來自山區的暴雨逕流順延河流而下，往往造成下游莫大的傷害，有鑑於此，加州政府花費不少精力保護洪泛區；加州的河川特性與台灣河

川差異甚多，河川大多平緩，大部份河川均有航運功能，此區防洪重點在於如何紓解巨大的洪峰流量，因其河川坡度尚稱平緩且幅員廣闊，所以防洪單位有足夠的時間預測及準備；此區內幾條主要河川集水區域遼闊，雨季暴雨奔洩而下的水量相當驚人，為有效疏導洪峰避免瞬間鉅量洪流，美國政府於菲勒河（Feather River）（圖三）上游建造多座複合功能水壩，其中奧力佛壩（Orroville Dam）（圖四）為全美第一高土石壩，庫容量達43億立方公尺，其功能不僅能削減洪峰流量、延滯洪峰時間，並為整體加州水資源運用之總樞鈕；經水庫削減之洪流於下游地區輔以疏洪道排洪（圖五），更能有效紓解水患，此區於過去數十載發生過無數次嚴重水患，造成人民生命財產損失甚鉅，尤其加州首府沙加冕度市（Sacramentle），由於區內兩大主要河川交會於此，經年發生水患，Devries 教授就曾經主持過此區的防洪計畫；雖然水患無可避免，但經州政府多年來的努力及水利技術的進步，大部份水患均能有效的控制，其整個防洪系統最足以令人樂道的為即時

水文預報系統、洪泛區開發限制及疏洪道排洪政策（附件一），本人於研習期間 Devries 教授正在主持一個洪患區的疏散計畫，透過先進水文模擬工具 HEC-HMS 及 HEC1 配合 HEC-RAS 再加上 GIS 系統，使整個水文模擬結果更趨實用性，經 Devries 教授指導並實際操作使本人更能進一步了解水文模擬的技巧。

整個加州地區防洪工作的成功歸功於準確的水文預報系統

與地方救災組織的嚴密結合，能在第一時間掌握洪水的大小及災害的估計，更進一步能確切執行疏散計畫，整個加州防洪操作過程如下：

（1）每年模擬檢討洪患可能發生地區，擬定洪患限制發展地區，並透過立法嚴格實施。

（2）規劃出固定且容量足夠之疏洪區，以為洪水來臨時之疏導路徑。

（3）加築堤防以防高水位時之水患（美國地區堤防護岸大都採自然工法）

（4）透過國家氣象中心（National Weather Service）

與加州水資源局（Department of Water Resource）合作，由即時水文（降雨與河川水位等）資料，隨時掌握洪峰資訊以作最有效的指示，加州地區的水文預報系統為台灣所要學習的模式，雖然台灣地區已有多處集水區設有水文即時系統，但僅於小區域，未能於鄰近區域甚或跨流域連結且準確度有待提昇，故未能於洪水期間充分發揮有效的預警作用；因此意外事故頻傳，如八掌溪事件。

（5）於平時即未雨綢繆預估萬一洪水發生時之因應措施，各地區均有其疏散計畫且確實執行，以其洪水發生時損失降至最小。

心得：

其實世界各地解決水患方法不外乎分洪、減洪及消洪，但方法的運用與政策執行的徹底與否即為成敗的關鍵，洪水為自然現象，做再多的準備都無法完全避免，故僅能盡最大的努力以求最小的損失；加州地區防洪系統已臻完善，實為世界各國學習的對象，基於此點對於台灣的防洪工作有如下的

建議：

（1）台灣地區地文特性有異於美國加州，因河川特性，無法像加州大區域模式操作，應以局部地區為防洪重點，尤其以流域為分界。

（2）應限制洪患區土地開發，並嚴格實行以確保不再有另一個汐止鎮的水患地區。

（3）由於台灣地區土地成本高昂無法像美國加州地區有大區域的疏洪土地，故應配合河川行水區的管制開發加上堤防工程的購築以確保損失降至最低。

（4）全國水文資料庫的建立：美國地區之所以能有效的預測洪水除了徹底執行政策外，其豐富且精確的水文資料便成為其一大利器，美國水文資料均有特定單位收集整理並釋出於網路上供全國使用，如全美各地流量資料即為美國地質測量所（USGS）所量測整理，在美國水文資料量測工作者薪資非常高，其需要嚴格的訓練及專業的知識，台灣此類工作者的素質與之相差甚多，所以量測精度也相差甚多，往往造成水

資源規劃上的不準確度，且台灣地區尚未有一機構整合全部的水文資料，並有一套有效的更正系統，是故成立全國水文資料中心以為水資源永續發展之基礎。

（5）引進美國水文即時預報系統及長期預測技巧：台灣地區河川坡陡流急，降於山區雨量數小時內即奔洩入海，要有效掌握洪峰流量事件困難的工作，因此除藉水文預報系統之效用外一輔以長期水文預測結果，在美國往往有超過 100 年的水文資料可運用，藉著龐大的水文資料庫多少可歸類一相似的法則以為水文預測的基準。

（6）引進國外先進水文模擬技巧並培養專業人才：近年來歐美先進國家於水資源管理方面已逐漸偏向非結構性（Nonstructure Method）方法，其中模擬軟體的發展為主流，美國陸軍工兵團水文發展中心（Corps Of Engineer Hydraulic Engineering Center)為美國地區水文模擬軟體之發展重鎮，其發展出的水文模擬方法為全世界

廣泛使用，藉著模擬成果，該中心亦為美國水資源政策的重要諮詢者，本人於研習期間曾於該中心接受專業課程訓練，雖時間不長但受益良多，因有此感觸所以建議國內應積極培養此方面的人才，甚或成立一個類似的機構以發展適合我國使用的水文模擬模式，使台灣水資源的經營與管理更臻完善。

（二）輸水及發電

加州地區水源分佈極不平均，降水大多集中於北加州地區及與內華達州交界之喜愛拉內華達（Sieera Nevada）山區，此區人口並不多，人口集中的中央縱谷及南加州地區則有水資源不足的現象，尤以洛杉磯地區超過一千萬人口，僅有科羅拉多河河水供水資源運用，實不足以因應民生需求；加州政府有鑑於此於五十年代即開始著手全州水源供應計畫，由於北加州地區雨量豐沛，加上人口並不稠密且雨季時常常造成水患，是故如何有效利用剩餘的水量以供應有需求的地區遂為水資源單位重要的議題，加州地形為狹長形，南北距離近一千公里要將北方

的水引導至南方使用為一高難度技術，途中地形起伏不均更增添操作的困難，所幸美國政府起步很早加上執行徹底才有今日的成就，該地區輸、配水工程為吾人見過最偉大的水源工程（附件二），茲將其運作的方式與資料詳述於下：

整個加州水利工作為加州水資源局（DWR）執行，其中輸水工作由州水資源計畫處（SWP）負責，系統相關資料:

奧力佛水庫（OROVILLE DAM）為整個系統總樞鈕，主要供水地區有 DELTA、SAN LUIS、SAN JOAQUIN 和南加州地區（如圖六），供水人口約為二千萬人（加州人口的三分之二），其中儲水設備 32 座（總容量 72 億立方公尺），渠道及管線總長約 1065 公里，有五座水力發電工程（年輸出能量 49 億千瓦），加壓站 17 座（最高加壓高度 587 公尺）。

整個系統由北加州菲勒河（Feather River）上游的奧力佛水庫（Oroville Dam）蓄水控制（圖七），該水庫壩高 235 公尺，容量為 43 億立方公尺為一超大型

多功能水庫，除了防洪蓄水效益外尚有遊憩功能，其下游 Hyatt Powerplant（圖八）水力發電廠年產電量 49 億千瓦，為北加州地區能源提供不少的貢獻。來自水庫的釋放水量沿著菲勒河向西南方流下至舊金山灣區，此區為河流沖積而成之三角洲 DELTA 地區，灣北藉加壓站將水送至高程較高之 NAPA 地區 NAPA TURNOUT 水庫儲存以供當地農業使用（圖九），NAPA 地區為世界聞名的葡萄酒產地，其所生產的葡萄酒在世界大賽中屢獲大獎，由此可知該水源工程之貢獻；灣南地區游 Bethany 水庫集水統籌運用，一方面經過加壓站將水送至儲水桶以供使用，經一系列的輸水工程將水送至西南方 Santa Clara Terminal 水庫儲存使用，另一方面向南行供應此區南部地區（圖十），接續此段渠道為 San Luis 水庫（圖十一），此為 San Luis 地區供水之樞紐亦為加州地區北水南送之重要中繼站，該水庫集水區並無入流量，由於地勢較高所以來自北加州的水需加壓入庫；水續向南行至 San Joaquin 地區（圖十二），一部份向西行供應加州中南部沿海地區，另一路線續

向南行至本系統最終點南加州地區以供應洛杉磯及聖地牙哥地區（圖十三）。

本系統最引人樂道的為長距離的輸水渠道及大高差的加壓送水，由於加州地區幅員寬廣，多處地區空曠荒蕪所以輸水渠道用地的取得並不困難，但加壓送水實為一偉大工程（圖十四），雖然整個系統有多處水力發電廠但加壓站使用的電力更多，為了紓解用電負荷於空曠區設置多處風力發電廠（圖十五），此亦為加州地區景觀的特色。

心得：

在美國水為國家所擁有之天然資源，非個人或特殊團體可擁有，政府有足夠的權利調配水源的供給，州水資源計畫處（SWP）共和 29 個用水單位簽約，以確保各標的用水之不虞匱乏，如於枯旱年則可依不同標的的重要性調配供水，為符合公平原則，各地水價不一，水質亦不同，如南加州地區水價高於北加州地區，舊金山市區水質較其他地區優良（水源來資優勝美地山區）

建議：

（1）台灣地區土地成本高昂無法仿效美國北水南送的方式，故適採區域性調配。

（2）政府應立法保障水權，水為國家資源理應由政府統一調配，執政單局應展現魄力全面檢討各標的水源需要，收回全國水權統一調配。

（3）水價重新檢討：

台灣地區水價一致，但供水成本各地相差甚多，應回歸社會公平原則，使用者付費。

（4）調整水資源開發觀念：

對於抽水蓄存方式的水資源開發應予已考慮，並配合水力發電設施使水資源利用能永續發展。

（三）生態保護：

近年來全美環保意識抬頭，許多人不遺餘力的投入環境生態的保護的工作，使得這個議題愈發重要；水利工作影響最甚的為水域生態，近十年來美國水利設施的興建均配合生態的保育使的環境的衝擊減至最低，其中對於魚類的保護更是投入不少的經費，加州地區由於水文條件分配不均，有水患亦有

枯旱，洪泛區堤防護坡均採自然工法施做以美化環境回歸自然，為了供水建造了許多欄河堰及抽水站，為了保護迴游生物於水利設施旁建造了魚梯及復育場（圖十六），於抽水站附近建造魚類引導渠道及試驗場（圖十七）以保護其不受高壓抽水所傷，每年亦花費大量金額委託學術單位從事魚類保護工作。

心得及建議：

第一次參觀魚類保護場時感慨萬千，心想大概只有美國才做的出來，其為了生態保育所投入的經費令人咋舌；近年來台灣保育觀念抬頭時有人高喊環保，政府亦投入許多經費於此，但依我看來效用不大，以水利工程而言，每個水中構造物於設計規劃時多會加設一個魚梯以供於類迴游，立意很好但不切實際，因為不了解該河段的魚種，所以魚梯的作用僅是應付環保單位的疑問並無實際的效用，吾人建議政府應針對全台各溪流於類作一完整圖鑑資料，並針對不同魚種所需配合得水工構造物作一歸類以供水利工程師參考，雖然目前無法做到和美國

一樣的先進但基本的保護總是需要的，重要的是花錢一定要有效果。

二、參訪

在研習期間經該校土木系安排，至丹佛 USBR（美國內政部墾務局）總部及科羅拉多州水利設施地點參觀訪問，並事逢加州水資源年會，經該校推薦得以參加盛會。

（1）丹佛參訪行程：期間由八十九年九月十八日至二十二日，因事逢水利處人員拜訪墾務局，除拜訪 USBR 總部及丹佛字來水公司聽取相關簡報外（圖十八），並在 USBR 人員安排下參觀了舒壯泉水庫（Strotia Spring Dam）（圖十九）、墾務局工程中心（圖二十）及附近流域集水區。

心得：

於聽取其對水庫淤積問題的簡報及參觀若干水利設施後感慨很深，美國河川坡度大多平緩，水庫淤積問題並不嚴重，目前雖無立即迫切解決但該局早已針對其所轄的水庫進行淤積監測，且採最新 GPS 測量技術逐年監控，並時時改進以求精確；台灣河川坡陡流急水庫淤積

問題嚴重，加上水庫興建有愈來愈難的趨勢，如何提高現有水庫的使用能力實為目前台灣水利工程師所需迫切解決的首要議題；墾務局工程中心為全美頂尖之水利設計中心，許多著名的水利設施均出至於此（如胡佛壩），該中心試驗室非常壯觀，幾為全美各大水庫的縮版，但近年來生態環境逐漸為民眾所注重，故該中心目前有多項環境生態保育的計畫，顯示出未來水利工作的趨勢。

建議：

（1）政府應派員至美國墾務局學習水庫淤積測量的先進技術，並與其技術合作以吸取水庫永續經營之觀念。

（2）台灣大部份水庫均無排砂功能，於未來規劃水庫開發時應予已考慮排砂道的設計（如舒壯泉水庫）。

（3）政府應可仿照墾務局模式成立依功能健全的設計中心，將水利開發與生態環境結合，以因應未來的趨勢。

三、蒙特利加州水資源年會：

會議於八十九年十一月二十七日至十二月一日在蒙特力召開，參加人員主要為加州及鄰近地區水利工作

者、各地區義工協會、學者專家及政府人員，討論議題非常廣泛，從專業技術到地方百姓意見均有，並有水利義工於各地區流域努力成果發表，意義頗深，其中有一主題主講人為一農夫讓我感受深刻，整個年會討論內容並不僅於學術上的發表，更強調集水區保護觀念上的傳承與政府民間的溝通（圖二十一），真的有異於台灣的學術年會，僅侷限於論文的發表，對實際水利的貢獻並不大。

心得及建議：

其實我國也可仿效美國模式每年舉辦類似研討會，邀請各界與會並將會議內容導向於實務，政府可朝資助各地區環境保護組織，如美濃愛鄉協會、藍色東港溪協會等，鼓勵當地居民對其居住環境的關切，並定期舉辦整府與民間的溝通使人民了解政府的政策，且舉辦相關教育訓練讓民眾對水資源的經營與管理有更進一步的認識，使政府與民間的對立不再，以透明化的原則誠心與民眾溝通，並與民眾共同研究如何減輕水資源開發所帶來的環境衝擊，製造雙贏的契機，使的美濃水庫激烈抗爭的場面不再，政府實應未雨綢繆，及早準備。

四、美國陸軍工兵團水文中心訓練課程：

HEC 為全球著名之水文模擬系統發展中心，其發展出的水文應用軟體廣泛為全世界水利工程師所使用，其中以 HEC1 至 HEC6 等一系列的應用軟體最為有名，近年來由於大型水利設施開發困難，故如何有效運用現有水利設施達到最有效水資源運用為目前努力的目標，所以水文系統的模擬益行重要，該中心為水文模擬系統發展重鎮亦為近年來美國水資源政策的諮詢單位（附件三）；本人經加州大學推薦參加其水文頻率分析（附件四）及 GIS（附件五）於水資源經營上之運用二個專業課程，雖然只有短短二週但有幸與全美水利工作者同堂研習並交換意見學習先進的技術真是受益良多。

心得及建議：

HEC 水文中心網羅各地水利精英共同發展實用的模擬軟體，以供全美水利工作者使用，貢獻不凡，但其程式發展大多以美國本土水文狀況為依據，若運用於其他國家地區恐需些許修正，台灣雖有很多水利人員使用其發展軟體，但卻無人從事相關水文係數的修正工作，吾

曾經和 HEC5 程式的重要發展人 Rcchard Hyess 討論該程式應用於台灣南部水資源聯合用運的可能性，該程式為水資源供應模擬的通用程式，廣為全世界所運用，但經我描述台灣南部複雜的水資源系統亦令其訝異功能強大的 HEC5 竟無法解決我的問題，我們花了三天晚上討論這個問題，他亦給我許多寶貴的意見，使我萌生一想法一我國亦可比照美國模式成立類似的水文發展中心，針對台灣特殊的水文環境發展適用的程式，雖然投資會很大，但對於大型水庫開發困難的台灣如何有效運用現有水資源為當前最重要的工作，且水資源經營市永續的，我想該為後代子孫做點事吧！

參、心得

台灣雖已漸邁入開發國家之林，但對於水利工作仍有改進的必要，此次美國之行讓我感觸良多，其治水的觀念實有值得我們仿效之處，台灣地狹人稠，天然資源貧乏加上經濟發展，用水的急迫性可見一般；先天的水文條件不加讓台灣的治水工程多了一分困難性，每年汛期頻繁的水患（如八掌溪事件、基隆、汐止淹水等）困擾著政府與民間，雖天然災害無所避免但應盡力減輕損失於最低；台灣南部為農工業重鎮，人口日益稠密，都會區發展迅速，對於水源的需求甚般，但由於環保意識的抬頭，重大水資源開發案往往受阻於民意，使得未來供水目標執行充滿了不確定性；美國的水資源經營與管理政策實值得我們學習，水利工作事件吃力不討好的差事，須要長時間的努力才可見到實績，執政者往往疏忽了這一點，所以總是頭痛醫頭腳痛醫腳，無法作整體的規劃，加上公權力不彰，治水效率無法看到，政府應下定決心好好的作一通盤的考量，擬定一完整的水資源政策並確實執行，或許十年後才能見到實績，不管執政者為誰只要貫徹政策，相信很多目前所遭遇的水利問題均

可迎刃而解，為了後代子孫著想，此應為政府目前最急迫解決的事情。

肆、建議

此次赴美研習學到很多，視野以便廣了尤其是整體水資源經營與管理的觀念，使得吾人未來從事台灣水利工作時有另樣的思考方向，茲將吾所見所聞直得國內參考學習與改善的部份條列於後：

一、全國水文資料中心的建立－此為所有水利工作的起源，為了有效準確的掌握水文狀況，該中心的成立各不容緩。

二、各地環境保護協會的成立－仿效美國的模式由政府輔導資助其運作，鼓勵地方百姓參與，培養對鄉里國家的熱愛，亦為政府與民間重要的溝通橋樑，台灣地狹人稠，天然資源缺乏，政府與民間實不應資源開發立場的不同而對立，取而代隻的應為良性的溝通以求雙贏的局面。

三、人員的培訓－科技日益先進為了跟上時代潮流，實應鼓勵水利從業人員出國進修以廣見聞，並時常邀請國外專家來台授課以吸取其寶貴的經驗。

水利設施的串聯運用－由於環保意識的抬頭，大型水庫興建困難，為有效運用現有水源實應朝水資源

四、聯合運用方向努力，美國加州成功的案例給我們很大的信心，南部地區現有高屏溪堰與南化水庫聯合運用，將來努力的目標為全部水庫與欄河堰的串聯運用，以達最有效的水資源運用。

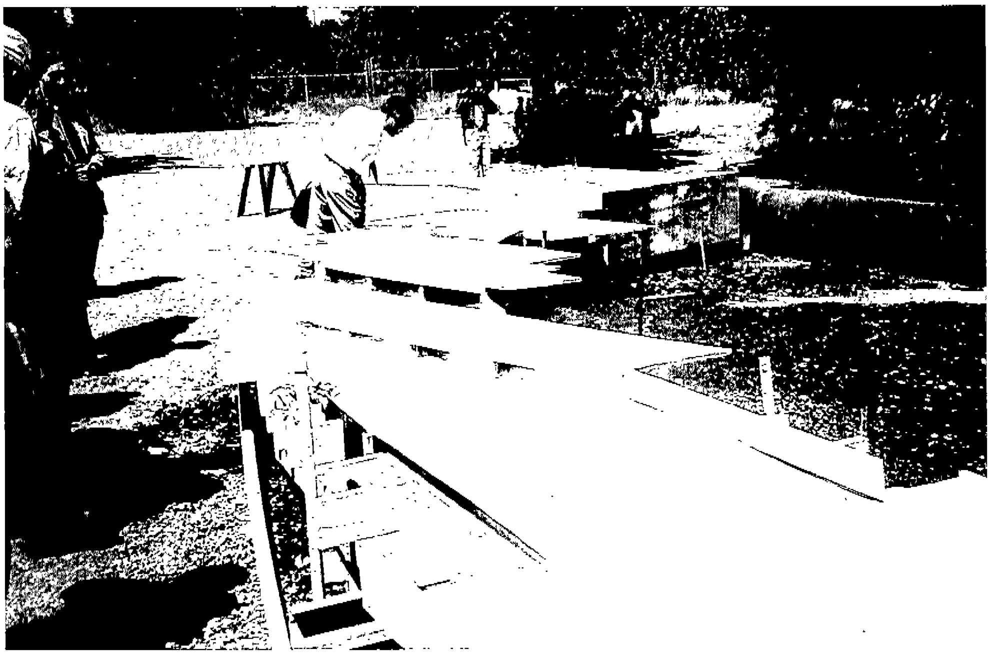
五、在工程設計上在安全無慮的條件下應多些自然的設計，使得工程完工後能吸引遊客參訪達成宣傳、教育等目標，畢竟，水資源保護應為全民的責任，教育是很重要的一環。

六、使用者付費的觀念－為符合公平正義原則合理的水價制定為目前重要工作，隨著水資源開發成本日益高漲，用水費用之調整為一刻不容緩的工作，台灣水價偏低民眾對水源的不珍惜造成供水單位常捉襟見肘，應該以價制量以達水資源用續利用的目標。

七、公權力的伸張－水為國家所有天然資產非個人或團體所擁有，尤其地方政府更是擁水自重，政府應發揮公權力收回並掌握全國水權統一調配，這樣才能雨露均沾，有效的運用水資源。

八、發展台灣自身適合的水文模式－水利技術日新月異，雖國際上各國發展很多，但水文條件很多因地而異，

各地有各地的水文特性，台灣應可仿效美國 HEC 水文中心發展本國適合的模式，以更精確的應用水資源。

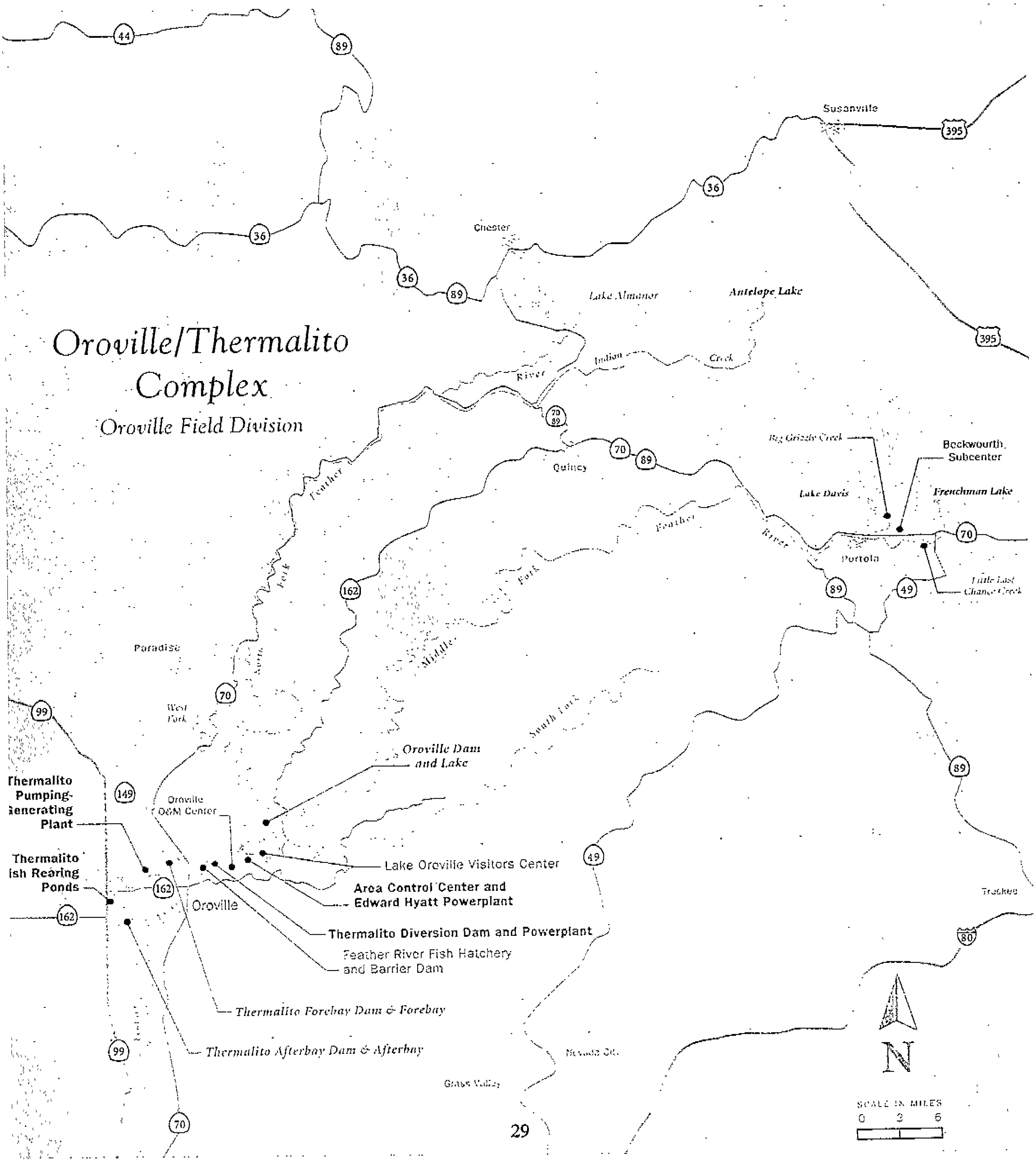


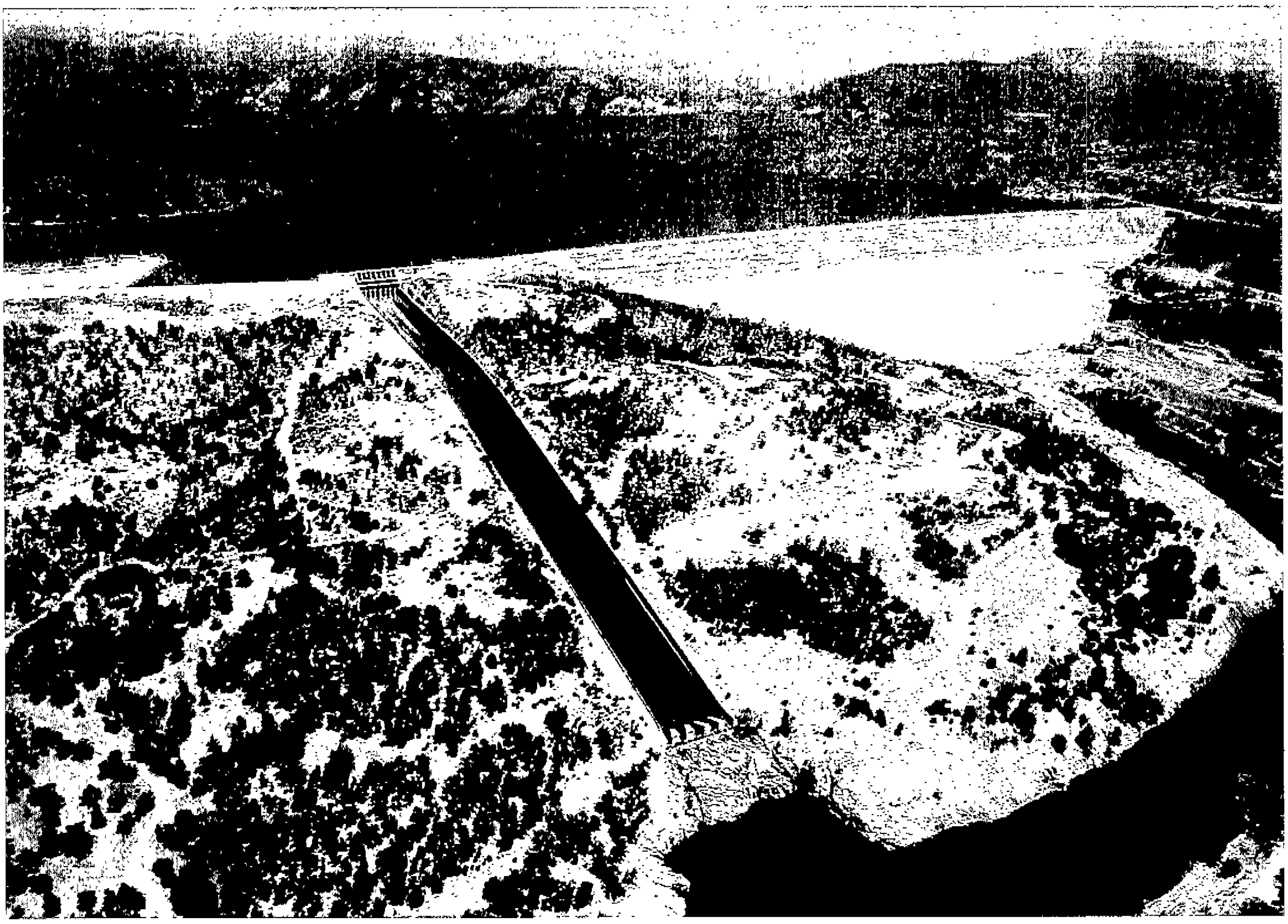
圖一 加州大學戴維斯分校水工試驗場

圖二　中央縱谷地區洪水景象

Floods were commonplace in the Central Valley The Flood of 1955 breached levees at Yuba City (above) and caused widespread destruction and more than 60 deaths Because the Oroville project would help control such floodwaters, the State Legislature quickly approved money in 1956 for its final design.

圖三　菲勒河（Feather River）上游水庫位置圖

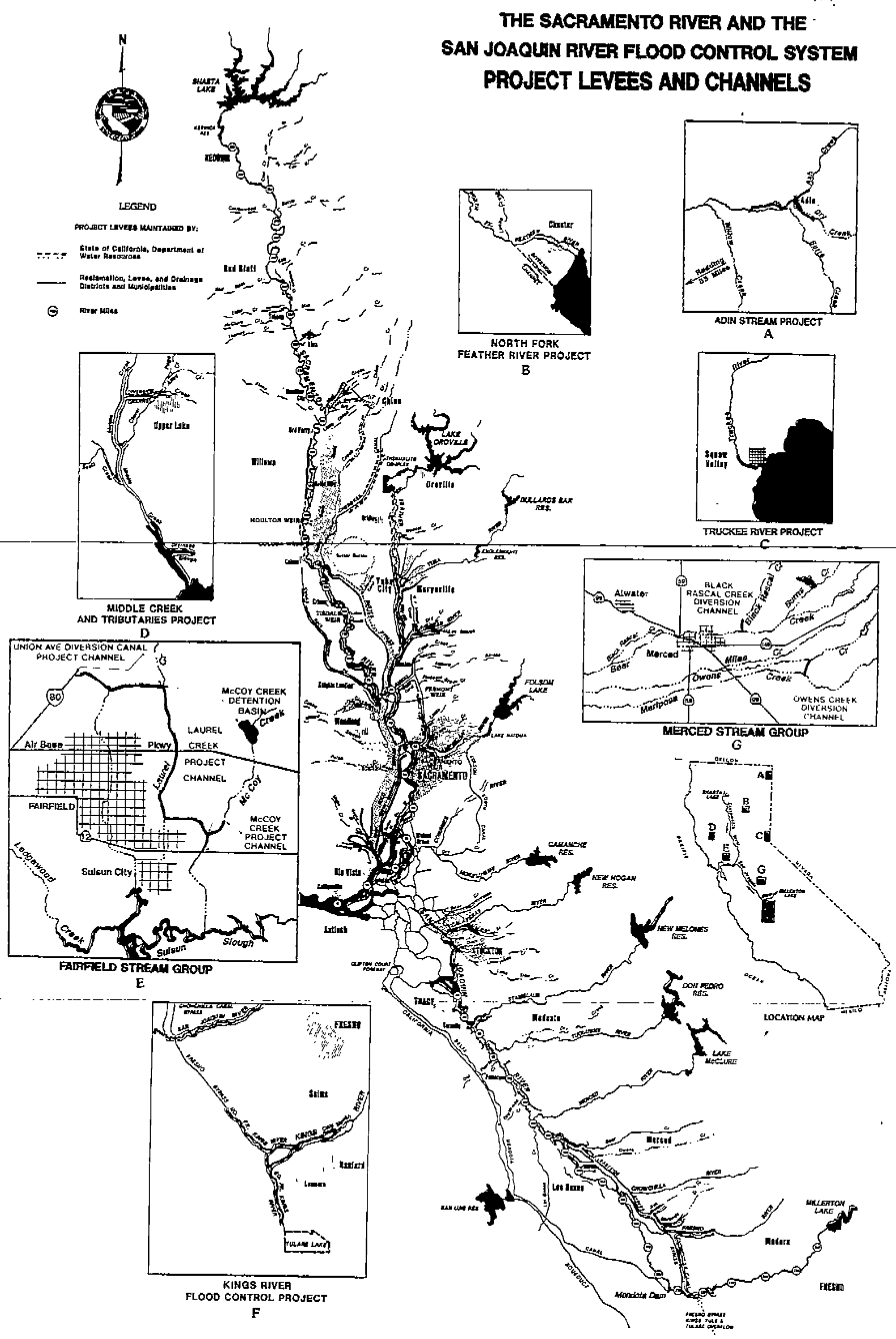


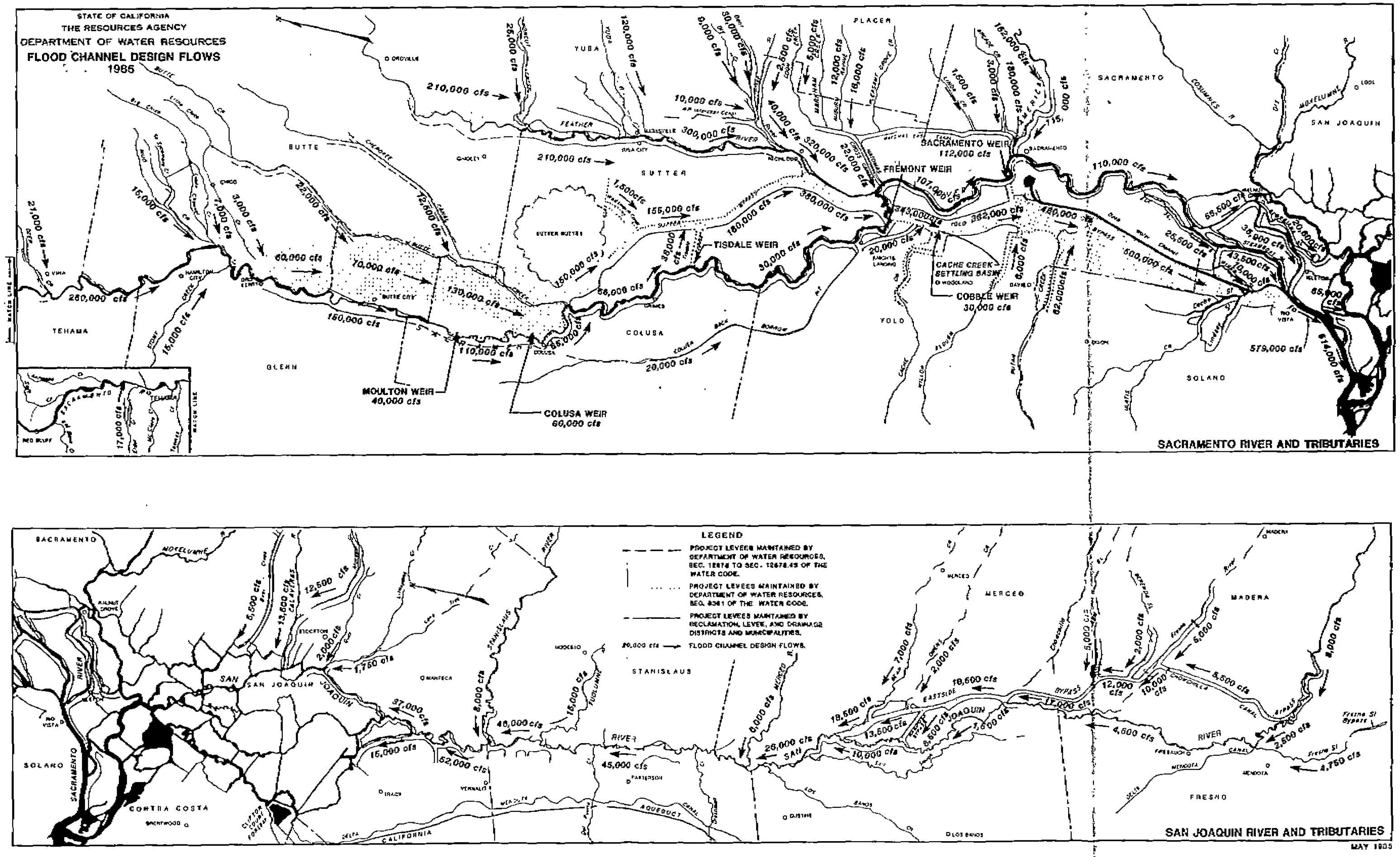


Construction of the Oroville complex on the Feather River started in 1957 and ended in 1971. The dam itself took nearly six years to complete (1962-68) Part of it was building the 283,000-cubic-yard concrete core block (bottom right photo) and preparing the foundation for the main dam (bottom left photo) Final cost of the dam was $135.3 million when DWR signed the completion documents on April 26, 1968. Today, Lake Oroville (top photo) is the SWP's largest reservoir and the second largest reservoir in California.

