US Army Corps of Engineers
Hydrologic Engineering Center

Proceedings of a Hydrology & Hydraulics Workshop on

Riverine Levee Freeboard

27 - 29 August 1991
Monticello, Minnesota

SP-24

REPORT DOCUMENTATION PAGE

Form Approved OMB No. 0704-0188

1. REPORT DATE *(DD-MM-YYYY)* 27 - 29 August 1991	**2. REPORT TYPE** Seminar Proceedings	**3. DATES COVERED** *(From - To)*
4. TITLE AND SUBTITLE Hydrology & Hydraulics Workshop on Riverine Levee Freeboard	**5a. CONTRACT NUMBER** **5b. GRANT NUMBER** **5c. PROGRAM ELEMENT NUMBER**	
6. AUTHOR(S) CEIWR-HEC	**5d. PROJECT NUMBER** **5e. TASK NUMBER** **5F. WORK UNIT NUMBER**	
7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES) US Army Corps of Engineers Institute for Water Resources Hydrologic Engineering Center (HEC) 609 Second Street Davis, CA 95616-4687	**8. PERFORMING ORGANIZATION REPORT NUMBER** SP-24	
9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)	**10. SPONSOR/ MONITOR'S ACRONYM(S)** **11. SPONSOR/ MONITOR'S REPORT NUMBER(S)**	

12. DISTRIBUTION / AVAILABILITY STATEMENT
Approved for public release; distribution is unlimited.

13. SUPPLEMENTARY NOTES
Collection of papers presented at a workshop held at Riverwood Conference Center, Monticello, Minnesota on 27 - 29 August 1991.

14. ABSTRACT
The Hydraulics and Hydrology Branch, HQUSACE and the Corps Hydrology Committee co-sponsored a workshop on Riverine Levee Freeboard on 27 - 29 August 1991. The St. Paul District hosted the workshop at the Riverwood Conference Center, Monticello, Minnesota. The Hydrologic Engineering Center (HEC) was responsible for the technical program and workshop coordination.

The workshop consisted of four one-half day sessions and one evening session. There were twenty-seven invited workshop participants who presented a total of thirteen papers and eleven panel discussions. These proceedings are a compilation of the papers and panel discussions. Participants included representatives from headquarters, division and district offices, the Hydrologic Engineering Center, the Institute for Water Resources (IWR), and the Waterways Experiment Station (WES). A representative from the Federal Emergency Management Agency (FEMA) and a private consultant from Water Engineering and Technology, Inc. also participated.

The major objectives of the workshop were to 1) identify issues related to present procedures for determining riverine levee project height, including freeboard, 2) discuss alternative procedures for determining levee height, and 3) establish a direction for implementing new policy and procedures for determining levee height that would eliminate the concept of freeboard, and incorporate risk and uncertainty in the analysis.

15. SUBJECT TERMS
Initial Project Management Plans (IPMP), feasibility phase, reconnaissance phase, hydrology, hydraulics

16. SECURITY CLASSIFICATION OF: a. REPORT	b. ABSTRACT	c. THIS PAGE	**17. LIMITATION OF ABSTRACT**	**18. NUMBER OF PAGES**	**19a. NAME OF RESPONSIBLE PERSON** / **19b. TELEPHONE NUMBER**
U	U	U	UU	286	

Standard Form 298 (Rev. 8/98)
Prescribed by ANSI Std. Z39-18

Proceedings of a Hydrology & Hydraulics Workshop on

Riverine Levee Freeboard

27 - 29 August 1991
Monticello, Minnesota

Attendees:
Corps of Engineers
Federal Emergency Management Agency
Michael Baker, Jr., Inc.
Water Engineering & Technology

Sponsored By:
US Army Corps of Engineers
Institute for Water Resources
Hydrologic Engineering Center
609 Second Street
Davis, CA 95616

(530) 756-1104
(530) 756-8250 FAX
www.hec.usace.army.mil

SP-24

TABLE OF CONTENTS

TABLE OF CONTENTS (continued)

TABLE OF CONTENTS (continued)

TABLE OF CONTENTS (continued)

Page
No.

FOREWORD

The Hydraulics and Hydrology Branch, HQUSACE and the Corps Hydrology Committee cosponsored a workshop on Riverine Levee Freeboard on 27 - 29 August 1991. The St. Paul District hosted the workshop at the Riverwood Conference Center, Monticello, Minnesota. The Hydrologic Engineering Center (HEC) was responsible for the technical program and workshop coordination.

The workshop consisted of four one-half day sessions and one evening session. There were twenty-seven invited workshop participants who presented a total of thirteen papers and eleven panel discussions. Participants included representatives from headquarters, division and district offices, HEC, the Institute for Water Resources (IWR), and the Waterways Experiment Station (WES). A representative from the Federal Emergency Management Agency (FEMA) and a private consultant from Water Engineering and Technology, Inc. also participated.

The major objectives of the workshop were to 1) identify issues related to present procedures for determining riverine levee project height, including freeboard, 2) discuss alternative procedures for determining levee height, and 3) establish a direction for implementing new policy and procedures for determining levee height that would eliminate the concept of freeboard, and incorporate risk and uncertainty in the analysis. These proceedings include a compilation of the paper and panel discussions presented at the workshop.

ACKNOWLEDGEMENTS

The St Paul District is commended for the excellent job they did in selecting a conference site. A special thanks goes to Robert Engelstad, Chief, Hydrology Section and Pat Foley, Chief, Hydraulics Section, for coordinating the accommodations and other necessary arrangements. They served as outstanding hosts throughout the workshop. Opening remarks were made by Helmer (Bud) Johnson, Chief, Geotechnical and H&H Branch, Engineering Division, St. Paul District. His support and participation was very much appreciated.

Lew Smith, HQUSACE, provided several excellent suggestions for topics and participants for the workshop. His coordination efforts contributed significantly to the overall success of the workshop. Darryl Davis, Director, and Michael Burnham, Chief, Planning Analysis Division, HEC, contributed significantly to the format and content of the workshop. Their unending support and encouragement is very much appreciated. Michael Burnham served as workshop monitor. Harry Dotson, Planning Analysis Division, HEC, was responsible for developing the technical content and format of the workshop, making arrangements with presenters, and publishing the workshop proceedings. Last, but certainly not least, the individual participants, who's excellent presentations made this workshop the successful and exceptional experience it was, deserve special acknowledgement.

HYDROLOGY AND HYDRAULICS WORKSHOP
on
RIVERINE LEVEE FREEBOARD

EXECUTIVE SUMMARY

INTRODUCTION

The Hydraulics and Hydrology Branch, HQUSACE and the Corps Hydrology Committee cosponsored a workshop on Riverine Levee Freeboard on 27 - 29 August 1991. The St. Paul District hosted the workshop at the Riverwood Conference Center, Monticello, Minnesota. The Hydrologic Engineering Center (HEC) was responsible for the technical program and workshop coordination.

The workshop consisted of four one-half day sessions and one evening session. There were twenty-seven invited workshop participants who presented a total of thirteen papers and eleven panel discussions. Participants included representatives from headquarters, division and district offices, HEC, the Institute for Water Resources (IWR), and the Waterways Experiment Station (WES). A representative from the Federal Emergency Management Agency (FEMA) and a private consultant from Water Engineering and Technology, Inc. also participated.

The major objectives of the workshop were to 1) identify issues related to present procedures for determining riverine levee project height, including freeboard, 2) discuss alternative procedures for determining levee height, and 3) establish a direction for implementing new policy and procedures for determining levee height that would eliminate the concept of freeboard, and incorporate risk and uncertainty in the analysis. This summary provides an overview of the issues discussed during the paper and panel presentations and the conclusions reached at the close of the workshop.

EXISTING LEVEE FREEBOARD POLICY

General Corps' guidance on levee freeboard states that freeboard is provided to ensure that the desired degree of protection will not be reduced by unaccountable factors that effect water surface elevation. Freeboard is the added increment in levee height that accounts for the uncertainties in the design water surface elevation. The guidance is interpreted to include, as part of the freeboard, the final levee grade adjustments necessary to plan the location and manner of overtopping.

Freeboard is to be designed and the final results supported and documented. Site specific analyses of the uncertainties involved in the determination of the design water surface elevation are required. Final levee grade is to be engineered to allow for overtopping at least hazardous locations and in a safe and controlled manner.

Minimal guidance exists that addresses how to account for uncertainties in discharge and economic aspects of levee sizing.

CURRENT PROCEDURES

Several different approaches to determining freeboard for a host of projects were identified during workshop paper and panel presentations. The types of projects discussed included low levees, urban levees, agricultural area levees, levees for high velocity channels, levees for containing dam impoundments, and the evaluation of both new and existing levees. Methods varied from simply applying a fixed minimum (often three feet) to determining the appropriate freeboard by attempting to account for the uncertainties in the factors that have a significant impact on water surface elevation computations. Factors most often identified include channel roughness, channel restrictions and associated expansion and contraction coefficients, channel bed form stability, ice and debris, waves and super elevation at bends. Typically, the design water surface elevation is determined from the best estimates of these factors. Next, the increase in water surface elevation resulting from incorporating values that represent reasonably high conveyance losses that could occur given the uncertainty of the best estimates is computed. The increase in water surface elevation establishes minimum freeboard. Final adjustments to the levee grade are made to meet design objectives of safety and performance, including the assurance that the levee will overtop in a safe manner and at the least hazardous locations.

CURRENT ISSUES

Several important issues were surfaced during the paper and panel presentations. The following summarizes the major problems associated with applying current levee freeboard policy and procedures that were identified.

- If three feet is the generally accepted minimum standard for freeboard, how can this be appropriately applied when the determined base design levee height for a specific project is low (e.g., five feet or less, where the cost applicable to freeboard is a disproportionately large share of the total cost) and a lower minimum cannot be justified?

- If levee freeboard is to be designed, as opposed to being taken as a minimum, how can the uncertainties of the hydrologic, hydraulic, and economic parameters best be quantified to preserve reasonableness, consistency, and acceptability?

- If a project involves an existing levee upgrade, how can we quantify the existing condition and with project flood hazard, while preserving consistency and appropriately accounting for freeboard?

- What type of analysis should be applied to evaluate levee overtopping floods and what detail do we need to appropriately identify when and how a failure will occur, where the water goes, and at what velocity and depth?

- Given the current cost sharing environment and the difficulty in defining the return for the required investment in freeboard, how can we better represent ultimate levee height to our cost sharing partners who need to know what protection their dollars are buying and the cost and associated risks of projects larger or smaller than the Corps' "recommended" plan?

- If the Corps revises current policy and procedures for determining ultimate levee height, and in effect eliminates the concept of "freeboard" as it is now applied, how will this impact FEMA's administration of the National Flood Insurance Program (NFIP) regarding levee certification?

RISK ANALYSIS FRAMEWORK APPROACH

An alternative approach was presented in the final session of the workshop which proposes sizing levees based on risk and uncertainty without identification or addition of freeboard. The risk analysis framework proposed uses statistical distributions of the error in the estimates of the several important variables including discharge-frequency, stage-discharge, stage-damage relationships, and perhaps cost, which are used to determine levee height. The statistics quantify the uncertainties in the relationships. Exhaustive trials are made using Monte Carlo simulations to account for all the possible combinations of the errors in the relationships. The result of the risk analysis framework approach is a matrix of levee height, probability distributions of the several variables, expected cost and benefit, and exceedances for selected design heights. The National Economic Development (NED) plan is identified in accordance with current policy. The matrix is further used as appropriate, to evaluate project sizes larger or smaller than the NED plan. The performance of the selected plan would be expressed in reliability terms such as, "the levee has a 90% chance of protecting against the .005 chance exceedance (200-year) flood event, should it occur and a 75% chance of protecting against a .002 chance exceedance (500-year) flood event, should it occur", in lieu of stating level-of-protection, as is the current procedure.

After the base levee size is selected final levee grade is engineered to assure that the levee overtops in the least hazardous location and in a safe, gradual manner, and provides superiority at critical locations such as at pumping stations or other locations where integrity is essential to project performance. These final adjustments or additions to levee grade are made to meet the engineering and design objectives assuring project safety and performance and will no longer be considered "freeboard".

CONCLUSIONS

The identified issues and the described new approach were discussed at length during the final session of the workshop. Details on what needed to be accomplished to implement the risk analysis framework approach were discussed in an open forum lead by HQUSACE. Major conclusions and recommendations are listed below:

- The risk analysis framework approach for determining levee size without reference to freeboard is the recommended procedure and will be phased into Corps' levee project formulation. Traditional use of the concept of "freeboard" and use of the term will be phased out.

- Interim policy defining the phasing in of the risk analysis framework approach will be in the form of an EC jointly issued by Policy & Planning and Engineering Divisions, HQUSACE.

- Although present knowledge is considered adequate to apply the new procedure now, research is needed to further develop and refine the procedure so it can be more effectively applied in the field.

- An appropriate means for technology transfer was discussed. Possible approaches identified were: 1) a formal or informal training program, 2) field office workshops, and 3) a seminar similar to what was done for flood warning and preparedness programs. In the interim, the risk analysis framework approach will be announced to those disciplines involved at every opportunity (meetings, training sessions, etc.).

- The Corps will continue to work closely with FEMA in support of their management of NFIP and will coordinate implementation of new levee project formulation policy.

SUMMARY OF SESSION 1: LEVEE FREEBOARD POLICY

Overview

This session included three papers and a panel on current levee freeboard policy and issues related to the application of current policy. Participants included representatives from HQUSACE, a Division Office, and the Federal Emergency Management Agency.

Paper Presentations

Paper 1. Earl Eiker, Chief H&H Branch, Engineering Division, HQUSACE, presented a paper entitled, "Freeboard Design for Urban Levees and Floodwalls." He emphasized that the purpose of freeboard is to achieve specific design objectives and to account for the uncertainties in the computation of water surface profiles. Design objectives include assurance that initial overtopping will occur in the least hazardous location, prevention of chain failure and providing for levee superiority at critical locations. Uncertainties in water surface profile computations are due to errors attributable to inadequacies of procedures and data, instabilities in stage- discharge relationships and other factors. Mr. Eiker described what he considers an adequate approach to account for uncertainties in water surface profile computations under current levee freeboard policy. The procedure, described in detail in the paper, involves estimating the stage and discharge using conservative losses to establish the minimum levee grade at the point of initial overtopping and engineering the levee grade upstream of this point to assure gradual, non-hazardous overtopping for higher discharges.

In discussions during and following the paper Mr. Eiker made it clear that the focus of the workshop would be to move away from using the concept of "freeboard" and to establish a direction for incorporating risk and uncertainty in the analysis for determining levee height.

Paper 2. Robert Daniel, Chief, Economic and Social Analysis Branch, Policy and Planning Division, HQUSACE, submitted a paper that described the need and a proposal for a risk analysis approach. Mr. Daniel's presentation was essentially an open discussion which he began by emphasizing the urgency of establishing guidance and procedures for determining levee height by a risk analysis framework approach and without using the concept of "freeboard". He indicated that the levee freeboard approach is just not workable from an economic analysis viewpoint. Levee freeboard issues are upon us now and a risk analysis framework approach has already been promised as a remedy.

During the discussion it was stated that levees should be categorized in terms of probability and level of assurance of passing a given frequency flood. For example, a given levee would have a 90% chance of protecting against the .005 chance exceedance (200-year) flood event, should it occur and a 75% chance of protecting against a .002 chance exceedance (500-year) flood event, should it occur, in lieu of stating that the levee has a specific level-of-protection, as is currently done. Several issues pertaining to the new approach were discussed.

Paper 3. John Matticks, Asst. Administrator, Office of Risk Assessment, Federal Insurance Administration (FIA), Federal Emergency Management Agency (FEMA), presented a paper entitled, "FEMA Levee Accreditation Procedures." In summary, the paper describes the evolution of FEMA's levee policy, current levee accreditation procedures, differences between procedures adopted by FEMA and those used by the Corps of Engineers, and on-going and future activities regarding levee evaluation. Key evaluation criteria is that a levee must have a minimum of three feet of freeboard above the base one percent chance (100-year) flood unless certified to provide protection from the one percent chance flood by the Corps or other Federal agency. The National Flood Insurance Program (NFIP) also provides partial credit through the Community Rating System for levees with less than one percent chance flood level of protection. In answer to a question as to how a new risk based approach to determining levee height in the Corps may affect the NFIP, John indicated that if the Corps certifies that the levee will provide one percent chance flood level of protection, FEMA would most likely accept it as is currently the case. It would be up to the Corps to decide what level of risk would be acceptable to make the certification. Mr. Matticks was available during the entire workshop to answer questions regarding the relationship between NFIP and Corps levee freeboard policy and contributed significantly to the overall success of the workshop.

Panel 1: Policy Issues

Four panel members made short presentations based on their individual experiences that highlighted specific issues related to current levee freeboard policy.

A. James Mazanic, North Central Division (NCD), discussed existing levee freeboard policy and how it relates to situations involving low levees with minimal risks. When levee projects are relatively low (three feet or less) the required minimum freeboard height should be less than the three feet that has been the "standard" minimum in the past. The cost of an additional three feet of freeboard for a low levee project would typically render the project economically infeasible. Based on experiences in NCD Jim would recommend that any new levee freeboard policy include a provision for relaxing the three foot minimum requirement for low levee projects where it can be shown that hazards from an overtopping flood would be minimal.

B. Kenneth Zwickl, Flood Plain Management Services and Coastal Resources Branch, Policy and Planning Division, HQUSACE, made a presentation on, "Levee Freeboard Policy Issues Impacting on Corps Support to the National Flood Insurance Program (NFIP)." He discussed the differences between Corps and FEMA levee freeboard requirements, FEMA's levee certification requirements, and the ramifications of the Corps providing levee certifications to FEMA. A key point made during the presentation was that changes in Corps procedures for establishing levee height could impact on FEMA's administration of the NFIP and that the Corps must continue to coordinate policy changes closely with FEMA to minimize negative impacts on either agencies programs related to the NFIP. Ken noted that differences in the application of levee freeboard requirements between the Corps and FEMA are generally handled satisfactorily on a case-by-case basis. He emphasized that the Corps should continue to make every effort to comply with the levee certification process and to ensure that levee level-of-protection is not over or under stated.

C. Lew Smith, H&H Branch, Engineering Division, HQUSACE, discussed, "Freeboard, Overtopping and Safety for Levees with Low Levels of Protection." His main concern was that the effects of overtopping and public safety issues be fully addressed for low level of protection levees which may be subject to frequent overtopping. His presentation described overtopping concerns and safety considerations that are particularly important for these types of projects.

D. Tom Munsey, H&H Branch, Engineering Division, HQUSACE, presented a panel discussion entitled, "Flood Overtopping of Levees - Intent and Implications of Current Design Guidance and Possible Modifications." Tom outlined several questions pertaining to current local protection levee project overtopping design that, in his mind, are not addressed in current guidance. Included were questions pertaining to where the levee will fail and in what manner, where the water will go and how fast will it flow, and how do we analyze these factors. He indicated that these questions will remain unanswered whether new methods for determining levee height are adopted or not.

FREEBOARD DESIGN FOR URBAN LEVEES AND FLOODWALLS[1]

Prepared by
Roy G. Huffman[2]

Edited and Presented by
Earl E. Eiker[3]

BACKGROUND

This paper discusses the hydrologic engineering analysis required to design a top of containment profile for levees and floodwalls protecting urban areas from riverine flooding. The design process described herein will result in providing a marginal height above the design flood water surface profile to the top of the containment elevation. This marginal height is called freeboard. Designed freeboard is an integral component of a levee floodwall and is necessary to provide, insofar as practical, a safe and functional project. It is Corps policy that freeboard will be designed to achieve specific purposes with as much care as any element of a major project.

FREEBOARD PURPOSES

Conceptually, freeboard is provided to reasonably assure that the project design flow will be contained, given the uncertainty of water surface profile computations, and to minimize the damages and any threat to life in the event the levee is overtopped. In the design of freeboard it is convenient to consider that freeboard has two primary purposes (1) to achieve specific design objectives, and (2) to allow for the uncertainty inherent in the computation of a water surface profile.

1. Design Objectives. An increment of freeboard is provided to achieve design objectives. Some examples of design objectives are:

a. Adjustments in levee grade to insure initial overtopping at the most desirable (least hazardous) location and a gradual extension of the length of levee experiencing overtopping. This may require computing the actual slope of the water surface profile for an incipient overtopping flood.

b. Prevent chain failure; i.e. the failure of a levee causing failure of a downstream levee and so on.

c. Reduce volume of wave overtopping and/or likelihood of levee failure due to wave action.

---

[1]The original paper was part of a briefing for MG Kelly, Director of Civil Works in Feb 90.

[2]Chief, Hydrology Section, H&H Branch, HQUSACE (CECW-EH-Y) Washington, DC, (Retired)

[3]Chief, H&H Branch, HQUSACE (CECW-EH) Washington, DC

d. Adjustments in levee grade to provide superiority in grade at critical locations (e.g. at a pumping station) and/or one levee over another.

e. To extend the interval between major maintenance such as removal of tree growth and sediment deposition from the river channel.

2. Water Surface Uncertainties. A freeboard allowance is provided to insure, as fully as practicable, against overtopping due to uncertainties in determination of a flood profile. Those allowances that are not otherwise specifically accounted for in the design flood flow line because they are considered too small to require specific determination or because they are too intractable to be quantified are also included. Matching computational procedures, basic data, and assumptions with reality is generally only partly achieved, resulting in some error in computed water surface profiles. Some examples of factors contributing to uncertainty in a water surface profile computation, insofar as these have not been specifically accounted for in hydraulic computations involved in a design flood profile estimate, are as follows:

a. Errors in profile computation attributable to inadequacies of procedures, basic data, etc.

b. Dynamic effects and short-period discharge fluctuations (Surges, transient flood waves, wind tides, waves).

c. Short term instabilities in stage-discharge relationships resulting from changes in bed forms, flow retardance by debris, and ice, etc.

d. Uncertainties due to braids, meander, abrupt expansions and constrictions, etc.

e. Uncertainties of project maintenance interval and quality.

An appropriate allowance for uncertainty may be established analytically by examining the sensitivity of the water surface elevation to assumptions, data, etc. Or the uncertainty allowance may be estimated from experience and judgment. In most cases some combination of informed judgment and sensitivity analysis will be the most appropriate approach. Each factor contributing to uncertainty does not increase the freeboard allowance for uncertainty in an additive manner. Some of the factors may cancel each other to some extent, or not be effective at the same time.

FLOOD PROFILE

Those factors that influence the water surface profile, and level of protection, and that can reasonably be quantified are included in the design flood flow line; not the freeboard. Some examples of what these factors may be are as follows:

1. Changed conveyance due to sedimentation or scour, and vegetation growth or removal.

2. Super elevation and standing waves.

3. Transverse slope due to water flowing out of or into the channel or differences in velocity head between the channel and overbank locations.

4. Energy losses due to changing flow area; e.g. constrictions (bridges), expansions, and bends.

5. Future changes in flood flows due to changes in the watershed.

FLOOD FLOW UNCERTAINTY

Uncertainty associated with a flood flow rate estimate is not included in freeboard. Flood flow uncertainty is more appropriately treated in the context of sensitivity and risk and considered in the selection of a level of protection.

SETTLEMENT

Settlement is identified as a separate increment added to the levee for that purpose. For more information on settlement analysis and allowances see EM 1110-2-1913, paragraphs 6-1b, 6-9, 6-10, 7-3e, and 8-10c.

FREEBOARD DESIGN

Freeboard design should be refined as a study progresses and not left to the last design phase such as PED. The amount of effort and corresponding refinement for a given phase is a function of the importance and cost of freeboard relative to the overall plan. For an early reconnaissance phase it will generally be satisfactory to use quickly estimated freeboard values. As the study progresses, these early estimated values will be replaced by values arrived at by a careful design process. Freeboard design for a main stem levee conceptually proceeds in five typical steps as follows:

1. Normal Rating. (P.D.F., Elevation A and Profile A on the Plate)

   - Estimate project design flood (P.D.F.) water surface profile using reasonable losses, debris, etc. Those factors listed in **FLOOD PROFILE** paragraph above that are significant and can be quantified are included.

2. Initial Overtopping Section

   - Select location where initial overtopping would be least hazardous.

   - Usually located at downstream end of a long levee or where overtopping would be least hazardous.

3. High Rating. (Elevation B on the Plate)

   - Estimate project design flow water surface elevation using conservative (but not extreme) losses, debris, etc.

- This elevation (along with allowance for waves, extending maintenance interval, and small or intractable factors discussed in the Design Objectives. paragraph) sets minimum levee grade in the initial overtopping reach. (Elevation B on the Plate)

4. High Flow (Flow C and Profile C on the Plate)

   - Estimate flow capacity at top of minimum levee grade (Elevation B) using reasonable losses. (Flow C)

   - Estimate water surface profile upstream of initial overtopping reach for high flow and reasonable losses. (Profile C)

5. Overtopping Rising Flood Water Profile Slope (Profile D on the Plate)

   - Estimate water surface profile slope for the rising side of an overtopping flood. A flood larger than Flow C, when it overtops the levee, will have a slope steeper than Profile C.

   - Add an increment to upstream end of levee to insure levee initially overtops at least hazardous location; downstream location.

   - Between the levee low point (Elevation B) and the high point at the upstream end (Profile D), adjust the levee grade so that the length of levee subject to overtopping will gradually increase as flood stages continue to rise.

   - Consider adjustments in levee grade for superiority in grade at critical locations.

   - Consider relative rates of change of stage and flow rate. Considering potential consequences from an overtopping flood, establish final top of levee or floodwall design (Profile D).