圖四　奥力佛壩（Oroville Dam）

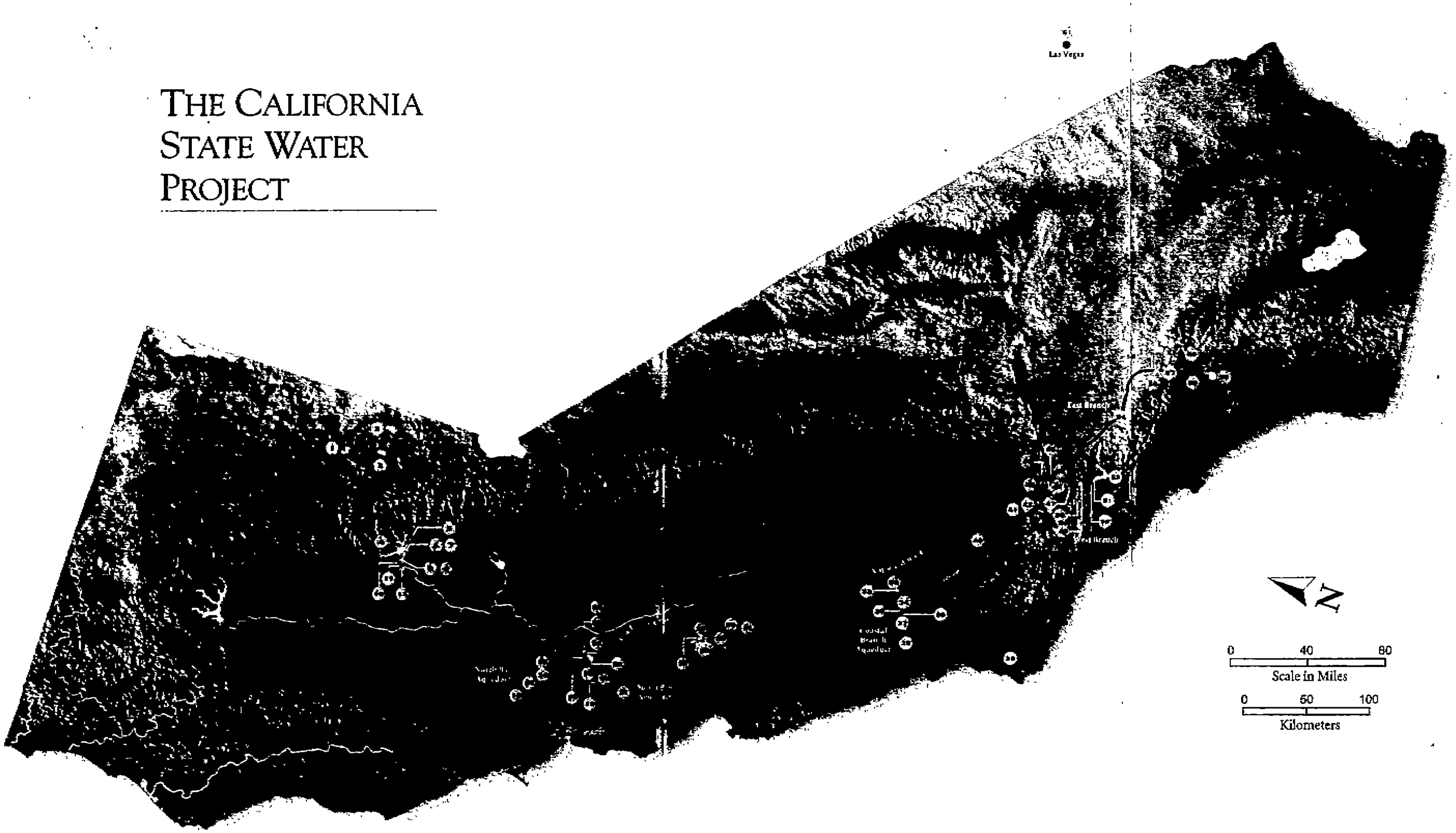
圖五之一　Sacramento 和 San Joaquin 防洪系統圖





圖五之二　Sacramento 和 San Joaquin 防洪系統圖

THE CALIFORNIA
STATE WATER
PROJECT



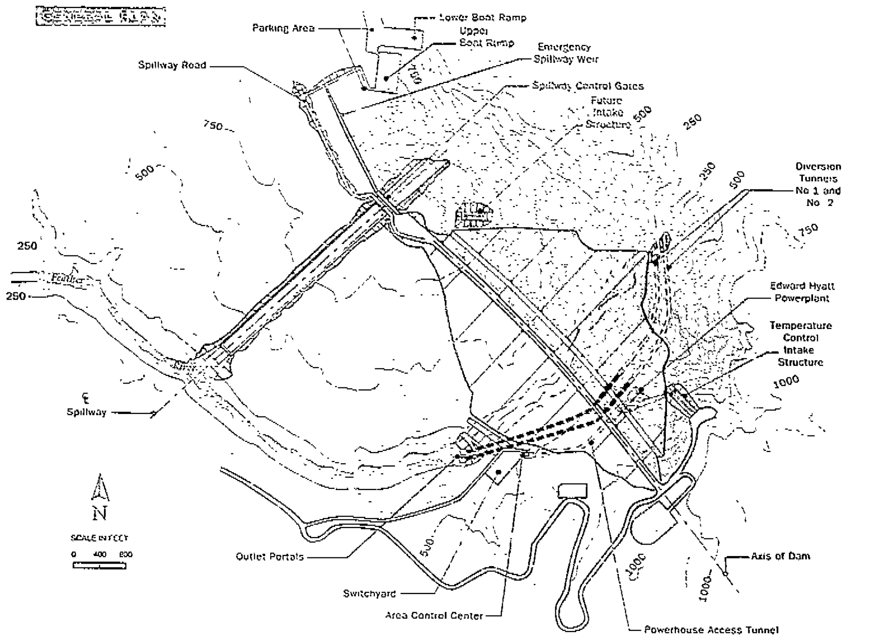
圖六　加州地區供水系統圖

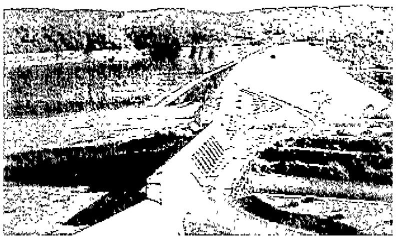
圖七　奧力佛水壩（Oroville Dam）平面圖

OROVILLE DAM & LAKE OROVILLE

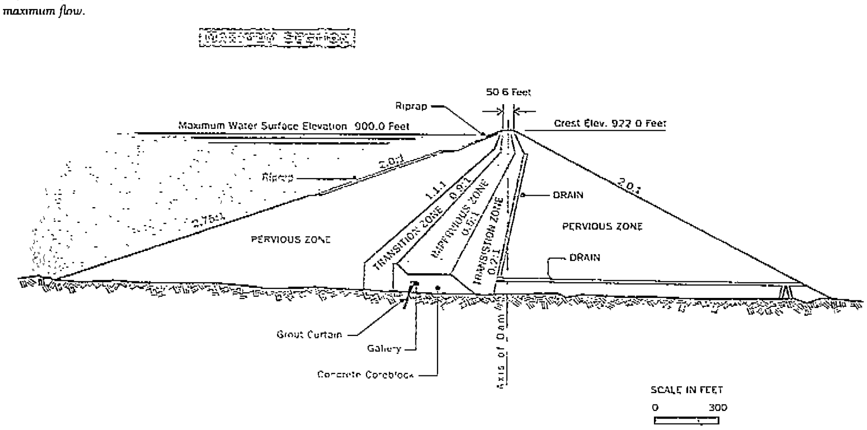
Oroville Dam and Lake Oroville lie in the foothills on the western slope of the Sierra Nevada and are one mile downstream of the junction of the Feather River's major tributaries. The lake stores winter and spring runoff which is released into the Feather River to meet the Project's needs. It also provides pumped-storage capacity, 750,000 acre-feet of flood control storage, recreation, and freshwater releases to control salinity intrusion in the Sacramento-San Joaquin Delta and for fish and wildlife enhancement.

Construction first began in 1957 on relocating what is now Highway 70 and the Western Pacific Railroad. Work on the dam site began in 1961. The embankment was topped out in 1967.





To protect the dam embankment, the Oroville Dam spillway was designed to pass the probable maximum flow.



DAM

Type: zoned earthfill (highest in U.S.)
Embankment volume . . . 80,000,000 cubic yards
Height. . . . . 770 feet
Crest length . . . . 6,920 feet
Crest elevation . . . . 922 feet

LAKE

Maximum operating storage . 3,537,580 acre-feet
Water surface elevation @ mos . . . . . . 900 feet*
Water surface area @ mos . . . . 15,810 acres
Shoreline @ mos. . . . . 167 miles
**maximum operating storage*

圖八之一 Hyatt Powerplant 平面圖

EDWARD HYATT POWERPLANT

Located in rock in the left abutment near the axis of Oroville Dam, Edward Hyatt Powerplant is an underground, hydroelectric, pumping-generating facility. Construction of the plant began in 1964 and was completed in 1967. Hyatt Powerplant maximizes power production through a pumped-storage operation where water, released for power in excess of local and downstream requirements, is returned to storage in Lake Oroville during off-peak periods and is used for generation during peak power demands.

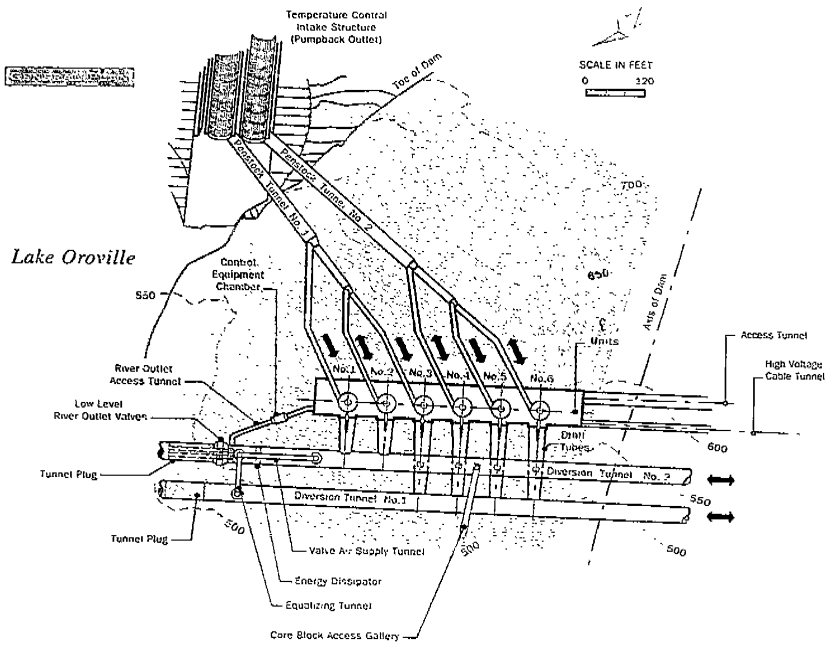
Water from the lake is conveyed to the units through penstocks and branch lines. After passing through the units, water is discharged through the draft tubes to one free surface and one full-flow tailrace tunnel.

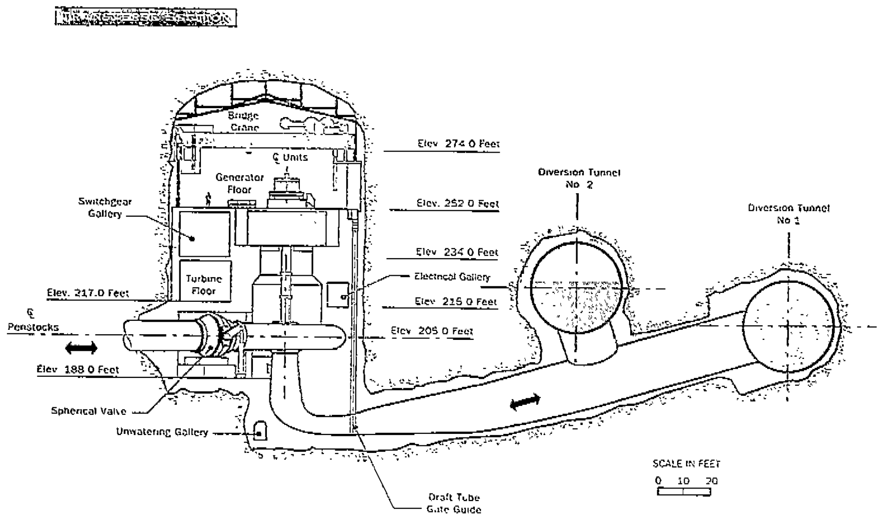
PUMPING

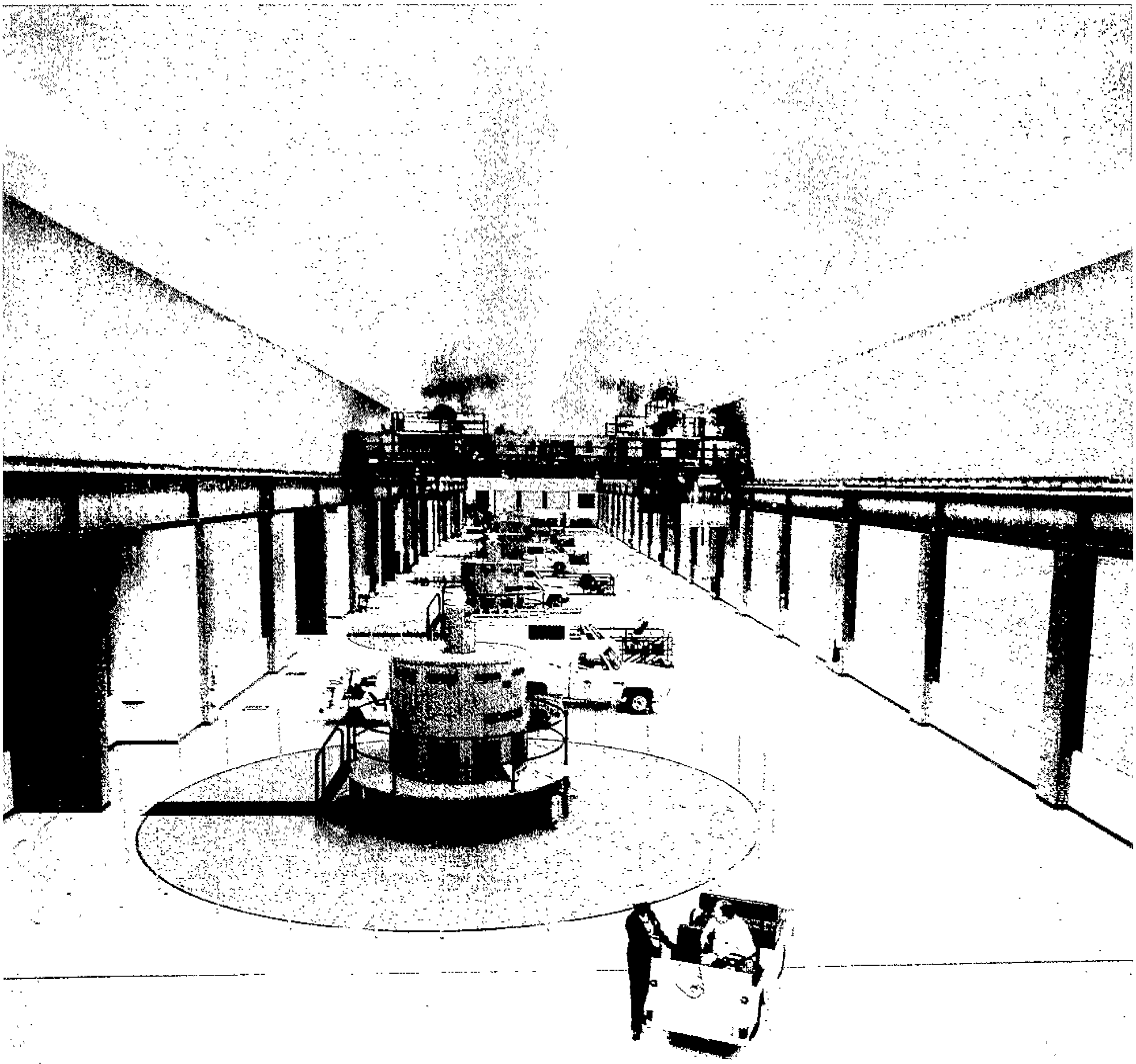
Installed capacity .... *5,610 cfs, 519,000 hp*
Normal static head .... *500-660 feet*
Design dynamic head .... *592 feet*
No. of units .... *3 (p/g)*
Unit Size .... *1,870 cfs,173,000 hp*

GENERATING

Installed capacity .... *819 MVA, 16,950 cfs*
Normal static head .... *410-676 feet*
Design dynamic head .... *615 feet*
Number of units .... *6 (3 g, 3 p/g)*
Unit size .... *3 @ 132 MVA, 2,800 cfs*
3 @ 141 MVA, 2,850 cfs
Penstock/diameter .... *2 @ 22 feet*

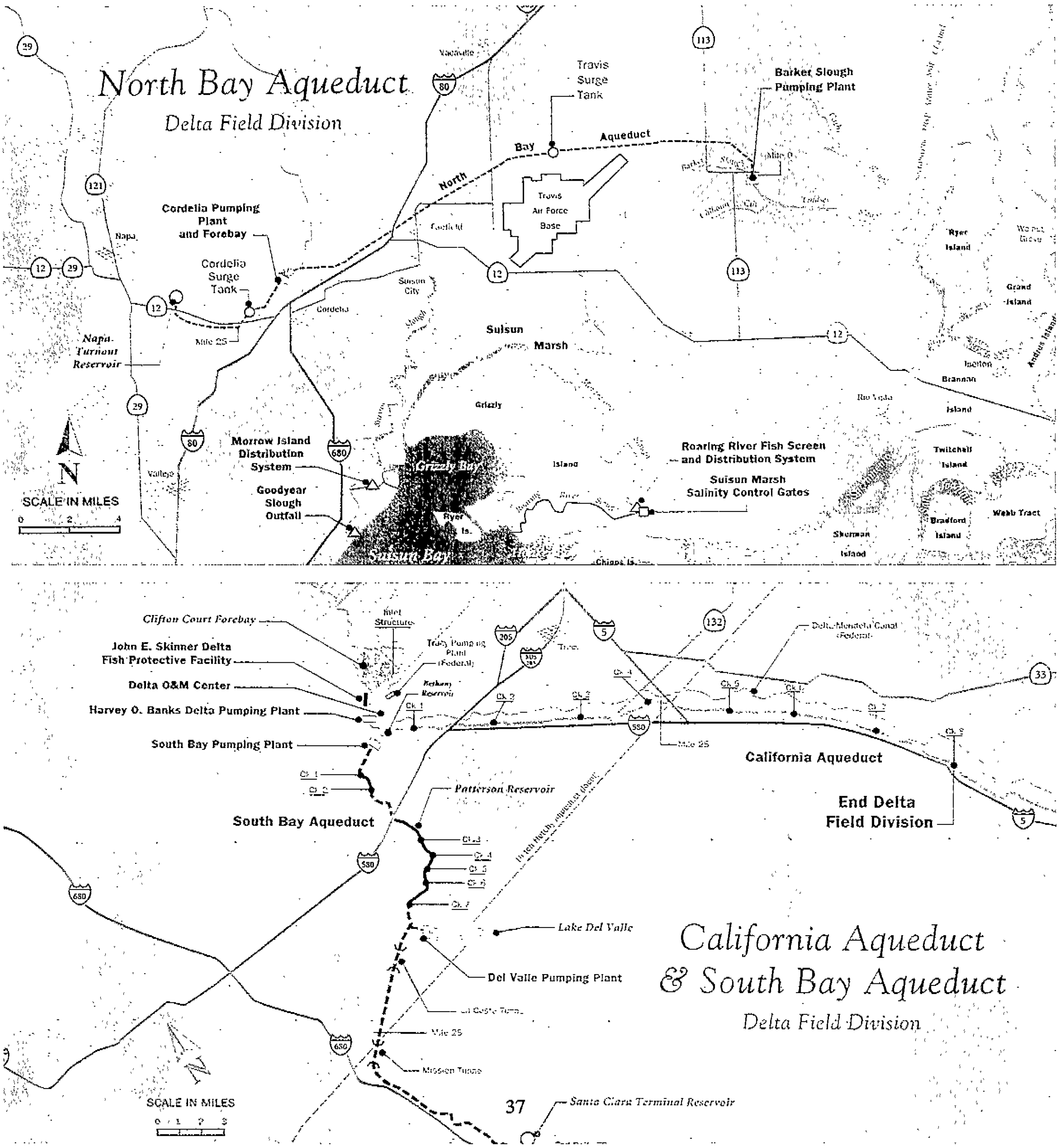






圖八之二　Hyatt Powerplant 內部設備圖

圖九　舊金山北灣供水系統圖



圖十　舊金山南灣供水系統圖

SOUTH BAY AQUEDUCT

South Bay Aqueduct was the first delivery system completed in the State Water Project and has been conveying water to Alameda and Santa Clara counties since 1962 and 1965, respectively. Constructed between 1958 and 1969, the 42.9-mile system consists of 8.4 miles of canals, 32.9 miles of pipeline, and 1.6 miles of tunnels.

Water agencies served by the aqueduct are the Alameda County Water District, the Alameda County Flood Control and Water Conservation District (Zone 7), and Santa Clara Valley Water District. They can receive up to 188,000 acre-feet annually.

PIPELINES

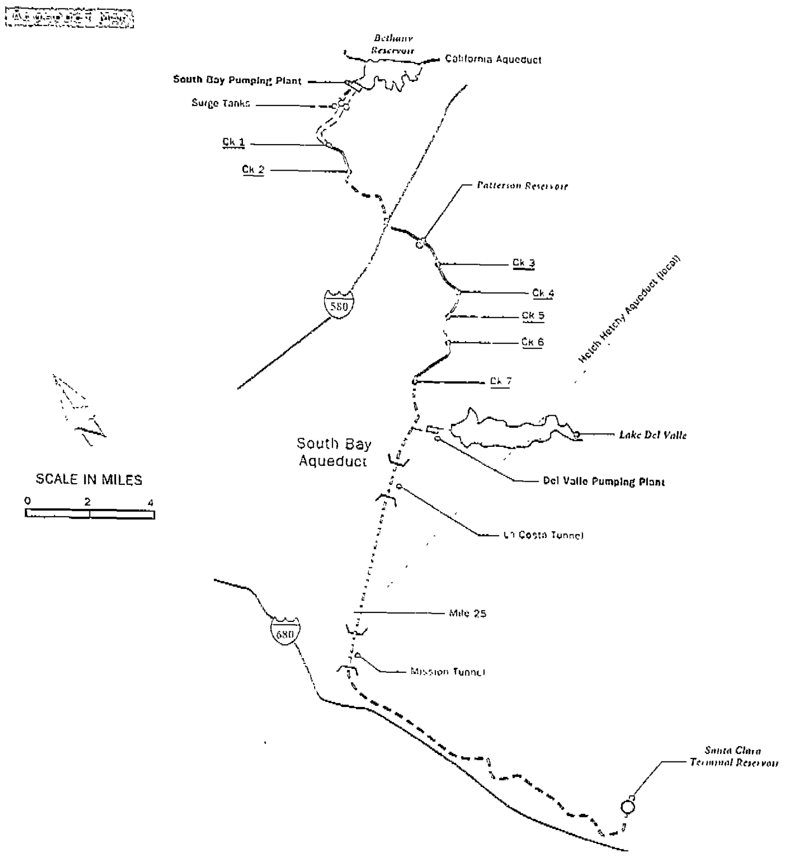
Type: 32.9 miles of buried reinforced concrete, cylinder, and steel pipes
Diameter . . . . *varies from 42 to 90 inches*
Capacity . . . . *varies from 120 to 363 cfs*

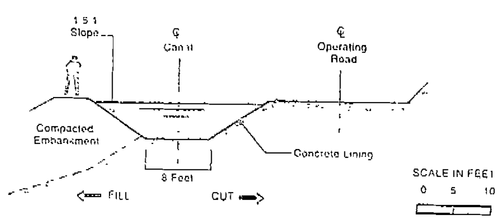
CANALS

Type: 8.4 miles of concrete-lined, trapezoidal, checked
Capacity . . . . *300 cfs*
Water depth . . . . *4.7 and 5.6 feet*
Side slope *1 1/2:1*
Width . . . . *8 feet (bottom)*

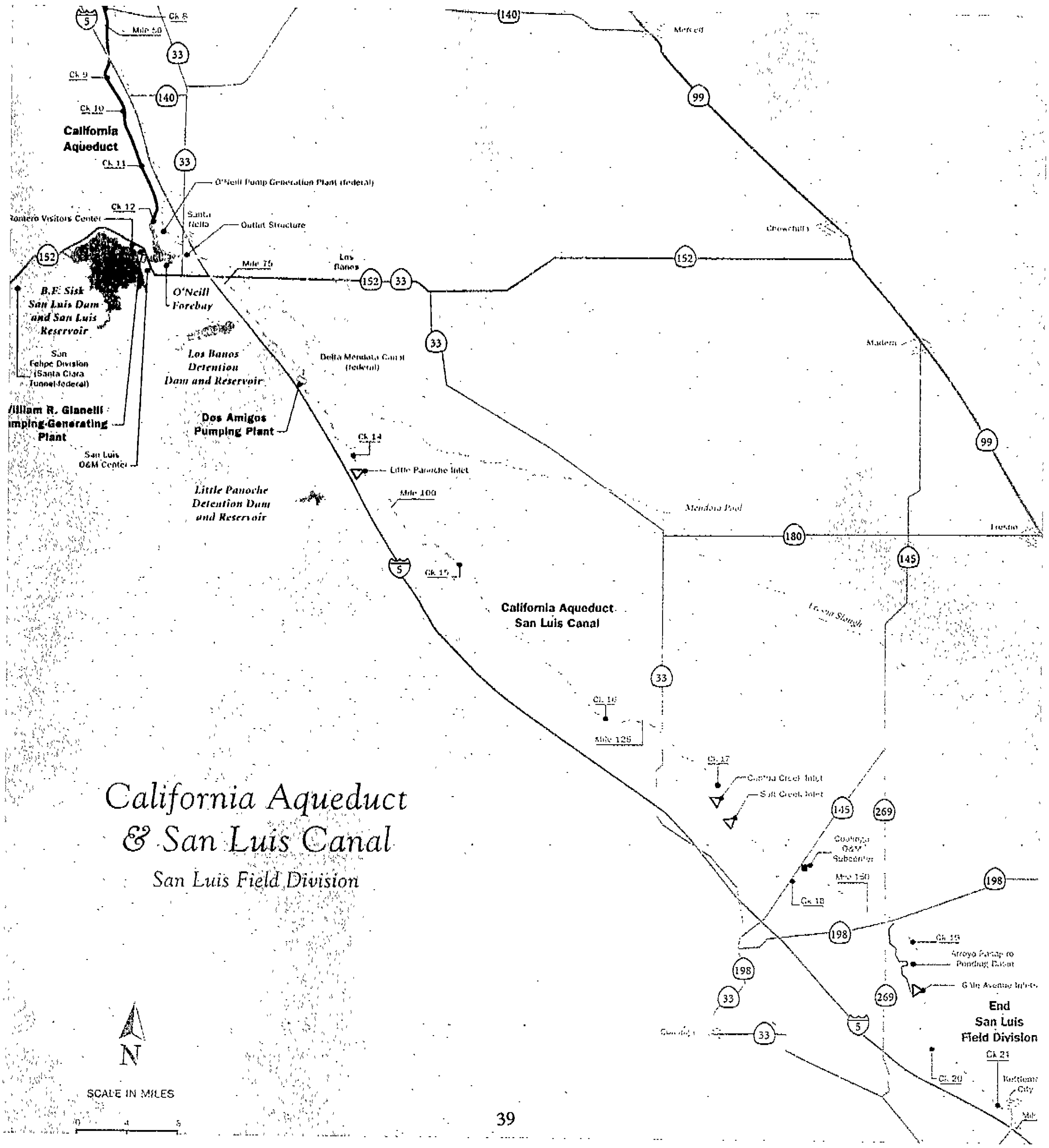
TUNNELS

Type: 1.6 miles of concrete-lined horseshoe tunnel, finished to a circular shape
Diameter . . . . *93 inches*
Capacity . . . . *305 and 255 cfs*

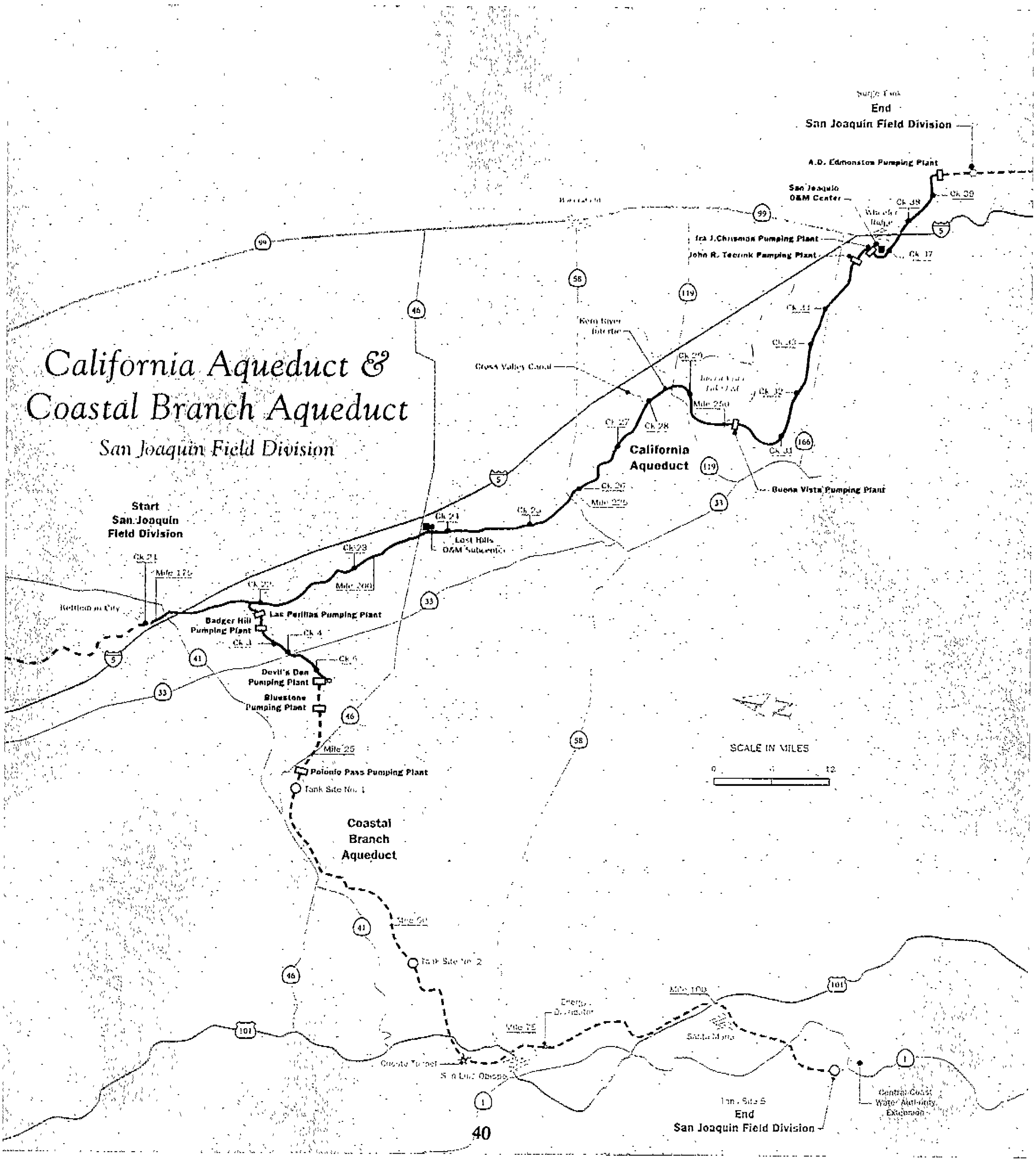




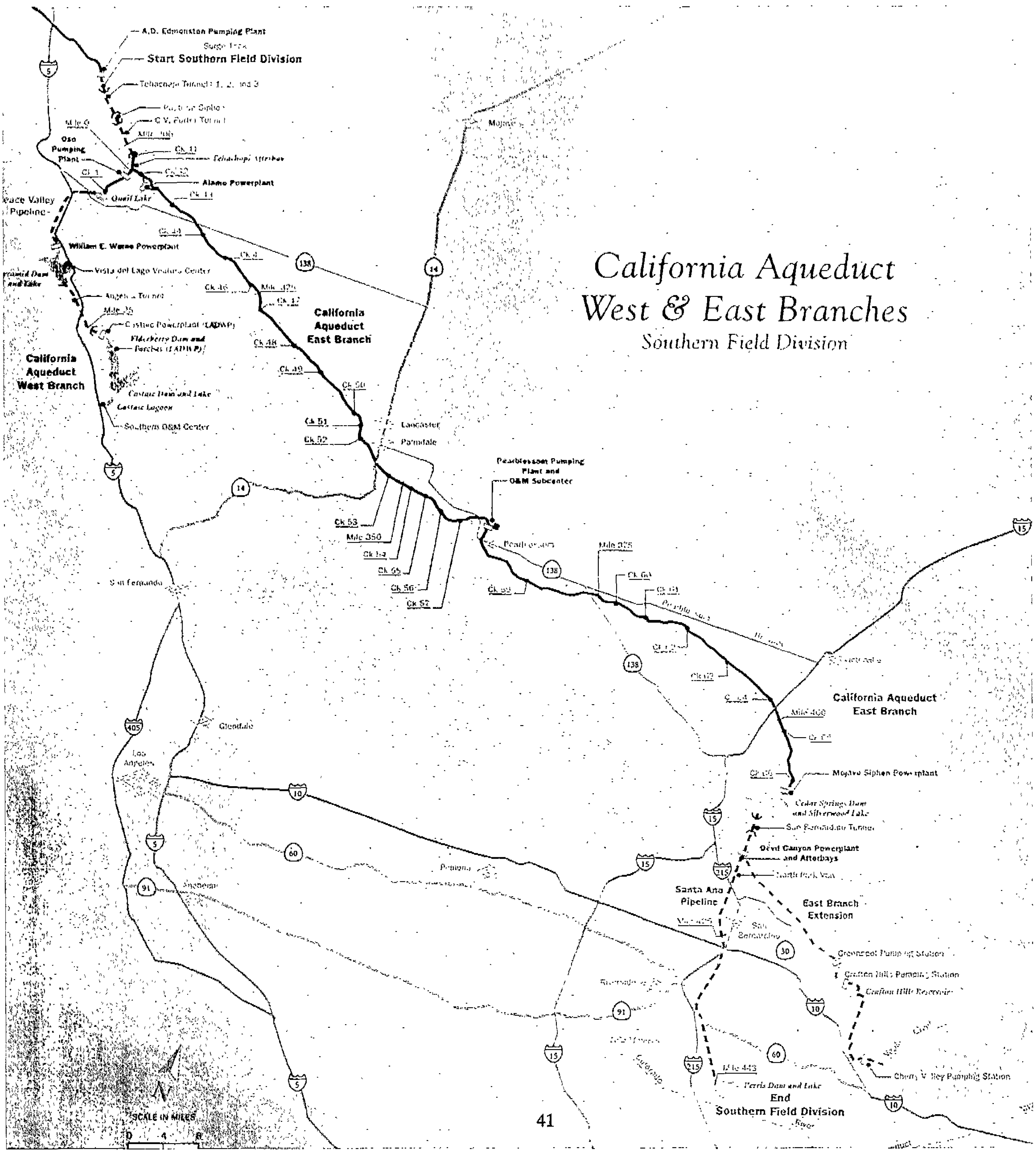
圖十一 San Luis 地區供水系統圖

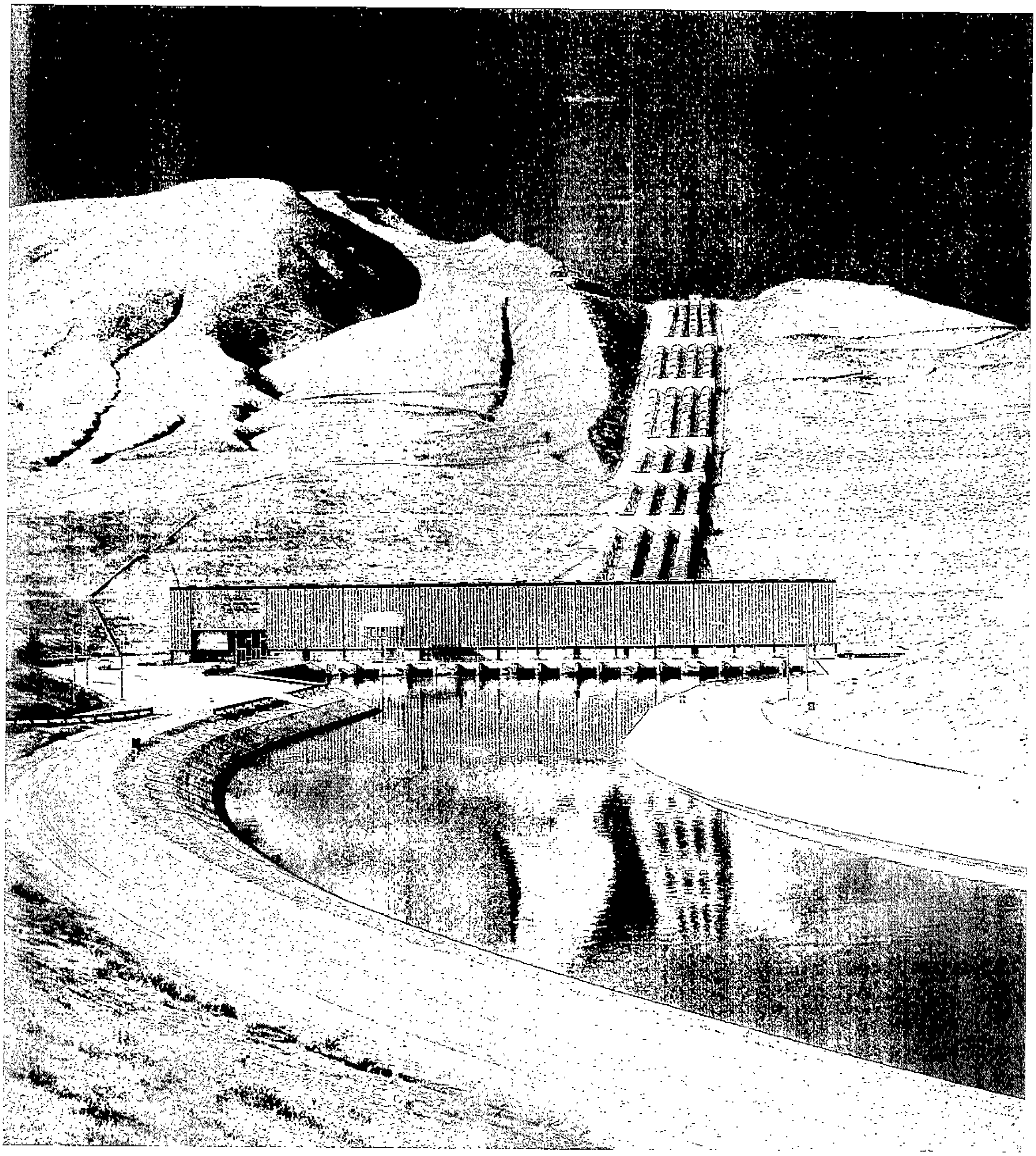


圖十二 San Joaquin 地區供水系統圖



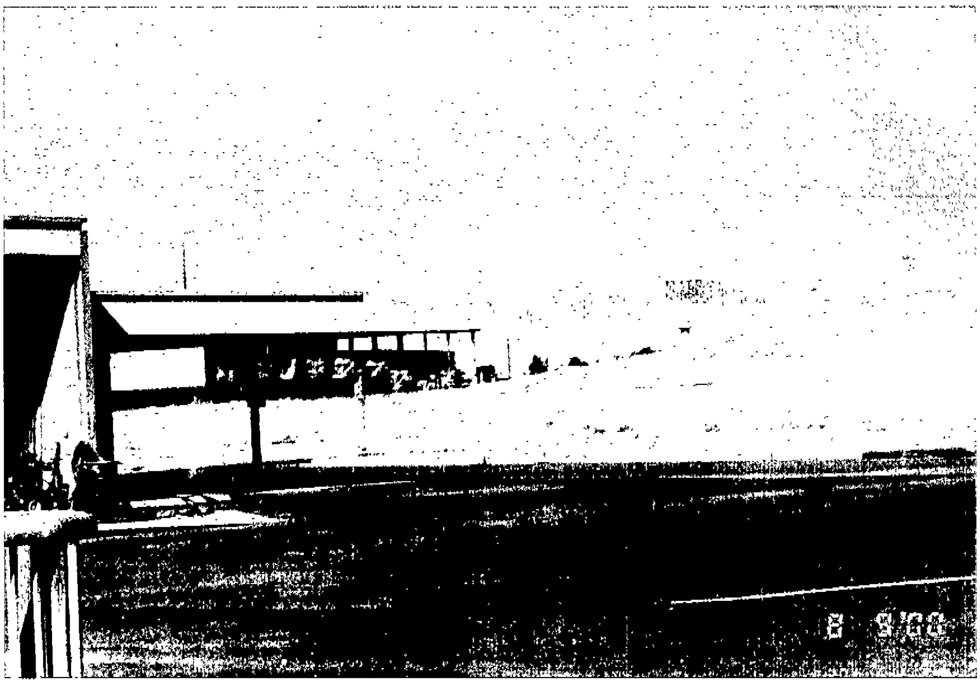
圖十三 南加州供水系統圖





圖十四之一 Ira J. Chrisman Gap Pumping Plant

圖十四之二 Buena Vista Pumping Plant



圖十五 風力發電圖

Fish & Wildlife Protection

ajor facilities built for fish and wildlife protection are the Feather River Fish Hatchery, the Suisun Marsh Salinity Control Gates, and the Skinner Fish Protective Facility. Compensating for lost spawning grounds on the Feather River near Oroville, the hatchery raises about 20 million fall-run and spring-run chinook salmon and steelhead annually. These fish are released into Northern California lakes, rivers, and the Delta.