FLANK LEVEE DESIGN

Flank levees complete the line of protection by connecting the ends of a main stem levee to high ground. Parallel flank levees extend along both sides of a tributary which joins the main stem river between two levee units. Chain failure of two or more adjacent levee units have occurred when the first levee unit was overtopped along the upstream flank, resulting in filling of the first leveed area, overtopping of the downstream flank, and erosion of the next adjacent levee downstream, etc. When levee units are in series, special provisions, including but not limited to an additional freeboard allowance, will be designed into the flank levees to prevent chain failure. When the height of a flank levee is influenced by tributary flows, the flank levee design will follow the same five steps as outlined for the main stem levee, except for any additional provision to prevent chain failure.

APPLICATION CONCERNS

Additional thoughts are appropriate in the application to all of the special formulation and topographic issues of levees and floodwalls.

1. Overtopping Design.

a. Limit damages to the levee and property by providing sufficient overtopping depth and time for adequate filling of the interior. Include damages from velocity (destruction) not just from inundation.

b. Minimize hazards to people by restricting the rate of rise and velocity in the interior.

c. Consider time and cost required to repair the levee.

d. For long levees consider multiple initial overflow sections.

e. Initial overtopping should occur at the least hazardous location.

f. Never allow sudden overtopping of a long reach of levee; design so that the length of levee being overtopped gradually increases.

2. Freeboard Design Miscellanea.

a. Levee freeboard should extend to high ground.

b. Freeboard should not have gaps, except when there are no practical alternatives, or when little damage would result from flow through the gaps.

c. If sandbag closures are necessary, provide permanent monuments to mark the location and height.

d. Levees consisting of all or mostly freeboard are strongly discouraged.

SUMMARY

Freeboard is an integral component of a levee. Freeboard design, assessment of uncertainty factors, and overtopping analysis, is a creative process not a cookbook exercise. To the veteran engineer this means the exercise of sound engineering judgment and experience. The freeboard design process results in a top of levee that, insofar as possible, provides a safe and functional project.

BACKGROUND GUIDANCE, CORPS PUBLICATIONS

1. Civil Works Engineer Bulletin 54-14, 23 April 1954,
   Improvements in Design and Construction Practices in Civil Works.

"Experience gained from actual floods and hydraulic model studies within recent years have shown conclusively that freeboard allowances for levee floodwalls should be "designed" with as much care as any other element of major projects, and that the allowance should be varied to conform with requirements at various points along a protection line."

2. MFR, 15 Jan 1962
   Quantitative Estimates of Wave - Overtopping of Levees and Flood Walls.

3. EM 1110-2-1601, 1 July 1991,
   Hydraulic Design of Flood Control Channel, Para 2-6.a.

4. EM 1110-2-1913, 31 March 1978
   Design and Construction of Levees, Para 6-1b, pg 6-1, Para 6-9, pg 6-8, Para 7-3e, pg 7-7, Para 8-10c, pg 8-13

5. ER 1110-2-1405, 30 September 1982
   Hydraulic Design of Local Flood Protection Projects, Para 6-h, pg 4

6. ETL 1110-2-299, 22 August 1986
   Overtopping of Flood Control Levees and Floodwalls

7. EP 1165-2-1, 15 February 1989
   Digest of Water Resources Policies and Authorities, Para 13-5, pg 13-4

8. EM 1110-2-2502, 29 September 1989
   Retaining and Flood Walls, Para 7-2, pg 7-1

9. ER 1105-2-100, 28 December 1990, Para 4-11c, pg 4-22, Para 6-166b, pg 6-147 (one-half of benefits for freeboard)

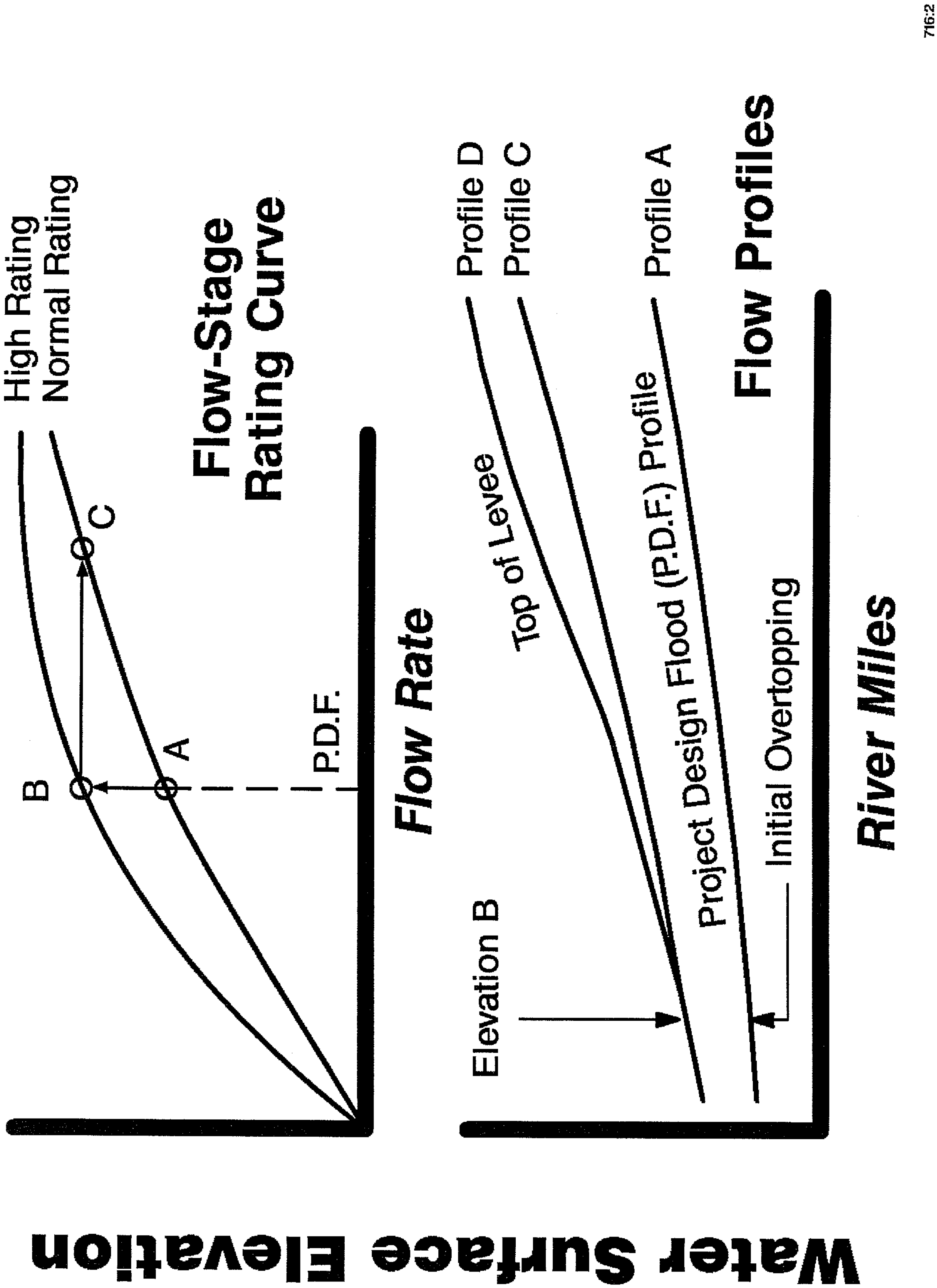
BACKGROUND GUIDANCE, TEXT BOOKS

1. River Engineering, Margaret S. Peterson, Prentice-Hall 1986
   Page 426, Embankment Design

2. Open Channel Hydraulics, Ven Te Chow, McGraw-Hill 1959
   Page 159, Para 7-5, Freeboard

3. Design of Small Canal Structures, USBR, 1974
   Page 14, para 1-20, Freeboard
   Page 28, para 2-10e, Canal Freeboard and Erosion Protection
   Page 102, Figure 2-33, Freeboard in Stilling Pool
   Page 105, para 2-34, Design Considerations

   USBR requires 1 to 3 feet of freeboard for 300 to 16,000 cfs.
   Controlled ideal X-section
   Controlled construction with uniform losses
   Failure not catastrophic



Water Surface Elevation
High Rating
Normal Rating
B
C
A
P.D.F.
Flow Rate
Flow-Stage
Rating Curve
Elevation B
Project Design Flood (P.D.F.) Profile
Top of Levee
Initial Overtopping
Profile D
Profile C
Profile A
River Miles
Flow Profiles
716:2

Plate

LEVEE FREEBOARD
DEJA VU ALL OVER AGAIN

by

Brad Fowler[1]

presented by

Robert M. Daniel[2]

1. Introduction. It's been now nearly ten years since the last great debate on freeboard. Then it was mostly planners and economists who were dissatisfied. The discussions, which included hydrologists and hydraulic engineers, resulted in planning guidance, first EP 1105-2-45 in about 1982, and now ER 1105-2-100, informally known as the Planning Guidance Notebook. The guidance, which I am told was agreed to despite the better instincts of at least some H&H folks, says for economic evaluation purposes take half the benefits in the freeboard zone.

a. Like the great Missouri Compromise, which attempted to resolve the conflict over slavery by admitting Missouri to the Union as a slave state and Maine as a free state, this second Missouri (now pronounced misery) compromise is not going to stick. It's not going to stick because it doesn't solve anything either. I think I'm safe in saying that today hardly anyone is satisfied with freeboard. To be more precise, hardly anyone is satisfied with the way freeboard is treated in analysis.

b. I use the neutral term "analysis" to sidestep (some may think, evade) associations with the planning function in the COE. The freeboard "problem", however, is now so general that it is central to not only benefit evaluation for new project planning and levee/floodwall raises, but is becoming so for the PL 99 program and the dam safety program. It is likely to soon become a factor in the newly restarted major rehabilitation program. Traditional freeboard analysis is already causing confusion for levee reconstruction projects where there is concern with performance or reliability (geotech uncertainty), and has long been at least a minor irritant in COE/local sponsor/FEMA relations as far as designating 100-yr protection floodplains is concerned.

c. I think it is time, now, to do something about freeboard analysis. The thing to do now is not simply conduct more research to increase knowledge about the uncertainties giving rise to the need for freeboard in the first place. Such research is of course needed and should be carried out, but it does not solve any immediate problems.

---

[1]Economist, Economic and Social Analysis Branch, Policy and Planning Division, HQUSACE (CECW-PD), Washington, DC

[2]Chief, Economic and Social Analysis Branch, Policy and Planning Division, HQUSACE (CECW-PD), Washington, DC

d. I will propose an interim solution that will restore consistency to analyses involving freeboard. Consistency existed before the negotiated compromise of the early 80's. The analytical procedures were wrong, but at least they did not lead to the kinds of internal inconsistencies and other analytical difficulties we are experiencing today. The proposed solution is almost certainly not precisely correct. I believe it is approximately correct; I think it can be demonstrated to be so (by others, however, not me). Approximate correctness and consistency is a vast improvement over both the current situation and that prevailing before. But first I want to present a little history and a few examples of the kinds of analytical difficulties we are now experiencing.