The salinity control gates protect water quality in the Suisun Marsh, one of the largest contiguous brackish water marshes in the U.S. Its radial gates trap fresher water in the marsh, diluting saline waters from San Francisco Bay.

Located along the intake channel to Banks Pumping Plant, the Skinner Fish Facility uses louver screens and fish bypass systems to divert fish from entering the plant's pumps. These fish are counted, identified, recorded, and transported back to the Delta for release.

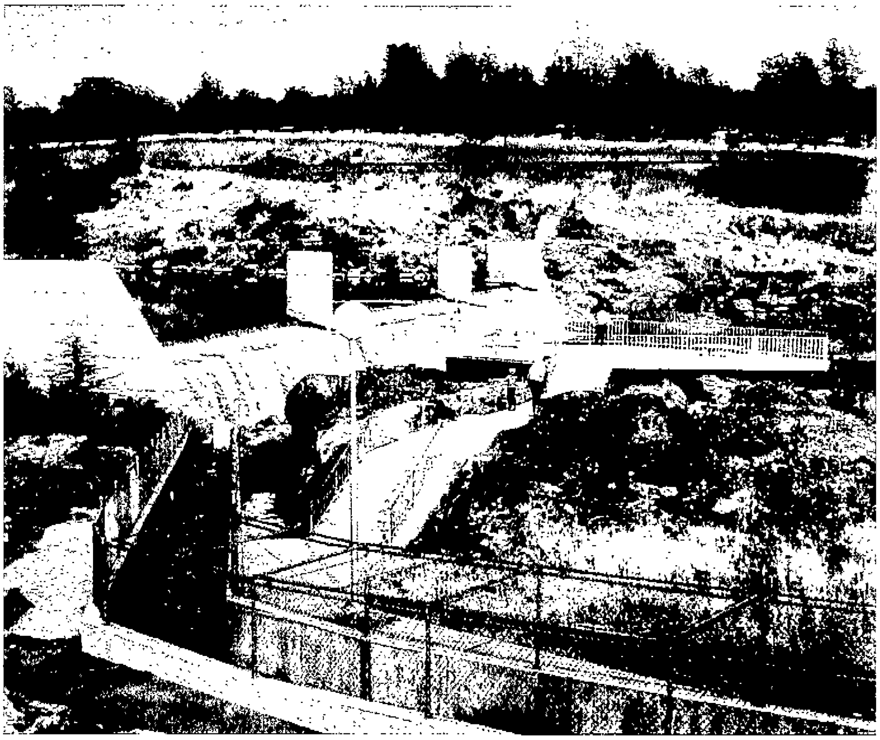
Other environmental protection measures include streamflow maintenance and temperature control, flow augmentation, restricted pumping schedules, fish screens, and mitigation agreements.

Salinity Control

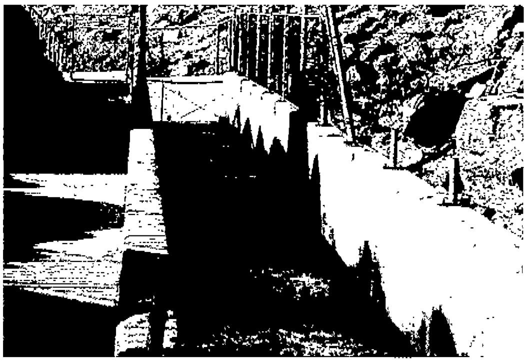
he State Water Project, in coordination with the federal Central Valley Project, is operated to limit salinity intrusion into the Delta and Suisun Marsh. This is accomplished by supplementing freshwater outflows to San Francisco Bay and limiting water exports from the Delta during specific times of the year.

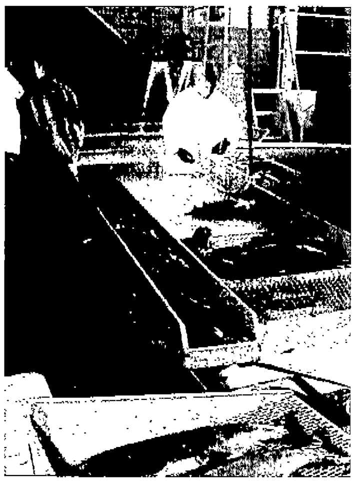
Funds from the SWP contractors are spent on a variety of fish and wildlife programs such as the Feather River Fish Hatchery. The fish ladder to the hatchery begins near the Fish Barrier Dam (top photo). Other programs include the Skinner Fish Protective Facility, and wildlife studies (bottom photo).

圖十六之一 Skinner Fish Protective Facility



Overlooking the Feather River is an observation platform Below is the start of the fish ladder leading into the hatchery Glass panels allow for an underwater view of fish swimming up the ladder

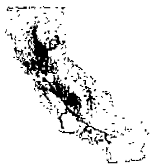




In the main hatchery building, the chinook salmon and steelhead are artificially spawned. Eggs, taken from the females, are mixed with milt from the males.

圖十六之二 鮭魚魚梯及產卵場

圖十七之一 John E. Skinner Delta Fish Protective Facility



JOHN E. SKINNER DELTA FISH PROTECTIVE FACILITY

Located two miles upstream of the Banks Pumping Plant, the Skinner Fish Facility contains a giant fish screen to keep most fish away from the pumps that lift water into the California Aqueduct. Large fish and floating debris are directed away by a 388-foot-long trash boom. Smaller fish are diverted from the intake channel (leading to the pumping plant) into bypasses by a series of metal louvers, while the main flow of water continues through the louvers and towards the pumps. These fish pass through a secondary system of screens and pipes into seven holding tanks, where they are later counted and recorded. The salvaged fish are then returned to the Delta in oxygenated tank trucks.

The Skinner Fish Facility, which salvages an average of 15 million fish a year, was constructed from 1966 to 1970. A second building with three holding tanks was built during 1991 to 1992. Operations of the initial facility began in 1968.

PRIMARY CHANNEL

Dimensions .... *383 feet long, 158 feet wide, 29 feet deep*
Capacity .... *10,300 cfs*
Velocity .... *1.5 to 3.5 fps*

LOUVER ASSEMBLIES

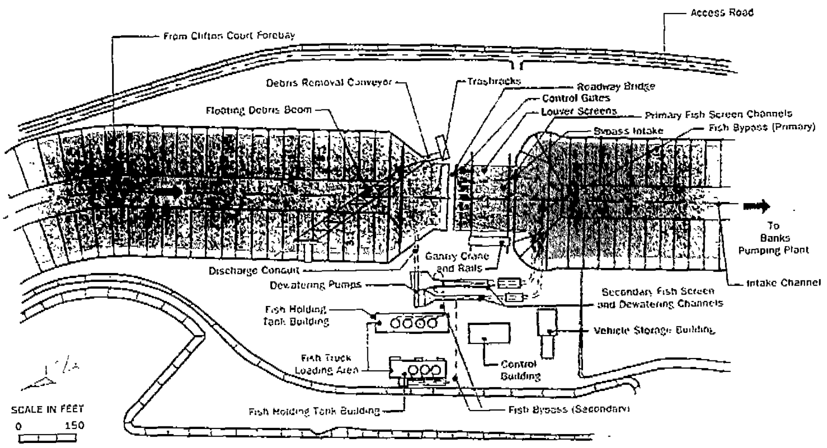
Type. aluminum alloy, closely spaced, parallel, vertical slats arranged in a sawtooth pattern to screen the primary channel
Height .... *26 feet*

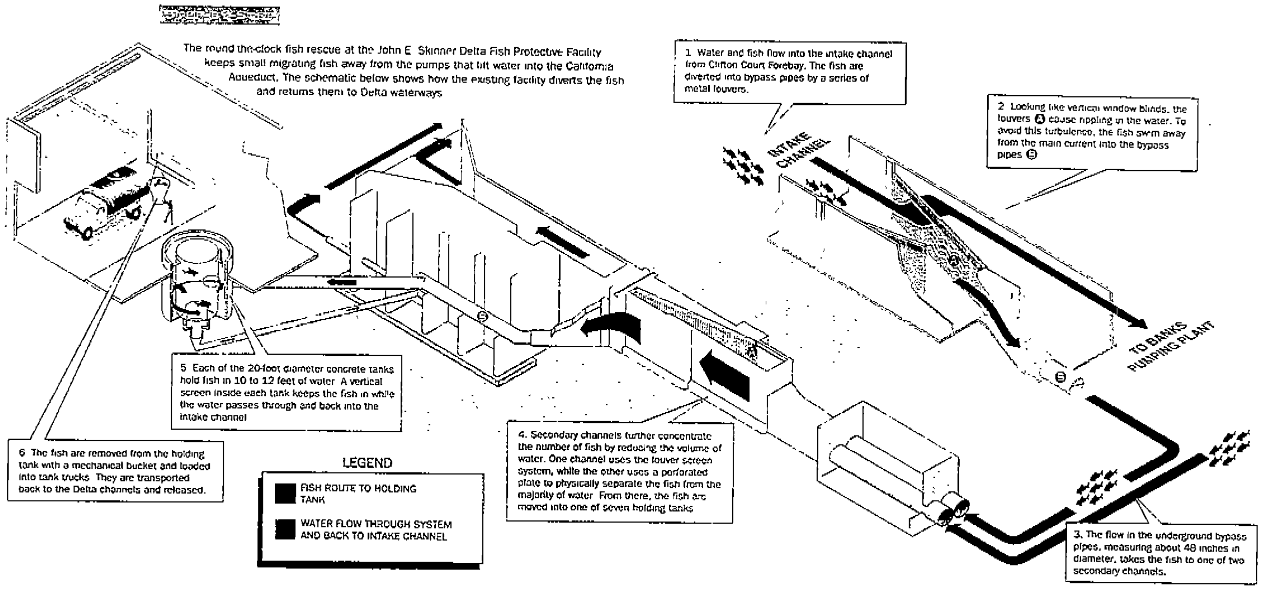
FISH HOLDING TANKS

Type: cylindrical, reinforced concrete
Number... 4 in first building, 3 in second building
Diameter .... *20 feet*
Depth...... .... *19.5 feet*

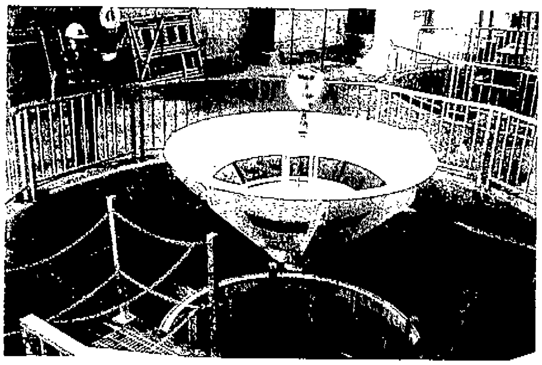
Skinner Fish Facility is operated and maintained by DWR, but the California Department of Fish and Game conducts fish collection and fish transport activities 24 hours a day

GENERAL PLAN





Metal louvers create a rippling in the water To avoid this turbulence fish are diverted from the main current into bypass slots where water draws them into underground pipes leading to one of seven 20-foot diameter concrete holding tanks



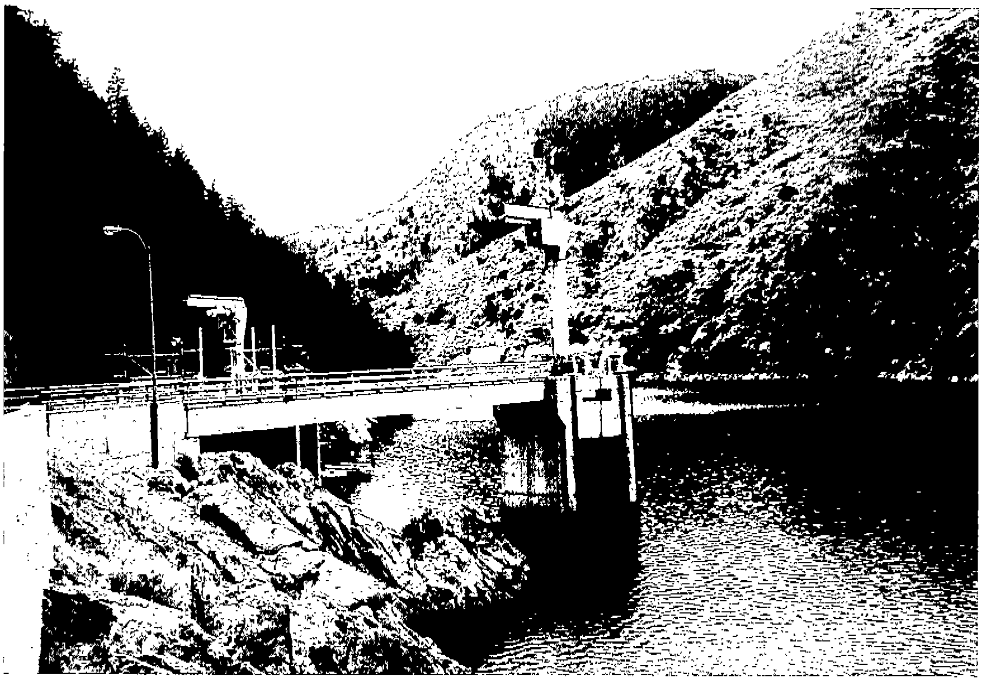
A conical bucket is lowered into the holding tank to remove the salvaged fish After they are counted and recorded, the fish are then moved to an oxygenated truck for transport back to the Delta.

圖十七之二 魚類保護導引渠道圖

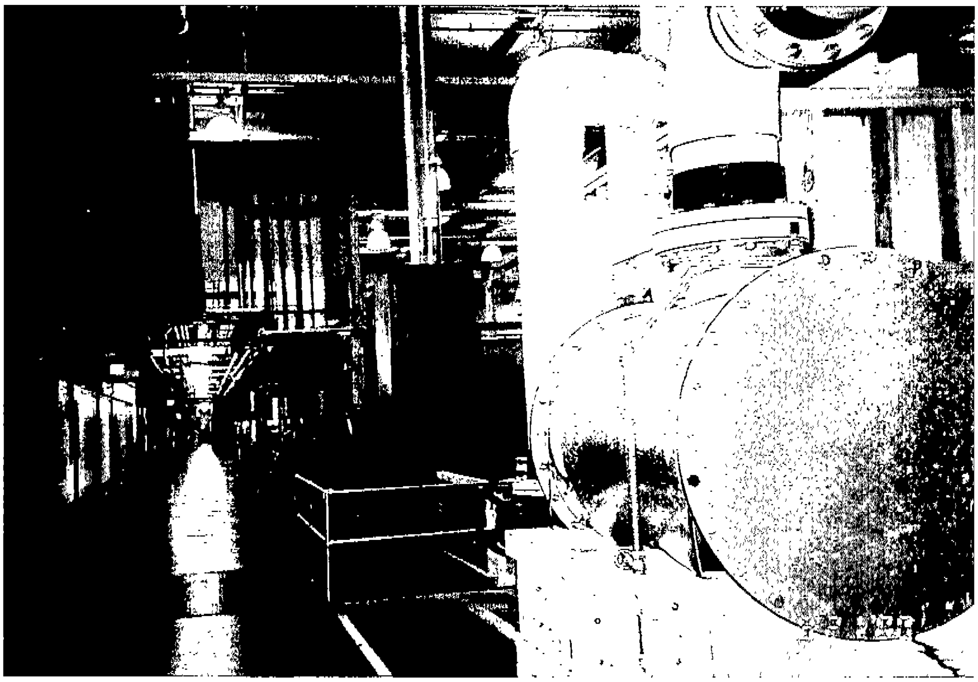
與墾務局人員及水利處人員合影於墾務局

圖十八　　聽取丹佛自來水公司簡報

圖十九　　舒壯泉水庫（Strotia Spring Dam）



胡佛壩水庫模型



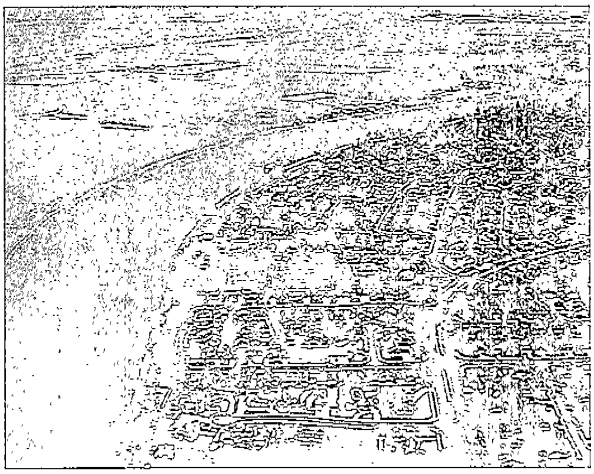
圖二十　　墾務局工程試驗中心

圖二十一　　水利義工於 Camel 河流域解說

Floodplains

Development is widespread on many floodplains of rivers and creeks throughout the West. Large parts of the alluvial valley areas of California are historic floodplains, low areas adjacent to waterways that flood during wet years. Urban areas developed in the floodplains because rivers were the main routes of commerce. In addition, floodplains often contain the best soils for agricultural crops because overflowing rivers leave behind layers of silt and topsoil.

Floods in these vulnerable areas have caused billions of dollars in damage. Congress created the National Flood Insurance Program (NFIP) in 1968 to reduce the loss of life, damage to property and rising disaster relief costs in these high risk areas. This incentive-based, voluntary program is administered by the Federal Emergency Management Agency (FEMA) and aims to end the expensive cycle of flooding and rebuilding. The program requires that new or replacement buildings in flood hazard areas be constructed to mitigate future flood damages. FEMA insists on assurances that local upstream flood repair measures and development in the floodplains will not exacerbate flooding in other communities. It guides future development away from flood prone areas and transfers the costs of flood losses from American taxpayers to floodplain property owners. Local officials are required to regulate development in these designated floodplain areas through the adoption of a floodplain management ordinances, which meet the minimum NFIP regulations. Ordinance requirements can include elevating the lowest floor of new homes at or above the 100-year flood elevation to prohibiting new buildings.



Aerial view of levees and housing development along the Sacramento River. These homes are protected by the coordinated efforts of many local, state and federal flood control agencies.

Participating communities identified with special flood areas are required to comply with certain development standards to qualify for federally subsidized flood insurance. (Property owners in these participating communities qualify for flood insurance whether or not their structure is in a floodplain. Federally subsidized flood insurance is available for older structures built prior to a floodplain designation. If new development does not comply with NFIP standards, the property owner pays higher flood insurance premiums.) Many people, however, do not purchase flood insurance and assume they will be protected by the federal government. Following the 1997 floods, FEMA reported only about 260,200 NFIP policy holders in California, a comparatively small amount given the state's population and number of flood-risk areas. The flood insurance program was reported to be more than $9 million in debt.

FEMA works with state and local governments to designate Special Flood Hazard Areas. At a minimum, these are areas determined to be within the 100-year flood boundary of a waterway. Local agencies may exceed these minimum standards and designate larger areas. Flood maps often are revised as new data are acquired and new areas are mapped, but not without considerable controversy. In addition, some contend that too much reliance is placed on the floodplain maps

Under pressure from FEMA, Sonoma County imposed stringent rebuilding restrictions in the disaster zone following the 1986 floods. Sonoma revised their ordinance to require that when flood damage costs exceeded 50 percent of market value of the structure, owners were required to elevate the building 1 foot above 100-year flood levels. These restrictions were very costly, but those who modified their homes weathered the 1995 floods better than their neighbors. Flood levels in 1995 were nearly as high as in 1986, and several hundred people were evacuated from Sonoma and Napa counties.

Parts of Sacramento were subjected to revised maps following the 1986 floods. Corps engineers reassessed the adequacy of the local levee system and estimated that a 100-year flood could cause $15 billion in damage and cost as many as 100 lives in the American River floodplain. The city of Sacramento and the adjoining community, including nearly 400,000 residents, were included in the revised floodplain map. The Natomas area at the junction of Interstates 5 and 80, which consists of 86 square miles targeted for new development, was included in the revised FEMA floodplain. It had existed for eons as a floodplain when it was part of the American River basin, before any levees were built. Extensive levee repair and upgrades were carried out along the Sacramento River system to increase protection to a 100-year level. Reoperation of Folsom Dam and reservoir to include more interim flood control storage was also factored into the plan. Based on levee improvements, the Sacramento City Council approved more development in the Natomas area in 1997. The Natomas area assessed itself to finance more levee improvements and is now considered less dangerous than other parts of the Sacramento area.

New maps for the Colorado River were released in 1990 that greatly restricted development in the 100-year floodplain below Davis Dam – just downstream from Hoover Dam – to the Mexican border.

In 1993, Congress required that FEMA develop a new special flood hazard area. This restoration or "AR Zone" designation will be given to newly designated 100-year floodplain areas where levee restoration is underway. The AR designation will affect flood insurance rates and the design of new structures, but will not preclude development. Development may occur in AR zones if structures are elevated to a certain height and NFIP regulations are applied. The AR Zone designation will be used nationwide by FEMA. It will be applied to floodplains in southern Los Angeles County, such as areas in Long Beach along the Los Angeles River, and to parts of Sacramento. In developed floodplains in southern California the designation applies to structures being substantially reconstructed.

Key Agencies Involved in Flood Control

U.S. Army Corps of Engineers
The Corps is the primary federal flood control agency. It develops guidelines for flood control storage in federally-funded reservoirs and monitors reservoir operations. The Corps also constructs flood control projects; operates multiple-purpose projects, provides DWR equipment and personnel for emergency flood fights and contributes funds to local flood control projects.

U.S. Bureau of Reclamation
The Bureau operates several multipurpose projects throughout the state, including the Central Valley Project (CVP) and the Colorado River system. The Bureau's flood hydrologists assist in interpreting flood-related data.

National Weather Service
The NWS issues weather forecasts and flood warnings. It helps communities establish flood warning systems and conducts flood hazard analyses and provides other technical assistance.

Federal Emergency Management Agency
FEMA administers the National Flood Insurance Program (NFIP) and disaster planning and recovery programs. FEMA works closely with states and communities and provides financial and technical assistance and flood hazard maps and data to better manage floodplains.

Department of Water Resources
DWR operates the State Water Project (SWP), runs the state-federal Flood Operations Center and assists NWS in flood forecasting. It is responsible for the operation and maintenance of the Sacramento and San Joaquin flood control projects. DWR funds flood control projects outside the Central Valley, carries out the state's floodplain management laws, and coordinates the floodplain management aspects of the FEMA in California.

State Reclamation Board
The board cooperates with the Corps in the planning, construction, operation and maintenance of flood control projects in the Central Valley. Once a project is completed, the board accepts legal responsibility for its maintenance and then turns it over to a local agency or DWR to maintain. The board also restricts, through a permitting process, development in designated floodways.

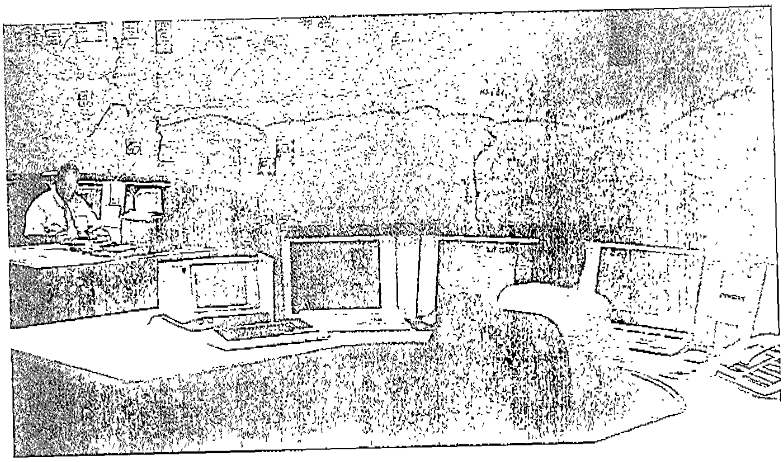
Office of Emergency Services
The state OES may allocate funds for investigation, estimates, reports and repairs regarding disaster recovery financial assistance for flood control works that do not come under the provisions of another authority. It administers FEMA's hazard mitigation program in California.

The state also has many local flood control agencies responsible for the day-to-day operations and maintenance of facilities, development and implementation of flood control and storm water drainage plans, and coordination with other state and federal agencies.

Flood Forecasting

Predicting the weather is an unsure science but enhanced computers that analyze data have improved the accuracy of short-term weather forecasts. Long-term forecasts (30 to 90 days in advance) usually are limited to general terms such as "wetter than normal," as opposed to delineating specific weather events. Long-term forecasts are about 55 percent accurate.

Determining when to issue flood warnings is fraught with uncertainty because of the unknowns inherent in weather forecasting. As one DWR official noted: "We have to be careful not to cry wolf too often or we lose credibility. But at the same time, we don't want to lose any lead time that could be used to safely evacuate people." Part of the uncertainty stems from the fact that there are only 100 years of climate data upon which to base weather predications.



At the State/Federal Flood Control Operations Center in Sacramento, hydrologists, weather forecasters and others work together to coordinate flood fights and reservoir releases.

Flood control managers, however, often are able to predict with a high degree of accuracy when local flooding is likely to take place. Their forecasts combine storm runoff from unregulated tributaries with reservoir releases to predict river levels. In northern California, the joint state-federal forecasting is done by the National Weather Service (NWS) in cooperation with DWR's Division of Flood Management at the California-Nevada River Forecast Center in Sacramento. Federal and state hydrologists can estimate high river stages within 12 to 24 hours before the event because of updates on weather, precipitation amounts, and reservoir and river levels. The information is distributed by the State/Federal Flood Operations Center in Sacramento, which cooperates with local flood control and emergency services agencies. Two days before the full force of the 1997 subtropical series of storms was unleashed, forecasters predicted that 40 inches of rain would fall in the upper Feather River watershed, which was within 1 inch of the actual total downpour.

On large, slower-moving rivers such as the Sacramento and San Joaquin, forecasters can predict high river stages more than 48 hours in advance. This is because releases from upstream dams can take days to flow down the river to the Delta. In contrast, on smaller, faster-moving rivers. such as the Eel and Smith on the north coast of California, and most southern California waterways, officials can produce accurate flood warnings no more than 12 hours ahead of time.

Weather patterns that worry climatologists and flood managers are those arising from El Niño and La Niña weather conditions. El Niño, known formally as El Niño-Southern Oscillation (ENSO), refers to unusually warm currents along the coasts of Peru and Ecuador, which heats other parts of the ocean and atmosphere. This weather phenomenon can increase tropical water temperatures in the eastern Pacific by as much as 5 degrees Celsius and cause heavier and more frequent storms, particularly in the south coast regions and areas south of San Francisco. In 1982-1983, which was an El Niño event, more than 26 inches of rain fell and a record 37.7 million acre-feet flowed through the Sacramento River system. However, most of the impact was along the coastal regions because of high tides and waves. There was $100 million in damage to coastal homes, businesses and public recreation facilities.

La Niña, which is the inverse of El Niño, causes intense, wet warm storms. The January 1997 deluge, along with the floods of 1955 and 1964 were a result of La Niña, which is caused by the interaction of cold surface ocean water near the equator and air.

Although weather patterns are discernible, the nature of a given storm and the regional impacts are nearly impossible to predict. As a precautionary measure, legislation was enacted in 1997 that allocated $7.5 million to better prepare California to cope with potentially severe storms associated with the El Niño that battered parts of California in 1998. The funds were for repairing and upgrading levees battered by the New Year's floods, increasing staff to monitor flood forecasts and reservoir operations and for advanced deployment of personnel in the event of flooding. During both the 1982-1983 and late 1997 El Niño events, tropical fish were found swimming in the waters off the Golden Gate.

Local Flood Warnings

In regions more susceptible to flash flooding, local entities often take over flood warning responsibilities. They work closely with the River Forecast Center or the nearby NWS office. In the past decade, many areas in southern California and along the central and north coasts developed cooperative programs called ALERT (Automated Local Evaluation in Real Time). Under the ALERT programs, precipitation is measured by gauges in the watershed linked to computer models adapted to local situations, which determine expected runoff.

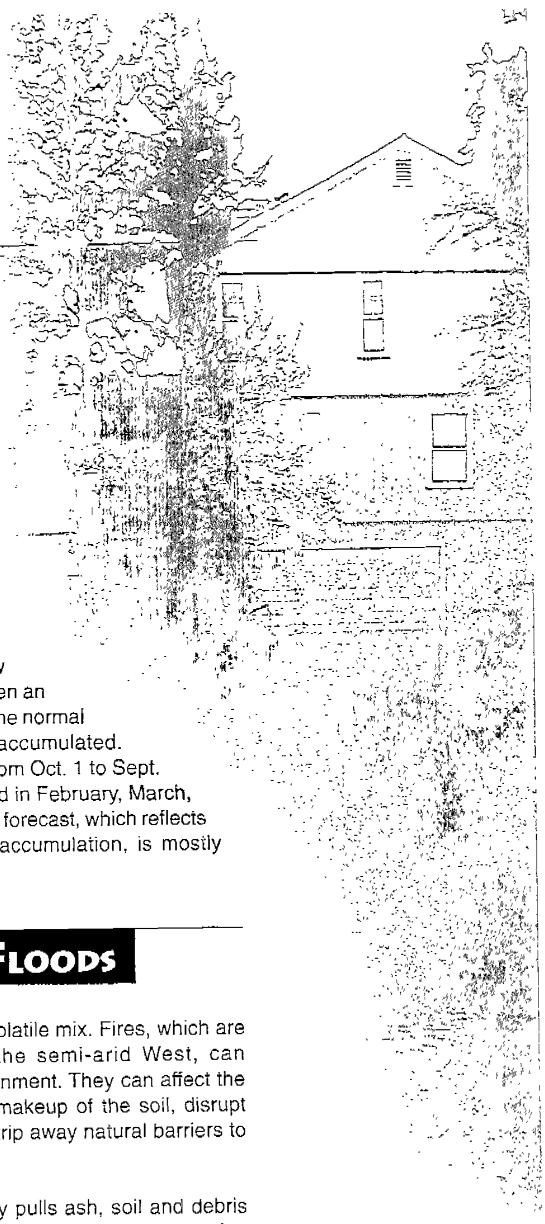
With adequate calibration and distribution of rain gauges within the watershed, an ALERT system provides timely information to help determine whether to evacuate. It also pinpoints areas of greatest concern, allowing more effective use of emergency personnel, the probable extent of the flooding and a response plan.

The system has been credited with saving lives and property during floods in California communities. When a storm struck the city of Petaluma on Feb. 14, 1983, city officials were told at 4 a.m. that flooding was imminent according to the ALERT system. Officials contacted emergency crews and by 5:15 a.m. an emergency center was set up and evacuation vehicles were dispatched. Less than 40 minutes later, police and firefighters were going door-to-door to 321 residences in low-lying areas to deliver evacuation warnings. In less than an hour, flood waters raced through the residential streets and poured into the houses in the evacuated areas. Residents who did not immediately evacuate after notification were rescued by boats. No loss of life was reported, and the ALERT system was given much of the credit.

However, the Petaluma system consists of only one simple watershed. Los Angeles County's ALERT system, on the other hand, consists of 33 watersheds which contain 15 dams and extensive downstream channel systems. Forecasting with this system is extremely complex, requiring extensive monitoring of the flows going into the dams and heading downstream.

An essential element of coastal weather forecasting is monitoring of high tides and waves. Flood damage along the coast is caused primarily by storm driven waves and high tides, which erode cliffs and cause loss of beach sand.

A significant factor in spring flooding of southern Sierra rivers is snow melt. Through the California Cooperative Snow Survey Program, each year the state measures how much water content is in the Sierra Nevada snowpack. Forecasts of spring snow melt begin on Feb. 1, when an average of two-thirds of the normal snowpack has usually accumulated. (The "water year" runs from Oct. 1 to Sept. 30.) Forecasts are issued in February, March, April and May. The April 1 forecast, which reflects the normal peak snow accumulation, is mostly used by water planners.



Fires and Floods

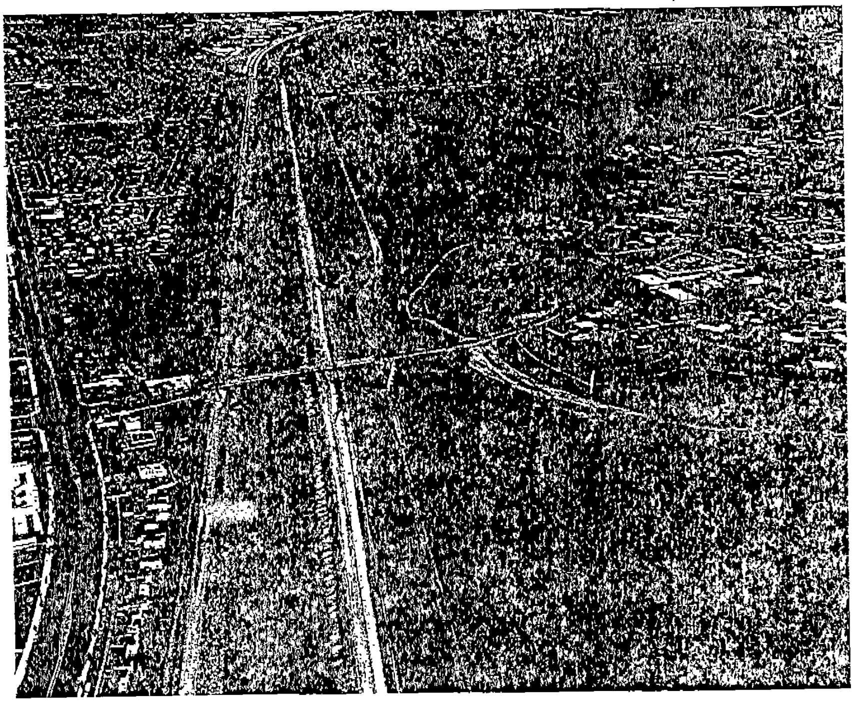
Fires and floods are a volatile mix. Fires, which are common throughout the semi-arid West, can radically alter the environment. They can affect the physical and chemical makeup of the soil, disrupt vegetative cycles and strip away natural barriers to erosion.

On steep slopes, gravity pulls ash, soil and debris into drainages, choking channels and threatening the area's ability to absorb runoff. High temperature fires sometimes form "hydrophobic soil," which repels water. When winter rains fall on the burned area, rainwater cannot percolate to deeper layers of the soil, increasing the amount of rainwater runoff that carries the top few inches of soil and rock.

Seeding burnt areas with annual grass and other quick-growing vegetation can help decrease erosion, but often there is not enough time for the plants to take hold before storms hit.

Strenuous efforts averted mudslides in Berkeley after the Oakland Hills fire of 1992. Mudslides were a problem in Malibu and other southern California communities following the fires in 1993.

Ongoing and Proposed Projects



A $1.3 billion project is underway to protect urban development near the Santa Ana River (the channel on the left) in southern California.

In the past, many of the West's large water projects such as California's federal CVP, were fully funded by the federal government. In 1986, Congress was faced with a growing national deficit and changed the terms and ratio of cost-sharing for Corps flood management projects with local and state governments under the Water Resources Development Act. Increased cost-sharing was seen as a way to stretch federal dollars over a large number of projects while eliminating less economical ones. It also would enable local agencies to have more influence over the selection of projects.

Cost-sharing ratios may vary, depending upon the project phase. Preliminary studies – called reconnaissance studies – on proposed projects are usually paid 100 percent by the Corps. If the project proceeds and future feasibility and environmental studies are warranted, the cost is shared on a 50-50 basis with non-federal entities.

The 1986 Water Resources Development Act, amended in 1996, also set forth requirements for the non-federal sponsors of flood control projects. Under the act non-federal sponsors must:

- Provide lands, easements and rights-of-way needed for project construction and operation;
- Perform relocations and alterations of buildings, utilities, highways, bridges, sewers and other facilities required for construction of the project;
- Pay at least 25 percent but not more than 50 percent of the total cost;
- Operate and maintain the project after construction and hold the federal government harmless.

There are about 100 flood control projects underway in California, most of them involving smaller streams. The largest project is the $1.3 billion Santa Ana River Mainstem Flood Control Project that aims to protect people and property in San Bernardino, Riverside and Orange counties. More than 20 miles of the 23-mile river were channelized and levees are being built or rehabilitated. A dam on the upper Santa Ana River, the Seven Oaks Dam, is under construction and expected to be completed in 1999. The last phase of the project will involve significantly increasing the capacity of Prado Dam spillway. Mitigation for environmental damage by the project includes the purchase of about 1,700 acres of land to be set aside for the preservation of salt marsh habitat for endangered species.

A decades-long, $196 million flood control rehabilitation project along the Sacramento River was begun after the 1986 floods. One of the proposals to lessen the flood danger to Sacramento, along with rebuilding levees, is to permanently increase the flood storage space behind Folsom Dam from 400,000 acre-feet to as much as 670,000 acre-feet. However, increasing flood storage lessens space available for water storage, potentially causing water shortages in dry years. Less water also is available for hydropower production. In addition, the lower water levels could threaten valuable fish and wildlife habitat and decrease recreational opportunities on the river and reservoir. Temporary reoperation is underway and it, along with increased storage space behind Folsom, was attributed with averting inundation of Sacramento during the 1997 New Year's storms.

Some suggest that the ultimate flood control solution for Sacramento is Auburn Dam, first authorized for construction in 1965 as a multipurpose project. Construction of the dam, located on the north fork of the American River above Folsom Dam, was stalled in 1975 after an earthquake struck the Oroville area, 41 miles north of Auburn. New seismic considerations were built into the design but changes in federal cost-sharing rules, as well as a growing disagreement among project proponents, environmental groups, national taxpayer organizations and wildlife agencies over long-term impacts, caused further delays.