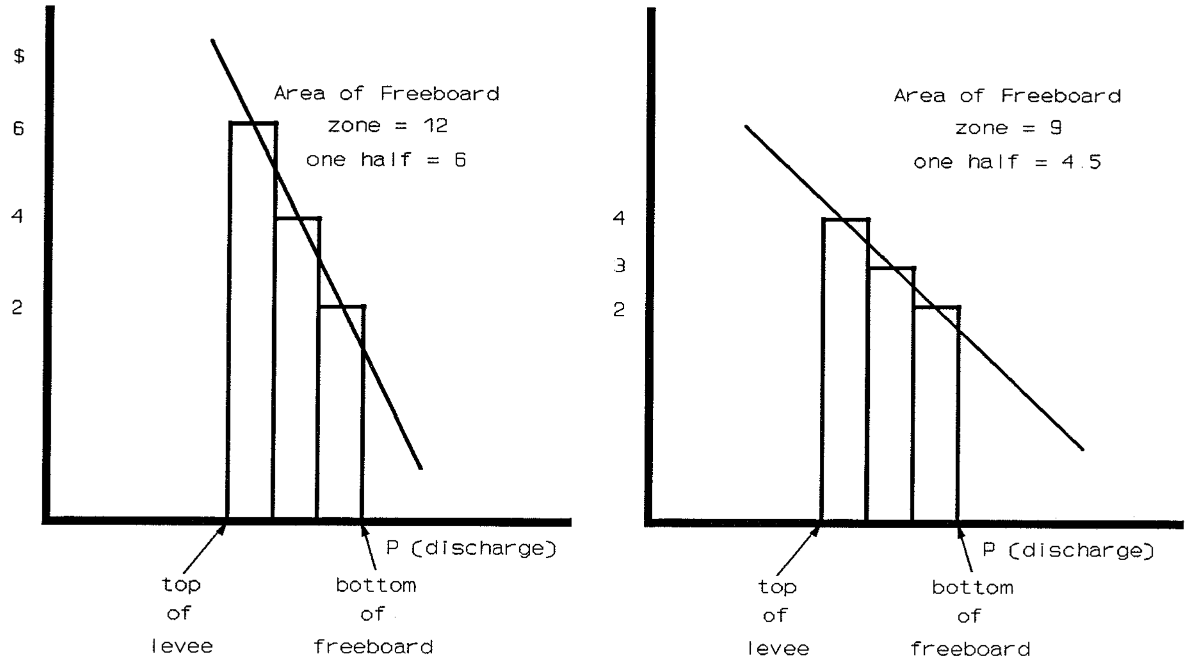
2. 1984 - The Second Time Around. The proximate cause for the COE having any planning guidance for freeboard was a desire to enhance a specific project's feasibility. What often is true for guidance driven by the need to justify a given project is: (1) The guidance isn't very good; and (2) it's adopted as general guidance anyway. This is exactly what happened for planning's freeboard analysis guidance.

a. What is good and not good guidance lies sometimes mostly in the eye of the beholder. The fact that the guidance makes little analytical sense may be irrelevant. The main thing is that the guidance be useful. The COE's freeboard guidance has been useful. It is easy to understand and implement, and the absence of better procedures has not until recently caused insurmountable problems.

b. The reason the existing guidance doesn't make a lot of sense is that the only consistent hydraulic assumption I can think of which can result in claiming one-half of the benefits in the freeboard zone is that all design exceedance events are equally likely to overtop the levee. Attempts to impose the reasonable expectation that the bigger the discharge event the greater the likelihood of overtopping leads to the nonsensical conclusion that the overtopping likelihood depends on the shape of the probability-damages curve. To see this consider the diagrams below. Before any H&H types snicker at planning guidance, remember that the economic conclusion is reasonable; it's the implied hydraulic behavior that doesn't make much sense. Also, H&H agreed to it. Please see following page.

c. In other words our guidance to date says nothing illuminating about the nature of the real problem. The real problem is not benefit evaluation per se, but is to somehow characterize the probability of overtopping events. Failure to reasonably characterize these probabilities leads to problems in benefit evaluation.

d. It was this realization, and the thought that some progress could be made using a purely statistical/probabilistic or a combination statistical/probabilistic and policy approach, that lead in 1984 to the writing of a paper titled, appropriately enough, "Levee Freeboard." The 1984 paper was perhaps regarded as NewSpeak by some, and in any case went nowhere except into my files. I'll summarize the arguments and incorporate them as an modest proposal in a later section.



P(·) = Probability of not overtopping given discharge

P(2)(2) + P(4)(4) + P(6)(6)	P(2)(2) + P(3)(3) + P(4)(4)
= (1)(2) + ½(4) + ⅓(6)	= (1)(2) + (½)(3) + ¼(4)
= 2 + 2 + 2	= 2 + 1.5 + 1
= 6	= 4.5

The probabilities of discharge are ignored as they just complicate the example. Thus the probability of overtopping given discharge depends on the shape and position of the damages curve.

3. <u>Problems, We Get Problems</u>. The absence of a consistent and reasonably accurate analytical approach to the question of the probability of overtopping leads to some disconnects in both benefit and cost evaluation. It also leads to the failure to even begin thinking about how one would handle situations in which both levee/floodwall reliability concerns (geotech or structural uncertainty) and freeboard concerns (hydraulic uncertainly) exist.

a. I've described the hydraulic problem as one of determining the probability of overtopping because that description puts a fine edge on it. The problem is really more general however: it is what is the total probability of the water surface exceeding a given elevation? It is partly because the freeboard zone was so large in the past, so that there was a fairly high degree of confidence that the design discharge would not overtop, that this question could be avoided. I don't mean to try and lay the whole fault at the feet of the H&H folks however. It was also because the COE as an organization has in the past been "casual" about what counted as a project cost and how those costs were estimated that this question could be avoided.

b. Benefit evaluation. Current freeboard analysis almost certainly underestimates benefits for new projects and overestimates benefits for levee/floodwall raise projects. Taking one half the benefits of the freeboard zone must be recognized as incorrect as soon as it is granted that the probability of overtopping is positively related to the discharge.

c. Cost Evaluation. Except when overtopping would be very rare, the cost of project rehabilitation after overtopping must be included as a project cost. For new projects, raises and major rehabs we would now include such costs. This was is not necessarily true in the past and is not now true for the PL 99 program. There is considerable doubt that much PL 99 work is economically justified because these costs are omitted. To include these costs the probability of overtopping must be estimated. If the top of levee profile is used in making the estimate of rehabilitation costs then it must also be used for benefit evaluation. But such an approach denies the reason for freeboard in the first place, hydraulic uncertainty.

d. Levee Reliability. Geotechnical uncertainty, leading to the possibility of levee non performance prior to overtopping, is another area in which it is critical to be able to better describe water surface elevations in probabilistic terms. If a levee is thought to be at risk of failure when the water surface reaches a certain elevation on it, it is not possible to accurately appraise the benefits of the existing levee unless an additional arbitrary assumption about freeboard is made. It is also not possible to estimate without project condition costs as these depend on a prediction of when the levee is likely to fail. This case is parallel to that of overtopping, except that the critical point is not the top of the levee but some point lower on it.

If a levee is thought to be potentially unstable over a range of water surface elevations, there are two distinct physical phenomena subject to risk, the actual water surface elevation given discharge (hydraulic uncertainty), and the integrity of the structure given the water surface elevation (geotechnical uncertainty). This is the case that would be generally expected and it requires use of joint probability distributions.

e. Dam Safety. Although not a levee freeboard consideration, freeboard does enter into dam deficiency analyses. Freeboard is added to account for some of the same uncertain factors that give rise to the desire for levee freeboard: waves, wind setup, etc. The principle difference is that for questions of dam safety freeboard is added after a whole array of so called conservative assumptions are built into the analysis. Dam safety design practice makes even current levee design analyses appear to be the cutting edge of sophisticated assessments. Dam safety investigations can benefit immensely from even modest to moderate adoption of an alternative analysis framework.

4. A Modest Proposal. So far I've described the levee freeboard issue in narrow terms, by posing it as a question of estimating the probability of levee overtopping, of determining the total probability of a water surface elevation or of estimating the joint probability of water surface elevations and levee failure. I've listed a number of benefit and cost estimation problems, and suggested that an alternative analysis framework can be helpful. That alternative framework uses as one tool the probability analyses discussed above. The framework is called a risk analysis framework. My modest proposal is:

(1) Adopt a risk analysis framework for levee design in feasibility studies.

(2) Do it NOW. Right now, at this conference.

(3) Issue guidance implementing the risk analysis framework within sixty (60) days of the close of this conference.

b. I've been told recently that the levee freeboard problem has been solved, or will be soon, by other participants in this conference, and that the solution is adoption of a risk analysis framework. I believe this, and it is truly wonderful and miraculous: therefore there should be little difficulty in issuing guidance in sixty days.

c. How To Do It Now While Acknowledging There May Be A Small Risk My Belief Above Is Ill Founded. The 1984 paper suggested this approach to a risk framework for levee freeboard:

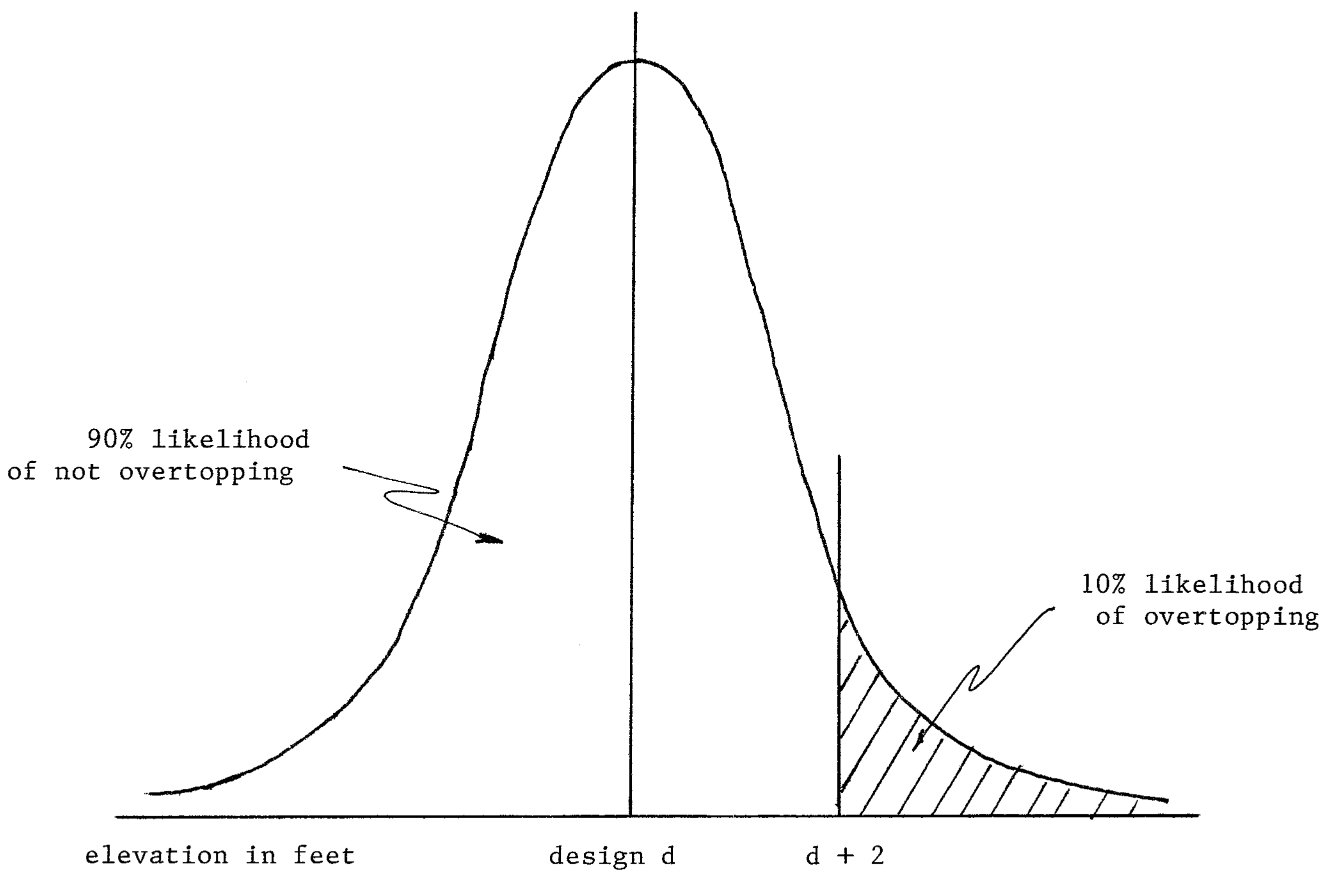
(1) Treatment of the "uncertain" factors [that is those factors which are not explicitly accounted for in the hydraulic modelling or which may be subject to variation] in a very general probabilistic sense [this means they are all handled at once in a summary fashion].

(2) Interpretation of the COE standards for freeboard allowance as design accommodations which reduce the likelihood of overtopping, due to the uncertain factors, to acceptable levels.