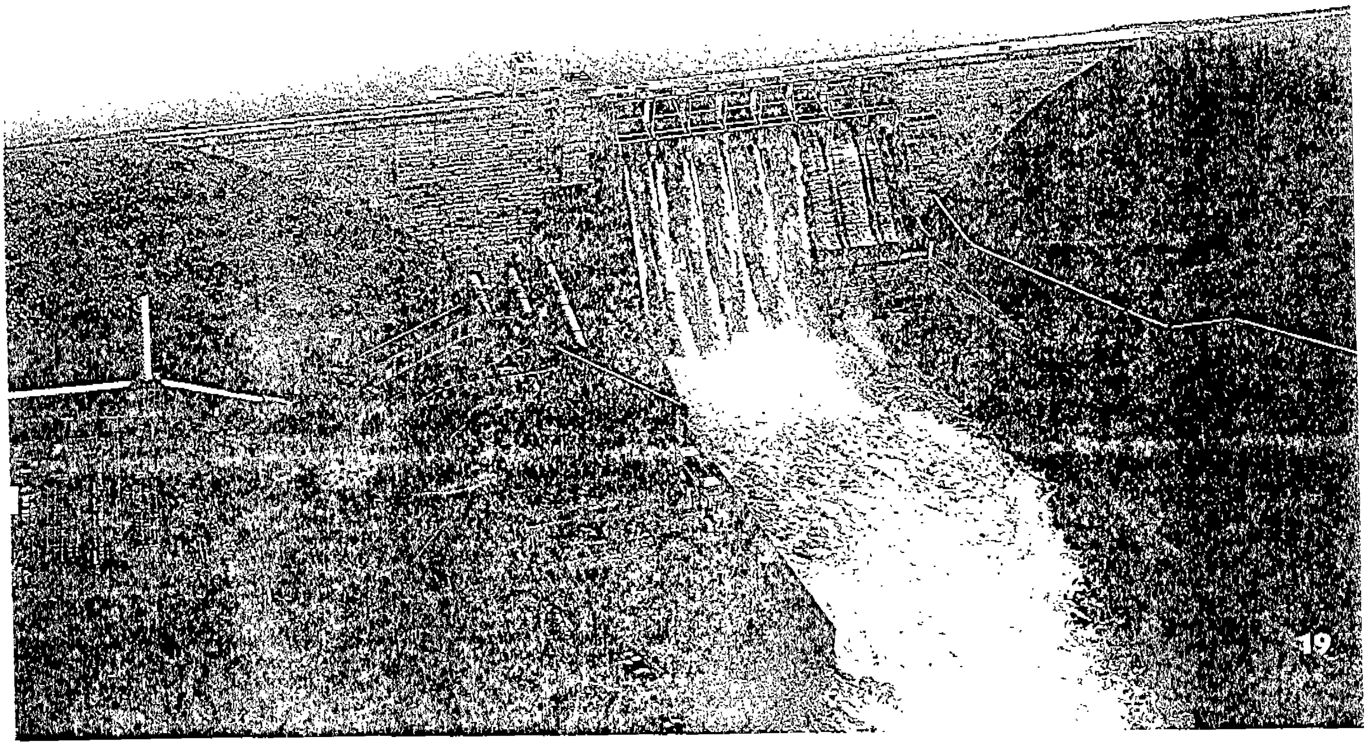
Sacramento area flood leaders have twice selected a flood detention dam upstream of Folsom to protect the Sacramento area from flooding. Other proposals included a downstream plan involving Folsom Dam, reservoir modifications and downstream levee improvements. However, Congress has rejected the dam, instead authorizing $44 million to fund the "common elements" of the alternative plans, which includes fixing downstream levees. In 1998, Sacramento area officials were once again considering which flood control plan to forward to Congress for possible funding. In addition to Auburn Dam, the other options were to modify Folsom Dam's outlets so it can release more water earlier in a storm or to combine dam modifications with raising and strengthening downstream levees.

In southern California, a study was conducted on the adequacy of Los Angeles County's flood control, which is estimated to provide between 25 to 40 year level of protection. The report released in 1991 by the county Department of Public Works (DPW) incorporated input from local and environmental organizations to increase water conservation, environmental and recreation resources. Projects have been initiated to enhance wildlife habitat and recreational opportunities, including the establishment of a green belt along the river, which has been extensively channelized with concrete. There is an ongoing controversy between DPW and the Corps and environmental groups over how to increase flood protection along the Los Angeles and San Gabriel river systems. The public agencies are pushing for higher levee walls, whereas environmentalists advocate watershed management measures, including widespread concrete removal, dry wells, cisterns, mulching to increase the absorption of the compacted land along the river banks and tree planting.

The concept of "greening" the banks with riparian vegetation is gaining support. Proponents of greening the river banks believe that concrete channels increase downstream flooding. Concrete channels and small tributaries to the river route the runoff into the main riverbed simultaneously. Advocates of greening are encouraging water managers to look at the desirability of allowing runoff to flow more slowly along natural, ragged banks, and by trapping it in basins so it can percolate and restore groundwater supplies.

Other future flood control projects statewide include a flood containment and environmental restoration of Redwood Creek in Humboldt County; an increase in the channel capacity of the Guadalupe River to increase protection for the city of San Jose; and possible improvements for flood control in the Whitewater River Basin near Palm Springs.

Folsom Dam releasing flood flows.



Linear Programming for Flood Control in the Iowa and Des Moines Rivers

By Jason T. Needham,[1] David W. Watkins Jr.,[2] Jay R. Lund,[3] Associate Members, ASCE, and S. K. Nanda,[4] Fellow, ASCE

Abstract: This study addresses questions related to flood-control operating procedures followed by the U.S. Army Corps of Engineers, Rock Island District. Application is presented of a mixed integer linear programming model for a reservoir system analysis of three U.S. Army Corps of Engineers' projects on the Iowa and Des Moines rivers. A strategy for evaluating the value of coordinated reservoir operations is developed. Results of this study suggest that operating Coralville Reservoir, on the Iowa River, for flood control on the Mississippi River does not provide appreciable benefits and, therefore, an operation plan coordinating releases from Coralville Reservoir with the two reservoirs on the Des Moines River may be unnecessary. Damage-minimizing results were obtained by operating the three reservoirs independently for 8 of the 10 largest flood events on record. Also, a review of the operating procedures for the flood of 1993 illustrates how much damage could have been reduced if inflows could be predicted months in advance or if the existing operating rules were more averse to extreme flood events.

INTRODUCTION

Record floods in the past decade have caused enormous economic damage and human suffering. In particular, the Great Midwest Flood of 1993 along the Upper Mississippi River and its tributaries caused an estimated 48 fatalities and \$15–\$20 billion in economic damages, surpassing all floods in the United States in modern times (U.S. Department of Commerce 1994). With increasing environmental concern and decreasing public support for large-scale flood-control structures, a growing number of engineers and hydrologists are concentrating on developing computer models for optimizing the operation of existing systems rather than proposing/designing new flood-control projects. Better forecasting methods are being developed to provide the most accurate data possible for these models. Along these lines, this paper describes an application of deterministic optimization to assess flood-control operations for the Iowa/Des Moines River Reservoir System and to provide insight for possible modifications to the current operating plan [U.S. Army Corps of Engineers (USACE) 1999].

Developing optimization models for analyzing operating policies of multiple reservoir systems has been a popular area of research for >30 years. Yeh (1985) and Wurbs (1993) presented in-depth reviews of reservoir management and operations models that contain extensive references in this area. Labadie (1997) presented a thorough discussion and formulation of reservoir optimization models and comments on reasons for the gap between theoretical developments and real-world implementation. The USACE has applied optimization methods in studies of reservoir system operations on both the Missouri and Columbia rivers (USACE 1994, 1996). These studies focused on seasonal and long-term operations using the Hydrologic Engineering Center (HEC) Prescriptive Reservoir Model, which uses a 1-month time step, limiting its capabilities for assessing flood-control operations because decisions need to be made on a daily or even hourly basis during flood events.

Optimization models for reservoir flood-control operations have not been applied as vigorously. One approach that has been applied is dynamic programming (Glanville 1976; Beard and Chang 1979). Dynamic programming is popular because it can directly accommodate the nonlinear and stochastic features that characterize many water resources systems. However, discretization of state, input, and decision variables, especially for multiple reservoirs, causes dimensionality problems. Wasimi and Kitanidis (1983) avoided dimensionality problems through the application of linear quadratic Gaussian control for the optimization of real-time daily operation of a multireservoir system under flood conditions. Their approach, however, is valid only under moderate flood conditions when capacity constraints are not likely to become binding. Windsor (1973) formulated a linear programming (LP) model that includes storage and release capacity constraints, along with some theoretical discussion of how it could be used with the latest forecast information to adjust reservoir operations during a flood event. The model presented in this work is similar to that of Windsor.

IOWA/DES MOINES RIVER RESERVOIR SYSTEM

The Iowa/Des Moines River Reservoir System consists of three reservoirs, one on the Iowa River main stem and two on the Des Moines River main stem, as shown in Fig. 1. The reservoirs are operated and maintained by the USACE, with the Rock Island District responsible for day-to-day decision making. Operators follow guidelines described in the reservoir regulation manuals that have been prepared as part of the design of the system (*Master* 1983, 1988, 1990).

Authorized purposes for these reservoirs include flood control, low-flow augmentation, fish/wildlife, water supply, and recreation. In each case, access and facilities are provided for recreation, but water is not controlled for that purpose (USACE 1992). Total capacities and average inflows for the three reservoirs are shown in Table 1. Other pertinent characteristics of the Iowa and Des Moines rivers are shown in Tables 2 and 3, respectively.

Table 2 illustrates that Coralville Reservoir can regulate no more than 25% of the total average annual flow entering the Mississippi from the Iowa River. Because of this, one could expect that Coralville Reservoir's flood-control effectiveness below the confluence of Cedar River and on the Mississippi

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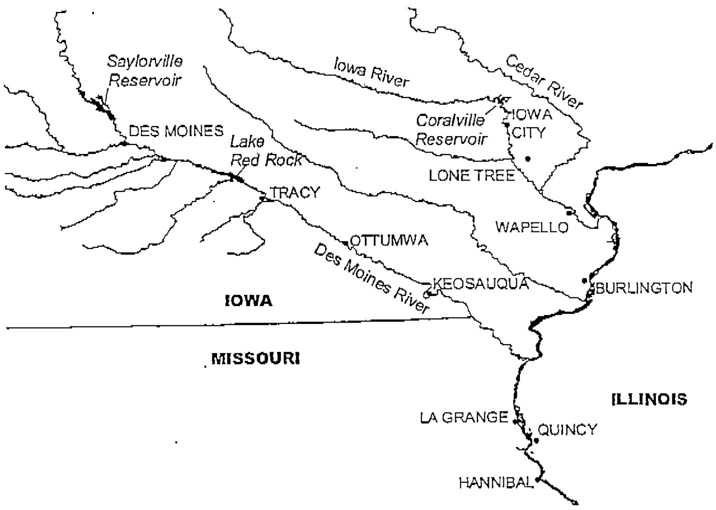

FIG. 1. Map of Iowa/Des Moines River Reservoir System

TABLE 1. Capacities of, and Average Inflows to, Three Reservoirs

Reservoir (1)	Inflows (acre-ft/year) (2)	Capacity (acre-ft/year)			
		Conservation (3)	Flood control (4)	Total (5)	Percent[a] (6)
Coralville (Iowa River)	1,271,800	25,900[b]	435,300	461,200	18
Saylorville (Des Moines River)	1,540,600	90,000	586,000	676,000	20
Red Rock (Des Moines River)	3,568,000	265,500[b]	1,494,900	1,760,400	62

[a]Percent of total federal project flood storage in Des Moines/Iowa system.
[b]Varies seasonally, value is minimum which corresponds to maximum flood storage.

TABLE 2. Iowa River Characteristics

Location (1)	Drainage area (sq mi) (2)	Mean inflow (cfs) (3)
Coralville Reservoir	3,115	1,760
Iowa River (confluence with Cedar River)	4,770	2,360
Cedar River (confluence with Iowa River)	7,870	4,230
Iowa River (confluence with Mississippi River)	12,980	7,120
Mississippi River (confluence with Iowa River)	89,000	49,000

TABLE 3. Des Moines River Characteristics

Location (1)	Drainage area (sq mi) (2)	Mean inflow (cfs) (3)
Saylorville Reservoir	5,823	2,200
Red Rock Reservoir	12,323	4,928
Des Moines River (confluence with Mississippi River)	14,540	8,210
Mississippi River (confluence with Des Moines River)	119,000	64,520

River is limited. Conversely, as illustrated in Table 3, Saylorville and Red Rock reservoirs regulate more than half of the average flow entering the Mississippi River from the Des Moines River. The main tributaries of the Des Moines River join the main stem upstream of, or at, Lake Red Rock. An important tributary is the Raccoon River, which converges in the southern part of the city of Des Moines and has a large effect on the stage there. The hydrographs at Ottumwa and Keosauqua are similar because no major tributaries join th Des Moines downstream of Ottumwa.

Coralville Reservoir was completed and placed in operation during 1958 as a unit in the general flood-control plan for the Upper Mississippi River Basin. Under current operations, Coralville Reservoir is to be operated for flood control at Lone Tree and Wapello on the Iowa River and Burlington, Iowa, on the Mississippi River (*Master* 1990). Presumably, when operated in conjunction with the reservoirs on the Des Moines River, the flood peaks can be offset enough to cause a significant difference in the water levels on the Mississippi River during flooding.

Saylorville Reservoir and Lake Red Rock projects also are associated with the comprehensive flood-control plan for the Upper Mississippi River Basin. Lake Red Rock was completed in 1969, and Saylorville Dam was completed in 1975. According to the reservoir regulation manuals, Saylorville Reservoir is operated not only to reduce flood damage in the city of Des Moines, but it is also operated in tandem with Red Rock Reservoir to reduce flood damage at Ottumwa and Keosauqua on the Des Moines River and at Quincy, Ill., on the Mississippi River (*Master* 1983, 1988).

MIXED-INTEGER LP MODEL

A mixed-integer LP model, termed HEC-FCLP, has been developed at HEC to assist with USACE's flood management studies. This model is based on work done by Ford (1978), with mixed-integer variables added to the formulation to incorporate nonconvex hydraulic relationships (Watkins et al. 1999). The model treats the flood-operation problem as one of finding a system-wide set of releases that minimize total system penalties for too much or too little release, storage, an flow. A simulation model embedded in the LP model uses given releases to compute storage and downstream flows, accommodates reservoir continuity and linear channel routing (e.g., Muskingum routing), and accounts for hydraulic limitations such as reservoir outlet capacities.

HEC-FCLP reads a description of the flood control system from an ASCII text file and generates a set of linear equations that constitute the LP model. Using IBM/OSL, a general-purpose, large-scale LP/MIP solver (*Optimization* 1995), HEC-FCLP calculates the optimal values of decision variables and then translates the LP results into terms familiar to hydrologic engineers (e.g., release, flows, storage values). HEC-FCLP is linked to the HEC Data Storage System (USACE 1995), from which it reads incremental flow data and to which it writes the model results.

The model constraint set includes continuity constraints for each reservoir and control point, along with constraints on reservoir release capacity, in each time period. The objective function includes penalties for too much or too little storage, release, or flow in each time period.

The general form of the reservoir continuity constraints, for reservoir j, time period i, is

$$\frac{1}{\Delta t}[S_{i,j} - S_{i-1,j}] + f_{i,j} - \sum_{k,k\in\Omega}\sum_{t=1}^{i} c_{t,k} f_{t,k} = I_{i,j} \quad (1)$$

where $S_{i-1,j}$ and $S_{i,j}$ = storage at the beginning and end of period i, respectively; $f_{i,j}$ = total release in period i; Ω = set of all control points upstream of j from which flow is routed to j; $f_{t,k}$ = average flow at control point k in period t; $c_{t,k}$ = linear coefficient to route period t flow from control point k to control point j for period i; and $I_{i,j}$ = inflow to the reservoir. The routing coefficients are found directly from the Muskingum model coefficients.

To model desired storage-balancing schemes among reservoirs, the total storage capacity of each reservoir in the system is divided into zones. The total storage at any time i is the sum of storage in these zones

$$S_{i,j} = \sum_{l=1}^{NLF} S_{i,j,l} \quad (2)$$

where l = index of storage zone; and NLF = number of storage zones. Substituting this in the continuity equation yields

$$\frac{1}{\Delta t}\left[\sum_{l=1}^{NLF} S_{i,j,l} - \sum_{l=1}^{NLF} S_{i-1,j,l}\right] + f_{i,j} - \sum_{k,k\in\Omega}\sum_{t=1}^{i} c_{t,k} f_{t,k} = I_{i,j} \quad (3)$$

The storage in each zone l is constrained as

$$S_{i,j,l} \le S\max_{j,l} \quad (4)$$

The maximum reservoir release physically possible is limited by the hydraulic properties of the reservoir outlet works. This limitation is expressed as a piecewise linear function of the storage in the reservoir. That is, the maximum release from reservoir j for period i is specified as

$$f_{i,j} \le \sum_{l=1}^{NLF} \frac{\beta_{j,l}}{2}\cdot(S_{i-1,j,l} + S_{i,j,l}) \quad (5)$$

where $\beta_{j,l}$ = slope of the storage-discharge capacity relationship in storage zone l. To correctly represent nonconvex storage-discharge functions, critical under forced spill conditions, the following binary variable and logical constraints must be added for each reservoir j:

$$\sum_{l=1}^{2} S_{i,j,l} \ge Y_{i,j}\sum_{l=1}^{2} S\max_{j,l} \quad (6)$$

$$S_{i,j,3} \le Y_{i,j} S\max_{j,3} \quad (7)$$

$$Y_{i,j} \in \{0,1\} \quad (8)$$

These constraints ensure that, for example, storage zones 1 and 2 are filled before water is stored in zone 3.

The continuity constraint for each control point other than a reservoir takes the following general form:

$$f_{i,j} - \sum_{k,k\in\Omega}\sum_{t=1}^{i} c_{t,k} f_{t,k} = I_{i,j} \quad (9)$$

where $f_{i,j}$ = average control-point flow during period j; and $I_{i,j}$ = local inflow during period j. For proper representation of the damage function, control-point flow may also be divided into zones. The control-point continuity equation then takes the form

$$\sum_{l=1}^{NF} f_{i,j,l} - \sum_{k,k\in\Omega}\sum_{t=1}^{i} c_{t,k} f_{t,k} = I_{i,j} \quad (10)$$

where l = index of discharge zone; and NF = number of discharge zones.

Penalties for too much or too little storage represent operators' aversion to storage levels outside of a target range. The penalties are specified for each reservoir as a piecewise linear convex function of volume of water stored in the reservoir during the period. The total penalty for storage SP is defined as

$$SP_j = \sum_{t=1}^{i}\sum_{l=1}^{NLF} A_{j,l} S_{t,j,l} \quad (11)$$

where $A_{j,l}$ = slope of the storage penalty function in zone l of reservoir j.

Penalties for changing release rates too rapidly quantify negative impacts such as bank sloughing or inadequate response time to changing conditions downstream. Changes in release rates may also be limited by the equipment available to change gate or outlet settings. To impose this penalty, the LP model includes a set of auxiliary constraints that segregate the release for each period into the previous period's release, plus or minus a change in release. If the absolute value of this change in release exceeds a specified maximum, a penalty is imposed.

The auxiliary constraints relate the release for each period to release in the previous period by the equation

$$R_{i,j} = R_{i-1,j} + [Ra^{+}_{i,j} + Re^{+}_{i,j}] - [Ra^{-}_{i,j} + Re^{-}_{i,j}] \quad (12)$$

where $Ra^{+}_{i,j}$, $Re^{+}_{i,j}$ = acceptable and excessive release increase, respectively; and $Ra^{-}_{i,j}$, $Re^{-}_{i,j}$ = acceptable and excessive release decrease, respectively. $Ra^{+}_{i,j}$ and $Ra^{-}_{i,j}$ are constrained not to exceed the user-specified desirable limits, and a penalty, RP, is imposed on $Re^{+}_{i,j}$ and $Re^{-}_{i,j}$ at reservoir j as follows:

$$RP_j = \sum_{t=1}^{i} B_{i,j}Re^{+}_{i,j} + \sum_{t=1}^{i} D_{i,j}Re^{-}_{i,j} \quad (13)$$

where $B_{i,j}$ = penalty per unit flow for a positive change in release greater than the user-specified limits; and $D_{i,j}$ = penalty per unit flow for a negative change in release greater than the user-specified limits.

Flow penalties are specified as a piecewise linear convex function of downstream flow, which is the sum of local runoff and routed reservoir releases. The penalty for flow QP is given by

$$QP_k = \sum_{t=1}^{i}\sum_{l=1}^{NF} E_{k,l} f_{i,k,l} \quad (14)$$

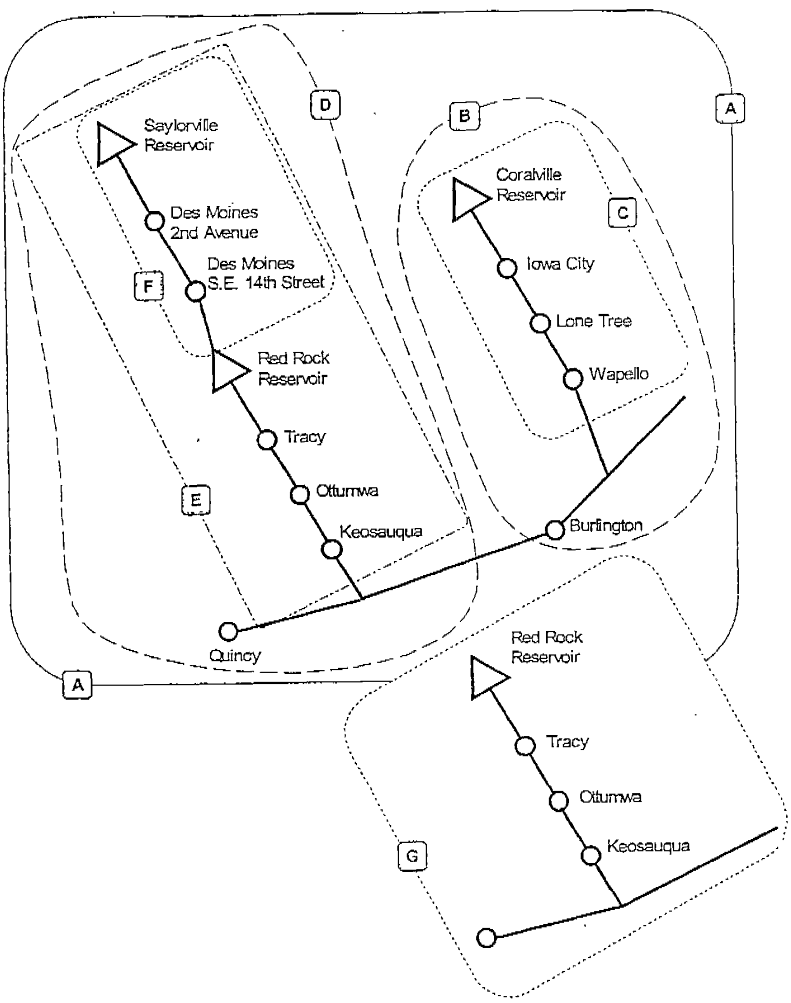

FIG. 2. System Decomposition

where $E_{k,l}$ = slope of the penalty function in flow zone l at control point k.

Incorporating penalty terms given by (11), (13), and (14), the objective function is as follows:

$$\min \mathrm{TP} = \left[\sum_{k,k\in\Psi} QP_k + \sum_{j,j\in\Phi} RP_j + \sum_{j,j\in\Phi} SP_j\right] \quad (15)$$

where TP = total penalty; Ψ = set of all control points; and ϕ = set of all reservoirs. The release schedule that yields the minimum total penalty is the optimal schedule.

It should be noted that HEC-FCLP makes release decisions for all periods simultaneously, with perfect knowledge of the complete flow hydrographs. Despite their inherent optimism, results from this type of deterministic model have proven useful for inferring general reservoir system operational policies (Lund and Ferreira 1996). Historical operation of a reservoir can be compared with the optimal operation determined by the model to identify possible shortcomings in current procedures, and questions regarding the operation of multiple reservoirs or the effects of changing physical aspects of the system can be addressed quickly.

ANALYSIS STRATEGY

The first step in the analysis is to select a number of flood events out of the approximately 70 years of record. Since the water year and the calendar year are similar in this region, the 10 years with the largest flood events were identified based on a combination of peak flow and total volume at each gauge. For each of the selected years, beginning and ending dates of the flood events were estimated visually from hydrographs.

To estimate the benefits from operating the reservoirs as a coordinated system, the larger Iowa/Des Moines/Mississippi River System was divided into various smaller subsystems as illustrated in Fig. 2. By optimizing the operations of each subsystem independently, the benefits from operating the three reservoirs as a system can be evaluated, and the question of whether or not these benefits are significant and obtainable can be addressed.

System A, the most complex, consists of the three reservoirs located on the Iowa and Des Moines rivers and all 10 control points, two of which are on the Mississippi River. System B isolates the Iowa River, which causes Coralville Reservoir to operate only for damage locations on the Iowa River, pl

Burlington on the Mississippi River. System C is similar to System B except that Burlington is removed from consideration. This illustrates the potential effect of Burlington on the operation of Coralville Reservoir. System D represents the two reservoirs on the Des Moines River operating in tandem for control at all damage locations on the Des Moines River and Quincy on the Mississippi River. System E is identical to System D except that Quincy is not considered. Dividing System D just upstream of Red Rock Reservoir to form Systems F and G helps illustrate the effect of operating Saylorville Reservoir and Red Rock Reservoir independently. Possible combinations of these systems include A, BD, CD, BE, CE, BFG, and CFG.

MODEL APPLICATION

Application of HEC-FCLP to the Iowa/Des Moines River System required the collection of flow data and the estimation of a number of model parameters. Daily incremental (local) flows and Muskingum routing parameters (e.g., Ponce 1989) for each river reach are estimated from USGS stream gauge data. Initial storage levels in each reservoir are set as the top of the conservation pool, and reservoir storage pools were divided into five zones: drought pool, conservation pool, flood-control pool, emergency flood-control pool, and flood surcharge pool. Storage-discharge capacity relationships are derived from outlet and spillway rating curves. All values are obtained from the master reservoir regulation manuals (*Master* 1983, 1988, 1990).

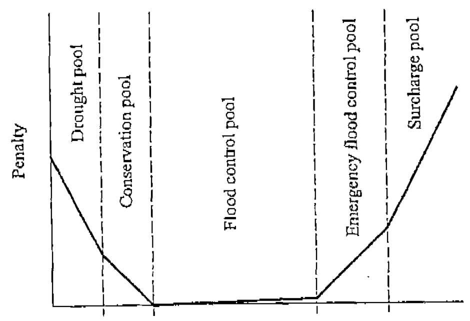

FIG. 3. Example Storage-Penalty Function

Penalties for high flow are based on economic data found in the reservoir regulation manuals and subsequent surveys conducted by the Rock Island District. The penalty functions used in this study represent the total penalty at each location, which is a combination of urban, rural, and agricultural damage. Penalty functions are developed by approximating the nonlinear flow-damage relationships with convex piecewise linear functions. Flows are divided into zones based on vertices of the penalty functions, and the same penalties are used for all flood events studied.

Rate of change of release penalties are difficult to determine. The reservoir regulation manual for Saylorville (*Master* 1983) states that a maximum change of 3,000 cfs/day is allowable during normal flood operations. This limits bank sloughing in the reservoir and along the downstream channel. A relatively large penalty of 0.1 dollars/cfs for rates of change >3,000 cfs/day is set to discourage larger rates of change but still allow them when necessary. Maximum desirable rate of change values of 3,000 cfs/day for Coralville and 6,000 cfs/day for Lake Red Rock were determined through discussions with the Rock Island District and comparisons with historical observed reservoir storage data.

Storage penalties are set to force the model to operate within the flood-control pool when feasible. The penalty prescribed when storage enters the emergency flood-control pool or the surcharge pool represents the risk associated with uncontrolled spills. A small "persuasion" penalty is placed on storage within the flood pool so that reservoir levels return to the top of the conservation pool when downstream flows recede below flood stage. Fig. 3 illustrates an example storage-penalty function.

A more detailed discussion of parameter and penalty function estimation is provided in USACE (1999).

MODEL RESULTS

The flood of 1993 is important not only because of the record-breaking flows, but also because of the time it covered.

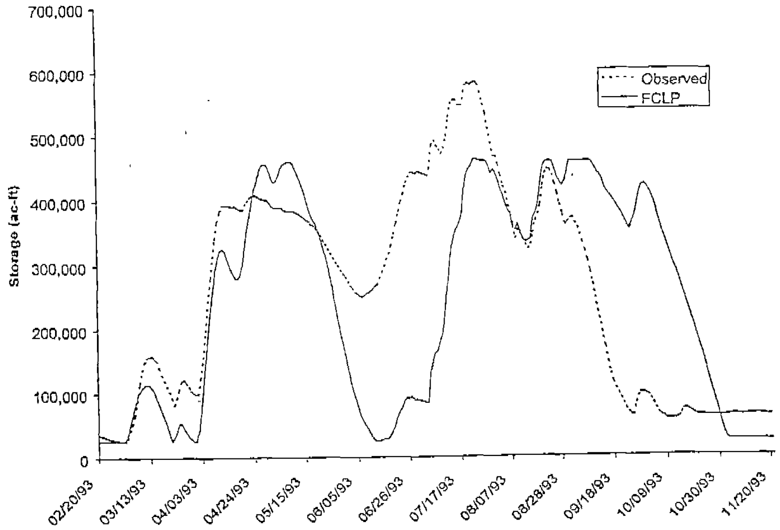

FIG. 4. Coralville Reservoir Storage—Flood of 1993

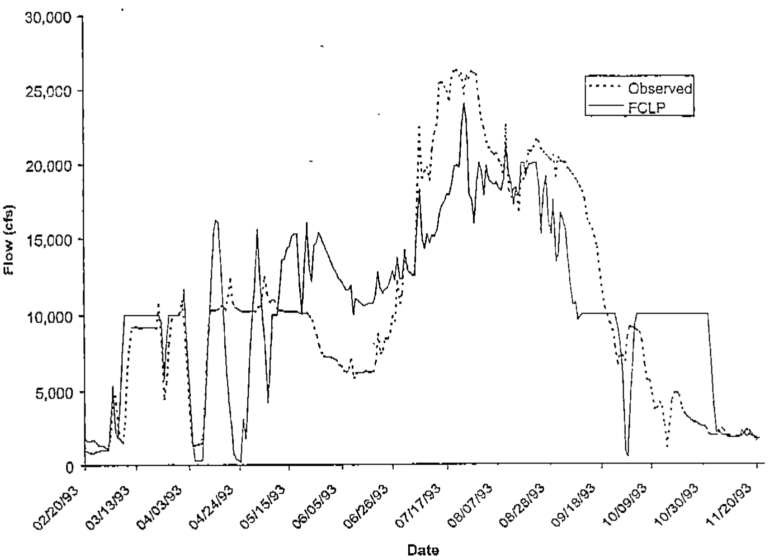

FIG. 5. Iowa City Hydrograph—Flood of 1993

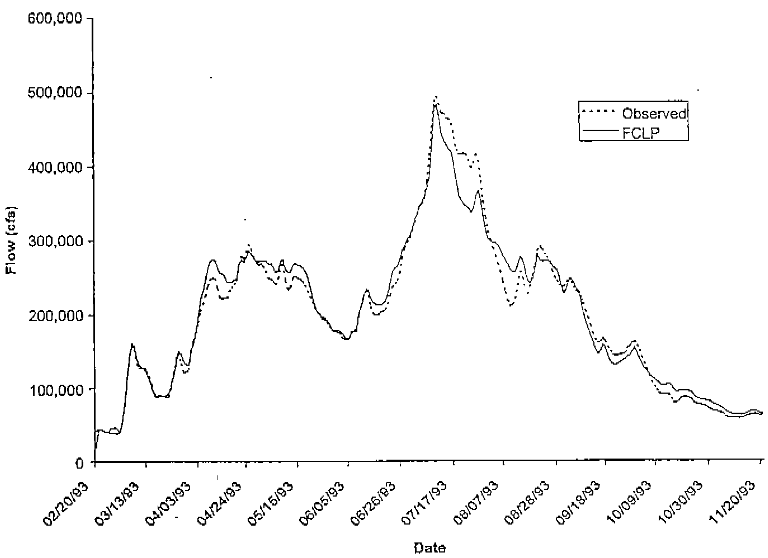

FIG. 6. Quincy Hydrograph—Flood of 1993

To consider the full impact of these flows, the model was configured to run from February 20 through November 25, a total of 282 days, which resulted in a model with >18,000 continuous decision variables, exactly 282 integer (0–1) variables, and >5,600 constraints. Figs. 4–6 illustrate results from the HEC-FCLP model and how they differ from observed data for the flood of 1993.

One would assume that the largest benefit will come from operating the reservoirs as one coordinated system. However, operating as a coordinated system also will lead to the most complex operating procedures, as well as increased hydrologic uncertainty when reservoirs are operated for points far downstream. Therefore, upon comparing the model-computed penalties resulting from the different operating schemes, the simplest operating scheme that leads to penalty values within 2% of those from System A is considered optimal. For example, if Schemes BD and BFG lead to penalty values within 2% of Scheme A, then BFG would be selected as the optimal system.

For the 1993 flood, penalty values resulting from optimal operation of the various Subsystems are listed in Table 4. Since Subsystems BD, CD, BFG, and CFG lead to essentially the same optimal penalty value as System A, there would have been little potential benefit from operating the reservoirs as a system. The only noticeable benefit would have resulted from operating Red Rock Reservoir for flood control at Quincy, indicated by Subsystems BE and CE having significantly higher penalty values than BD and CD, respectively. In 1993, the reservoir system was simply overwhelmed, and the rela

TABLE 4. Flood of 1993 Calculated Penalties (Subsystems Shown in Fig. 2)

Site (1)	Observed (2)	A (3)	BD (4)	CD (5)	BE (6)	CE (7)	BFG (8)	CFG (9)
Iowa City	69	33	31	31	31	31	31	31
Lone Tree	19	13	13	13	13	13	13	13
Wapello	313	278	278	278	278	278	278	278
2nd Avenue	212	148	148	148	148	148	148	148
14 Street	0	0	0	0	0	0	0	0
Tracy	450	422	422	422	422	422	422	422
Ottumwa	854	853	853	853	853	853	853	853
Keosauqua	102	89	89	89	97	97	89	89
Burlington	154	145	145	145	145	145	145	145
Quincy	2,208	1,427	1,430	1,431	2,006	2,008	1,430	1,431
Coralville	—	10	9	9	9	9	9	9
Saylorville	—	4,414	4,414	4,413	4,414	4,414	4,407	4,407
Red Rock	—	199	200	201	61	61	208	208
Total without storage	4,381	3,408	3,409	3,410	3,993	3,995	3,409	3,410
Total with storage	—	8,031	8,032	8,033	8,477	8,479	8,033	8,034

TABLE 5. Optimal Combinations of Subsystems

Flood year (1)	Optimal system (2)	Within 2% of optimal (3)
1993	CFG	A, BD, CD, BFG
1965	BFG	A, BD
1947	CFG	All
1973	CFG	All
1991	CE	A, BD, CD, BD
1960	CFG	All
1990	CFG	All
1979	CE	A, BD, CD, BE
1974	CFG	All
1944	CFG	All

tively small amount of flood storage provided by the three projects could not be coordinated to make an appreciable difference throughout much of the basin.

Similar reasoning was used to determine the optimal operating scheme for each of the other 10 flood events (USACE 1998). Table 5 summarizes the most basic optimal system (set of subsystems) for each flood event listed, from the most severe flood to the least severe flood.

Table 5 illustrates that during most flood events it is best to operate the three reservoirs independently. Thus, Coralville Reservoir operations are only concerned with flooding on the Iowa River, Saylorville Reservoir flood storage is used only for flood control in the city of Des Moines, and Red Rock Reservoir is operated to control flooding on the lower Des Moines River and at Quincy, Ill. Model results indicate that this policy would be the easiest to implement while still providing near-optimal results.

For the flood of 1991, and to a lesser extent the flood of 1979, potential benefits exist from operating Saylorville Reservoir and Red Rock Reservoir in tandem for flood control on the Des Moines River and at Quincy. Operating Saylorville Reservoir for flood control downstream of Red Rock Reservoir leads to a more complex release policy than the previous one, but in these cases benefits could be realized. Coralville Reservoir operates only for damages on the Iowa River for both of these events.

DISCUSSION

Model runs for the flood of 1965 are the only results in which an appreciable difference (14%) was observed at Burlington with and without flood control from Coralville Reservoir. This is due to the combination of large magnitude flows on the Mississippi River and relatively small flows on the Iowa River. During this event, operators would have been able to use the majority of Coralville Reservoir's storage for flood control at Burlington. For all other events, the penalty at Burlington was reduced by <2%.

Table 6 shows the release priorities for Coralville Reservoir flood-control operations, derived by comparing the penalty function slopes on the Iowa River and at Burlington and arranging them in descending order. Hydrographs of 1965 model results show a release >10,000 cfs from Coralville, which causes damage at Iowa City, to make space in the reservoir to dampen an upcoming peak at Burlington. This operation reduces the flow at Burlington by approximately 2,000 cfs. The 1993 flood event also recorded a peak flow above 265,000 cfs at Burlington; in this case however, Coralville Reservoir's flood control space was needed to reduce the flow at Iowa City below 20,000 cfs. From these results, it appears that operating Coralville Reservoir for flood control at Burlington is beneficial only under very special circumstances—when flows at Iowa City and Wapello can be maintained below 20,000 and 48,500 cfs, respectively.

Table 7 lists the operating priorities for the reservoirs on the Des Moines River, again based on penalty function slopes. According to this list, Saylorville Reservoir's entire flood-control pool should be used to ensure that flow at 2nd Avenue is <40,000 cfs. Moving down the priority list, if the flow at 2nd

TABLE 6. Coralville Release Priorities

Priority (1)	Keep flow less than (cfs) (2)	Location (3)
1	20,000	Iowa City—Iowa River
2	48,500	Wapello—Iowa River
3	265,000	Burlington—Mississippi River
4	10,000	Iowa City—Iowa River
5	17,500	Lone Tree—Iowa River
6	30,000	Wapello—Iowa River
7	150,000	Burlington—Mississippi River

TABLE 7. Des Moines River Flood Control Priorities

Priority (1)	Keep flow less than (cfs) (2)	Location (3)
1	40,000	2nd Avenue—Des Moines River
2	107,000	Ottumwa—Des Moines River
3	335,000	Quincy—Mississippi River
4	19,400	2nd Avenue—Des Moines River
5	19,000	Ottumwa—Des Moines River
6	270,000	Quincy—Mississippi River
7	90,000	Keosauqua—Des Moines River
8	13,000	Tracy—Des Moines River
9	28,000	Keosauqua—Des Moines River

TABLE 8. Effects of Tandem Operation of Des Moines River Reservoirs

Year (1)	Total Penalty—Des Moines River and Quincy, Ill.		
	Independent (2)	Tandem (3)	Savings (%) (4)
1993	2,943	2,943	0.0
1991	248	194	21.7
1990	33	33	0.0
1979	87	77	11.5
1974	27	27	0.0
1973	183	107	41.5
1965	158	154	2.5
1960	68	67	1.5
1947	211	206	2.4
1944	64	64	0.0
Total	4,022	3,872	3.7

Avenue is not in danger of surpassing 40,000 cfs, the flood-control space of both reservoirs should be utilized to keep the flow at Ottumwa <107,000 cfs. The remaining priorities are more complicated. For example, if the first two priorities are met, then both reservoirs should be used to keep the flow at Quincy <335,000 cfs. However, the question remains as to whether the flood-control burden should be placed evenly on the two reservoirs, or should Lake Red Rock control most of the flows and allow Saylorville Reservoir's flood storage to remain empty for protection of the city of Des Moines?