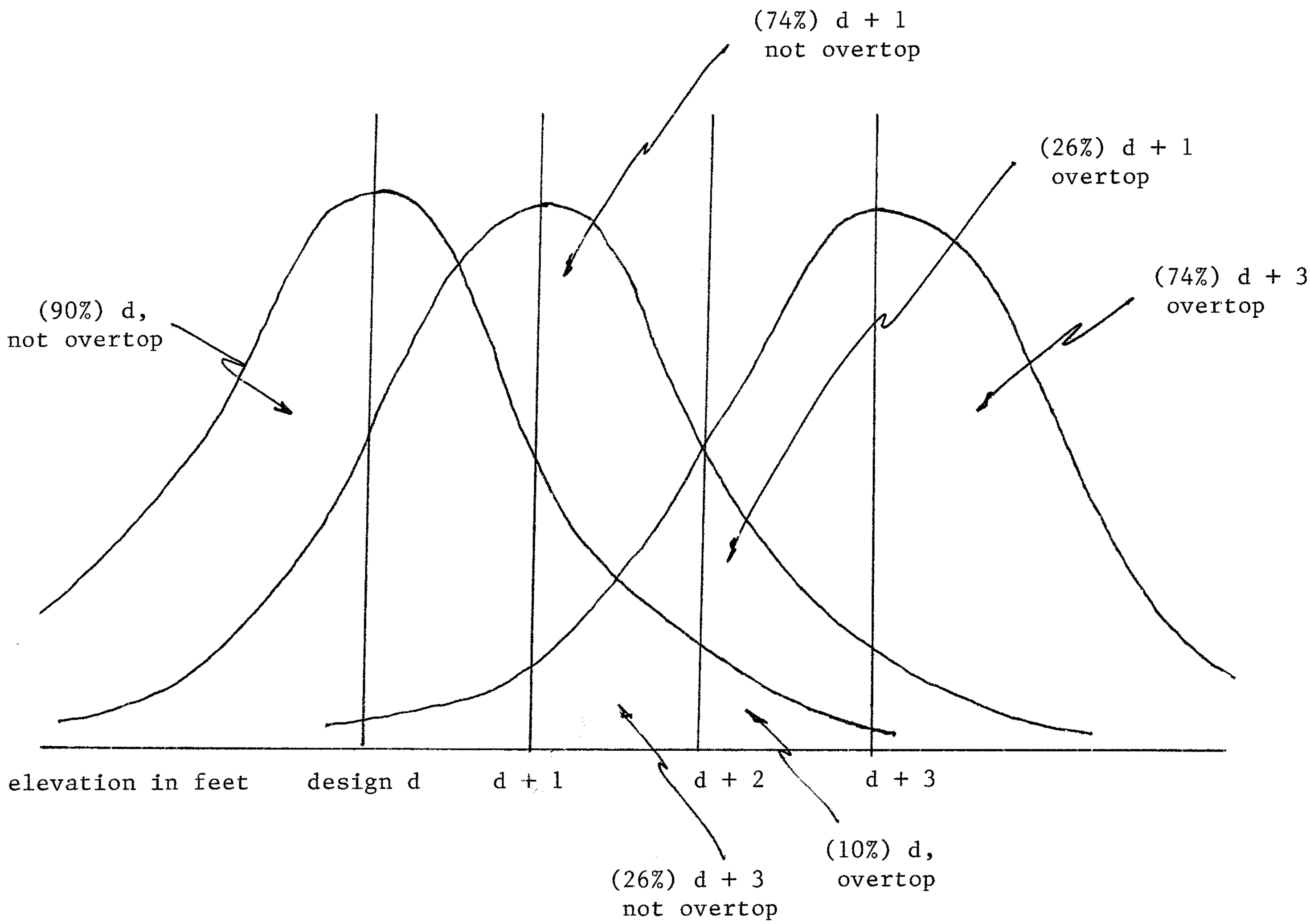
d. Only two things need to be done to make this general approach into an operational risk framework for levee freeboard analysis.

(1) Pick a probability distribution (or distributions) to represent the variation in water surface elevation about its expected (mean) value. Good candidates may be the normal or log normal distributions.

(2) Define "acceptable levels." The purpose of defining acceptable levels is to establish the variance of the distribution of the water surface elevation about its mean value. If, for example, it can be agreed for a given project that nine out of ten design discharges are expected to be contained within two feet of freeboard, sufficient information has been developed to establish a probability density function for overtopping, as below.



e. If this can be extended to other discharges it will be possible to estimate the benefits and costs of the freeboard region, and in addition use this information to modify freeboard recommendations. See below.



5. Giant Steps. Adoption of such a simple procedure as this will enable giant steps in analysis improvement for all of the problems mentioned above. In addition, it will get Corps planners and H&H folks used to dealing with a simple risk framework, which experience will be beneficial when they have the real risk analysis framework soon to be provided by the other participants in this conference.

FEMA LEVEE ACCREDITATION PROCEDURES

Prepared by:

Colleen S. Guthrie[1], Robert C. Henchbarger, P.E.[1], Ernest D. Lepore, P.E.[1], and Karl F. Mohr[2]

Presented by:

John L. Matticks[3]

INTRODUCTION

Although the Federal Emergency Management Agency (FEMA) levee policy is based, in part, on U.S. Army Corps of Engineers (COE) design criteria, many perceive FEMA requirements to differ from those of the COE. This paper will describe the evolution of FEMA's levee policy, outline the current procedures for evaluating and recognizing the protection that levees provide, address the differences between the purposes for which FEMA and the COE evaluate the protection provided by levees, and describe the solutions that FEMA has developed, and is continuing to develop, to address the problems associated with the evaluation of levees.

EVOLUTION OF THE LEVEE POLICY

Before the National Flood Insurance Program (NFIP) was established, the national response to flood disasters was generally limited to constructing flood-control works such as dams, levees, and seawalls, and providing disaster relief to flood victims. Building techniques to reduce flood damage and management of development in floodprone areas were often given secondary consideration. In addition, many flood-protection systems actually compounded the risk of future flood disaster by encouraging development in floodprone areas protected by those systems. The intent of the NFIP was to mitigate future flood damage through non-structural measures, encouraging floodplain management regulation that would guide development away from floodprone areas and establish effective standards for the development that continued to occur in the floodplain.

[1]Civil and Water Resources Division, Michael Baker, Jr., Inc., Alexandria, Virginia

[2]Risk Studies Division, Office of Risk Assessment, Federal Insurance Administration, Federal Emergency Management Agency, Washington, D.C.

[3]Assistant Administrator, Office of Risk Assessment, Federal Insurance Administration, Federal Emergency Management Agency, Washington, D.C.

Because the NFIP emphasizes non-structural measures, no formal guidance for evaluating and accepting existing flood-control works, such as levees, or flood-fighting systems was provided when FEMA began the initial identification and mapping effort. When the initial Flood Hazard Boundary Maps were prepared for the approximately 20,000 communities that were identified as having some areas of special flood hazard, the protection provided by a particular structure or system was assessed based on available floodplain studies and on evaluations performed by other Federal agencies, such as the COE. FEMA depended heavily on the judgment of the contractor that prepared the map. In most cases, due to time and cost constraints, a levee having crown elevations that exceeded the estimated 100-year flood levels was credited with providing protection from the 100-year flood. Little or no consideration was given to freeboard, stability, or maintenance.

After a significant number of Flood Insurance Studies had been performed, the need for a formal levee policy became more apparent. Study contractors began to request guidance for the evaluation of levees, and developers began requesting that FEMA outline the standards that levees must meet in order to be recognized as providing protection from the base (100-year) flood. In addition, levees were being constructed for the sole purpose of removing flood insurance requirements. By recognizing levees as providing protection from the 100-year flood simply because their crown elevations exceeded the 100-year flood levels, FEMA may have been encouraging the construction of levees to no more than this minimum standard, contradicting the original intent of the NFIP.

FEMA identified the following concerns, which demonstrated the need for a formal levee policy:

1) Levees built with crown elevations at the computed 100-year flood elevation may not provide protection from an actual 100-year event because of the uncertainty involved in establishing 100-year flood elevations, changing hydrologic conditions, and the possibility of structural failure before overtopping.

2) The degree of protection provided by a levee designed to protect against the 100-year flood is less than that obtained by elevating individual buildings to the 100-year flood elevation because of the possibility of levee failure during smaller floods and the greater depths of flooding experienced in unelevated structures upon levee overtopping or failure.

3) Crediting a levee system with protection against the 100-year flood would remove essentially all floodplain management requirements, lender notification requirements, and insurance purchase requirements within the leveed area; therefore, the accuracy of the levee evaluation is of utmost importance.

4) Without a formal policy, FEMA had no instrument by which to ensure the consistent evaluation of the design, operation, and maintenance of levees nationwide.

To address these concerns, FEMA developed an interim levee policy and contracted with the National Academy of Sciences, National Research Council, to make recommendations for a comprehensive policy to be used in evaluating levees for the purposes of the NFIP.

Interim Levee Policy. The interim levee policy, issued on February 10, 1981, outlined criteria to be used to evaluate levee systems in all ongoing and future analyses of flood hazards conducted by FEMA. These criteria included freeboard requirements, to allow for all of the uncertainties in analysis, design, and construction; guidelines for field evaluation of levee stability and maintenance; and requirements that levee operation not depend on human intervention (e.g., sand-bagging, flood-fighting). The interim policy did not provide specific criteria for the evaluation of design plans and "as-built" certifications for proposed or newly constructed levees. Such situations were to be handled on a case-by-case basis.

National Academy of Sciences Study. Under contract with FEMA, the Commission on Engineering and Technical Systems established a committee of recognized experts with experience in hydrologic, hydraulic, and geotechnical engineering; levee and dam design; construction and operation of flood-control works; floodplain management; and law. The objectives of the committee, which included liaison representatives from FEMA, the COE, the Soil Conservation Service, and the U.S. Water Resources Council, were to provide authoritative recommendations for the treatment of levees under the NFIP in the following areas:

1) Minimum design standards, including level of protection and freeboard requirements

2) Nature and extent of the inspection and evaluation to be performed to ensure conformance with minimum design standards

3) Requirements to be placed on communities who receive credit for protection afforded by levees

4) Assessment of risk in areas protected by levees, for use in establishing flood insurance requirements

5) Portraying risks associated with flooding in leveed areas on study and map products

The resulting report, entitled "A Levee Policy for the National Flood Insurance Program" (National Academy of Sciences, 1982), presented the committee's recommendations. The recommendations that FEMA chose to adopt included the following:

1) Levees should be recognized for the purpose of reducing insurance rates where they provide protection against 100-year or larger floods and where they meet specified structural design criteria, including freeboard requirements. In addition, FEMA should confirm its interim policy that does not recognize human intervention, such as sandbagging, as augmenting a levee's design level of protection.

2) For the purposes of levee inspection and evaluation, FEMA should pursue the possibility of using the services of Federal or State agencies having water resources experience.

3) A specific operation and maintenance plan should be formally adopted by the levee owner in order for the levee to be recognized by the NFIP.

4) FEMA should require the elevation of new structures in all areas protected by levees unable to contain the 100-year flood.

5) The location of all levees, dikes, or floodwalls credited as providing 100-year protection or more should be clearly denoted on all future FIRMs.

Formal Levee Policy. After evaluating the committee's recommendations and COE engineering and design manuals, a formal policy for evaluating levees was developed. In accordance with the standard procedure, the policy was published in the Federal Register as a proposed rule for comment by interest parties. All comments received were addressed, and the policy was published in the Code of Federal Regulations at 44 CFR, Chapter I, Part 65, Section 65.10. Section 65.10, which became effective on October 1, 1986, outlines levee design criteria, operation and maintenance requirements, and certification requirements. According to the NFIP regulations, FEMA will only reflect the existence of levees on an NFIP map upon receipt of one of the following:

1) Technical data indicating that the freeboard, structural stability, internal drainage, operation, and maintenance requirements, as detailed in Section 65.10 of the NFIP regulations, have been met.

2) Certification from a Federal agency, such as the COE, that the existing levee system or improved levee has been adequately designed and constructed to provide protection from the 100-year event and data demonstrating that the internal drainage, operation, and maintenance requirements have been met.

To satisfy the requirements of Section 65.10 of the NFIP regulations, riverine levees that are not certified by a Federal agency must provide a minimum freeboard of 3 feet above the water-surface level of the base flood. An additional 1 foot above the minimum is required within 100 feet on either side of structures (such as bridges) riverward of the levee or wherever the flow is constricted. An additional 0.5 foot above the minimum at the upstream end of the levee, tapering to not less than the minimum at the downstream end of the levee, is also required. For coastal levees, the freeboard must be established at 1 foot above the height of the 1-percent wave or the maximum wave runup associated with the 100-year stillwater surge elevation at the site, whichever is greater.

COMPARISON OF FEMA AND COE LEVEE EVALUATIONS

Similarities. As noted above, the FEMA levee policy was based, in part, on COE criteria. In fact, the freeboard requirements for riverine levees, as presented in Section 65.10 of the NFIP regulations, were taken from Subsection 6-1.b., Crown Elevation, of the COE engineering manual entitled "Design and Construction of Levees" (U.S. Department of the Army, 1978). That manual is referenced in the portion of Section 65.10 that concerns embankment and foundation stability. In addition, the portion of Section 65.10 that concerns settlement references a second COE engineering manual entitled "Soil Mechanics Design--Settlement Analysis" (U.S. Department of the Army, 1990). The fact that the FEMA levee policy uses established COE criteria and references COE manuals indicates that FEMA levee evaluation criteria are compatible with COE design criteria.

Differences. Despite the similarities in the COE and FEMA levee criteria, many perceive those criteria to differ as the result of situations in which FEMA has not recognized an existing levee system as providing protection from the 100-year flood, and yet the COE is unable to identify a levee-upgrading project to be economically feasible. This situation can cause confusion to local sponsors of flood-protection works and the general public because property owners behind existing levees are required to purchase flood insurance while additional flood protection is deemed economically unjustifiable by the COE. Although this confusion is real, it is not caused by differences in the levee criteria of the two agencies but rather by differences in their missions.

One of the NFIP's major objectives is to identify and map flood hazards throughout the United States for flood insurance and floodplain management purposes. This identification and mapping effort is based on a uniform 100-year standard, which was adopted for the NFIP after various alternatives were considered. The 100-year standard constitutes a reasonable compromise between the need for building restrictions to minimize loss and the economic benefits to be derived from floodplain development. As a result of identifying areas subject to 100-year flood hazards, FEMA makes flood insurance available to individual property owners and floodplain management information available to community officials.