Analysis of results from Subsystems D and FG can help to answer this question. As illustrated in Table 8, subsystem results indicate that modest benefits can be obtained from operating the two reservoirs on the Des Moines River in tandem. When operating the reservoirs independently, releases from Saylorville Reservoir are regulated only for the city of Des Moines. For most events, only a small portion of Saylorville Reservoir's flood storage capacity is utilized, since inflows into the reservoir are rarely enough to fill the flood-control pool. Model results show that only for the 1965 and 1993 flood events would Saylorville Reservoir reach capacity when operated independently of Red Rock Reservoir. Flooding would be so widespread for these events that Saylorville Reservoir's flood pool would best be used mainly for control in the city of Des Moines whether operated in tandem or independently. When operated in tandem with Red Rock Reservoir, Saylorville Reservoir's flood-control pool would be filled during every event except for 1974. However, reservoir operators do not have perfect foresight, as HEC-FCLP does, and typically it would be imprudent to use the full capacity of Saylorville Reservoir's flood-control space with the city of Des Moines directly downstream.

According to Table 8, a significant benefit would have been obtained through tandem operation in 1973. During this event, large flows entered the Des Moines River System downstream of the city of Des Moines, while flow into Saylorville Reservoir was low. The rare hydrologic conditions of 1973 would have allowed Saylorville Reservoir's flood pool to be used to reduce damages downstream of Lake Red Rock. If operated independently, Lake Red Rock would not have had the flood-control capacity needed to control this flood. During most other events studied, however, the flood pool of Lake Red Rock alone would have been large enough to control the flood flows.

Since the risk assumed is large when filling Saylorville Reservoir's flood pool for control downstream of Lake Red Rock, and Lake Red Rock's flood storage is large enough to contain most floods, the majority of Saylorville Reservoir's flood pool should probably be reserved for flood control in the city of Des Moines. A possible solution would be to divide the Saylorville Reservoir flood pool into two "virtual" pools—one for flood control downstream of Lake Red Rock and the other for flood control in the city of Des Moines.

TABLE 9. Effects of Operating Des Moines Reservoir for Flood Control at Quincy

Year (1)	Total Damage—Des Moines River Tandem Operation		
	Without Quincy (2)	With Quincy (3)	Savings (%) (4)
1993	3,526	2,942	16.5
1973	109	108	0.9
1965	205	155	24.4
1960	69	67	2.9

The results endorse operating the Des Moines Reservoirs for flood control on the Mississippi River at Quincy, Ill. Benefits at Quincy are seen in all four of the years studied that had damaging flows on the Mississippi River. Table 9 illustrates the flow-related penalty reduction for these four events when the Des Moines Reservoirs operate for flood control at Quincy.

HEC-FCLP model results and observed 1993 operations have many significant differences. Although Table 4 shows that the total flow-related penalty could have been reduced by nearly 25%, it is incorrect to conclude that current operating procedures are inadequate without first looking at where and why the differences in penalty occur.

The most notable difference between observed and model results during the flood of 1993 is in the operation of Coralville Reservoir. With perfect foresight, HEC-FCLP drew down the reservoir much more in the first few weeks of June than was recorded, as illustrated in Fig. 4. Historical data show releases were cut back to prepare for the planting season downstream although the reservoir was still relatively full. The additional HEC-FCLP drawdown allowed Coralville Reservoir to provide more protection from the large inflows that occurr in late July and August. Were this policy of rapidly drawi down Coralville Reservoir following a flood event adopteu every year, it would likely result in greater agricultural losses. The operating procedures for Coralville Reservoir should be reviewed with this in mind.

It is impossible to predict hydrologic conditions 2–3 months in the future with enough certainty to justify making damaging prereleases. Even with much shorter lead times, it is often difficult to convince the general public that they should be flooded today in order to reduce potential system-wide damage in the future. As shown in Fig. 7, such a policy was proposed by the model for the flood of 1993, when the Lake Red Rock flood-control pool was kept empty for 3 months in order to dampen the mid-July flood peak. The optimization results are impracticable in this regard, serving mainly to represent the lower bound of flood damage from a flood event.

LIMITATIONS

As with all reservoir model applications, the implications of the results from this study are limited by the approximations necessary to model an existing physical system. The most widely recognized limitation inherent to LP is that all relationships in the model, such as routing equations and penalty functions, must be approximated with linear or piecewise linear functions. Nonlinear flood routing techniques—those using variable routing coefficients—may provide more accurate results than do linear techniques over a range of discharges [e.g., Ponce (1989)]. Similarly, a nonlinear and/or discontinuous objective function may be more appropriate than the linear (additive) function used in this study.

Additional limitations specific to HEC-FCLP that should

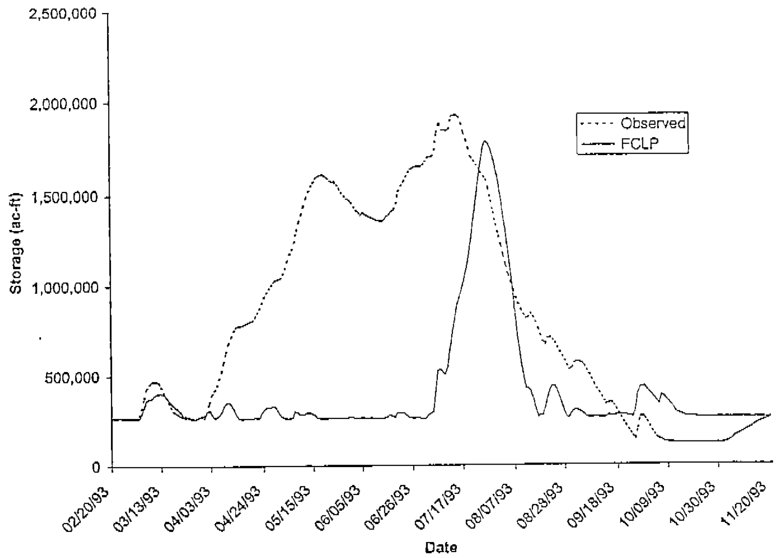

FIG. 7. Lake Red Rock Storage—Flood of 1993

considered during the analysis of practical problems include the following:

- Penalties on change in release, flow, and storage are assessed for each period. This duration-based solution does not directly account for flood damage, especially urban flood damage that is primarily a function of peak discharge, but it does capture the desire of reservoir operators to reduce the flow in flooded areas as soon as possible. The duration-based penalty leads to more realistic results during nonpeak periods than does a penalty function based solely on peak flows. HEC-FCLP is currently being modified to incorporate a combination of peak-based and duration-based penalties.
- HEC-FCLP does not account for the seasonal variation of damage potential that is often associated with agricultural areas. Consequently, for a flood event that spans two or more seasons, it cannot properly optimize the seasonal allocation of flood storage capacity that is common in flood-control systems.
- FCLP is a deterministic model, which means it implicitly assumes that future flows are known with certainty. Thus, as illustrated by this study, HEC-FCLP draws down reservoirs in anticipation of future floods, perhaps even months in advance. While this is not necessarily a limitation, it is an important feature that should be taken into account when using results from deterministic optimization models.

When used for planning studies, computational time is not a limitation of HEC-FCLP. In this study, a global optimum solution to each mixed-integer LP problem was obtained in <15 min using a 200-MHz Pentium II PC. However, this solution time might limit the use of HEC-FCLP for real-time decision support.

CONCLUSIONS

The method of dividing the system into various smaller systems produces results that quantify the potential benefits of making reservoir releases based on selected control points. For the majority of flood events studied, the optimal operational policy would be to operate each reservoir independently. Results indicate that Coralville Reservoir could be operated for flood control on the Iowa River with little consideration for Burlington, Saylorville Reservoir's flood capacity could be used mainly for flood protection in the city of Des Moines, and Lake Red Rock could be operated for flood control on the Lower Des Moines River and at Quincy.

Operating Coralville Reservoir for flood control on the Mississippi River is risky because flood-control space is consumed that could prove more valuable to Iowa City. It is acceptable to operate Coralville Reservoir for flood control on the Mississippi River as long as current and forecasted flows in the Iowa River are low. Optimization results illustrate that this scenario occurred only once during the historical record.

By dividing the Des Moines River just upstream of Lake Red Rock, the effect of operating the two reservoirs, Red Rock and Saylorville, in tandem was illustrated. When operated in tandem, most of Saylorville Reservoir's flood control capacity was used for protection downstream of Lake Red Rock. When operated independently, the majority of Saylorville Reservoir's flood pool capacity was rarely used, and the resulting flows were regulated by Lake Red Rock. Penalty values obtained from tandem and independent operation were within 3% for most of the flood events studied. Since the city of Des Moines is potentially one of the highest damage locations on the river, it would be more risk averse to save a majority of Saylorville Reservoir's flood storage for the city of Des Moines and use Lake Red Rock for flood control downstream.

Review of operations during the Great Flood of 1993, a very rare event, shows that damages could have been reduced if inflows were known months in advance. Obviously, this is not possible with current forecasting technology. However, the damage could also have been reduced during the 1993 flood if current reservoir operation were more averse to extreme events. Release decisions during the flood of 1993 were made based on knowledge of previous events. With new data and a better understanding of the runoff these drainage areas can produce, the release rules should be modified to account for events of this magnitude in the future.

Deterministic optimization models are useful for evaluating the potential benefits of a reservoir system when analyzing operating procedures, but results from these models need to be kept in perspective. Detailed simulation modeling should always accompany these omniscient optimization procedures when developing operating rules for a reservoir or set of reservoirs, since simulation models can give a more accurate estimate of the system performance given a set of operating policies.

Optimization models are only as good as their penalty functions and constraints. Establishing these penalty functions and producing a standard set of historical inflows is an important, though time-consuming task. Not only has this study led to increased understanding of the Iowa/Des Moines Reservoir System—the potential flood control value of each reservoir and the potential damage at various locations—but it has also produced a standardized set of data that will prove invaluable in future studies.

ACKNOWLEDGMENTS

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State-Federal Flood Operations Center

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Overview

The mission of the Division of Flood Management is to prevent loss of life and reduce property damage caused by floods and to assist in recovery efforts following any natural disaster. The State-Federal Flood Operations Center (FOC), located in Sacramento, California, is a component of the Division's Emergency Response Section. Year-round, the FOC is the focal point for the gathering, analysis, and dissemination of flood and water-related information. During emergencies, the FOC provides a facility from which DWR can effectively coordinate emergency response.

River Forecasts

As major storm systems approach California, forecasters from the National Weather Service and Department of Water Resources forecast the location, amount, and timing of expected precipitation, and make initial river forecasts. Once the storm arrives and runoff begins, updated forecasts are issued as necessary. Reservoir operators adjust flood control releases as inflows increase or downstream channels swell with runoff. If runoff is sufficient to raise streams to threatening levels, the NWS and DWR issue these forecasts as official public bulletins. Automated NWS and DWR computer systems disseminate bulletins and FOC staff make high water notification calls to selected agencies. Depending on the event's severity, DWR may declare a Flood Alert or Flood Mobilization.

High Water Notifications

When streams are forecast to rise above certain levels, FOC personnel make high water notification calls to appropriate local flood system maintaining and emergency response agencies. On leveed streams, maintaining agencies are required to patrol their levees on a 24-hour basis as long as the level is at or above monitor stage, and until no threat remains to the levees.

Flood Alert

Forecasts of sustained storm patterns and resulting flood potentials, the need for coordinated field operations, or requests for technical support from local agencies may require the Flood Operations Branch Chief to declare a Flood Alert to officially activate the Flood Operations Center under the Standardized Emergency Management System (SEMS). Flood Management personnel from the Flood Operations and Hydrology Branches, the National Weather Service Sacramento Forecast Office, and when applicable, the Information Services Branch of the Office of Water Education expand their regular duties to meet these needs. If additional personnel are needed they are first obtained from within the Division of Flood Management then from other areas in the Department.

Flood Mobilization

Additional Department personnel, equipment, material, and financial resources may be needed for extended periods to respond to sustained severe storms and flooding. The Director may declare a Flood Mobilization to meet this need. The Division of Flood Management is authorized to use any Department personnel and expenditures beyond budgeted funding during a Flood Mobilization.

Standardized Emergency Management System (SEMS)

Developed by the Governor's Office of Emergency Services, SEMS is the framework for coordinating emergency response in California. SEMS utilizes the Incident Command System originally developed by fire agencies for managing wildfire response. State and local agencies use SEMS to improve the mobilization, deployment, utilization, communication, tracking, and demobilization of mutual aid resources used in an emergency. SEMS is organized into the following five functions at the FOC:

Management – directs all FOC activities, responsible for overall policy and coordination, coordinates emergency assistance requests, provides Public Information support

Operations – coordinates with Incident Command Posts, Mobilization Centers, and other field operations units including flood fight, technical assistance and emergency repairs

The Operations Section coordinates and dispatches joint DWR / Corps of Engineers technical teams to investigate potential flood threats. Flood fight crews are dispatched when necessary, with a majority of the crews provided by the California Department of Forestry and Fire Protection and the California Conservation Corps.

Planning & Intelligence – collects, evaluates, documents and disseminates flood emergency information, tracks personnel and equipment resources, and provides event planning assistance

Information includes river and weather conditions and forecasts, reservoir operations, levee and other flood-related incident reports and maps.

Logistics – provides services, personnel, equipment and facilities to support operations

Finance & Administration – overall responsibility for fiscal accounting, compensation and claims

Cooperating Agencies

In addition to the National Weather Service, many agencies cooperate with DWR during flood emergencies and some send representatives to work at the FOC. These agencies include:

National Weather Service (NWS): – The mission of the NWS Hydrologic Services Program is to: (1) provide river and flood forecasts and warnings for the protection of lives and property, and (2) provide basic hydrologic forecast information for the nation's environmental and economic well being. Eight Weather Forecast Offices located in Medford (OR), Eureka, Reno (OR), Sacramento, Monterey, Hanford, Oxnard and San Diego, and the California-Nevada River Forecast Center accomplish this. Both Sacramento offices are co-located with the FOC at the Joint Operations Center.

Governor's Office of Emergency Services (OES) – coordinates the emergency activities of all State agencies. When requested by county Operational Areas, OES will direct (through the assignment of mission numbers) those State agency resources necessary to support flood fight operations. OES shall request, as directed by the Governor, a Presidential Emergency and/or a major disaster declaration. DWR representatives are dispatched as needed to OES's Inland (Sacramento), Coastal (Oakland) and Southern (Los Alamitos) Regions, while OES representatives are assigned to the FOC.

California Department of Forestry and Fire Protection (CDF) – CDF provides a majority of the crews used in flood fight activities. CDF also assists OES by setting up Mobilization Centers, mobile kitchens and other facilities. CDF's expertise in the Incident Command System is a valuable resource during flood emergencies.

California Conservation Corps (CCC) – the CCC provides personnel for flood fight crews and levee patrols during emergencies. Standby crews are frequently stationed near sites where problems are anticipated due to storm activity, high river stages, high tides or heavy reservoir releases.

California State Water Project (SWP): – The SWP Operations Center, located one floor above the FOC, participates in daily briefing and planning activities and coordinated reservoir operations with respect to Lake Oroville and other SWP facilities.

U.S. Army Corps of Engineers – In instances when the nature of the disaster exceeds the capabilities of State and local interests, the Corps may provide assistance under Public Law 84-99 to save human life, prevent immediate human suffering or mitigate residential and commercial property damage. Assistance includes: acquisition of flood fight materials; geotechnical evaluation of levees and other flood operations structures, contracts for emergency flood fight and temporary repairs, clearance of drainage channels or blocked structures; technical assistance for development of plans; and upon request, inspection of non-Federal dams and flood control projects. The Corps also has jurisdiction over storage capacity seasonally reserved for flood control on most major reservoirs throughout the State.

U.S. Bureau of Reclamation – Although the Bureau of Reclamation is primarily involved in the irrigation and hydropower purposes of its federal water projects, many USBR reservoirs also provide flood control storage. In the Central Valley, these projects include Shasta Dam on the Sacramento River; Folsom Dam on the American River; New Melones Dam on the Stanislaus River, and Friant Dam on the San Joaquin River. The Central Valley Operations Office, located one floor above the FOC, participates in daily briefing and planning activities and coordinated reservoir operations.

U.S. Geologic Survey – The Survey participates in a flood emergency by measuring, processing and sharing streamflow data. The Survey cooperates with DWR and NWS in establishing telemetered stream gages necessary for flood operations.

Levee Maintaining Agencies – Local agencies have primary authority for both maintenance of levees and flood fighting. Levee maintenance is provided by public levee districts, local government entities, private levee owners, and is some cases DWR. Collectively these agencies are referred to as Levee Maintaining Agencies or LMA's. Some levees are not maintained by private or public agencies.

Operational Areas – representatives from the Sacramento Operational Area (Sacramento County being the geographic boundary) may be assigned to the FOC when county streams and levees are threatened. The FOC provides intelligence and planning support to Operational Areas as needed throughout a flood emergency.

Contacts

The Flood Operations Center provides a telephone bank of Flood Information Specialists to answer flood-related questions, assist with high water notification calls, and direct calls to other appropriate personnel. Public Information Officers from the Office of Water Education coordinate Media briefings and interviews. This information is available on request to other agencies, the media and the public, either by telephone or via the California Data Exchange Center operational hydrologic database.

Toll-free public lines .... (800) 952-5530, recordings
(24-hour emergency contact via answering service)
TDD service .... (800) 900-3582

Eureka Flood Center .... (707) 445-6576
North Coast Flood and River Information.................................. (707) 445-7855, recording

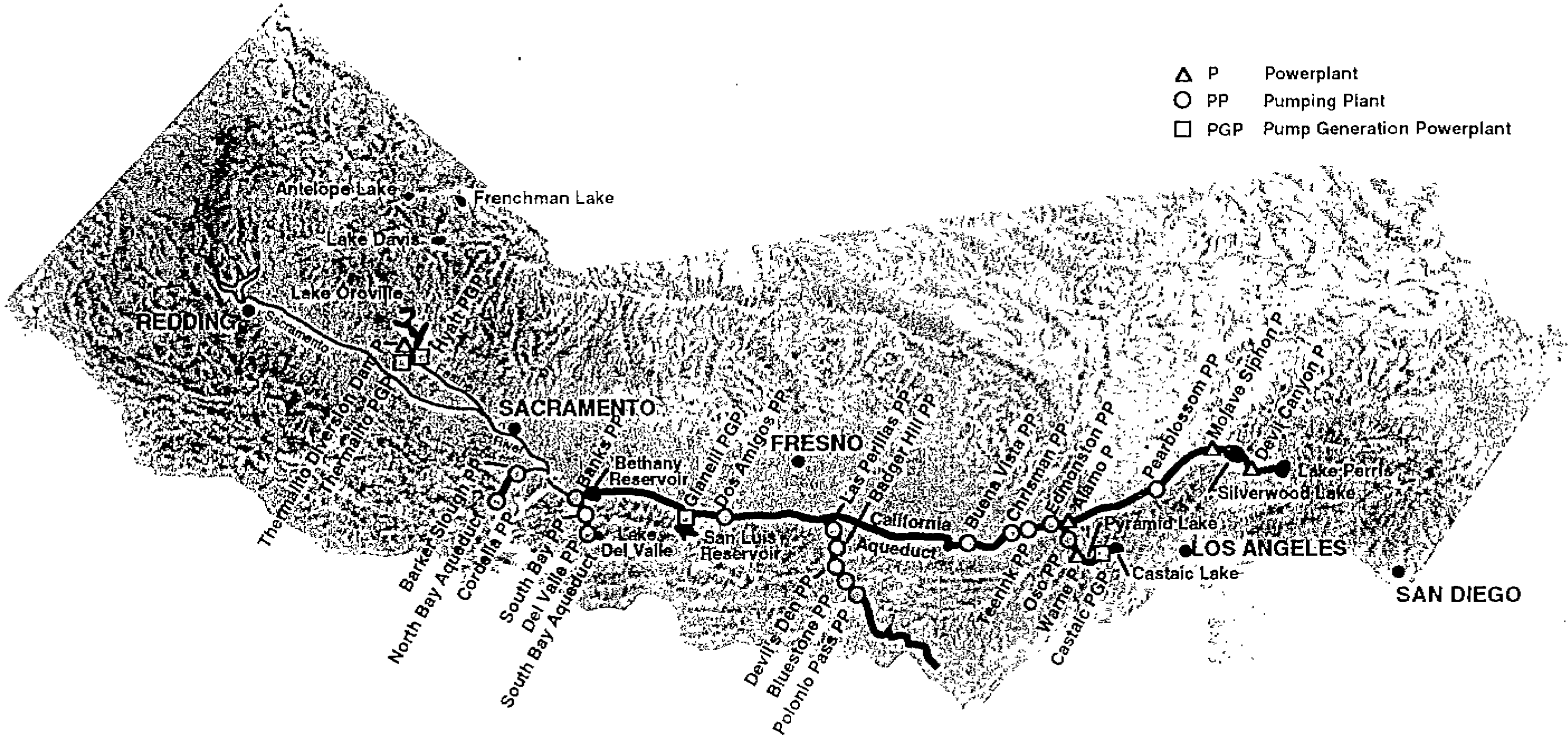
DWR Office of Water Education .... (916) 653-6192

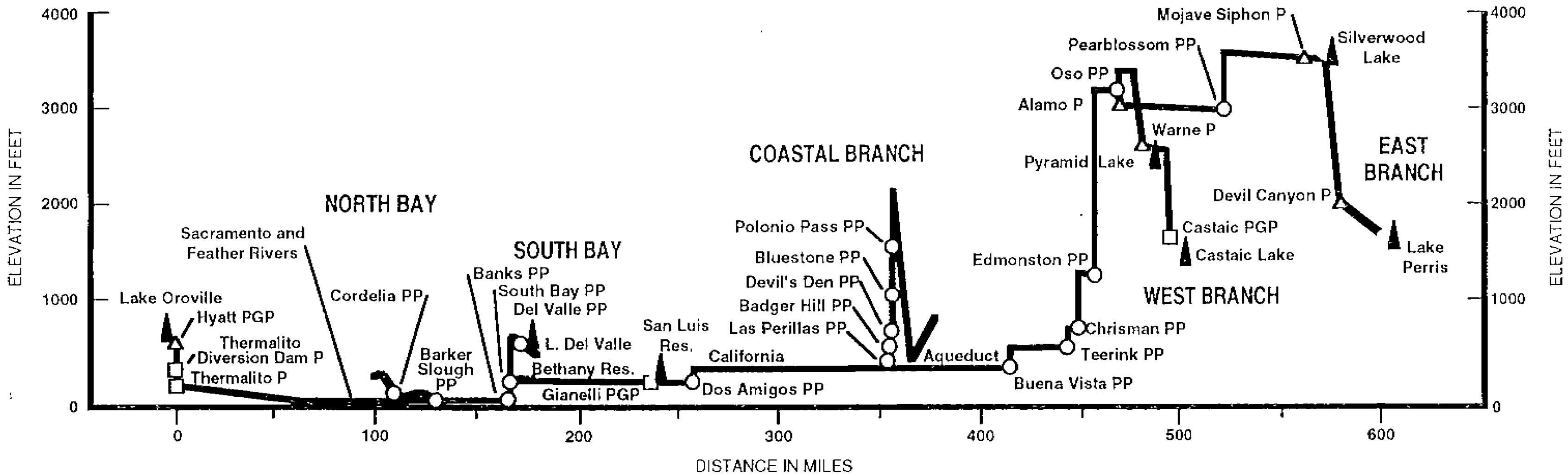
California Data Exchange Center Website.................................. http://cdec.water.ca.gov
CDEC Help Desk .... (916) 574-1777
California Flood Information (CERES) Website .... http://www.ceres.ca.gov/flood/

NWS Weather Forecast Office Website .... http://www.wrh.noaa.gov/sacramento/
NWS River Forecast Center Website .... http://www.wrh.noaa.gov/cnrfc/

附件二

Location and Profile of State Water Project Facilities





The California State Water Project

WHAT IS IT?

Planned, designed, built, and operated by the California Department of Water Resources, the State Water Project (SWP) is the largest state-built multipurpose water project in the U.S. It is a water delivery system of 29 storage facilities, 18 pumping plants, four pumping-generating plants, five hydroelectric power plants, and approximately 600 miles of canals and pipelines.

WHY WAS IT BUILT?

California's water supply varies from year to year, season to season and area to area. The major water sources are in northern California, while the major urban centers and agricultural lands are in the northern Bay area, central valley, and southern California. Seventy percent of the total stream runoff is north of Sacramento, but 80 percent of the water demand is south of that city. The SWP's main purpose is water supply. The project diverts and stores surplus water during wet periods and distributes it to areas of need in Northern California, the San Francisco Bay area, the San Joaquin Valley, the Central Coast, and Southern California.

Other project benefits are flood control, power generation, recreation, fish and wildlife enhancement, and water quality improvement in the Sacramento-San Joaquin Delta.

WHERE IS IT?

The Project extends for more than 600 miles north to south through the State. Water first stored in Lake Oroville in Butte County flows through the Hyatt and Thermalito hydroelectric plants and reenters the natural channel of the Feather River. From here, the water winds its way to the Sacramento River and to the Delta.

The 444-mile California Aqueduct begins at the Delta Pumping Plant in the south Delta. The Aqueduct carries water southward through the San Joaquin Valley over the Tehachapi Mountains and into southern California, where the Aqueduct divides. The West Branch terminates at Castaic Lake in north Los Angeles County, while the East Branch ends at Lake Perris in Riverside County.

WHO PAYS FOR IT?

The 29 contracting agencies that receive SWP Water are paying for its major costs. A $1.75 billion bond issued in 1960 provided the initial funding. Payments received from the contracting agencies are paying off the bonds.

WHEN WAS IT BUILT?

Construction of the Project began in 1957. Although the initial facilities were completed in 1973, the expansion of SWP facilities continues.

HOW MUCH WATER IS DELIVERED?

With its existing facilities, the Project can deliver 2.3 million acre-feet*, somewhat more in wet years. At full capacity, the Project would eventually deliver 4.2 million acre-feet a year.

*An acre-foot is 326,000 gallons, enough supply for one to two average families a year. Revised 6/97

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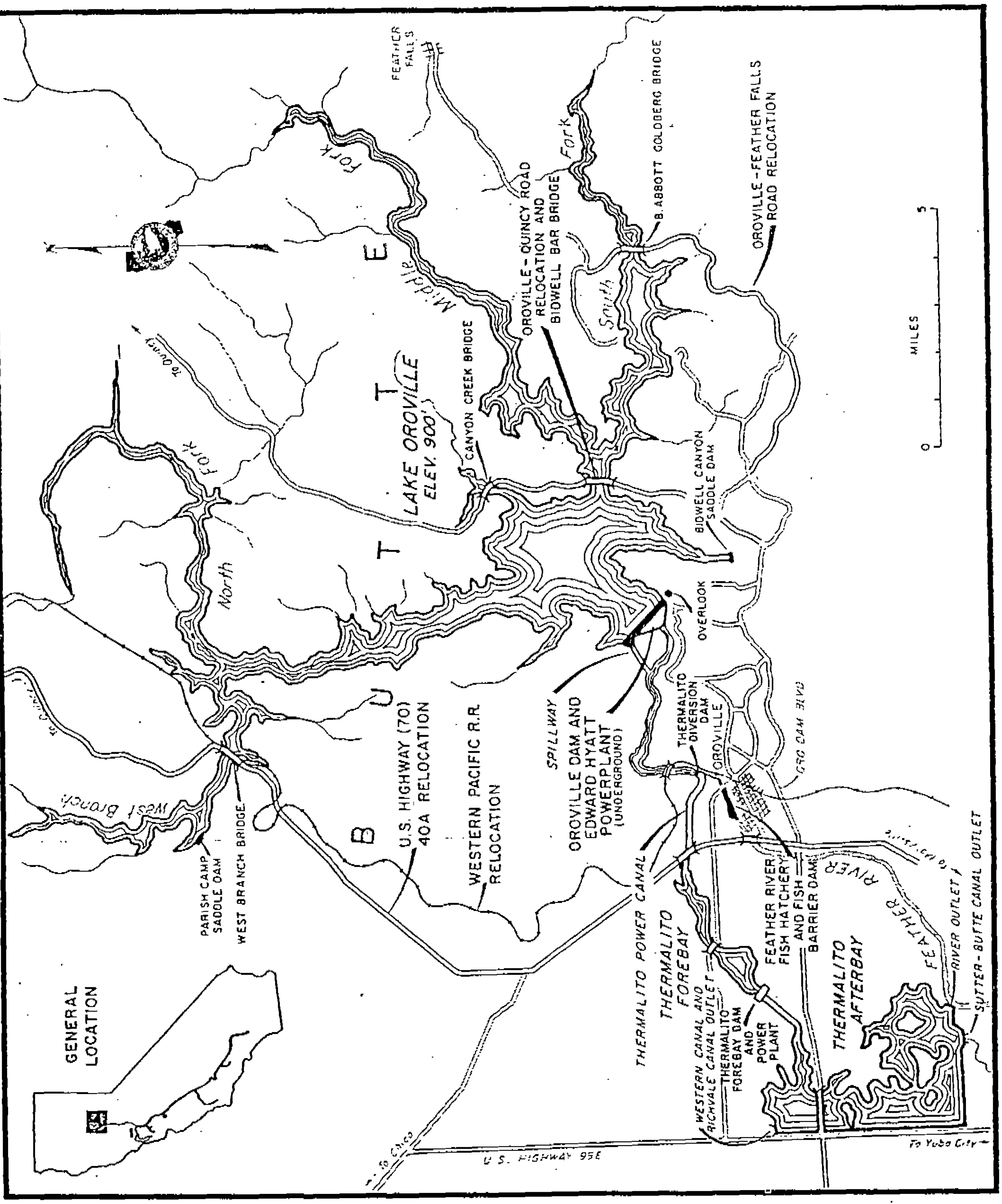

GENERAL LOCATION
PARISH CAMP SADDLE DAM
WEST BRANCH BRIDGE
West Branch
B U T T E
U.S. HIGHWAY (70) 40A RELOCATION
WESTERN PACIFIC R.R. RELOCATION
SPILLWAY
OROVILLE DAM AND EDWARD HYATT POWERPLANT (UNDERGROUND)
North Fork
LAKE OROVILLE ELEV. 900'
CANYON CREEK BRIDGE
To Quincy
Middle Fork
OROVILLE - QUINCY ROAD RELOCATION AND BIDWELL BAR BRIDGE
South Fork
B. ABBOTT GOLDBERG BRIDGE
FEATHER FALLS
OROVILLE - FEATHER FALLS ROAD RELOCATION
BIDWELL CANYON SADDLE DAM
OVERLOOK
THERMALITO DIVERSION DAM
OROVILLE
THERMALITO POWER CANAL
THERMALITO FOREBAY
WESTERN CANAL AND RICHVALE CANAL OUTLET
THERMALITO FOREBAY DAM AND POWER PLANT
FEATHER RIVER FISH HATCHERY AND FISH BARRIER DAM
THERMALITO AFTERBAY
FEATHER RIVER
RIVER OUTLET
SUTTER-BUTTE CANAL OUTLET
U.S. HIGHWAY 99E
To Chico
To Yuba City
MILES
0
5

GEOLOGICAL ORIGINS

OF THE FEATHER RIVER CANYON

The Feather River Canyon is the result of the volcanic activity and water erosion in a relatively recent geological period. Events which led up to this development, however, began in a much more remote period of time.

The evolution of the Sierra Nevada began perhaps 120 to 130 million years ago, in the late Jurassic Period, when the western half of a great sedement-filled trough was crumpled into mountains. Available evidence indicates that this took place beneath the sea and that the mountains thus formed appeared as islands, to be "welded" into the continent at a later time. The height of these mountains is not known, but it is estimated to have been about 6,000 to 7,000 feet above sea level.

About a mile below the surface, meanwhile, the formations of igneous rock, solidified from molten masses, developed a huge body of granitic rock under a very large part of the range. This rock, under tremendous pressure, was forced upward through weak spots in the earth's crust to form the large masses of granite subsequently exposed by erosion.

By the beginning of the Cenozoic Era, about 60 million years later, the mountains had been so eroded that they were nearly inconspicuous and in places the ocean had approached the base of the range. The land was so low that western winds carried moisture over it and into the land to the east. This area, so arid at the present time, was then characterized by luxuriant vegetation.

During the Eocene Epoch, the second part of the Cenozoic Era, the land was bowed upward, creating a low mountain barrier into which streams cut deep gorges. About 10 million years later it was again uplifted to sufficient height to catch and hold the moisture from the western winds.

Much more vigorous disturbances in the Miocene, Pliocene, and Pleistocene epochs - comprising approximately 30 million years - elevated the range to its present height.

Volcanic activity and water erosion played the principle parts in the development of the Sierra Nevada in more recent epochs. Glacier action within the last one million years affected parts of the range greatly, as is evidenced by Mono Lake, Yosemite Valley and other places. The Feather River Canyon however, is non-glaciated it is one of many valleys throughout the range which were formed by water erosion.

About 20 million years ago volcanic eruptions blocked the huge 500 square mile watershed which is now drained by the North Fork of the Feather River, forming a large lake. Gradually a notch began to develop, permitting the water to slowly drain out, which in turn enlarged the notch. The erosion developed in this way cut through lava rock and clay to create the beautiful Feather River Canyon as we know it.

Sediment deposited on the former lake bottom had formed a broad flat area which was named Big Meadows early in the American period of California history. The construction of Big Meadows Dam in 1914 created modern Lake Almanor, which stores water for regulated release for hydroelectric generation downstream.

STATE OF CALIFORNIA
THE RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

DATA SHEET
OROVILLE FIELD DIVISION
STATE WATER PROJECT

A. OROVILLE DAM *

1. General

Drainage Area	3,611 sq.mi.	9,354 sq.km.
Runoff, Estimated Full Natural		
Average Annual Flow(1912-1957)	4,138,000 ac.ft.	5,104,223,000 cu.m.
Runoff, Estimated Project Impaired		
Average Annual Flow(1921-1951)	3,490,000 ac.ft.	4,304,915,000 cu.m.
Maximum Instantaneous Flood Peak		
1907 (Estimated)	230,000 c.f.s.	6,513 c.m.s.
1955	203,000 c.f.s.	5,748 c.m.s.
1964	245,000 c.f.s.	6,941 c.m.s.
1986	266,000 c.f.s.	7,532 c.m.s.

2. Reservoir

Pool Elevation Above Sea Level(U.S. Coast & Geodetic Survey, 1929 Adjustment)		
Minimum Operable Power Pool	640.0 ft.	195.1 m.
Flood Control Pool(Secondary)	847.6 ft.	258.4 m.
Flood Control Pool(Primary)	874.8 ft.	266.7 m.
Normal Pool	900.0 ft.	274.4 m.
Spillway Flood Pool	917.0 ft.	279.6 m.
Reservoir Area at Elev. 900'	15,500 ac.	6,275 ha.
Reservoir Shore Line at 900'	167 mi.	268.7 km.
Reservoir Storage		
Minimum Operable Power Pool	834,600 ac.ft.	1,029,479,100 cu.m.
Flood Control Pool(Secondary)	2,734,200 ac.ft.	3,372,635,700 cu.m.
Flood Control Pool(Primary)	3,109,200 ac.ft.	3,835,198,200 cu.m.
Normal Pool	3,484,000 ac.ft.	4,297,514,000 cu.m.
Spillway Flood Pool	3,625,000 ac.ft.	4,471,437,500 cu.m.
Conservation Pool		
Summer	3,484,000 ac.ft.	4,297,514,000 cu.m.
Winter	2,734,000 ac.ft.	3,372,389,000 cu.m.
Flood Storage	750,000 ac.ft.	925,124,000 cu.m.
Mandatory Release (Irrigation,Fish,Domestic)	3,700 c.f.s.	104.8 c.m.s.

DATA SHEET
OROVILLE FIELD DIVISION
STATE WATER PROJECT

Oroville Dam

Type: Graded Gravel and Rolled Earthfill		
Height:	770 ft.	234.7 m.
Volume:	80,600,000 c.y.	61,256,000.0 cu.m.
Crest Elevation:	922 ft.	281.0 m.
Crest Length:	6,922 ft.	2,109.8 m.
Base Width:	3,570 ft.	1,088.1 m.
Diversion Tunnels (2)		
Length No.1	4,300 ft.	1,310.6 m.
Length No.2	4,600 ft.	1,402.0 m.
Diameter	35 ft.	10.6 m.

Oroville Dam Spillway

Control Gates:	8 (17.6'x33.5')	
Controlled Capacity:	150,000 c.f.s.	4,247.5 c.m.s.
Emergency Capacity:	477,000 c.f.s.	13,507.2 c.m.s.
Chute Length:	3,055 ft.	931.2 m.
Chute Width:	179 ft.	54.5 m.
Chute Depth:	28 ft.	8.5 m.
Emergency Spillway		
Crest Length:	1,730 ft.	527.3 m.
Crest Elevation:	901 ft.	274.6 m.
Maximum Release:	359,000 c.f.s.	10,165.8 c.m.s.

Edward Hyatt Power Plant

Type: Underground Rock Chamber		
Excavation:	1,400,770 cu.y.	1,101,506.0 cu.m.
Length:	550 ft.	167.6 m.
Width:	69 ft.	21.0 m.
Depth:	137 ft.	41.7 m.
Units:		
(3) Conventional:		140 mw.
(3) Reversible:		130 mw.
Transformers:	6	230/13 kv.
Intake Structure:		
TYPE: 2 Sloping Rectangular Channels		
Height: Elevation	614 ft. to 900 ft.	187.1 m. to 274.3 m.
Length:	650 ft.	198.2 m.
Penstock Tunnels and Branches:		
Diameter:		
Main Penstocks:	2 at 22 ft.	2 at 6.7 m.
Branches:	6 at 12 ft.	6 at 3.6 m.
Capacity of Branches:		
Turbines	3 at 3,400 c.f.s.	3 at 96.3 c.m.s.
Pump Turbines	3 at 2,600 c.f.s.	3 at 73.6 c.m.s.