While the floodplain management standards of the NFIP are directed primarily at new development, the COE is oriented toward the design and construction of economically justifiable civil works projects to protect existing development. When evaluating an existing levee system for possible upgrading and estimating the potential for flood damage, the COE must consider the flood-protection levels afforded by the existing levee. Those evaluations are not tied to any specific frequency event.

When the projected flood damage is annualized and compared with the estimated project cost annualized over the life of the project, the COE will often find that the benefits that would be realized by constructing the project do not outweigh the costs, and the project cannot be justified economically. Estimates of annualized flood damage are extremely sensitive to the damage figures obtained from flood events more frequent than the 100-year event. If an existing levee system prevents damage by more frequent events, such as the 25- or 50-year flood, the annualized flood damage is greatly reduced; therefore, an improvement project will, in all likelihood, not be considered economically justifiable. In addition, an improvement project that would increase a levee's level of protection so that it would prevent damage by these more frequent events, but would not provide protection from the 100-year event, may be considered economically justifiable.

Because of the sense of security that a levee can create among the owners and potential purchasers of property, development often occurs in the area the levee is intended to protect. As a result, a large number of homes may be at risk if the levee fails. Also, the flooding that results from levee failure is more like the instantaneous flooding from a dambreak than the flooding caused by a gradual rise of floodwaters in an unprotected floodplain area. FEMA therefore considers the flooding from a failed levee to be potentially catastrophic and, for the purposes of the NFIP, generally assumes that all levees that provide less than 100-year protection cannot be considered in the assessment of flood risk.

Although the COE may determine, based on the factors described above, that a levee-upgrading project is not economically justifiable, that determination does not necessarily indicate that the project provides protection from the 100-year flood. In fact, as noted previously, if the COE, or any Federal agency, certifies that an existing levee system has been adequately designed and constructed to provide protection from the 100-year event, FEMA will credit that system with providing protection on the appropriate NFIP maps. It is because FEMA recognizes the levee evaluation experience and knowledge of local conditions of other Federal agencies, including the COE, that this provision was included in Section 65.10 of the NFIP regulations.

While it is the missions of the two agencies that differ and not the levee criteria, the general public does seem to receive contradictory messages. FEMA has taken action to assuage the frustration that communities and property owners may experience as the result of the contradiction they perceive.

Assistance to Communities Protected by Uncertified Levees. One way that FEMA assists communities protected by levees that provide a certain degree of protection but less than that required for recognition on NFIP maps is through the Community Rating System (CRS). Although FEMA does not recognize levees as providing partial protection on NFIP maps, the CRS enables a community to earn credit for levees that provide less than 100-year protection. FEMA created the CRS to provide a new incentive for communities to pursue activities that reduce flood losses and support the sale of flood insurance. Any community participating in the NFIP may apply for CRS classification in order to obtain flood insurance premium rate credits for its residents. Levee Safety is one of 18 activities for which communities may earn points for CRS credit.

There are 10 community classes in the CRS, based on the total number of points earned by a particular community. To earn points, the community must demonstrate that it is implementing floodplain management and public information activities that exceed the minimum requirements of the NFIP. Residents of Class 1 communities receive the largest premium credit, a 45-percent reduction in flood insurance premiums for property subject to special flood hazard. Residents of Class 10 communities receive no premium credit. To be designated Class 1, the community must earn a total of 4,500 points. Communities that do not apply for CRS classification are designated Class 10.

Under the Levee Safety activity, communities earn points by maintaining and operating certain levees that provide protection less than that required for 100-year certification. Levees recognized under the CRS are those which provide protection from at least the 25-year event and which satisfy all of the requirements presented in Section 65.10 of the NFIP regulations, with the exception of the requirement that the crown of the levee be at least 3 feet above the elevation of the 100-year flood. To receive credit points, communities must also have developed emergency response plans to guide situations when the levees are threatened with overtopping or failure. The credit for this activity is not intended to encourage construction of new flood-control structures. Accordingly, the CRS only recognizes levees that were built before January 1, 1991. The maximum credit awarded under the Levee Safety activity is 900 points.

The number of CRS points earned for maintaining and operating a specific levee is calculated with consideration given to the each of the following factors:

1) The levee protection level (LPL) = 30 + the flood recurrence interval at the flood protection level, which is 3 feet below the lowest point of the crown.

2) The number of pre-FIRM buildings (built before the date of the FIRM) that the levee protects (bLP).

3) The number of buildings in the Special Flood Hazard Area (bSF).

4) Whether the levee is being operated and maintained in accordance with the community's floodplain management plan (pLP).

Therefore, a community earns CRS credit for a particular levee based on the specific characteristics of that levee. The formula used to compute the number of points awarded for maintaining and operating a levee system under the CRS is as follows:

$$LPL \times (bLP/bSF) \times pLP = \text{Points Awarded}$$

For example, consider a levee with an LPL of 105 (30 + 75), a bLP of 77, and a bSF of 150. If the community has no comprehensive floodplain management plan, the pLP is 1. Using the equation above the community would be awarded 54 points:

$$105 \times (77/150) \times 1 = 54$$

These 54 points would then be added to the points awarded to the community for other CRS activities to determine the total number of points and the corresponding class designation and reduction in flood insurance premiums. In a sense, this reduction in flood insurance rates to account for levees that protect against more frequent events is similar to the COE's practice of not considering annual damages for structures located behind certain levees. Additional information about the CRS is provided in the "National Flood Insurance Program Community Rating System Coordinator's Manual" (FEMA, 1990).

FEMA also assists communities through the provision promulgated at Section 61.12. Under this provision, adequate progress toward the construction of a flood-protection system involving Federal funds, such as a levee system designed by the COE, is recognized for flood insurance purposes. If the Administrator, Federal Insurance Administration, determines that a community has made adequate progress toward the construction of such a system, the insurance rates charged to owners of property located within the area intended to be protected directly by the system will be reduced. A special flood insurance rate zone has been developed for this purpose. Although the owners of property within these zones are required to purchase flood insurance until the flood-protection system is complete, the risk premium rates charged are those that would be applicable upon completion of the system.

The Federal agency involved with the project assists the community in receiving a determination of adequate progress by providing backup information. For the purposes of Section 61.12, adequate progress means that the community has provided documentation that the following steps have been completed.

1) 100 percent of the total financial project cost of the completed flood-protection system has been authorized.

2) 60 percent of the total financial project cost of the completed flood-protection system has been appropriated.

3) At least 50 percent of the total financial project cost of the completed flood-protection system has been expended.

4) All critical features of the flood-protection system, as identified by the Administrator, are under construction, and each critical feature is 50 percent completed as measured by the actual expenditure of the estimated construction budget funds.

5) The community has not been responsible for any delay in the completion of the system.

Through this provision and credit provided through CRS, FEMA assists communities that are not protected by levees from the 100-year flood but nonetheless have demonstrated a willingness to cooperate with FEMA to increase flood protection.

FUTURE FEMA ACTIVITIES REGARDING LEVEE RECOGNITION

<u>Levees No Longer Providing 100-Year Protection</u>. FEMA has encountered a number of situations in which local flood-control systems previously credited with providing 100-year flood protection could no longer be credited with providing that level of protection. Such situations arise for various reasons, as follows:

1) Changes in hydrologic and/or hydraulic conditions that lead to higher base flood elevations

2) Flooding events which affect the stability of the structure due to embankment erosion and/or sedimentation of river channels

3) Lack of proper operation and maintenance procedures

4) Improved hydrologic estimates of base flood levels due to the availability of additional years of stream gage and rainfall data

5) Improved hydrologic and/or hydraulic modeling techniques

FEMA is obligated to issue an NFIP map that accurately reflects the flood risk when it determines that a local flood-control system no longer provides protection from the 100-year flood. However, in fulfilling this obligation, FEMA must also determine how best to address the problems that may accompany such action. In particular, if a levee that protects a highly developed area is determined no longer to provide 100-year protection, the economic and psychological effects on the community and individual property owners could be significant.

FEMA's experience in the Sacramento, California, metropolitan area provides an example of the problems that must be addressed when, due to changes in hydrologic conditions, a levee no longer provides the 100-year flood protection with which it was credited. In 1986, the Sacramento area experienced extensive flooding; therefore, the U.S. Congress directed the COE to reassess the level of protection provided by the existing flood-control system. Because the COE study revealed that the system provided significantly less than 100-year protection, FEMA contracted the COE to perform the analyses necessary to support revisions to the NFIP maps.

The COE analyses indicated that areas previously designated as subject to no significant flood hazard should actually be identified as subject to 100-year flooding. Such a change in the assessment of flood risk to the area could have serious repercussions on real estate sales, development, and reconstruction and could result in an increase in flood insurance rates that would be devastating to many property owners. Any new structure or substantial improvement to an existing structure within an identified area of special flood hazard would have to be constructed so that the lowest floor of the structure (including basement) and the lowest grade adjacent to the structure are at or above the 100-year flood elevation, which could be as much as 15 feet above the existing ground in some locations.

In response to public outcry and to prevent serious economic disruption in the area, Congress passed Section 1086 of the McKinney Homeless Assistance Act of 1988, which delays regulatory action by FEMA for a period of 4 years to give communities in the Sacramento area an opportunity to improve existing flood-control systems. To reflect the results of the COE analyses on the NFIP maps for communities in the area while adhering to the intent of the McKinney Act, FEMA has identified those areas on the NFIP maps as it would identify areas for which the adequate progress criteria presented in Section 61.12 of the NFIP regulations had been met.

Although the communities in the Sacramento area have not improved the levee system to meet these criteria at this time, adequate progress must be demonstrated by the end of the 4-year period. If, at the end of this period, adequate progress toward the improvement of the flood-control system is demonstrated, the special zone designation will be retained on the NFIP maps, and flood hazard area designations will be removed from the maps as the flood-control structures are completed. However, if adequate progress has not been made, FEMA will initiate revisions to the NFIP maps to reflect current flood risk data, and insurance rates will be adjusted accordingly.

Based on its experience in Sacramento, FEMA has evaluated whether to adopt a policy of providing rectification periods to other communities in which levees are determined to provide less protection than that with which they are credited on NFIP maps. During such rectification periods, communities would be required to formulate detailed plans for and either complete or make adequate progress toward completing improvements that would restore their levee systems to a minimum 100-year design level of protection.

An area in which it has been determined that 100-year protection no longer exists is the Los Angeles, California, metropolitan area, specifically, the area along the Los Angeles River. Revisions to the NFIP maps for the communities in the Los Angeles area to reflect the decrease in flood protection revealed by a recent restudy could affect as many as 500,000 structures and 1,000,000 local residents. As in the Sacramento area, flood elevation requirements may increase as much as 15 feet in some places. Because of the extent of the area that would be affected and based on its experience in Sacramento, FEMA has recognized the possibility of congressional intervention.

Although FEMA provided a rectification period for Sacramento, as mandated by Congress, the agency has not determined that it is appropriate to take this action in all situations where levees are determined to provide less protection than previously estimated. Of primary concern is whether providing such rectification periods is consistent with FEMA's responsibility to identify existing flood hazards and encourage the mitigation of flood losses unless there is a virtual guarantee that the level of protection will be upgraded within a reasonable period of time (in accordance with Section 61.12).

If a policy of providing rectification periods is adopted by FEMA, the agency will be allowing construction of structures well below the 100-year flood elevations for extended periods of time. It is possible that proposed flood-protection structures to improve the level of flood protection will never be built or will take 10 or more years to complete. FEMA must also consider the risk it would be taking by allowing the purchase of flood insurance at a reduced rate or identifying an area as not requiring flood insurance. If the area should experience flooding during the rectification period, the NFIP could suffer financially because lower flood insurance premiums were paid, if premiums were paid at all. Other flood insurance policy holders and taxpayers throughout the country would be paying for the flood losses. Due to the complexity of the issues that surround the adoption of a policy that would provide rectification periods for the improvement of existing levee systems, FEMA has not adopted such a policy at this time.

<u>Future Levee Evaluation Criteria</u>. To best fulfill its obligation of providing communities that participate in the NFIP with maps which accurately identify the flood risk, FEMA must constantly search for new and improved ways of assessing that risk. The following suggestions for revision to the levee evaluation process may be worthy of consideration as the NFIP continues to evolve:

1) A specific alternative to the general freeboard requirement for non-Federal levees, apart from the existing provision for exceptions, could be incorporated into the levee policy. Such an alternative could allow for levees that may provide less freeboard but provide protection from a flood of a lesser frequency, such as the 200-year event. Revising the existing policy in this manner would provide an alternative to the freeboard requirement while still ensuring the adequacy of the levee to provide 100-year protection.

2) Section 65.10 of the NFIP regulations could be revised to incorporate a requirement that communities reevaluate existing hydrology as part of their levee maintenance activities. By adopting such a provision, FEMA could be assured of the adequacy of a levee without incurring additional costs. This requirement would also enable FEMA to identify changes in hydrology more efficiently so that NFIP maps would present the most up-to-date information.