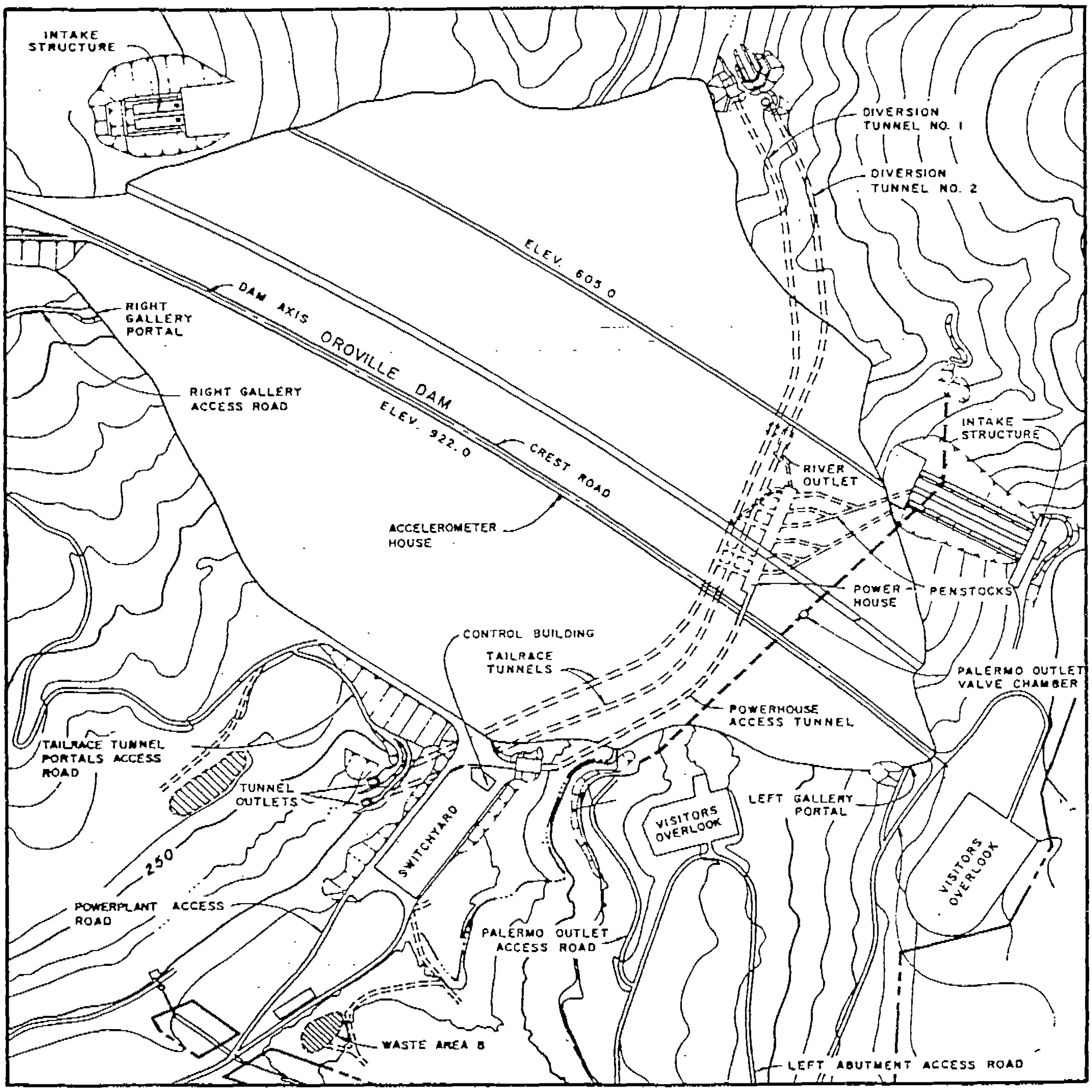
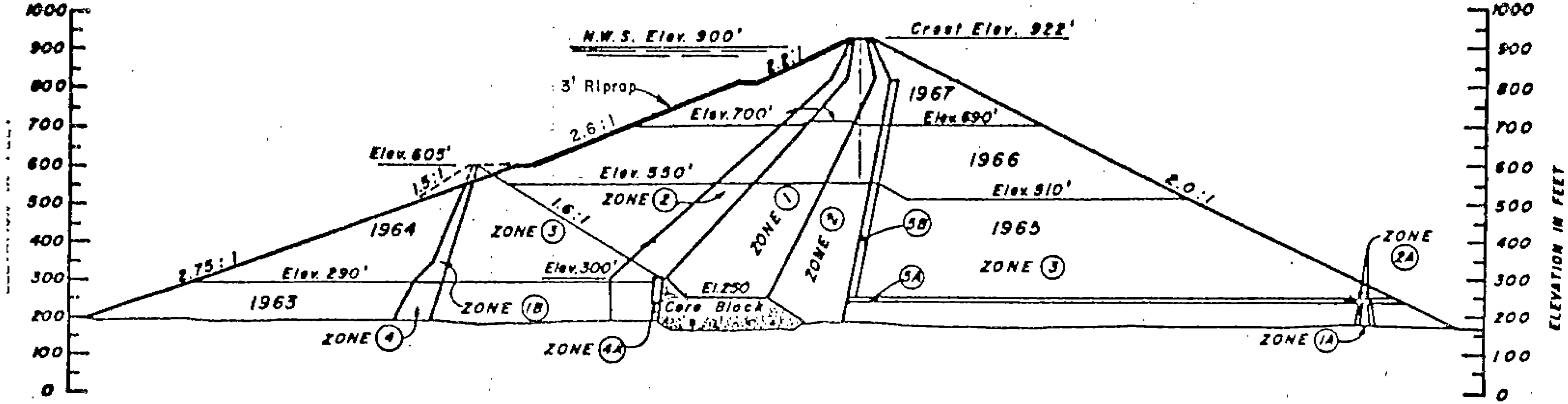


Figure 61. General Plan



SCHEDULE OF MATERIALS REQUIREMENTS, CUBIC YARDS

CONSTR. YEAR	RIPRAP	PERVIOUS	TRANSITION	IMPERVIOUS	TOTAL	CONCRETE
1962	0	0	0	0	0	0
1963	0	1,600,000	200,000	200,000	2,000,000	290,700
1964	0	12,600,000*	300,000	400,000	13,300,000	0
1965	5,000	16,000,000	3,500,000	3,100,000	22,600,000	0
1966	114,000	16,500,000	3,200,000	2,600,000	22,400,000	0
1967	294,000	14,400,000	2,300,000	2,700,000	19,700,000	0
TOTAL	413,000	61,100,000	9,500,000	9,000,000	80,000,000	290,700

* MINIMUM 1964 PERVIOUS REQUIREMENT TO CONSTRUCT EMBANKMENT TO ELEVATION 605 IS 9,455,000

OROVILLE DAM

MAXIMUM SECTION SHOWING

PROPOSED CONSTRUCTION

SCHEDULE

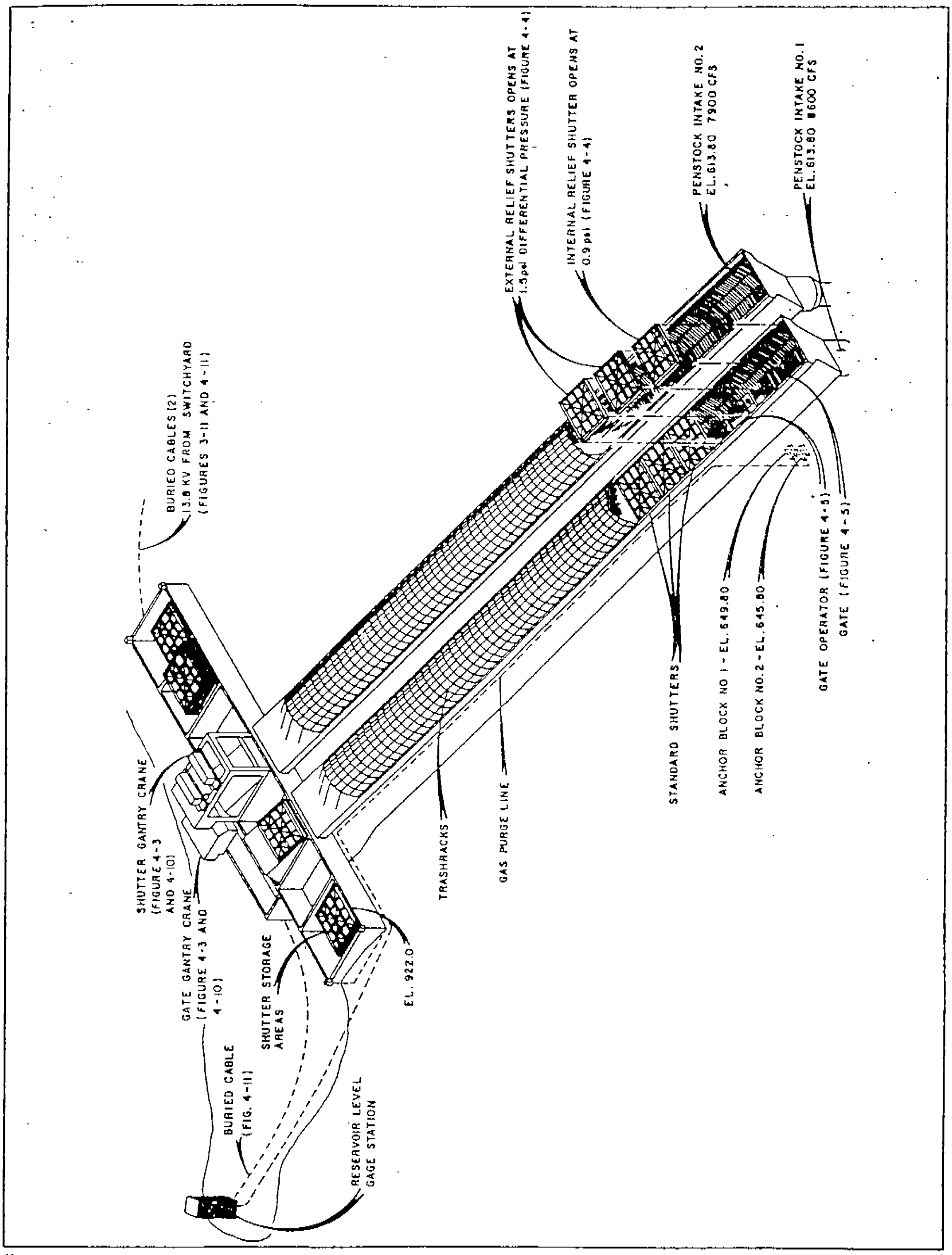


Figure 64. Intake Structure

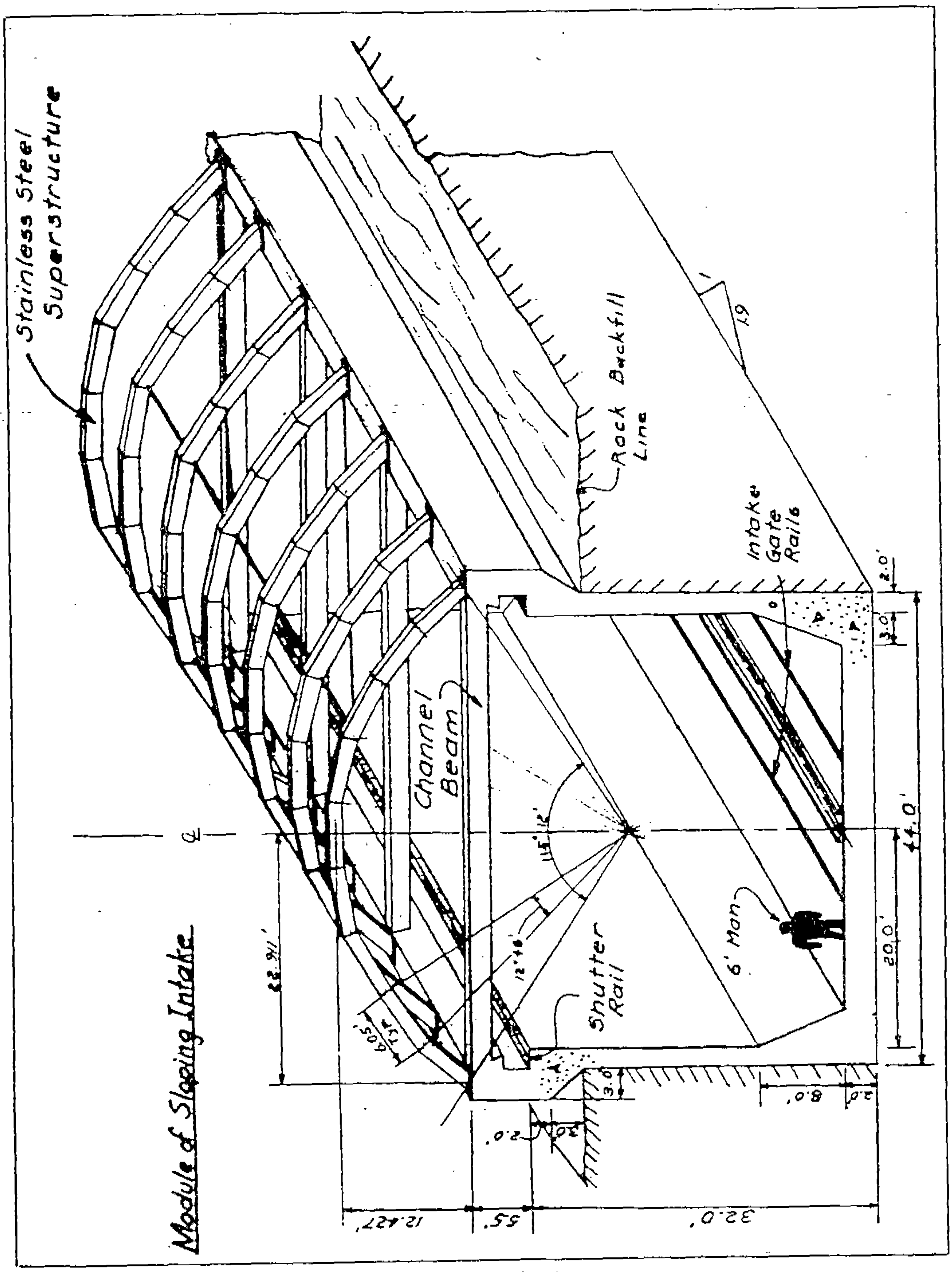
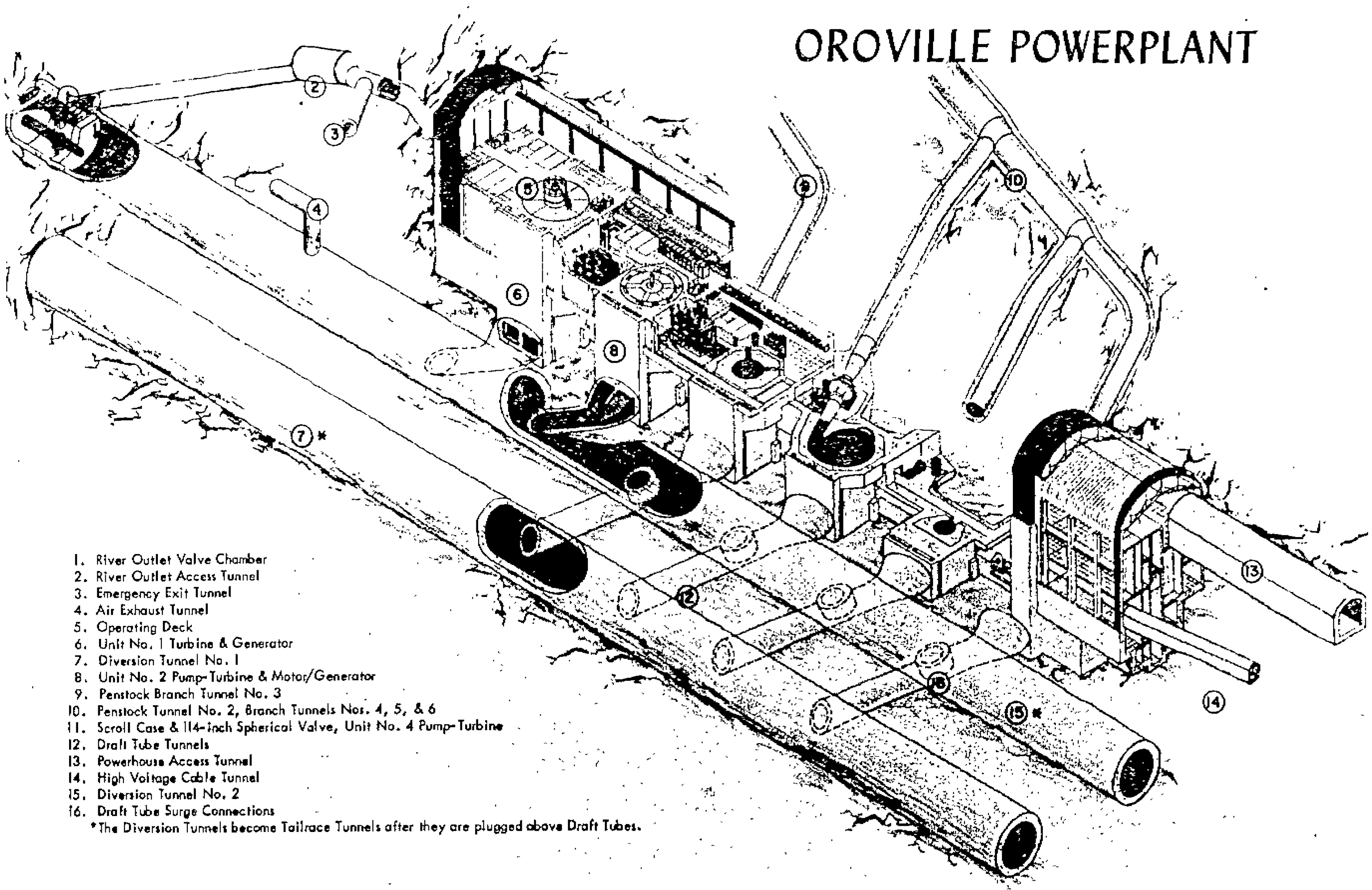
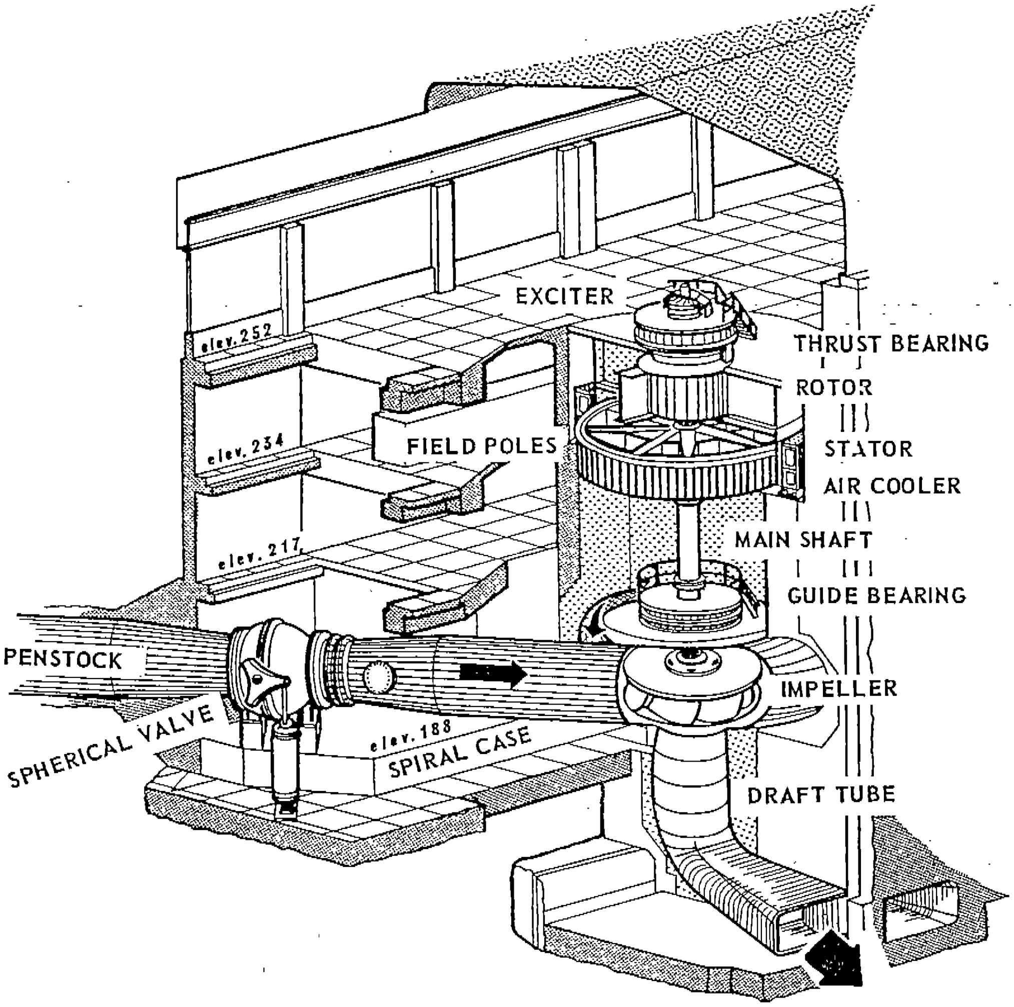


Figure 65. Channel Section

OROVILLE POWERPLANT





PUMP-GENERATOR

EDWARD HYATT POWERPLANT, OROVILLE DAM

GENERATORS AND MOTOR/GENERATORS - HYATT POWERPLANT

The three generators and three motor/generators are of a vertical-shaft type, connected directly to hydraulic turbines or pump/turbines respectively. All units are synchronous machines with directly connected rotary exciters. The combination of the thrust bearing and upper guide bearing is located above the rotor.

The auxiliary equipment of each unit consists of a voltage regulator with static magnetic amplifier, an excitation cubicle, surface air coolers in a closed ventilation system, air housing heaters, a surge protection cubicle, and a neutral grounding equipment cubicle.

Each generator is rated 123,157 KVA, 95 percent power factor (overexcited) , 3-phase, 200 rpm, 60 cycle, designed for counterclockwise rotation when looking down on the unit, and for 12,500 volts Line-to-line voltage.

Each motor/generator is rated 173,000 HD with a unity power factor as a motor, and 115,000 KVA with 85 percent power factor (overexcited) 3-phase, 189.5 rpm, 60 cycle, designed for counterclockwise rotation as a generator and clockwise rotation as a motor. The line-to-line voltage rating is 12,500 volts for a generator mode of operation

Excitation rated voltage for all units is 250 volts.

Stator windings of all units are connected in wye, suitable for either grounded or ungrounded neutral operation.

The thrust bearing, upper guide bearing and main exciter are supported by the upper bearing bracket. The lower guide bearing and the combination of air breaks and hydraulic jacks are supported by the lower bearing bracket. The turbine or pump/turbine parts and the lower bearing bracket can be removed through the Stator bore.

Technical Aspects of Wetlands

Wetland Hydrology, Water Quality, and Associated Functions

By Virginia Carter[1]

The formation, persistence, size, and function of wetlands are controlled by hydrologic processes. Distribution and differences in wetland type, vegetative composition, and soil type are caused primarily by geology, topography, and climate. Differences also are the product of the movement of water through or within the wetland, water quality, and the degree of natural or human-induced disturbance. In turn, the wetland soils and vegetation alter water velocities, flow paths, and chemistry. The hydrologic and water-quality functions of wetlands, that is, the roles wetlands play in changing the quantity or quality of water moving through them, are related to the wetland's physical setting.

Wetlands are distributed unevenly throughout the United States because of differences in geology, climate, and source of water (fig. 17). They occur in widely diverse settings ranging from coastal margins, where tides and river discharge are the primary sources of water, to high mountain valleys where rain and snowmelt are the primary sources of water. Marine wetlands (those beaches and rocky shores that fringe the open ocean) are found in all coastal States. Estuarine wetlands (where tidal saltwater and inland freshwater meet and mix) are most plentiful in Alaska and along the southeastern Atlantic coast and the gulf coast. Alaska has the largest acreage of estuarine wetlands in the United States, followed by Florida and Louisiana.

Inland (nontidal) wetlands are found in all States. Some States, such as West Virginia, have few large wetlands, but contain many small wetlands associated with streams. Other States, such as Nebraska, the Dakotas, and Texas, contain many small isolated wetlands—the lakes of the Nebraska Sandhills, the prairie potholes, and the playa lakes, respectively. Northern States such as Minnesota and Maine contain numerous wetlands with organic soils (peatlands), similar in origin and hydrologic and veg-

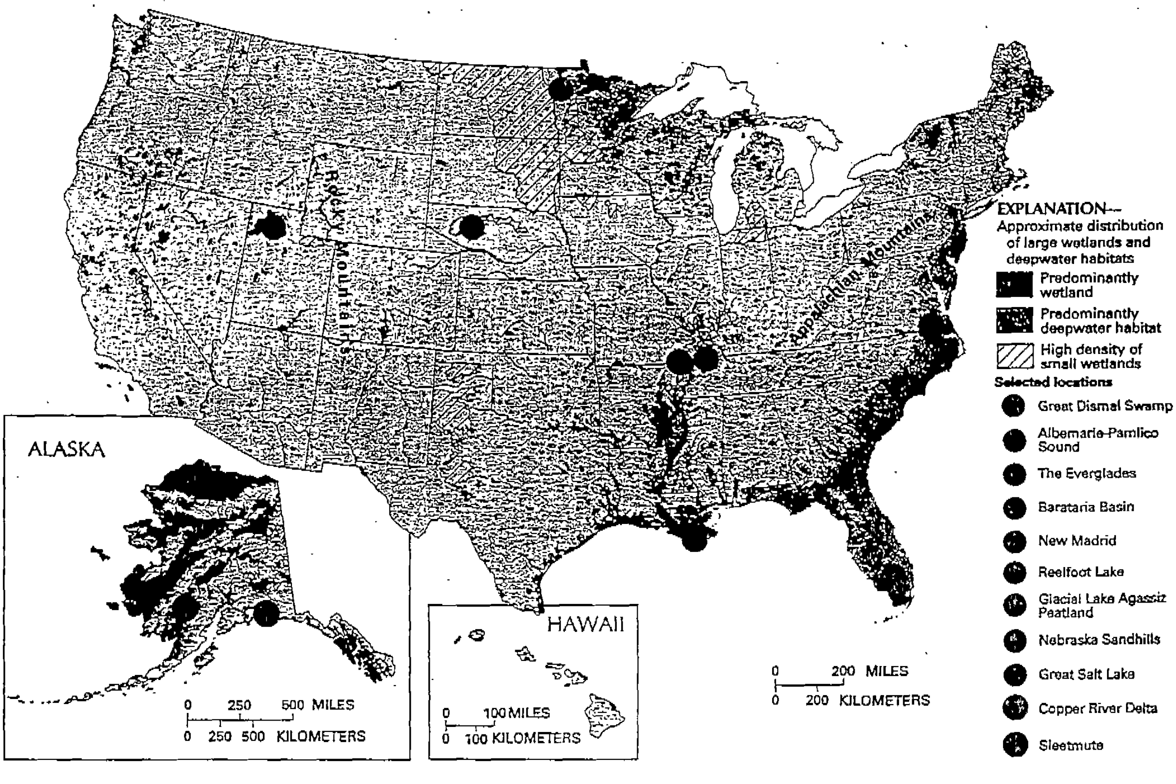
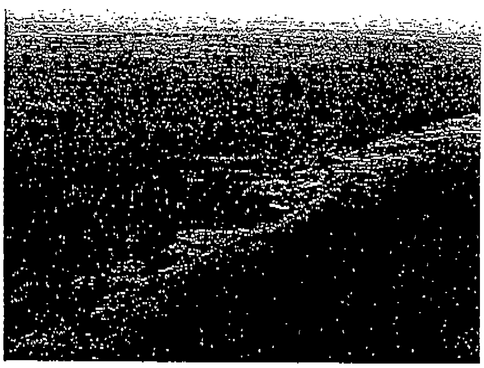


Figure 17. Major wetland areas in the United States and location of sites mentioned in the text. *(Source: Data from T.E. Dahl, U.S. Fish and Wildlife Service, unpub. data, 1991.)*

[1] U.S. Geological Survey.

Typical prairie pothole wetland in North Dakota. *(Photograph by Virginia Carter, U.S. Geological Survey.)*



Glacial Lake Agassiz peatland, Minnesota. *(Photograph by Virginia Carter, U.S. Geological Survey.)*

etative characteristics to the classic bog and fen peatlands of northern Europe. However, peatlands are by no means limited to Northern States—they occur in the Southeastern and Midwestern United States wherever the hydrology and chemical environment are conducive to the accumulation of organic material.

Wetlands occur on flood plains—for example, the broad bottom-land hardwood forests and river swamps (forested wetlands) of southern rivers and many of the narrow riparian zones along streams in the Western United States. Wetlands are commonly associated with lakes or can occur as isolated features of the landscape. They can form large complexes of open water and vegetation such as The Everglades of Florida, the Okefenokee Swamp of Georgia and Florida, the Copper River Delta of Alaska, and the Glacial Lake Agassiz peatland of Minnesota.

HYDROLOGIC PROCESSES IN WETLANDS

Hydrologic processes occurring in wetlands are the same processes that occur outside of wetlands and collectively are referred to as the hydrologic cycle. Major components of the hydrologic cycle are precipitation, surface-water flow, ground-water flow, and evapotranspiration (ET). Wetlands and uplands continually receive or lose water through exchange with the atmosphere, streams, and ground water. Both a favorable geologic setting and an adequate and persistent supply of water are necessary for the existence of wetlands.

The wetland water budget is the total of inflows and outflows of water from a wetland. The components of a budget are shown in the equation in figures 18 and 19. The relative importance of each component in maintaining wetlands varies both spatially and

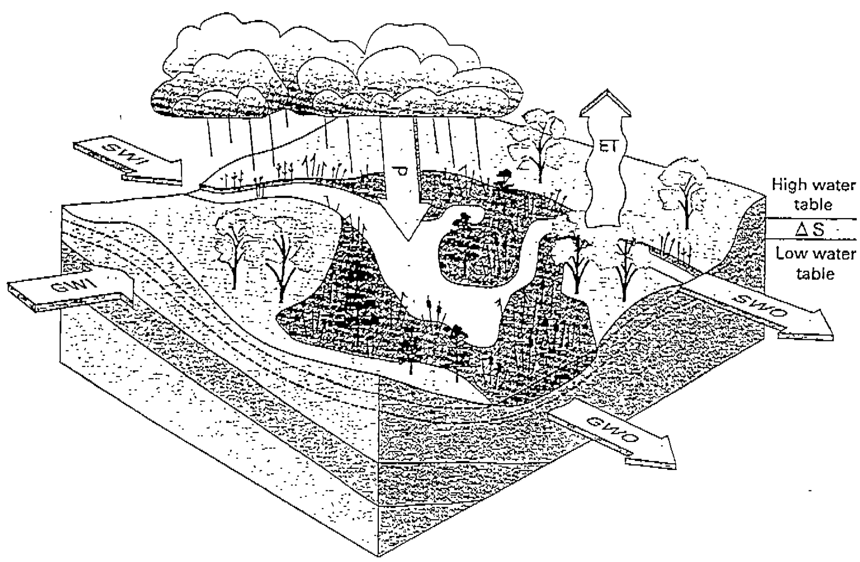


Figure 18. Components of the wetland water budget. ($P + SWI + GWI = ET + SWO + GWO + \Delta S$, where P is precipitation, SWI is surface-water inflow, SWO is surface-water outflow, GWI is ground-water inflow, GWO is ground-water outflow, ET is evapotranspiration, and ΔS is change in storage.)

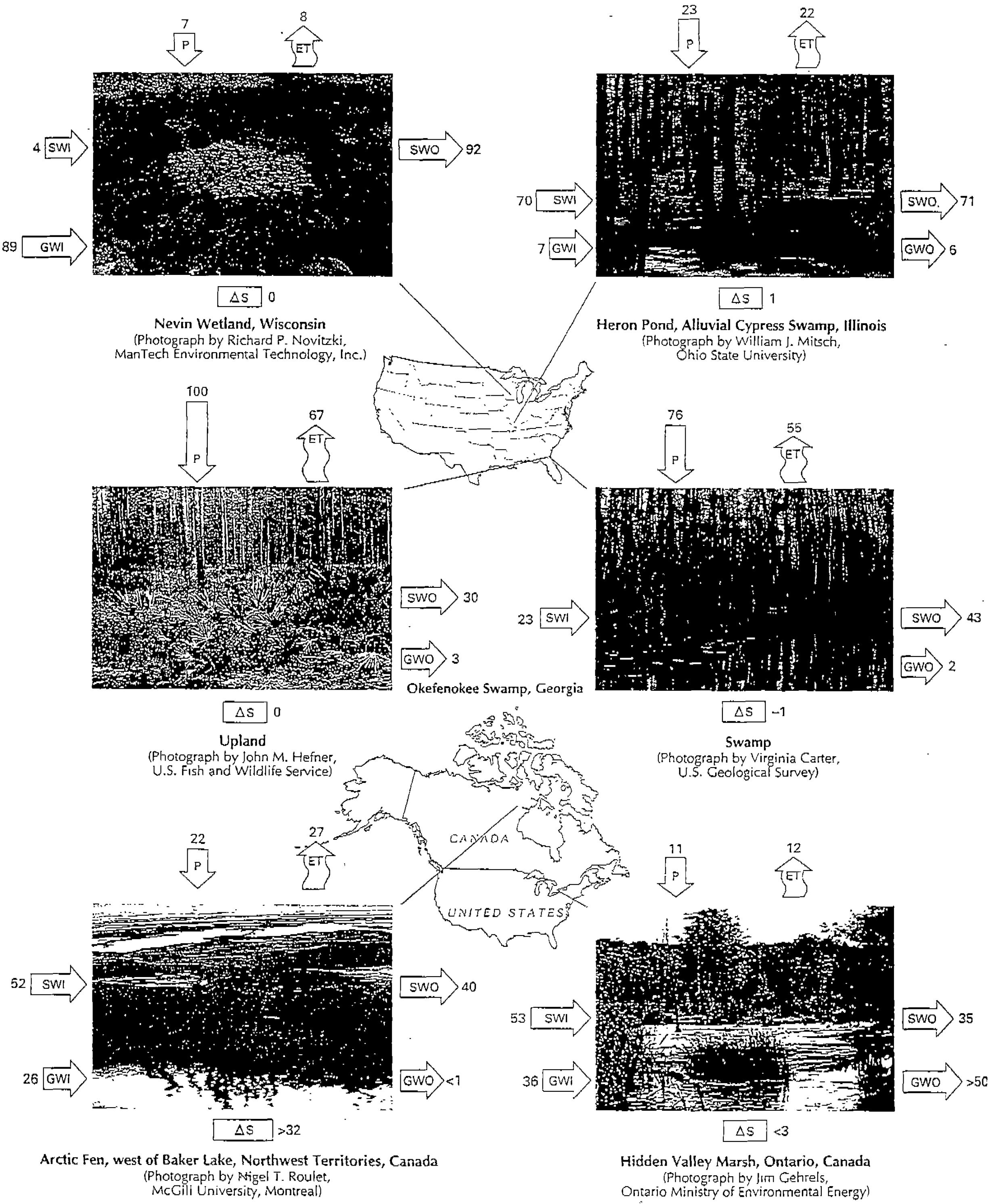


Figure 19. Water budgets for selected wetlands in the United States and Canada. ($P + SWI + GWI = ET + SWO + GWO + \Delta S$, where P is precipitation, SWI is surface-water inflow, SWO is surface-water outflow, GWI is ground-water inflow, GWO is ground-water outflow, ET is evapotranspiration, and ΔS is change in storage. Components are expressed in percentages. Abbreviations used: < = less than; > = greater than.) *(Sources from left to right and top to bottom: Novitzki, 1978; Roulet and Woo, 1986; Rykiel, 1984; Rykiel, 1984; Mitsch and Gosselink, 1993; and Gehrels and Mulamoottil, 1990.)*

flow from wetlands fed primarily by precipitation (fig. 21). This is because ground-water discharge tends to be relatively constant in quantity compared with precipitation and snowmelt.

In coastal areas, tides provide a regular and predictable source of surface water for wetlands, affecting erosion, deposition, and water chemistry. The magnitude of daily high and low tides is affected by the relative position of the sun and the moon—highest and lowest tides usually occur during full or new moons. Where tidal circulation is impeded by barrier islands (for example, in the Albemarle-Pamlico Sound in North Carolina, where tides are primarily wind-driven) or dikes and levees, tidal circulation may be small or highly modified. Strong winds and storms can cause extreme changes in sea level, flooding both wetlands and uplands.

Ground Water

Ground water originates as precipitation or as seepage from surface-water bodies. Precipitation moves slowly downward through unsaturated soils and rocks until it reaches the saturated zone. Water also seeps from lakes, rivers, and wetlands into the saturated zone. This process is known as ground-water recharge and the top of the saturated zone is known as the water table. Ground water in the saturated zone flows through aquifers or aquifer systems composed of permeable rocks or other earth materials in response to hydraulic heads (pressure). Ground water can flow in shallow local aquifer systems where water is near the land surface or in deeper intermediate and regional aquifer systems (fig. 22). Differences in hydraulic head cause ground water to move back to the land surface or into surface-water bodies; this process is called ground-water discharge. In wetlands that are common discharge areas for different flow systems, waters from different sources can mix. Ground-water discharge occurs through wells, seepage or springs, and directly through ET where the water table is near the land surface or plant roots reach the water table. Ground-water discharge will influence the water chemistry of the receiving wetland; whereas ground-water recharge will influence the chemistry of water in the adjacent aquifer.

Wetlands most commonly are ground-water discharge areas; however, ground-water recharge also occurs. Ground-water recharge or discharge in wetlands is affected by topographic position, hydrogeology, sediment and soil characteristics, season, ET, and climate and might not occur uniformly throughout a wetland. Recharge rates in wetlands can be much slower than those in adjacent uplands if the upland soils are more permeable than the slightly permeable clays or peat that usually underlie wetlands.

The accumulation and composition of peat in wetlands are important factors influencing hydrology and vegetation. It was long assumed that the discharge of ground water through thick layers of well-decomposed peat was negligible because of its low permeability, but recent studies have shown that these layers can transmit ground water more rapidly than previously thought (Chason and Siegel, 1986). Peatland type (fen or bog) and plant communities are affected by the chemistry of water in the surface layers of the wetland; the source of water (precipitation, surface water, or ground water) controls the water chemistry and determines what nutrients are available for plant growth. Ground-water flow in extensive peatlands such as the Glacial Lake Agassiz peatland in Minnesota may be controlled by the development of ground-water mounds (elevated water tables fed by precipitation) in raised bogs where ground water moves downward through mineral soils before discharging into adjacent fens (Siegel, 1983; Siegel and Glaser, 1987). Movement of the ground water through mineral soils increases the nutrient content of the water.

Coastal wetlands and shallow embayments represent the lowest point in regional and local ground-water flow systems; ground water discharges into these areas, sometimes in quantities large enough to affect the chemistry of estuaries (Valiela and Costa, 1988;

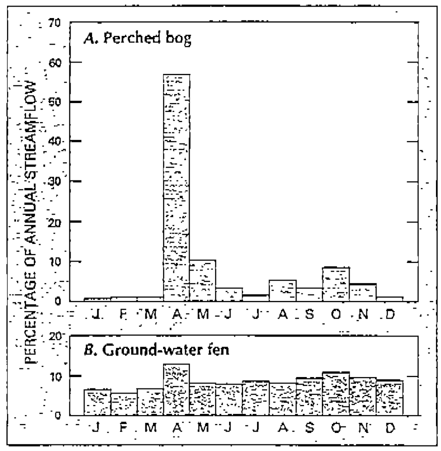


Figure 21. Monthly streamflow from two wetlands in northern Minnesota; *A*, a perched bog whose inflow component is primarily precipitation, and *B*, a fen whose inflow component is primarily ground water. *(Source: Modified from Boelter and Verry, 1977.)*

The hydrology of a wetland is largely responsible for the vegetation of the wetland.

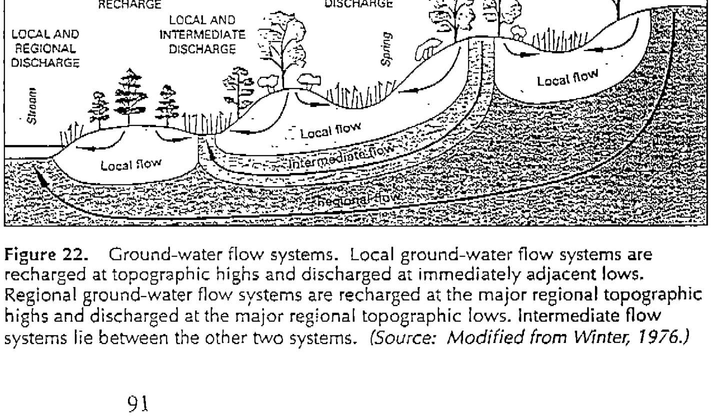


Figure 22. Ground-water flow systems. Local ground-water flow systems are recharged at topographic highs and discharged at immediately adjacent lows. Regional ground-water flow systems are recharged at the major regional topographic highs and discharged at the major regional topographic lows. Intermediate flow systems lie between the other two systems. *(Source: Modified from Winter, 1976.)*

flow from wetlands fed primarily by precipitation (fig. 21). This is because ground-water discharge tends to be relatively constant in quantity compared with precipitation and snowmelt.

In coastal areas, tides provide a regular and predictable source of surface water for wetlands, affecting erosion, deposition, and water chemistry. The magnitude of daily high and low tides is affected by the relative position of the sun and the moon—highest and lowest tides usually occur during full or new moons. Where tidal circulation is impeded by barrier islands (for example, in the Albemarle-Pamlico Sound in North Carolina, where tides are primarily wind-driven) or dikes and levees, tidal circulation may be small or highly modified. Strong winds and storms can cause extreme changes in sea level, flooding both wetlands and uplands.

Ground Water

Ground water originates as precipitation or as seepage from surface-water bodies. Precipitation moves slowly downward through unsaturated soils and rocks until it reaches the saturated zone. Water also seeps from lakes, rivers, and wetlands into the saturated zone. This process is known as ground-water recharge and the top of the saturated zone is known as the water table. Ground water in the saturated zone flows through aquifers or aquifer systems composed of permeable rocks or other earth materials in response to hydraulic heads (pressure). Ground water can flow in shallow local aquifer systems where water is near the land surface or in deeper intermediate and regional aquifer systems (fig. 22). Differences in hydraulic head cause ground water to move back to the land surface or into surface-water bodies; this process is called ground-water discharge. In wetlands that are common discharge areas for different flow systems, waters from different sources can mix. Ground-water discharge occurs through wells, seepage or springs, and directly through ET where the water table is near the land surface or plant roots reach the water table. Ground-water discharge will influence the water chemistry of the receiving wetland whereas ground-water recharge will influence the chemistry of water in the adjacent aquifer.

Wetlands most commonly are ground-water discharge areas; however, ground-water recharge also occurs. Ground-water recharge or discharge in wetlands is affected by topographic position, hydrogeology, sediment and soil characteristics, season, ET, and climate and might not occur uniformly throughout a wetland. Recharge rates in wetlands can be much slower than those in adjacent uplands if the upland soils are more permeable than the slightly permeable clays or peat that usually underlie wetlands.

The accumulation and composition of peat in wetlands are important factors influencing hydrology and vegetation. It was long assumed that the discharge of ground water through thick layers of well-decomposed peat was negligible because of its low permeability, but recent studies have shown that these layers can transmit ground water more rapidly than previously thought (Chason and Siegel, 1986). Peatland type (fen or bog) and plant communities are affected by the chemistry of water in the surface layers of the wetland; the source of water (precipitation, surface water, or ground water) controls the water chemistry and determines what nutrients are available for plant growth. Ground-water flow in extensive peatlands such as the Glacial Lake Agassiz peatland in Minnesota may be controlled by the development of ground-water mounds (elevated water tables fed by precipitation) in raised bogs where ground water moves downward through mineral soils before discharging into adjacent fens (Siegel, 1983; Siegel and Glaser, 1987). Movement of the ground water through mineral soils increases the nutrient content of the water.

Coastal wetlands and shallow embayments represent the lowest point in regional and local ground-water flow systems; ground water discharges into these areas, sometimes in quantities large enough to affect the chemistry of estuaries (Valiela and Costa, 1988;

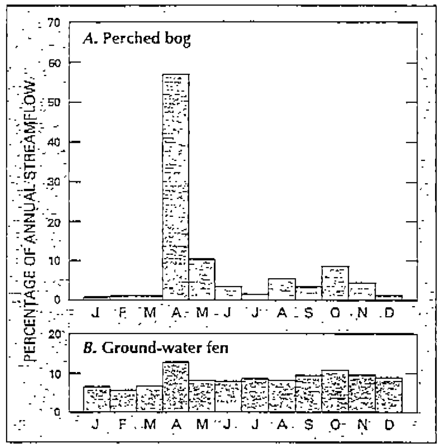


Figure 21. Monthly streamflow from two wetlands in northern Minnesota; *A*, a perched bog whose inflow component is primarily precipitation, and *B*, a fen whose inflow component is primarily ground water. *(Source: Modified from Boelter and Verry, 1977.)*

The hydrology of a wetland is largely responsible for the vegetation of the wetland.

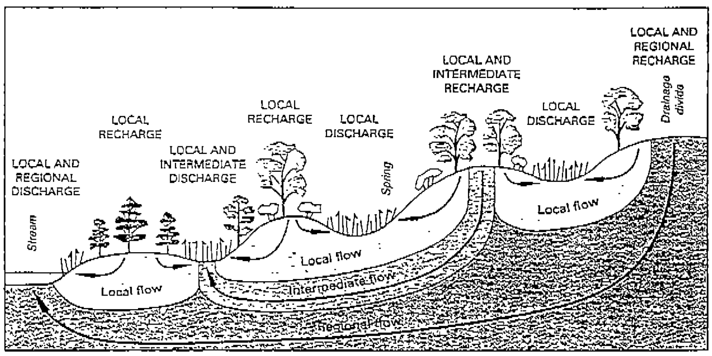


Figure 22. Ground-water flow systems. Local ground-water flow systems are recharged at topographic highs and discharged at immediately adjacent lows. Regional ground-water flow systems are recharged at the major regional topographic highs and discharged at the major regional topographic lows. Intermediate flow systems lie between the other two systems. *(Source: Modified from Winter, 1976.)*

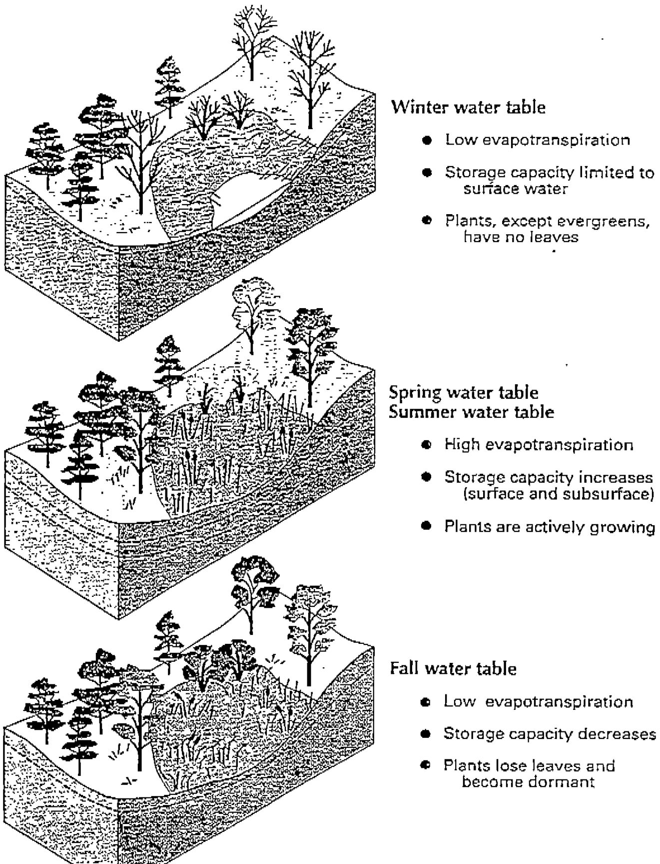


Figure 23. Seasonal changes in storage capacity and evapotranspiration (ET) in wetlands.

Valiela and others, 1990). The quantity of ground water discharged varies throughout the tidal cycle, affecting the water chemistry of the wetland soils (Harvey and Odum, 1990; Valiela and others, 1990).

Storage

The vegetation affects the value of the wetland to animals and people.

Storage in a wetland consists of surface water, soil moisture, and ground water. Storage capacity refers to the space available for water storage—the higher the water table, the less the storage capacity of a wetland. Some wetlands have continuously high water tables, but generally, the water table fluctuates seasonally in response to rainfall and ET. Storage capacity of wetlands is lowest when the water table is near or at the surface—during the dormant season when plants are not transpiring, following snowmelt, and (or) during the wet season (fig. 23). Storage capacity increases during the growing season as water tables decline and ET increases. When storage capacity is high, infiltration may occur and the wetland may be effective in retarding runoff. When water tables are high and storage capacity is low, any additional water that enters the wetland runs off the wetland rapidly.

SOME EFFECTS OF HYDROLOGY ON WETLAND VEGETATION

The hydrology of a wetland is largely responsible for the vegetation of the wetland, which in turn affects the value of the wetland to animals and people. The duration and seasonality of flooding and (or) soil saturation, ground-water level, soil type, and drainage characteristics exert a strong influence on the number, type, and distribution of plants and plant communities in wetlands. Although much is known about flooding tolerance in plants, the effect of soil saturation in the root zone is less well understood. Golet and Lowry (1987) showed that surface flooding and duration of saturation within the root zone, while not the only factors influencing plant growth, accounted for as much as 50 percent of the variation in growth of some plants. Plant distribution is also closely related to wetland water chemistry; the water may be fresh or saline, acidic or basic, depending on the source(s).

HYDROGEOLOGIC SETTINGS

The source and movement of water are very important for assessing wetland function and predicting how changes in wetlands will affect the associated basin. Linkages between wetlands, uplands, and deepwater habitats provide a framework for protection and management of wetland resources. Water moving into wetlands has chemical and physical characteristics that reflect its source. Older ground water generally contains chemicals associated with the rocks through which it has moved; younger ground water has fewer minerals because it has had less time in contact with the rocks. Which processes can and will occur within the wetland are determined by the characteristics of the water entering and the characteristics of the wetland itself—its size, shape, soils, plants, and position in the basin.

Because wetlands occur in a variety of geologic and physiographic settings, attempts have been made to group or classify them in such a way as to identify similarities in hydrology. For example, Novitzki (1979, 1982) developed a hydrologic classification for Wisconsin wetlands based on topographic position and surface water-ground water interaction; Gosselink and Turner (1978) grouped freshwater wetlands according to hydrodynamic energy gradients; and Brinson (1993) developed a hydrogeomorphic classification for use in evaluating wetland function. (See the articles "Wetland Definitions and Classifications in the United States" and "Wetland Functions, Values, and Assessment" in this volume.) Wetlands, like lakes, are associated with features where water tends to collect. They are commonly found in topographic depressions, at slope breaks, in areas of stratigraphic change, and in permafrost areas (fig. 24) (Winter and Woo, 1990).

Topographic Depressions

Most wetlands occur in or originate in topographic depressions—these include lakes, wetland basins, and river valleys (fig. 24*A*). Depressions may be formed by movement of glaciers and water; action of wind, waves, and tides; and (or) by processes associated with tectonics, subsidence, or collapse.

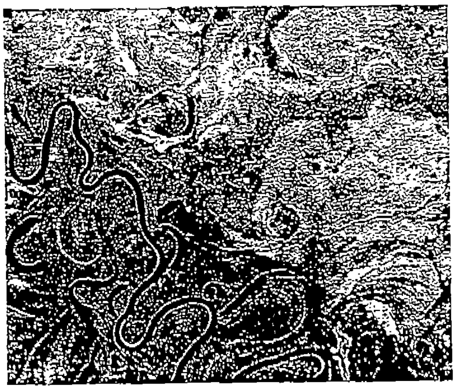
Glacial movement.—Glaciers shaped the landscape of many of the Northern States and caused wetlands to form in mountainous areas such as the Rocky Mountains and the northern Appalachians. As the glaciers advanced over the Northern United States they gouged and scoured the land surface, making numerous depressions, depositing unsorted glacial materials, and burying large ice masses. As the climate warmed, the glaciers retreated, leaving behind the depressions and the large masses of buried ice. As the temperatures continued to warm, the ice masses melted to form kettle holes. In many cases, water filled the depressions and kettle holes, forming lakes. As the lakes filled with sediments, they were replaced by wetlands.

Water movement.—Wetlands also are formed by the movement of water as it flows from upland areas toward the coast. The flow characteristics of water are partly determined by the slope of the streambed. On steeply sloping land, water generally flows rapidly through relatively deep, well-defined channels. As the slope decreases, the water spreads out over a wider area and channels usually become shallower and less defined. Shallow channels tend to meander or move back and forth across the flood plain. The changes in flow path sometimes result in oxbow lakes and flood-plain wetlands. When the river floods, the isolated oxbow lakes begin to fill with sediment, providing an excellent place for more wetlands to form. Obstruction to the normal flow of water also can cause the water to change course and leave gouges in front of or channels around the obstruction, or can cause water to be impounded behind the obstruction. Many lakes and wetlands are formed behind dams made by humans or beavers.

Wind, wave, and tidal action.—Wetlands are common in areas of sand dunes caused by wind, waves, or tides. Wetlands formed in the depressions between sand dunes are found in the Nebraska Sandhills, along the shoreline of the Great Lakes, and on barrier islands and the seaward margins of coastal States. In coastal States, tides, waves, and wind cause the movement of sand barriers and the closing of inlets, which often result in the formation of shallow lagoons with abundant associated emergent wetlands.

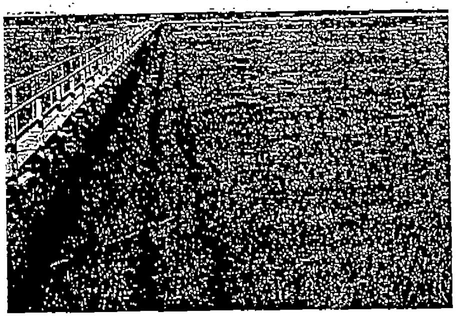
Tectonic activities.—Tectonic activity is responsible for depression wetlands such as Reelfoot Lake on the Mississippi River flood plain in Tennessee caused by the 1812 New Madrid earthquake. Earthquakes result when two parts of the Earth's crust move relative to each other, causing displacement of land. When this occurs, depressions may result along the lines of displacement or the flow paths of rivers may be changed, leaving isolated bodies of water. When a source of water coincides with these depressions, wetlands can form.

Subsidence and collapse features.—Land subsidence and collapse also can form depressions in which wetlands and lakes occur. In some areas, especially in the Southwest, pumping of ground water has caused the land above an aquifer to sink, forming depressions where water collects and wetlands develop. In karst topography (landscapes resulting from the solution of carbonate rocks such as limestone), such as is found in Florida, wetlands form in sinkholes. Collapse of volcanic craters produces



Infrared color photograph of oxbow lakes in the drainage area of Hoholitna River near Sleetmute, Alaska. *(Photograph courtesy of National Aeronautics and Space Administration.)*

Lotus in Reelfoot Lake, Tennessee. *(Photograph by Virginia Carter, U.S. Geological Survey.)*



Coastal marsh along San Francisco Bay, California. *(Photograph by Virginia Carter, U.S. Geological Survey.)*

This recently collapsed sinkhole, in central Florida, provides an ideal spot for a wetland to form. *(Photograph by Terry H. Thompson, U.S. Geological Survey.)*

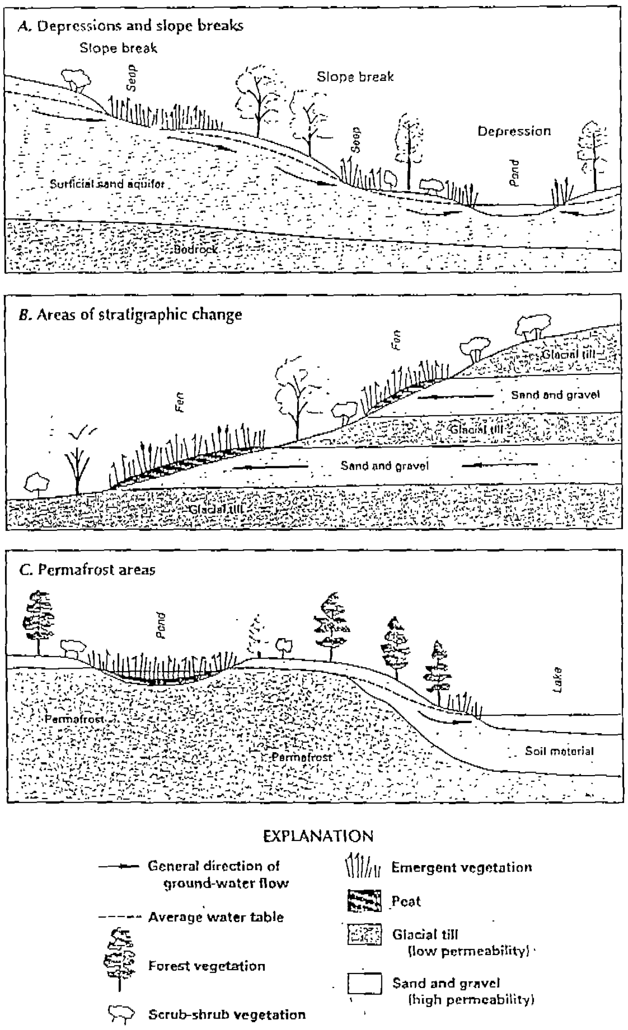


Figure 24. Cross sections showing principal hydrogeologic settings for wetlands; *A,* slope break and depression, *B,* area of stratigraphic change, and *C,* permafrost area.

calderas that fill with water and sediment and contain lakes or wetlands.

Slope Breaks

The water table sometimes intersects the land surface in areas where the land is sloping. Where there is an upward break or change in slope, ground water moves toward the water table in the flatter landscape (fig. 24*A*) (Roulet, 1990; Winter and Woo, 1990). Where ground water discharges to the land surface, wetlands form on the lower parts of the slope. Constant ground-water seepage maintains soil saturation and wetland plant communities. The Great Dismal Swamp of Virginia and North Carolina is maintained by seepage of ground water at the slope break at the bottom of an ancient beach ridge that runs along th western edge (Carter and others, 1994).

Areas of Stratigraphic Change

Where stratigraphic changes occur near land surface, the layering of permeable and less-permeable rocks or soils affects the movement of ground water. When water flowing through the more permeable rock encounters the less permeable rock, it is diverted along the surface of the less permeable rock to the land surface. The continual seepage that occurs at the surface provides the necessary moisture for a wetland (fig. 24*B*). Fens in Iowa form on valley-wall slopes where a thin permeable horizontal layer of rock is sandwiched between two less permeable layers and continual seepage from the permeable layer causes the formation of peat (Thompson and others, 1992).

Permafrost Areas

Permafrost is defined as soil material with a temperature continuously below 32°F (Fahrenheit) for more than 1 year (Brown, 1974); both arctic and subarctic wetlands in Alaska are affected by permafrost (figs. 24*C* and 25). Permafrost has low permeability and infiltration rates. As a result, recharge through permafrost is extremely slow (Ford and Bedford, 1987). In areas covered by peat, organic silt, or dense vegetation, permafrost is commonly close to the surface. In areas covered by lakes, streams, and ponds, permafrost can be absent or at great depth below the surface-water body. The surface or active layer of permafrost thaws during the growing season. In areas where permafrost is continuous, there is virtually no hydraulic connection between ground water in the surface layer and ground water below the permafrost zone. The imperviousness of the frozen soil slows drainage and causes water to stand in surface depressions, forming wetlands and shallow lakes.

In discontinuous permafrost areas (fig. 25), unfrozen zones on south-facing slopes (in the northern hemisphere) and under lakes, wetlands, and large rivers provide hydraulic connections between the surface and the ground water below the permafrost zone. Ground-water discharge to wetlands from deeper aquifers can occur through the unfrozen zone (Williams and Waller, 1966; Kane and Slaughter, 1973). In discontinuous permafrost regions, whether a slope faces away from or toward the sun can determine the presence or absence of permafrost and thus influence the location and distribution of wetlands (Dingman and Koutz, 1974). Permafrost is sensitive to factors that upset the thermal equilibrium. Thermokarst features (depressions in the land surface caused by thawing and subsequent settling of the land) may be caused by regional climatic change or human activities. These depressions formed by local thawing of permafrost are usually filled with wetlands.

WATER QUALITY IN WETLANDS

The water chemistry of wetlands is primarily a result of geologic setting, water balance (relative proportions of inflow, outflow, and storage), quality of inflowing water, type of soils and vegetation, and human activity within or near the wetland. Wetlands

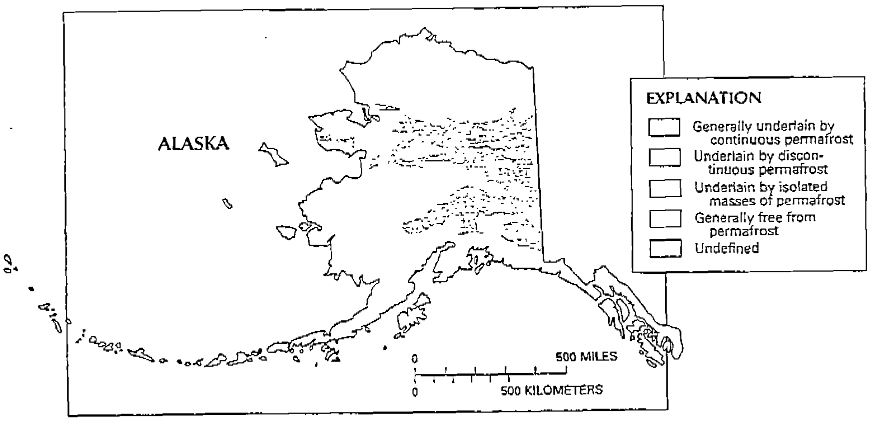


Figure 25. Continuous, discontinuous, and sporadic permafrost areas of Alaska. *(Source: Modified from Ford and Bedford, 1987.)*

dominated by surface-water inflow and outflow reflect the chemistry of the associated rivers or lakes. Those wetlands that receive surface-water or ground-water inflow, have limited outflow, and lose water primarily to ET have a high concentration of chemicals and contain brackish or saline (salty) water. Examples of such wetlands are the saline playas, wetlands associated with the Great Salt Lake in Utah, and the permanent and semipermanent prairie potholes. In contrast, wetlands that receive water primarily from precipitation and lose water by way of surface-water outflows and (or) seepage to ground water tend to have lower concentrations of chemicals. Wetlands influenced strongly by ground-water discharge have water chemistries similar to ground water. In most cases, wetlands receive water from more than one source, so the resultant water chemistry is a composite chemistry of the various sources.

Plants can serve as indicators of wetland chemistry. In tidal wetlands, the distribution of salty water influences plant communities and species diversity. In freshwater wetlands, pH (a measure of acidity or alkalinity) and mineral and nutrient content influence plant abundance and species diversity.

HYDROLOGIC AND WATER-QUALITY FUNCTIONS OF WETLANDS

Wetland hydrologic and water-quality functions are the roles that wetlands play in modifying or controlling the quantity or quality of water moving through a wetland. An understanding of wetland functions and the underlying chemical, physical, and biological processes supporting these functions facilitates the management and protection of wetlands and their associated basins.

The hydrologic and water-quality functions of wetlands are controlled by the following:

- Landscape position (elevation in the drainage basin relative to other wetlands, lakes, and streams)
- Topographic location (depressions, flood plains, slopes)
- Presence or absence of vegetation
- Type of vegetation
- Type of soil
- The relative amounts of water flowing in and water flowing out of the wetland
- Local climate
- The hydrogeologic framework
- The geochemistry of surface and ground water

Although broad generalizations regarding wetland functions can be made, effectiveness and magnitude of functions differ from wetland to wetland.

Natural functions of wetlands can be altered or impaired by human activity. Although slow incremental changes in the natural landscape can lead to small changes in wetlands, the accumulation of these small changes can permanently alter the wetland function (Brinson, 1988). Some of the major hydrologic and water-quality functions of wetlands—(1) flood storage and stormflow modification, (2) ground-water recharge and discharge, (3) alterations of precipitation and evaporation, (4) maintenance of water quality, (5) maintenance of estuarine water balance, and (6) erosion reduction—are discussed below.

The effectiveness and magnitude of a function varies from wetland to wetland.

Flood Storage and Stormflow Modification

Wetlands associated with lakes and streams store floodwaters by spreading water out over a large flat area. This temporary storage of water decreases runoff velocity, reduces flood peaks, and distributes stormflows over longer time periods, causing tributary and main channels to peak at different times. Wetlands with available storage capacity or those located in depressions with narrow outlets may store and release water over an extended period of time. In drainage basins with flat terrain that contains many depressions (for example, the prairie potholes and playa lake regions), lakes and wetlands store large volumes of snowmelt and (or) runoff. These wetlands have no natural outlets, and therefore this water is retained and does not contribute to local or regional flooding.

A strong correlation exists between the size of flood peaks and basin storage (percentage of basin area occupied by lakes and wetlands) in many drainage basins throughout the United States (Tice, 1968;

Hains, 1973; Novitzki, 1979, 1989; Leibowitz and others, 1992). Novitzki (1979, 1989) found that basins with 30 percent or more areal coverage by lakes and wetlands have flood peaks that are 60 to 80 percent lower than the peaks in basins with no lake or wetland area. Wetlands can provide cost-effective flood control, and in some instances their protection has been recognized as less costly than flood-control measures such as reservoirs or dikes (Carter and others, 1979). Loss of wetlands can result in severe and costly flood damage in low-lying areas of a basin.

Not all wetlands are able to store floodwaters or modify stormflow; some, in fact, add to runoff. Downstream wetlands, such as those along the middle and lower reaches of the Mississippi River and its tributaries, are more effective at reducing downstream flooding than are headwater wetlands, largely as a result of larger storage capacities (Ogawa and Male, 1986). Runoff from wetlands is strongly influenced by season, available storage capacity, and soil permeability. Wetlands in basin headwaters are commonly sources of runoff because they are ground-water discharge areas. Wetlands in Alaska that are underlain by permafrost have little or no available storage capacity; runoff is rapid and flood peaks are often very high.

Wetlands can influence weather and climate.

Ground-Water Recharge and Discharge

Ground-water recharge and discharge are hydrologic processes that occur throughout the landscape and are not unique functions of wetlands. Recharge and discharge in wetlands are strongly influenced by local hydrogeology, topographic position, ET, wetland soils, season, and climate. Ground-water discharge provides water necessary to the survival of the wetland and also can provide water that leaves the wetland as streamflow. Most wetlands are primarily discharge areas; in these wetlands, however, small amounts of recharge can occur seasonally.

Recharge to aquifers can be especially important in areas where ground water is withdrawn for agricultural, industrial, and municipal purposes. Wetlands can provide either substantial or limited recharge to aquifers. Much of the recharge to the Ogallala aquifer in West Texas and New Mexico is from the 20,000 to 30,000 playa lakes rather than from areas between lakes, ephemeral streams, and areas of sand dunes (Wood and Osterkamp, 1984; Wood and Sanford, 1994). Recharge takes place through the bottoms of some streams, especially in karst topography and in the arid West. Some recharge also takes place when floodwater moves across the flood plain and seeps down into the water-table aquifer. Cypress domes in Florida and prairie potholes in the Dakotas also are thought to contribute to ground-water recharge (Carter and others, 1979). Ground-water recharge from a wetland can be induced when aquifer water levels have been drawn down by nearby pumping.

Most estuarine wetlands are discharge areas rather than recharge areas, primarily because they are on the low topographic end of local and regional ground-water flow systems. As the tide rises, water is temporarily stored on the surface of the wetland and in the wetland soils, where it mixes with the discharging freshwater. The water moves back into the estuary or tidal river as the tide ebbs. Precipitation falling on nontidal freshwater wetlands on barrier islands may recharge the shallow freshwater aquifer overlying the deeper salty water.

Alterations of Precipitation and Evaporation

Wetlands can influence local or regional weather and climate in several ways. Wetlands tend to moderate seasonal temperature fluctuations. During the summer, wetlands maintain lower temperatures because ET from the wetland converts latent heat and releases water vapor to the atmosphere. In the winter, the warmer water of the wetland prevents rapid cooling at night; warm breezes from the wetland surface may prevent freezing in nearby uplands. Wetlands also modify local atmospheric circulation and thus affect moisture convection, cloud formation, thunderstorms, and precipitation patterns. Therefore, when wetlands are drained or replaced by impermeable materials, significant changes in weather systems can occur.

Maintenance of Water Quality

Ground water and surface water transport sediments, nutrients, trace metals, and organic materials. Wetlands can trap, precipitate, transform, recycle, and export many of these waterborne constituents, and water leaving the wetland can differ markedly from that entering (Mitsch and Gosselink, 1993; Elder, 1987). Wetlands can maintain good quality water and improve degraded water.

Water-quality modification can affect an entire drainage basin or it may affect only an individual wetland. Water chemistry in basins that contain a large proportion of wetlands is usually different from that in basins with fewer wetlands. Basins with more wetlands tend to have water with lower specific conductance and lower concentrations of chloride, lead, inorganic nitrogen, suspended solids, and total and dissolved phosphorus than basins with fewer wetlands. Generally, wetlands are more effective at removing suspended solids, total phosphorus, and ammonia during high-flow periods and more effective at removing nitrates at low-flow periods (Johnston and others, 1990). Novitzki (1979) reported that streams in a Wisconsin basin, which contained 40 percent wetland and lake area, had sediment loads that were 90 percent lower than in a comparable basin with no wetlands. Wetlands may change water chemistry sequentially; that is, upstream wetlands may serve as the source of materials that are transformed in downstream wetlands. Estuaries and tidal rivers depend on the flow of freshwater, sediments, nutrients, and other constituents from upstream.

Wetlands filter out or transform natural and anthropogenic constituents through a variety of biological and chemical processes. Wetlands act as sinks (where material is trapped and held) for some materials and sources (from which material is removed) of others. For example, wetlands are a major sink for heavy metals and for sulfur, which combines with metals to form relatively insoluble compounds. Some wetland mineral deposits (bog iron, manganese) are or have been important metal reserves in the past. Organic carbon in the form of plant tissues and peat

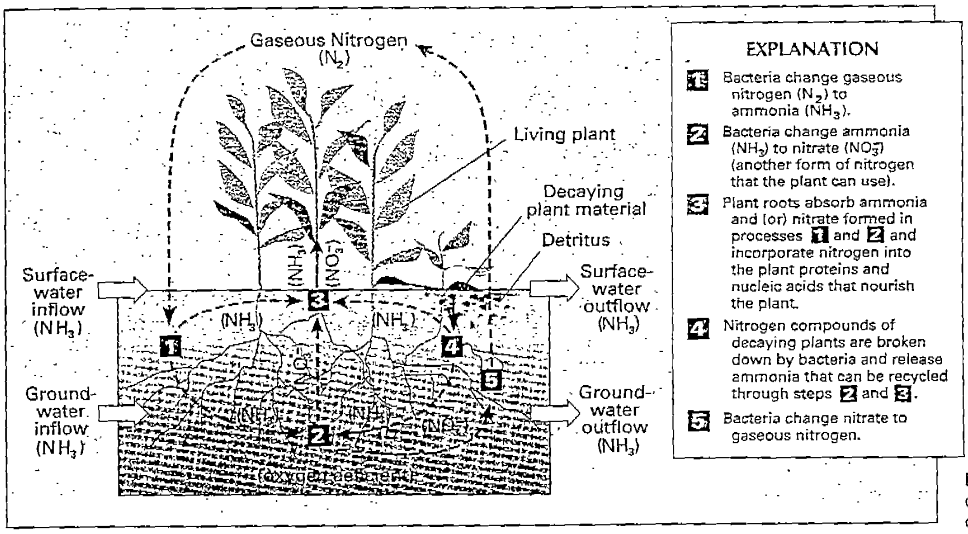


Figure 26. Simplified diagram of the nitrogen cycle in a wetland.

accumulates in wetlands creating a source of waterborne dissolved and particulate organic materials. Some materials, for example nutrients, are changed from one form to another as they pass through the wetland (fig. 26). Most stored materials in wetlands are immobilized as a result of prevailing water chemistry and hydrology, but any disturbance can result in release of those materials.

The water purification functions of wetlands are dependent upon four principal components of the wetland—substrate, water, vegetation, and microbial populations (Hammer, 1992; Hemond and others, 1987).

Substrates.—Wetland substrates provide a reactive surface for biogeochemical reactions and habitat for microbes. Wetland soils are the medium in which many of the wetland chemical transformations occur and the primary storage area of available chemicals for most plants (Mitsch and Gosselink, 1993). Organic or peat soils differ from mineral soils in their biogeochemical properties, including their ability to hold water and bind or immobilize mineral constituents.

Water.—Ground and surface waters transport solid materials and gases to the microbial and plant communities, remove the by-products of chemical and biological reactions from the wetlands, and maintain the environment in which the essential biochemical processes of wetlands occur. Flooding or soil saturation causes oxygen-deficient conditions that markedly influence many biological transformations.

Vegetation.—Wetland vegetation reduces the flow and decreases velocities of water, causing the deposition of mineral and organic particles and constituents attached to them, such as phosphorus or trace metals. Plants introduce oxygen to the generally oxygen-deficient soil environment through their roots, creating an oxidized root zone where bacterial transformations of nitrogenous and other compounds can occur (Good and Patrick, 1987). Plants also provide a surface for microbial colonization. Wetland plants remove small quantities of nutrients, trace metals, and other compounds from the soil water and incorporate them into plant tissue, which may later be recycled in the wetland through decomposition, stored as peat, or transported from the wetland as particulate matter (Boyt and others, 1977; Tilton and Kadlec, 1979; Hammer, 1992).

Microbes.—The microbial community, which includes bacteria, algae, fungi, and protozoa, is responsible for most of the chemical transformations that occur in wetlands. In order to meet their metabolic needs, microbes use up oxygen; transform nutrients, manganese, and iron; and generate methane, hydrogen sulfide gas, and carbon dioxide.

Wetlands serve as short-term or long-term sediment sinks. Floodwater spreading out across a wetland decreases in velocity, and sediments settle out and are trapped within the wetland. Some of this sediment may be transported out of the wetland during future flooding. Sediment deposition in estuarine wetlands provides a constant input that is of special importance for maintenance of wetlands acreage during periods of sea-level rise (Bricker-Urso and others, 1989).

The ability of wetlands to filter and transform nutrients and other constituents has resulted in the construction and use of artificial wetlands in the United States and other countries to treat wastewater and acid mine drainage (Hammer, 1989, 1992; Wieder, 1989). However, individual wetlands have a limited capacity to absorb nutrients and differ in their ability to do so (Tiner, 1985). A wetland's effectiveness in improving water quality depends on hydrologic patterns, amount and type of vegetation, time of year, and the constituent of concern (Zedler and others, 1985).

Estuarine Water Balance

Estuaries receive freshwater from precipitation, ground-water discharge, streamflow, and overland flow. Ground water discharges through shallow-water sediments of the estuary or through marsh soils and can affect the nutrient balance and salinity of the

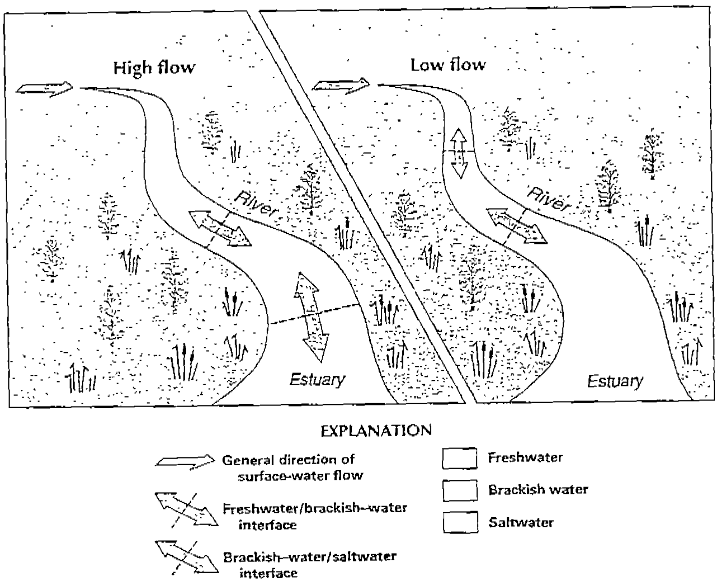


Figure 27. Movement of the freshwater-saltwater interface in an estuary during periods of high flow and during periods of low flow.

receiving waters (Valiela and others, 1978; Harvey and Odum, 1990). Estuarine salinity decreases during periods of high streamflow as the freshwater-saltwater interface moves down the estuary from the stream toward the sea (fig. 27). Estuarine salinity increases as streamflow decreases and the interface moves up the estuary. Estuarine plants and animals are well adjusted to these normal seasonal fluctuations in salinity. Water temporarily stored in flood-plain wetlands upstream from the estuary deposits sediment and nutrients, and water leaving these wetlands exports decomposition products and organic detritus to the estuary. This temporary storage of water and the concurrent decrease in flow velocity aid in controlling the timing and size of the freshwater influx to the estuary. For example, the freshwater wetlands of the Barataria Basin in Louisiana serve as a major freshwater reservoir for maintenance of favorable salinities in the brackish zone, and the major pulse of materials to the estuary coincides with the arrival of migrant fish for growth and spawning. Leaves that fall in flood-plain wetlands are broken down and enriched by microbial action and produce high-quality food for detrital based food chains in the estuary. Alterations in the timing and quality of streamflow and associated suspended particulate and dissolved material, caused by dams or artificial drainage, can alter the chemistry of coastal waters and affect the organisms that inhabit them.

Wetlands reduce the erosive forces of wind and waves.

Erosion Reduction

Wetlands reduce shoreline erosion by stabilizing sediments and absorbing and dissipating wave energy (Hammer, 1992). The ability of wetlands to stabilize and protect shorelines depends on their capacity to reduce the erosive forces of wind and waves. Beaches and shallow vegetated wetlands protect shorelines in moderate and small storms if the water does not carry excessive amounts of abrasive floating debris. Wetland vegetation decreases water velocities through friction and causes sedimentation in shallow water areas and flood-plain wetlands, thus decreasing the erosive power of the water and building up natural levees. Trees are excellent riverbank stabilizers and have been planted to reduce erosion along United States shorelines. Other wetland plants such as bulrushes, reeds, cattails, cordgrass, and mangroves can also successfully withstand wave and current action.

When vegetation is removed, streambanks collapse and channels widen and (or) deepen; removal of wetland vegetation can turn a sediment sink into a sediment source. The dissipation of erosive forces by vegetation differs from wetland to wetland and depends upon vegetative composition and root structure, sediment type, and the frequency and intensity of water contact with the bank.

SUMMARY

Wetlands are complex ecosystems in which ground water and surface water interact, but because ground water cannot be directly observed, its role in the hydrology of wetlands is sometimes more difficult to understand than that of surface water. Many wetlands owe their existence not only to poor drainage at the site but also to the discharge of ground water at the site. The hydrology of a wetland determines what functions it will perform. Each wetland is unique, but those with similar hydrologic settings generally perform similar functions.