These examples are only two areas that FEMA could investigate to improve its evaluation of the adequacy of levees. FEMA will continue to strive toward its goal of producing the most accurate maps in the most timely manner and, accordingly, will consider new procedures and policies that will facilitate the accomplishment of that goal.

Future Interagency Cooperation. One way that FEMA will be able to improve the accuracy of NFIP maps is by coordinating with other Federal agencies. In many cases, FEMA relies on data provided by other agencies, such as the COE, to update its maps. If Federal agencies can work together to identify a means of maximizing the usefulness of the studies produced while containing costs, both FEMA and the other agencies involved will benefit. For example, if Federal agencies could agree on a format of the initial study of the flood protection provided by a levee system that would satisfy the requirements of all of the agencies involved, FEMA could use that initial study to revise NFIP maps.

Such an arrangement would enable FEMA to reflect existing conditions on its maps in a more timely manner and better serve the communities that participate in the NFIP. In addition, other Federal agencies could eliminate the effort involved in reanalyzing the flood risk to satisfy FEMA requirements. Interagency cooperation and coordination as we near the end of this century will better prepare the parties involved to face the challenges that lie ahead.

REFERENCES

National Academy of Sciences, National Research Council, Commission on Engineering and Technical Systems, Water Technologies Board, Committee on a Levee Policy for the National Flood Insurance Program, "A Levee Policy for the National Flood Insurance Program," Washington, D.C., 1982.

U.S. Department of the Army, Corps of Engineers, "Design and Construction of Levees," Engineer Manual No. 1110-2-1913, Washington, D.C., March 31, 1978.

U.S. Department of the Army, Corps of Engineers, "Soil Mechanics Design--Settlement Analysis," Engineer Manual No. 1110-2-1904, Washington, D.C., January 1953, revised as Engineer Manual No. 1110-1-1904, September 30, 1990.

Federal Emergency Management Agency, Federal Insurance Administration, "National Flood Insurance Program Community Rating System Coordinator's Manual," FIA 15, Washington, D.C., October 1, 1990.

FREEBOARD REQUIREMENTS FOR LOW LEVEES

by

James Mazanec[1]

1. Introduction and Summary

Existing guidance in setting freeboard requirements for low levees where risk to life is minimal needs to be reconsidered. In situations where the levee heights are relatively small (less than three feet), and the uprush from wind and other hydraulic design factors are not expected to be excessive, consideration should be given to lower magnitude freeboard. This presentation is a brief overview of this issue from the North Central Division standpoint.

2. Background and Physical Setting

The Divisions and Districts do not set policy and establish general design guidance. However, it is anticipated that results and input at this conference will be used by HQUSACE to rethink and eventually enhance current guidance/policy on levee freeboard issues. In the current environment of limited dollar resources and in partnership with locals, the Corps needs to make sure that the design process is efficient and covers not only the large type projects but also small type projects. By small I mean design of low height structures not necessarily low cost projects or low gross benefit type projects.

3. Study Approach

a) Our primary goal is still to maintain reasonable engineering standards of project functionality and safety in design. Within the midwest, we have many watersheds with low stream gradients (approximately 5 feet per mile). These streams are generally not flashy in nature and due to existence of significant, and in many cases, regulated overbank natural corridors, flood hydrographs are significantly attenuated as the floodwave moves downstream.

b) Where communities are located adjacent to streams of this type, often the 1% (100 year) or .2% (500 year) overbank levels may only be a few feet in depth, yet significant damage can result. Further, more frequent nuisance flooding can be experienced annually. Many of these types of flooding situations can be effectively remedied with the construction of low levees.

[1] Hydraulic Engineer, North Central Division, U.S. Army Corps of Engineers

c) Current design requirements include engineering design leading to a base levee height and freeboard added with supplemental considerations for levee superiority. Typically levee freeboard methodologies have assumed compounding of uncertainty elements which in many instances leads to recommendations of freeboard on levees on the order of many feet. For low base levees this can result in levee freeboard components being recommended which are equal to or greater than the base levee height. In many instances allowing for high freeboard levels is prudent because of significant potential for instability of profile levels and/or safety concerns. This is especially the case when high velocities and/or substantial flood depths in the overbanks are encountered. However, in many other cases current analysis methods and minimum standards lead to significant freeboard additions, the merit of which is questionable.

4. Recommendations

a) Within NCD, we recommend that any new guidance by HQUSACE allow for accepting greater risk of overtopping for low levees where life threatening consequences can be shown to be small. This would result in more uncertainty in potential overtopping of these levees than we have in the past; however with appropriate caution and education of the sponsor and people being impacted, it can still result in projects with reasonable safety and effectiveness.

b) We recommend that for low levees where overbank flooding is less than 3 feet and velocities are low, that any future guidance on developing freeboard be based on limiting the compounding of analysis assumptions such that only one or two uncertainty factors are considered for establishing base freeboard. Thus, for watersheds exhibiting the right conditions, this could potentially lead to freeboard levels of as little as one foot. For such cases, levee superiority analyses would still be developed; however it could lead to a conclusion of no addition to force overtopping at specific locations first and thus, further reduce the freeboard requirements.

c) An example of this type of flooding is shown in these slides. The Corps (Chicago District) is currently evaluating low levels for many larger reaches of flooding on this river. The Corps may not be able to help these communities if in the final recommendation freeboard levels of 3 feet or greater are required.

5. Conclusion

In summary, we within NCD, would like HQUSACE to consider the potential benefits to be gained with relaxing freeboard requirements when dealing with low levees.

HYDROLOGY AND HYDRAULICS WORKSHOP
ON
RIVERINE LEVEE FREEBOARD

Levee Freeboard Policy Issues Impacting On Corps Support to the National Flood Insurance Program

by

Kenneth J. Zwickl[1]

GENERAL

The Federal Emergency Management Agency (FEMA) recognizes the Corps of Engineers' expertise in establishing freeboard levels for flood control projects, especially levees, and normally defers to us on decisions about adequate levels of freeboard. Needless to say, any changes in Corps procedures for establishing levee freeboard could have a pronounced effect on FEMA's administration of the National Flood Insurance Program (NFIP).

This paper discusses the role of the Corps of Engineers in support of FEMA and the National Flood Insurance Program. Included in the discussions are levee certification requirements of the NFIP and differences between the two agencies' policy on levee freeboard requirements.

SUPPORT TO FEMA FOR THE NATIONAL FLOOD INSURANCE PROGRAM

The Corps provides technical support to FEMA in support of the NFIP. Most of this support is fully reimbursable, and includes Flood Insurance Studies, the Community Assistance Program, and Limited Map Maintenance Program efforts. In addition to these reimbursable efforts, the Corps is often requested by FEMA to "certify" that existing levee systems can provide protection from the 100-year frequency flood for flood insurance rating purposes. Much of the Corps technical support to FEMA is managed, coordinated, and/or provided by the Corps Flood Plain Management Services (FPMS) Program.

FLOOD PLAIN MANAGEMENT SERVICES PROGRAM

The FPMS Program is the Corps means of using its technical expertise in flood plain management matters to help those outside the Corps, both Federal and non-Federal, to deal with floods and

---

[1]Civil Engineer, Policy and Planning Division, Headquarters, U.S. Army Corps of Engineers

flood plain related matters. The objective of the FPMS Program is to support comprehensive flood plain management planning with technical services and planning guidance at all appropriate governmental levels; and thereby, to encourage and to guide them toward prudent use of the Nation's flood plains for the benefit of the national economy and welfare.

The FPMS Program provides a full range of technical services and planning guidance on floods and flood plain issues within the broad umbrella of flood plain management. The FPMS Program provides indirect support to FEMA, by providing these services to state and local agencies and private persons for their administration of the NFIP.

Flood and flood plain related data are obtained or developed and interpreted, such as flood depth or stage, extent and duration of flooding, and flood frequency. This includes providing flood hazard determinations to individuals and agencies on NFIP-related matters, as well as determining flood protection levels for certification of levees for FEMA.

Planning assistance and guidance is provided for implementing or meeting requirements of flood plain regulations, flood warning and flood emergency preparedness, hurricane evacuation planning, flood proofing measures, permanent evacuation and relocation, the National Flood Insurance Program, and Executive Order 11988. This includes providing advice and assistance to communities for their participation in the NFIP's Community Rating System. The Community Rating System allows for flood insurance rate reductions based on a community's active involvement in reduction of flood losses through comprehensive flood plain management.

The FPMS Program also includes studies to improve methods and procedures for flood damage prevention and abatement, and preparation of guides and pamphlets on topics such as flood proofing, flood plain regulations, flood plain occupance, economics of flood plain regulations, and important natural flood plain values.

DIRECT SUPPORT TO FEMA FOR THE NFIP.

Direct support to FEMA for the NFIP falls into two broad categories: reimbursable technical analyses such as flood insurance studies, and special investigations, such as levee certifications.

Flood Insurance Studies and Limited Map Maintenance Program efforts are examples of reimbursable technical analyses typically performed for FEMA. These efforts normally require surveys, hydrologic and hydraulic analyses, flood hazard mapping, and an evaluation of existing flood control structures, including levees. The end product is a flood insurance rate map which depicts the flooding situation in the community and is used to set flood insurance rates for the community.

Another reimbursable effort performed by the Corps for FEMA is the Community Assistance Program effort. This program involves direct contact with communities to assist them in their administration of the requirements of the NFIP. Under this program, the Corps prepares and presents workshops on flood plain management related topics, including flood proofing, flood plain regulations, and coastal issues, and provides additional elevation reference marks for communities' flood insurance rate maps.

LEVEE CERTIFICATION FOR FEMA

FEMA, in setting flood insurance rates, recognizes only levees that are properly engineered, structurally sound, and have a reasonable expectation of protecting against the 100-year flood. When a Flood Insurance Study is being prepared by FEMA for a community, and a system of levees exists that impacts on flooding within that community, FEMA requires a certification that the levees provide protection to the 100-year flood or greater. FEMA often relies on the Corps to assist them in making these determinations regarding levee freeboard adequacy.

The levee certification is a determination that the design and maintenance of the levees in question are adequate to provide them with 100-year protection. The certification is not a warranty of performance, but merely the Corps current position concerning the levee system's design adequacy. The Corps position must also consider the amount of design levee freeboard. If this amount is less than FEMA's 3-feet requirement, a further determination must be made as to whether the levee system can protect against the 100-year flood with less than the FEMA 3-feet freeboard requirement.

Major considerations when performing a levee certification include the obvious factors such as levee freeboard, hydrologic and hydraulic conditions, and structural and geotechnical conditions. Social and political considerations are also a possibility when determining levee system viability. With the knowledge that certification of a levee system may either prevent or allow additional development in the flood plain fringe, the levee certification must be done cautiously and accurately so as to ensure that the community is correctly apprised of the level of protection of the levee system, as well as the risk of levee failure.

For levee systems designed and built by the Corps, the levee certification is normally provided with no charge to FEMA. For levee systems designed and built by other parties (state or local governments or private entities), the Corps normally does not provide a certification without gathering detailed information about the design, construction, and maintenance of the system. If a certification is to be provided, and FEMA is willing to reimburse the Corps, this effort can and should be undertaken.

LEVEE FREEBOARD - DIFFERENCES IN POLICY

Both Corps and FEMA levee freeboard requirements are intended to compensate for uncertainties inherent in the establishment of flood elevations, changing hydrologic and hydraulic conditions, and possible structural failure before the levee is overtopped. It is in the application of these requirements that the two agencies differ.

The Corps considers engineering design, structural stability and past perfomance in evaluating existing levee systems for determination of potential project benefits. Whether a 10-year or a 500-year design, the levee system is considered in benefit calculations provided that it meets the design, stablility and performance criteria. In other words, even if the levee system provides protection to a level less than the 100-year flood, the levees are considered to provide some protection and are included in the evaluation process.

As stated earlier, FEMA, in setting flood insurance rates, recognizes only levees that are properly engineered, structurally sound, and have a reasonable expectation of protecting against the 100-year flood. In cases where the levees are designed and constructed to protect against the 100-year flood but have less than three feet of freeboard, FEMA feels that their levee policy is flexible enough to address these area-specific circumstances. Their policy allows for an exception process by which the requirements could be relaxed if the community or a levee owner can provide documentation to demonstrate adequate protection from the 100-year flood with lesser freeboard. In addition, the minimum freeboard requirements can be waived if a Federal agency such as the Corps certifies that the levee has been designed, constructed and maintained to provide 100-year protection.

FEMA does not have provisions in its flood hazard analyses for recognizing levees that provide less than 100-year protection. In essence, the technical analyses which determine the flood elevations, extent of flooding and flood insurance rates, ignore the existence of the levees if they do not meet the minimum criteria for 100-year flood protection. FEMA has concluded that from an actuarial standpoint, recognition should not be given to these levees for flood insurance purposes.