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附件四

A Uniform Technique for Flood Frequency Analysis

By Wilbert O. Thomas, Jr.[1]

Abstract: In 1967 the U.S. Water Resources Council (WRC) published Bulletin 15 recommending that a uniform technique be used by all Federal agencies in estimating floodflow frequencies for gaged watersheds. This uniform technique consisted of fitting the logarithms of annual peak discharges to a Pearson Type III distribution using the method of moments. The objective was to adopt a consistent approach for the estimation of floodflow frequencies that could be used in computing average annual flood losses for project evaluation. In addition, a consistent approach was needed for defining equitable flood-hazard zones as part of the National Flood Insurance Program. In 1976 WRC published Bulletin 17 which extended and updated Bulletin 15 but still recommended the use of the "log-Pearson Type III" method. Since 1976, two updates of Bulletin 17 (17A and 17B) have been published which clarify or improve on this base method, or do both. This paper gives a brief historical review of the development of these bulletins and the motivation and justification for the adoption of this uniform technique. Special emphasis is given to Bulletin 17B, the current guidelines used by Federal agencies. Specific techniques examined are the development of regional skew, weighting of regional and station skew, the basis for the low- and high-outlier tests, and the basis for the adjustment of frequency curves using historical information.

Introduction

Each year floods cause the deaths of about 200 people, displace another 200,000 from their homes, and cause approximately $2 billion in property damage. In addition, several billion dollars are spent each year in the design and construction of transportation facilities such as bridges and culverts. The design and construction of dams, levees, and diversion channels for flood control and the definition of flood plains for land use planning represents another significant outlay of money. These huge expenditures necessitate having uniform and accurate methods for estimating the magnitude and frequency of flood discharges for gaged watersheds in order to make good design decisions. The development of a uniform and accurate technique for estimating the magnitude and frequency of floods has been an ongoing activity of several Federal agencies for approximately the last 20 years. This longstanding coordination effort culminated in September, 1981, with the publication of Bulletin 17B "Guidelines for Determining Floodflow Frequency" (15). The objectives of this article are to:

1. Justify the need for a uniform technique for estimating floodflow frequencies.
2. Describe the historical development of techniques leading up to Bulletin 17B.

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3. Describe in some detail the Bulletin 17B technique.
4. Provide some thoughts about the future direction of flood-frequency analysis within the Federal community.

Justification for Uniform Technique

There are several engineering/economic reasons that dictate the need for a uniform and accurate technique for estimating flood frequency. Among these reasons are the need for a consistent and accurate approach for:

1. Computing average annual flood losses for evaluation of flood-control projects.
2. Defining equitable flood-hazard zones as part of the National Flood Insurance Program.
3. Defining flood risk required for the economic design of highway drainage structures.

Several Federal agencies have the responsibility for the design and construction of flood-control projects such as reservoirs, levees, and improved channels. It would be difficult to equitably compute and compare the relative benefits of several flood-control projects proposed by the same or different Federal agencies without a uniform technique for estimating the floodflow frequencies. The same statement applies for defining flood elevations and profiles used to compute flood insurance rates. A uniform technique for estimating floodflow frequencies is desirable in order to compute equitable flood insurance rates. In addition, the Federal Highway Administration is encouraging the determination of flood risk as related to the design of highway drainage structures. Flood risk used in this context refers to determining the design frequency which yields the minimum sum of damages and construction costs for a given drainage structure. A uniform technique is again desirable for defining this flood risk.

Several Federal agencies make estimates of flood frequency in fulfilling their agency's mission. A uniform technique minimizes coordination efforts among agencies and permits a more cost-effective utilization of each agency's budget. In addition, a uniform technique minimizes public confusion and discourages legal litigation that might result from Federal agencies advocating different estimates of the same frequency flood.

Equally important as the economic motivation is the political motivation for a uniform technique. In August 1966 the 89th Congress passed House Document No. 465 entitled "A Unified National Program for Managing Flood Losses." This document recommended the establishment of a panel of the Water Resources Council (WRC) to "present a set of techniques for frequency analyses that are based on the best of known hydrological and statistical procedures." In response to House Document No. 465, the Executive Director of WRC in September 1966 assigned the responsibility for developing a uniform technique to the WRC Hydrology Committee. The Hydrology Committee established a Work Group on Flow-Frequency Methods comprised of members of the various Federal agencies to undertake this task. The next section of this

paper describes the activities of this work group and subsequent work groups of the Hydrology Committee.

Historical Development of Uniform Technique

The first Federal interagency attempt at developing a uniform flood-frequency technique was the publication of Bulletin 13 (27) in April, 1966. This Bulletin summarized and described several methods of determining flood frequency but did not recommend any particular methods. No testing or comparison of the various methods was attempted.

In December, 1967, the Work Group on Flow-Frequency Methods published Bulletin 15 "A Uniform Technique for Determining Flood Flow Frequencies" (1). Benson (5) provided additional details on the analysis and decisions that resulted in the publication of Bulletin 15. In Bulletin 15 six different flood frequency methods were applied to annual peak discharges at 10 long term stations (records greater than 40 years). They

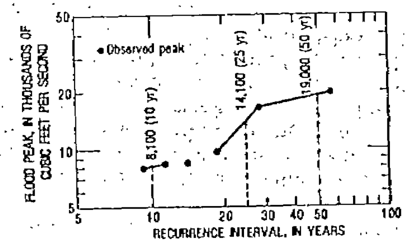

FIG. 1.—Example of Interpolating between Plotting Positions to Obtain Flood Discharges (5)

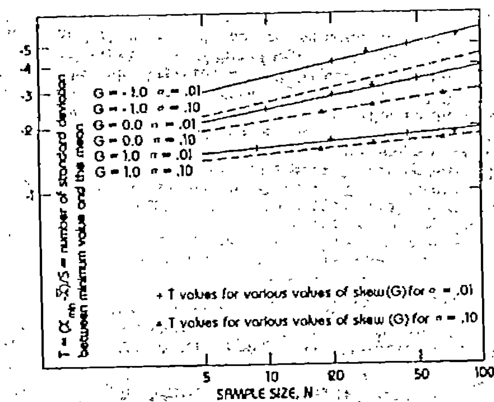

FIG. 2.—Relationships of Number of Standard Deviations (T) between Mean and Minimum Value to Stamp Size (N), Skew (G) and Level of Significance (α)

were: 2-parameter Gamma, Gumbel, log-Gumbel, log-Normal, log-Pearson Type III, and the Hazen method. The evaluation of the various distributions was based primarily on a comparison of flood discharges estimated by these six distributions to flood discharges estimated using probability plotting positions. Fig. 1, taken from Benson (5), illustrates how this comparison was made. Weibull plotting positions were determined for annual peak discharges for all 10 stations and were plotted on Gumbel probability paper. Straight lines were drawn between each plotting position and discharges for the 2-, 5-, 10-, 25- and, 50-year return periods were determined by interpolation. The percent difference between the interpolated value for each return period and the same value for each of the six distributions was computed by the following equation:

$$\text{percent difference} = \frac{Q_i - Q}{Q} * 100 \qquad (1)$$

in which Q_i = flood discharge estimated by distribution, i; and Q = flood discharge estimated by interpolating between plotting positions. It should be noted that the percent differences computed by Eq. 1 only examined the bias and not the precision of the various methods. In order to investigate the influence of the choice of plotting position and graph paper, the percent differences were also computed for selected return periods using Hazen plotting positions and log-Normal probability paper. The results were essentially the same. Of the commonly used plotting positions, the Weibull and Hazen plotting positions give the lowest and highest return periods, respectively, for a given quantile. Therefore, the results of the comparisons would not have changed if a distribution-free plotting position (such as Cunnane) had been used. On the basis of the percent differences, the log-Normal, Hazen and log-Pearson Type III methods gave comparable results and it was not possible to statistically discriminate between these methods. The recommendation in Bulletin 15 to fit the logarithms of the annual peak discharges to a Pearson Type III distribution using the method of moments was based on the statistical tests as well as the following reasons:

1. Some Federal agencies were already using the log-Pearson Type III distribution and computer programs were available.
2. The log-Normal is a special case of both the Hazen and log-Pearson Type III method with zero skew.
3. The log-Pearson Type III distribution utilizes skew as a third parameter and it was felt this would provide more flexibility and applicability in estimating flood discharges nationwide.
4. The Hazen method was partially graphical and required more subjectivity in its application.

The publication of Bulletin 15 was a significant event relative to flood-frequency analysis because for the first time a single method was recommended for use by all Federal agencies. However, it soon became evident that the Bulletin 15 technique was not as uniform as originally conceived. Some of the reasons were nonuniform treatment of outliers, nonuniform treatment of skew, and nonuniform treatment of historical

data. In the Bulletin 15 technique the inclusion or exclusion of extremely high or low annual peak discharges was at the discretion of the analyst. The primary problem was that the inclusion of very low peak discharges caused high negative skews and significantly affected the upper end of the frequency curve. Various analysts made different decisions concerning the inclusion of these low peak discharges resulting in nonuniform application of the technique. Most analysts used station skew in computing the frequency curve but some preferred to use zero skew or regional skew values. Finally, Bulletin 15 did not prescribe how to incorporate historical data into the analysis. Historical data in this context is the knowledge that a peak discharge occurring before, during or after systematic data collection is the largest for a period greater than the systematic data-collection period. Because no technique was recommended in Bulletin 15, many analysts did not use historical data in their studies. Many analysts used graphical techniques to incorporate historical data into the analysis and some used mathematical techniques such as the one proposed by Beard (3).

Bulletin 17

In January, 1972, the Hydrology Committee of WRC initiated review of Bulletin 15 and the need for more uniform guidelines. A new work group on floodflow frequency was established. In March, 1976, WRC published Bulletin 17 "Guidelines for Determining Flood Flow Frequency" (13). To correct problems noted with Bulletin 15, Bulletin 17 recommended the use of a low-outlier test, generalized skew, and a mathematical procedure for adjusting for historical data. Furthermore, Bulletin 17 recommended the continued use of the log-Pearson Type III distribution with the method of moments for parameter estimation. This recommendation was based primarily on a study by Beard (4) conducted for the work group on floodflow frequency. In Beard's study, eight combinations of distributions and estimation techniques were applied to annual peak discharges at 300 long-term stations (greater than 30 years of record). The eight different distributions and parameter estimation techniques—method of moments in log space (MM) and maximum likelihood (MLE)—were as follows: log-Pearson Type III (MM), log-Normal (MM), Gumbel (MLE), log-Gumbel (MLE), 2-parameter Gamma (MLE), 3-parameter Gamma (MLE), regional log-Pearson Type III (MM), and best linear invariant Gumbel. The regional log-Pearson Type III used a generalized value of skew (16) that was weighted with station skew. Each technique, after correction for average probability bias, was evaluated for accuracy and consistency using split-sampling techniques on the basis of:

1. The root-mean-square error of differences between the computed exceedance probabilities for one-half of the record (using the eight different techniques) and the same exceedance probabilities based on plotting positions for the other half.
2. The root-mean-square error of differences between the computed exceedance probabilities for the two record halves.

The regional log-Pearson Type III and the log-Normal distributions were shown to be most unbiased on the basis of (1) just noted. However, the regional log-Pearson Type III distribution was shown to be more consistent than the log-Normal on the basis of (2), also just noted. Other criteria for evaluation were the number of observed annual peak discharges exceeding the computed 1,000-year flood and the average observed exceedance probabilities for four different frequency floods for several regions of the country. Based on these various criteria, Beard (4) recommended the continued use of the log-Pearson Type III distribution but with the added provision that the station skew be weighted with generalized skew. Generalized skew is a regional value of skew based on several long term stations in a geographical area. Methods of determining generalized skew are examined later. In a more recent study Kuczera (20) also demonstrated that the weighting of generalized skew with station skew dramatically improved the performance of the log-Pearson Type III method of moments estimator.

Bulletin 17 represented a significant step toward achieving more uniformity in flood-frequency analysis. Several procedures suggested in Bulletin 17 to achieve this uniformity will be examined briefly for subsequent comparison with Bulletin 17B procedures.

A low-outlier test was provided in Bulletin 17 to give analysts a more uniform procedure for deciding when to include very low peak discharges in the analysis. The low-outlier test was of the following form:

$$\left|\frac{(X_{min} - \bar{X})}{S}\right| \geq \left[2.5 + 1.2 \log_{10}\left(\frac{N}{10}\right)\right](1 - 0.4\hat{G}) \quad \text{(2)}$$

in which X_{min} = logarithm of lowest value (values) in sample of N annual peak discharges; $\bar{X}$ = mean logarithm of all annual peak discharges; S = standard deviation of logarithms of all annual peak discharges; and $\hat{G}$ = generalized skew (not weighted with station skew). A low peak discharge (X_{min}) was considered an outlier if it satisfied Eq. 2. The way the test worked was that the right-hand side of Eq. 2 was computed as a low-outlier threshold. Using $\bar{X}$ and S based on the full sample, N, each peak discharge starting with the lowest value was tested by Eq. 2. The test was applied once but all peak discharges satisfying Eq. 2 were considered low outliers. The left side of Eq. 2 is the number of standard deviations the low-peak discharge is below the mean. The right-hand side of Eq. 2 was adapted from Grubbs and Beck (12) who studied the sampling distribution of minimum (and maximum) values occurring in normally distributed random samples of size N. Hardison (personal communication, 1977) modified the results given in Grubbs and Beck (12) for normal distributions to be applicable for a skewed Pearson Type III distribution thus obtaining the function in Eq. 2. What Hardison did was to:

1. Select the appropriate T-value from tables in Grubbs and Beck (12) for a given significance level and sample size.
2. Use the T-value from step 1 and a table of percentage points of the Pearson Type III distribution to determine the corresponding exceedance probability for zero skew.

3. Use the exceedance probability from step 2 and the given value of skew to determine the new T-value for the skewed distribution.

Fig. 2 relates the number of standard deviations (T) that a peak discharge is below the mean to sample size N, skew G, and significance level α. The right-hand side of Eq. 2 was computed from the linear relationship between T and N and G shown in Fig. 2 for a significance level of α = 0.01. Eq. 2 is a 1%, one-tail test which, on the average, should identify only one low outlier in every 100 samples tested if those samples are from a Pearson Type III distribution with skew coefficient, G. If one or more outliers are identified, the frequency curve is computed without the low outliers and then adjusted for the fact that a very low flood discharge(s) was censored, Jennings and Benson (19).

As noted earlier, Bulletin 17 recommended weighting station and generalized skew coefficient to obtain an improved estimate of skew. The weighted skew coefficient ($\check{G}$) was computed as follows:

$$\check{G} = \begin{cases} \hat{G} \quad \text{if} \quad N < 25 \text{ years} \\ \left(\frac{N-25}{75}\right) G + \left(1 - \frac{N-25}{75}\right)\hat{G} \quad \text{if} \quad 25 \leq N \leq 100 \text{ years} \\ G \quad \text{if} \quad N > 100 \text{ years} \end{cases} \quad \text{(3)}$$

in which $\hat{G}$ = generalized skew; G = station skew; and N = number of annual peak discharges. Eq. 3 was based on an accuracy assessment given by Hardison (16) in developing a comparable map on generalized skew. As can be seen from Eq. 3, equal weight is given to station and generalized skew when the observed record length N equals 62.5 years.

An adjustment procedure for historical floods was also provided in Bulletin 17 and is the same as the present adjustment procedure. The details of this adjustment procedure are described later in the section on Bulletin 17B.

Bulletin 17A

Shortly after Bulletin 17 was published it was noted that there was a discrepancy about the order of the historical adjustment and the determination of weighted skew. In June, 1977, Bulletin 17A, "Guidelines for Determining Flood Flow Frequency," (14) was published which clarified that the historical adjustment was to be applied before the weighting of skew. This clarification is the only significant difference between Bulletins 17 and 17A. A few editorial corrections were also made.

With time, problems with the Bulletin 17A methodology began to surface. These problems can be summarized as follows:

1. The low-outlier test did not adequately identify low outliers.
2. There was some confusion over the estimation and use of generalized skew.
3. There were inconsistencies in the use of the conditional probability adjustment for low outliers.

The low-outlier test in Bulletin 17A was a 1% significance level test. It did not always identify a low-peak discharge as an outlier even though this peak may be causing a large negative skew. Obviously a higher low-outlier threshold was needed to identify more outliers. Bulletin 17A contained a somewhat confusing flow chart for estimating generalized skew. In addition, it was not clear whether the Bulletin 17A generalized skew or the generalized skew developed by the analyst was the preferred value to use. In Bulletin 17A the conditional probability adjustment for samples with low outliers differed from the adjustment used with samples that had zero flow years and peaks below a gage base. This resulted in inconsistencies. Many analysts felt that the weighting procedure given by Eq. 3 gave too much weight to the generalized skew taken from the Bulletin 17A map. In addition, the weighting equation was based on the accuracy of the Bulletin 17A skew map and did not take into consideration the accuracy of generalized skew estimated from other procedures. Finally, a high-outlier test was needed to identify those large peaks occurring during systematic record that should be adjusted for historical information.

In September, 1981, Bulletin 17B, "Guidelines for Determining Flood Flow Frequency," (15) was published. Several technical changes were made in Bulletin 17B to correct problems noted in Bulletin 17A. The significant differences in the two Bulletins are:

1. Revised guidelines for estimating and using generalized skew.
2. A new procedure for weighting generalized and station skew.
3. A new test for detecting high outliers and a revised test for detecting low outliers.
4. Revised guidelines for the application of the conditional probability adjustment.

These revisions in Bulletin 17B are examined in the next section.

Bulletin 17B

Guidelines for the estimation of generalized skew were clarified in Bulletin 17B. The estimation procedure should utilize at least 40 stations or all stations within a 100-mile radius of the study area and with at least 25 years of homogeneous record. The generalized skew analysis for a given study area should include the following three methods:

1. Plot the station skew values on a map and draw isolines.
2. Compute a skew prediction equation based on watershed and climatic characteristics.
3. Compute the mean of the station skew values for the study area. The method with the lowest variance should be selected as the method for estimating generalized skew.

The skew map in Bulletin 17B is considered as an alternative for those who prefer not to develop their own generalized skew procedure. In the writers opinion, the skew map in Bulletin 17B should be used unless the analyst can demonstrate a significant improvement in the accuracy of generalized skew by using the analyst's own map or regression equation. The accuracy of the analyst's estimator should be compared to the accuracy of the Bulletin 17B map for the same geographical area. The accuracy of the Bulletin 17B map is not given for subregions of the United States and this must be determined by the analyst by computing the variation of the station skew values about the isolines of the Bulletin 17B map. The uniform application of Bulletin 17B suffers when several regional skew estimation procedures are developed. Therefore, analysts should coordinate the development of skew estimation procedures with all agencies with whom they coordinate flood-frequency estimates.

A new equation for weighting station and generalized skew was given in Bulletin 17B and is as follows:

$$G_w = \frac{MSE_{\bar{G}}(G) + MSE_G(\bar{G})}{MSE_{\bar{G}} + MSE_G} \qquad (4)$$

in which G_w = weighted skew coefficient; G = station skew; $\bar{G}$ = generalized skew; MSE_G = mean square error of station skew; and $MSE_{\bar{G}}$ = mean square error of generalized skew. The concept of weighting the station and generalized skew inversely proportional to their mean square errors was based on work by Tasker (34). Using Monte Carlo simulations, Tasker (34) demonstrated that more accurate estimates of flood frequency resulted when the station and generalized skew are weighted inversely proportional to their variances. The Bulletin 17B work group modified Tasker's approach to take into consideration the bias of the station skew coefficient. Eq. 4 is based on the assumptions that the station and generalized skew are independent estimates and that the generalized skew is unbiased.

The mean square error of the station skew (MSE_G) can be determined from Slack, Wallis, and Matalas (31). This mean square error can be computed from the equation

$$MSE_G = (\text{bias of skew coefficient})^2 + (\text{variance of skew coefficient}) \qquad (5a)$$

The bias and variance of skew coefficients for Pearson Type III random variables can be obtained from tables given in Slack, Wallis, and Matalas (31). In Bulletin 17B an equation is given for computing the MSE_G as a function of record length and magnitude of skew.

A revised low-outlier test and a new high-outlier test are given in Bulletin 17B. Low outliers are detected by the equation

$$X_L = \bar{X} - K_N S \qquad (5b)$$

and high outliers are detected by the equation

$$X_H = \bar{X} + K_N S \qquad (6)$$

in which X_L = low-outlier threshold; X_H = high-outlier threshold; $\bar{X}$ = mean logarithm of all peak discharges; S = standard deviation of logarithms of all peak discharges; and K_N = K values from Appendix 4 in Bulletin 17B which are equivalent to T values from Grubbs and Beck (12) for a one-sided 10% significance test. The order of testing for low and high outliers is based on the value of the systematic skew coefficient. If this skew coefficient is less than −0.4, then low outliers are detected and adjusted for first. If the skew coefficient is greater than 0.4, then high outliers are detected and adjusted for first assuming historical data are available. If skew coefficient is between −0.4 and 0.4, then the two tests are performed before any adjustments are made. The outlier tests shown in Eqs. 5a–b and 6 were selected on the basis of tests conducted with observed and simulated peak discharge data.

The following types of outlier tests were examined:

1. The Bulletin 17 outlier test as described earlier, as well as the Bulletin 17 outlier test with zero skew and with station skew substituted for generalized skew. (These tests were applied to low and high outliers.)
2. Low and high outlier detection based on Grubbs and Beck type tests for significance levels of 2.5, 5, and 10% with all combinations of station, zero, and generalized skew.
3. Low outlier detection based on the fact that the peak discharge is less than X% of the mean of next three lowest peaks. High outlier detection based on one of the Grubbs and Beck tests.
4. Low outlier detection based on an iterative computation of the frequency curve until there is less than a 10% change in the 2-, 10-, and 100-year flood discharges. High outlier detection based on one of the Grubbs and Beck tests.

Using various combinations of these procedures, approximately 50 tests were examined.

The first step in the analysis of the outlier tests was to apply the outlier tests to observed peak discharge data from 50 gaging stations for low outlier detection only. All work group members subjectively identified the number of low outliers for each station. On the basis of the number of low outliers identified by each test as compared to the consensus of the work group, the 50 outlier tests were reduced to 10 tests. The second step in this analysis was to use the 10 tests to detect high outliers as well and apply the tests to simulated log-Pearson Type III data. The simulated data had population skews ranging from ±1.5, but individual sample skews ranged from −3.67–3.25. The 10 outlier tests were applied to simulated samples and the sample estimates of the 2-, 10-, and 100-year flood discharges were compared to the true values. On the basis of the bias and root-mean-square of the 2-, 10-, and 100-year flood discharges, the 10 tests were reduced to 6 tests. The third and final step in the analysis was to apply the remaining 6 tests to observed peak discharge data. The observed skew values ranged from −2.2–2.8. As before, the work group identified the number of outliers (high and low) for each station. On the basis of the number of outliers identified by each test as compared to the consensus of the work group, the Grubbs and Beck tests using either zero skew or generalized skew at a 10 percent level of significance gave the most reasonable results. The Grubbs and Beck test with zero skew was selected as the best test because:

1. The test using zero skew is independent of generalized skew and, therefore, yields a more consistent test not affected by different generalized skew estimates.

2. Use of zero skew is consistent with the normal distribution assumption upon which the Grubbs-Beck tests are based.

The selected test performed as well as any other test but obviously is not a perfect test. There will be problems using the test in certain instances. However the new 10% test should be a significant improvement over the outlier test used in Bulletins 17 and 17A.

All editions of Bulletin 17 have contained the same historical adjustment procedure. Four different historical adjustment procedures were evaluated prior to publication of the original Bulletin 17. These were:

1. Compute the log-Pearson Type III frequency curve using only the peak discharges in the systematic record. Adjust the frequency curve on the basis of graphically plotted historical peaks.
2. Combine the historical peaks with the systematic peaks and compute the frequency curve giving equal weight to all peaks.
3. Assign plotting positions for flood discharges based on both periods of systematic and historical record, convert plotting positions to linear distances on the probability scale, and fit a regression line to the data (3).
4. The procedure adopted for Bulletin 17.

The procedure adopted for Bulletin 17 computes the logarithmic mean and standard deviation and the skew by giving more weight to systematic peaks than historic peaks. The procedure can best be explained by referring to Fig. 3. Assume a systematic data-collection period of N years with a high outlier at the end of the period and an extended period of H years during which a historical peak occurred. A historic threshold discharge Q_H is selected for which it is reasonably certain that no peak during the unobserved period (B-A) exceeded this threshold. The historic peak and high outlier are given a weight of 1.0 in computing the moments, and peaks in the periods A and (B-A) are given a weight of $W = B/A$. All systematic peaks excluding the high outlier are replicated during the period (B-A) under the assumption that these systematic peaks are representative of the unobserved flows during that period. Peaks below the gage base discharge (Q_{BB}) are also replicated during the period (B-A). This adjustment procedure creates a data set equivalent to H annual peaks by replicating the systematic flows less than Q_H. The procedure was suggested by Fred Bertle formerly of the U.S. Bureau of Re-

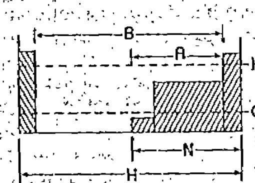

FIG. 3.—Graphical Representation of Historical Adjustment Procedure in Bulletin 17B

clamation (Bertle, personal communication, 1974).

An annual peak discharge may be identified as a low outlier or it may be a zero flow event, or may be undetermined because the peak stage was not high enough to be recorded at the gaging station. The flood-frequency curve is adjusted for the fact that these low (or zero) discharges occurred (19). All editions of Bulletin 17 have utilized this conditional probability adjustment but the procedure in Bulletin 17B was modified to achieve consistency in the flood-frequency curve regardless of whether or not low outliers or zero flow events occurred. The procedure consists of the following steps:

1. Compute the flood-frequency curve without low outliers, zero flow years, and peaks not recorded by the gage (i.e., use "above base" peaks).
2. Multiply the exceedance probabilities from this frequency curve by the ratio of the number of "above base" peaks to the total number of peaks.
3. Compute the mean, standard deviation, and skew for the adjusted frequency curve in step 2 using equations in Appendix 5 of Bulletin 17B.
4. Weight the computed station skew from step 3 with the generalized skew to obtain the final skew.

This adjustment procedure is described in greater detail in Appendix 5 of Bulletin 17B. The conditional probability adjustment procedures in earlier editions of Bulletin 17 differed depending upon whether or not low outliers or zero flow events were in the sample. The new procedure in Bulletin 17B should be an improvement toward achieving a more uniform and consistent technique.

The changes previously mentioned are the most significant improvements incorporated in Bulletin 17B. Other changes in Bulletin 17B from Bulletin 17A are a modification of the two-station comparison procedure (Appendix 7), a different procedure for computing confidence limits (Appendix 9; also see Ref. 33), and an expanded flood-risk table (Appendix II). These changes do not directly affect the computed flood discharges and are not described here.

In March 1982 several editorial corrections were made in Bulletin 17B and the guidelines were republished by the Hydrology Subcommittee of the Interagency Advisory Committee on Water Data. The new sponsorship of Bulletin 17B was necessitated by the dissolution of the Water Resources Council in the Fall of 1982. The Hydrology Subcommittee remained the same under the two different sponsors.

Future Direction of Flood Frequency Analysis in United States

The Bulletin 17B technique has evolved over the last 17 years since the publication of Bulletin 15. There is no uniform technique that could be adopted which, when rigidly applied to available data, would accurately define the flood potential of any given watershed. Engineering judgment should always be exercised in evaluating the results. In those cases where Bulletin 17B does not give reasonable estimates, other approaches may be used provided they are properly documented and comparisons are made with the Bulletin 17B results. In the writer's opinion

the Bulletin 17B technique has been developed to an acceptable operational level. The major problems noted earlier in Bulletins 17 and 17A have been corrected. However any technique can be improved and in 1984 a new Federal interagency work group was convened to decide if Bulletin 17B should be revised. At the writing of this paper it had not been decided if there was sufficient justification and resources to revise the guidelines. If it is decided to revise Bulletin 17B this future interagency work group should reconsider the questions of the appropriate distribution, the procedures for computing the parameters of the distribution, the utility of generalized skew, and even the logarithmic transformation. Kuczera (21), for example, suggested using the power transform proposed by Box and Cox (6) to normalize the annual peak discharges. Several new and interesting concepts related to flood estimation at gaging stations have recently been described in the literature and these need to be evaluated. Numerous papers have been written suggesting or advocating probability distributions other than the Pearson Type III. A distribution which received considerable attention was the Wakeby distribution described by Houghton (18) and again by Landwehr and others (23). The value of the Wakeby distribution for providing uniform and accurate estimates of flood frequency needs further evaluation. Tung and Mays (36) suggested that the sample mean, standard deviation, and skew be weighted with regionalized estimates of the same parameters to reduce the parameter uncertainty. They suggested using two nonparametric methods, (the jackknife and bootstrap methods), to compute the variances of the sample parameters. Rao (28) suggested using the method of mixed moments to estimate the parameters of the log-Pearson Type III distribution. In this approach the mean and standard deviation of the untransformed data and the mean of the logarithmically transformed data are used to compute the parameters of the log-Pearson Type III distribution. Condie (8) and Matalas and Wallis (25) examined the advantages and disadvantages of fitting the Pearson Type III distribution using maximum likelihood estimates. Greenwood and others (11) introduced the concept of probability weighted moments and examined their applications to certain types of distributions. Landwehr, Matalas, and Wallis (22) questioned whether it is feasible to construct regional skew maps of the logarithmically transformed annual-peak discharges as advocated in Bulletin 17B. Their study indicates that the geographical distribution of logarithmic skew values appears to be unrelated to the geographical distribution of the untransformed skew values. However the critical issue seems to be whether the mapping of logarithmic skew improves the estimates of selected design floods and not whether there is a one to one correspondence between logarithmic and untransformed skew. Beard (4) and Kuczera (20) have demonstrated that the use of regional skew does improve estimates of design flood discharges. McCuen (26) examined related issues and the relative precision of available skew maps. The references listed previously are not all-inclusive but only represent some of the publications that should be considered when new flood-frequency guidelines are developed.

In addition to the questions of the appropriate distribution, fitting procedures, generalized skew and data transformations, there are other areas or topics that need further study. Some of these are:

1. Evaluation of techniques for testing the homogeneity of the annual peak series. These techniques are needed to determine homogeneous periods of record for watersheds undergoing land use or channel changes. An example of using parametric and nonparametric tests is given by Hirsch and others (17).

2. Development of procedures to analyze a nonstationary annual peak series if in fact the watershed is undergoing some channel or land use change.

3. Identification and treatment of mixed distributions resulting from two or more types of hydrologic events such as flooding from snowmelt versus thunderstorm activity. Russell (29) and Singh (30) suggested that two separate distributions be combined to better fit the distribution of annual peak discharges.

4. Treatment of outliers both as to identification and computational procedures.

5. Alternative procedures for treating historic data. Leese (24), Condie and Lee (9), and Cohn and Stedinger (7) suggested consoring theory and maximum likelihood estimates as ways of more efficiently utilizing historic information. The computational disadvantages of the censoring theory/maximum likelihood estimates must be indicated against any improvement in accuracy.

6. Procedures to incorporate flood estimates based on precipitation or data available at nearby watersheds, or both, (for example a regional estimation procedure) into the frequency analysis at a given station.

Research is needed in all these areas to improve the hydrologist's ability to make more accurate and consistent estimates of flood frequency and flood risk.

There are other more general questions that should be addressed by any future Federal interagency work group. Stedinger (1983) again raised the question whether it is better to have an unbiased etimate of the design flood or whether it is better for the design flood to be exceeded, on average, with the intended frequency. He noted that the 100-year flood estimate based on a sample size of 16 from a log-normal distribution has an exceedance probability of 0.02 rather than the intended 0.01. Beard (2) and Thomas (35) also examined this problem. The advantages and disadvantages of using the partial duration series instead of the annual flood series should be reevaluated. Finally, the utility of using geomorphic, stratigraphic, and botanic techniques to evaluate the return periods of catastrophic floods warrants further investigation. A comprehensive review of the application of these techniques is given by Costa (10).

As evidenced by the list of recent publications, there has been considerable research conducted and published since the original Bulletin 17 was written. The writers of the original bulletin did not have this recent information available for evaluation. The scope of the Bulletin 17A and 17B work groups was to improve Bulletin 17 within the framework of the log-Pearson Type III-methods of moments-generalized skew methodology described in the original bulletin. Therefore, no major de-

viations from the Bulletin 17 methodology was investigated. It is the writer's opinion that a new Federal interagency work group should be established to evaluate recent research relative to its impact on new flood frequency guidelines.

Conclusions

Floods annually cause approximately $2 billion in property damage, the death of about 200 people, and displace another 200,000 people from their homes. Consequently, several billion dollars are spent each year on construction of transportation facilities and flood control structures and the definition of flood plains for land-use planning. Federal agencies concerned with flood frequency estimation have developed a uniform technique for flood frequency analysis in order that the flood control projects can be equitably evaluated and flood plains consistently defined. This uniform technique has evolved over the last 17 years with the publication of U.S. Water Resources Council Bulletin 15 (1), Bulletin 17 (13), Bulletin 17A (14), and Bulletin 17B (15). Each subsequent edition of these Bulletins achieved greater uniformity and consistency in the analysis techniques. The present guide for flood frequency analysis (Bulletin 17B) has been developed to an acceptable operational level. There is a Federal interagency work group presently evaluating whether Bulletin 17B should be revised. Recent publications have proposed innovative and interesting approaches to flood frequency analysis. This recent research should be evaluated by a new Federal interagency work group relative to the development of new flood frequency guidelines.

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Appendix II.—Notation

The following symbols are used in this paper:

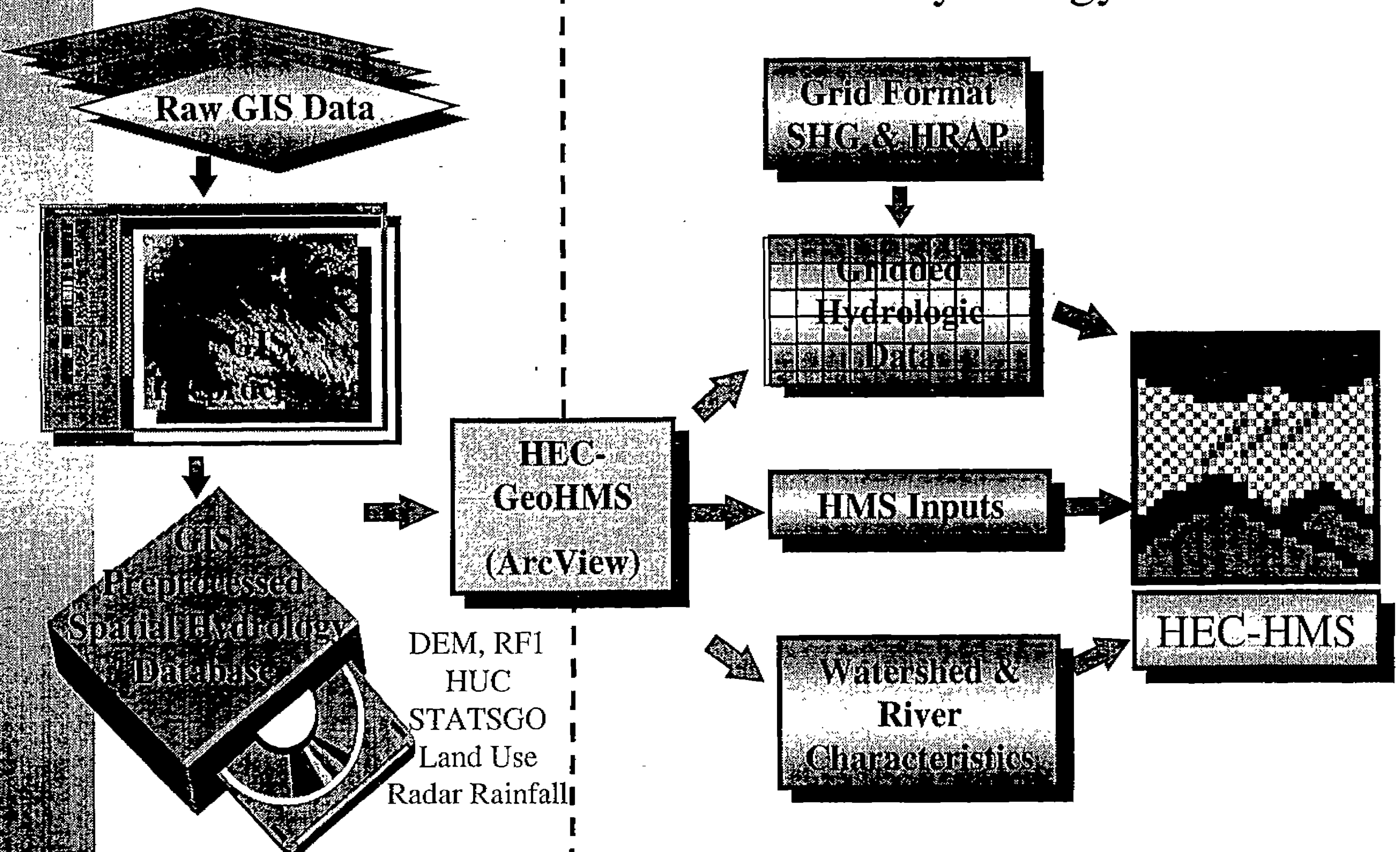
A = number of years in systematic record minus any high outliers;
B = number of years in historic period minus any high outliers or historic peaks;
G = station skew;
G = weighted skew;
$\hat{G}$ = generalized skew;
H = number of years in historic period;
K_n = K value of Pearson Type III distribution used in outlier tests;
MSE_G = mean square error of station skew;
$MSE_{\hat{G}}$ = mean square error of generalized skew;
N = sample size or number of annual peak discharges;
Q = flood discharge estimated by interpolating between plotting positions (Bulletin 15);
Q_{BB} = gage base discharge;
Q_i = flood discharge estimated by distribution, i (Bulletin 15);
S = standard deviation of logarithms of annual peak discharges;
T = number of standard deviations between the minimum annual peak and mean;
W = weight given to systematic peaks when adjusting frequency curve for historic data;
X_H = high-outlier threshold;
X_L = low outlier threshold;
X_{min} = logarithm of lowest value (values) in sample of N annual peak discharges;
$\bar{X}$ = mean logarithm of annual peak discharges; and
α = significance level for outlier testing.

附件五

Overview

GIS R&D

Watershed Hydrology R&D



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3

Hydrologic GIS Concepts

8-point pour model

32	64	128
16		1
8	4	2

Flow direction codes

78	72	69	71	58
74	67	56	49	46
69	53	44	37	38
64	58	55	22	31
68	61	47	21	16

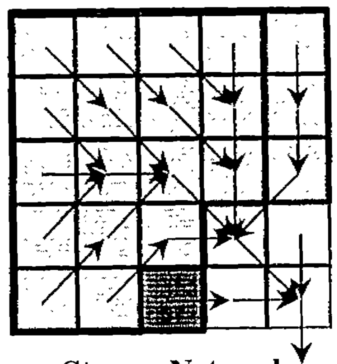
Digital elevation model (DEM)

2	2	2	4	4
2	2	2	4	4
1	1	2	4	8
128	128	1	2	4
128	128	1	1	4

Flow direction grid

0	0	0	0	0
0	1	1	2	1
0	3	8	5	2
0	1	1	20	0
0	0	0	1	24

Flow accumulation grid



Stream Network

L-1841/JHD/2000

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同意書

本人同意中華民國八十九年六月二十八日起至八十九年十二月二十三日共計六個月執行因公出國計畫「研習美國水資源經營與管理」所完成之出國報告書，其著作財產權歸屬中華民國。

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中華民國九十年三月十五日