In some cases, the existence of levees which do not meet FEMA's criteria, yet are considered by the Corps to provide at least some measure of flood protection, can result in significant differences in the two agencies' flood hazard estimates. All other parameters being equal, the Corps estimate for the extent and elevation of the 100-year flood may be considerably less than FEMA's estimates, due to the Corps inclusion of the existing levees in the analysis. In these cases, the community's FEMA-produced flood insurance rate maps indicate that the community has a widespread flooding problem, yet the Corps planning investigation indicates that there are insufficient flood damages to justify Federal interest in flood damage reduction measures.

Steps have been taken to address these inconsistencies. FEMA has implemented a Community Rating System (CRS) by which communities that perform certain specified flood plain management activities can qualify for premium credits toward individual flood insurance policies. One of the specified activities that would qualify a community for premium credits is a Levee Safety Program. Communities that have levees that offer protection for floods greater than the 25 year but still less than the 100-year flood would qualify for partial premium credits under this program. This aspect of the CRS would partially address the issue of recognition of levees that provide less than 100-year protection. It is important to note that only the flood insurance rates are affected by the CRS, not the flood elevations. The community would still be required to regulate future flood plain development within the FEMA-based 100-year flood plain.

The Corps and FEMA have met on a number of occasions to discuss differences in policy, including the levee freeboard issue. It has generally been agreed that the two agencies "agree to disagree" on the application of levee freeboard policy, in that the two agencies' missions are so different.

SUMMARY

Levee freeboard computation procedures have been in place for many years. Any proposed changes to the levee freeboard computation procedures must consider the overall impact to the Corps water resources mission, not just engineering or safety considerations. In addition, changes in Corps procedures for establishing levee freeboard could also have a pronounced effect on FEMA's administration of the National Flood Insurance Program (NFIP).

Differences in the application of levee freeboard requirements between the Corps and FEMA can sometimes cause difficulties in dealing with communities/local sponsors. These difficulties are recognized by both agencies, and are now handled on a case-by-case basis, usually with satisfactory results.

Concerning requests from FEMA for "certification" of existing levee systems, the Corps should make every effort to comply, and provide sufficient information for FEMA to determine the viability of the levee system in question. Extreme care should be taken when making the certification determination to ensure that flood protection levels are not overstated or undervalued. In addition, during the Corps planning process, if it is discovered that existing flood protection levels have changed from a previous study or notification to FEMA, FEMA must be notified as soon as possible to minimize the impact on the community's participation in the NFIP.

And finally, we must continue to coordinate levee freeboard issues and other technical issues closely with FEMA to ensure that future policy decisions have a positive impact on both Corps and FEMA programs.

FREEBOARD, OVERTOPPING AND SAFETY FOR LEVEES WITH LOW LEVELS OF PROTECTION

by

Lewis A. Smith[1]

APPROPRIATE CORPS' OVERTOPPING GUIDANCE.

Frequent overtopping of levees with low levels of protection will occur often as noted in ETL 1110-2-299. The costs and risks of these events must be considered in the design of the features; safety concerns of the population at risk; potential real estate taking; local operation, maintenance, repair, rehabilitation and replacement (OMRR&R) responsibilities; and full life cycle costs. The selection of the national economic development (NED) plan which maximizes the full life cycle net benefits is the goal. This is an engineering objective in accordance with paragraph 10.b.(1) on page 5 of EC 1110-2-268.

In addition to the NED feature selection process, overtopping of low levees should have full public disclosure of the performance and residual risks of the low levels of protection in accordance with paragraph 4-11.c. on pages 4-22 & 4-23 of ER 1105-2-100.

Selection of capacities of outlets for trapped interior waters should be in accordance with EM 1110-2-1413.

OVERTOPPING CONCERNS FOR LEVEES WITH LOW LEVELS OF PROTECTION

1. The OMRR&R cost estimate should include rehabilitation, replacement or repair costs to fix damages to levees with low levels of protection from one or more overtoppings during the normal economic life (50 or 100 years) of a project.

2. Overtopping of the levee may cause substantial ponding of flood waters trapped in interior areas long after the flood recedes. With many overtoppings during the economic life of the project, this may constitute a legal taking of property rights due to longer duration flooding of interior areas. Other criteria exist for taking and these should be considered also. Hydrology studies should identify real estate costs for potential takings due to trapped overtopping waters.

---

[1] Hydraulic Engineer, Hydrology Section, H&H Branch, CECW-EH-Y, US Army Corps of Engineers, Washington, DC

3. Computation of damages and costs associated with trapped overtopping flows allows formulation of alternates to reduce these expenses. Design of outlets for overtopping egress needs could result in larger capacities than outlets formulated only for residual interior flooding.

4. Overtopping or breaching flows should be routed thru interior areas to ponding areas or outlets. In large, relatively flat interior areas, these routings could be complex 2-D unsteady flow problems. The selected routing model should help quantify (1) the damages from interior ponding of trapped overtopping flows, (2) the potential real estate taking of property rights, and (3) the economic benefits of ponding stage reductions of various outlet alternates for egress of the trapped waters.

5. The population at risk from sudden rupture of the levee should be identified and alternates considered to improve safety. This applies for present conditions and future development scenarios. Safety alternates could involve set-back distances from the levees for any existing residential structures and/or prohibiting any new residential development in the future. This may involve a larger real estate "footprint" for the interior side of the levee alignment and/or local cooperation agreement (LCA) special provisions.

6. Costs of frequent rehabilitation, replacement or repair of the levees may be reduced by advanced purchase of permanent borrow areas of acceptable material in areas along the levee. Other cost savings to reduce the total life cycle costs should be considered. Since the OMRR&R for frequent levee overtopping can be significant, the NED plan should select features which maximizes total life cycle net benefits.

IS FREEBOARD APPROPRIATE FOR LEVEES WITH LOW LEVELS OF PROTECTION ?

Will freeboard help any of the above concerns? Probably not.

Will design of the initial overtopping location using freeboard superiority in accordance with ETL 1110-2-299 help? Probably not.

Will a combination of freeboard, overtopping design and flood warning preparedness plan help? Not entirely, since the dangers still exist but better lead time and planned evacuation could reduce risks.

Freeboard which doubles the levee height or some other large percentage increase in height may only make things worse. Better understanding of the design water surface profile and all the hydraulic concerns influencing the profile is part of the answer.

Understanding the safety aspects is the appropriate issue for levees with low levels of protection.

WHAT ARE APPROPRIATE SAFETY CONSIDERATIONS ?

The overriding concern is not one of freeboard selection but one of public safety understanding and policy associated with overtopping dangers.

1. The understanding route depends on educating the sponsor to allow them to make a fully informed decision about their project risks. Larry Holland (see references) addresses part of this with his paper for overtopping of a levee with just over a 1% chance level of protection on a fast rising mountain stream. The risks should be described using non-engineering vocabulary/presentation so that the population at risk can grasp the personal dangers of levee overtopping. The sponsor's education should start before the formulation process selects the NED plan. This is difficult and lacks positive incentives to effectively accomplish this task within the formulation process.

2. The other route is public safety policy. Applying risk and uncertainty analysis helps sort some financial concerns and justification needs. But to date, the NED formulation process using risk and uncertainty has not effectively resolved issues of public safety which have significant residual dangers. This formulation dilemma is further hindered since economics of personal danger of the population at risk from flooding has not been defined. Until this guidance is available any public safety policy flounders. Establishing an engineering objective for safety on projects helps apply common sense to the design. Thresholds of acceptable residual risks become inherit in the alternate selection and technical support for the recommended plan.

3. Projects with low level of protection levees for the NED plan still have frequent overtopping dangers and the potential OMRR&R costs could be significant. Recommended levels of protection above NED often are the only solution for levees. Other alternates may be much safer such as channels. Knowingly placing people at great risk is unacceptable regardless of NED recommendation or the sponsor's wishes.

4. The answer is

a. couple a full identification of the residual dangers and costs from levee overtopping with education of the sponsor to the decision about residual risks;

b. apply common sense and safety objectives to designs with effective technical justification to the highest level of protection; and

c. don't recommend projects with significant residual dangers.

ACKNOWLEDGEMENTS.

Thanks to Earl Eiker and Tom Munsey for their additional thoughts and editorial help.

REFERENCES.

1. ETL 1110-2-299, Overtopping of Flood Control Levees and Floodwalls, 22 Aug 86

2. ER 1105-2-100, Guidance for Conducting Civil Works Planning Studies, 28 Dec 90

3. EC 1110-2-268, Engineering and Design for Civil Works Projects, 1 July 1991

4. EM 1110-2-1413, Hydrologic Analysis of Interior Areas, 15 January 1987

5. Holland, Larry E., Safety Concerns for Levees and Ringwalls, published in HEC Seminar Proceedings No. 23, 7-9 Aug 1990

FLOOD OVERTOPPING OF LEVEES
INTENT AND IMPLICATIONS OF CURRENT DESIGN GUIDANCE
AND POSSIBLE MODIFICATIONS

by

Tom Munsey[1]

CURRENT GUIDANCE

ETL 1110-2-299 (22 August 1986) is the current guidance for including provisions for levee overtopping in freeboard design. Districts have adopted this guidance with little comment, presumably recognizing the need for some sort of allowance for the possibility that their levees may one day be overtopped by a flood. Since most LPP levees these days are designed for the 100-year exceedance interval flood, overtopping during the life of the project is a distinct possibility (63% chance without freeboard, for 100-year economic life.)

CURRENT INTENT

The intent of HQ overtopping design guidance is to to insure that project levee failure, if it has to occur, will occur in an area that will (1) put the least number of people at risk and (2) will do the least damage. Sometimes an overtopping point can be thought of as the last bell in the warning system: Once residents see that water is coming over the levee at that point, they know it's time to leave; and hopefully, with a properly designed overtopping scheme, they will have the time and a flood-free route to leave by.

CURRENT USE

While writing the ETL, we realized that, in the realm of events exceeding the design (usually 100-year) flood, prediction of flood behavior and the equivalent design procedures to accommodate that behavior are a bit murky. There is no clear direction as to what studies need to be done and what goals, other than to provide overtopping at the least hazardous point, are to be pursued. In spite of the murk, adherence to our intent has been good. Some Districts have gone somewhat beyond our intentions and have provided initial <u>overtopping</u> points without initial <u>failure</u> points; i.e., they have formalized their overtopping points with hardened spillways, arguing that they can't afford to have the levees actually fail at those points. Obviously, initial failure cannot occur at a hardened overtopping point. So what <u>does</u> this type of design do for us?

QUESTIONS

Using the above example as typical of current state of the art of overtopping design, take note of the number of questions this situation immediately brings up:

1. Does the water flowing over the overtopping point(s) pond up inside and adjacent to the levee or flow away from it?

2. How fast would the water rise inside the levee and how fast would it flow?

3. How much will the rate and volume of flow through the overtopping point(s) reduce the elevation of the flood flow in the channel on the levees upstream and downstream of the overtopping?

---

[1]Hydraulic Engineer, HQUSACE (CECW-EH-D), Washington DC 20314

4. Does the overtopping point encroach into the freeboard or is the freeboard redefined as being lower at this point? Will the locals flood fight at this point?

5. If the levee does not fail at these hardened overtopping points, what size flood will it take to fail the levee somewhere else at a non-hardened overtopping point?

6. At non-hardened overtopping points, what rate of failure is assumed? If needed, is it possible to guarantee quick failure at these points (fuse-plug levees)?

7. Different frequency floods and hydrographs with different shapes may behave in different ways vis-a-vis overtopping. At what return interval can one stop looking at hypothetical situations? <u>Question for workshop participants</u>: What if levee design incorporated some features of dam safety design? That is, assume the consequences of levee failure must be examined for all floods up to probable maximum flood; and, unless it can be shown that the population at risk with and without levee failure for a lesser flood are the same, design the levee height to eliminate the threat to the at-risk population.

8. Should unsteady, spatially varied flow methods be used to analyze the behavior of floods within flood control channels and through overtopping areas? Should two-dimensional flow analysis be used to analyze the behavior of floods on the landward side of the overtopping point? Are physical model studies currently the only reliable way to study these problems?

9. What analyses are used in the case of multiple overtopping points?

10. Does a re-entry point need to be designed to allow flood water ponded behind a levee to return to the channel?

NO ANSWERS

Although most of the above questions could be answered with the present state of the art, when you ask "<u>Should</u> they be answered?", HQUSACE's response at this time is: "We don't know!"

Are the questions concerned with events that are so unlikely to occur that we don't need to pay attention to them? That's certainly the current de facto policy. HQUSACE's position: "We don't know!"

Suppose the overtopping event is one of those frequent ones described a moment ago in Lew Smith's presentation. How frequent does the overtopping event have to be before we do begin to ask these questions? HQUSACE'S position: "W.d.k.!"

THE FUTURE

Even if, as a result of this workshop, the concept of freeboard is eliminated, overtopping design will still be with us. Although the engineering talent available to address some of the design questions listed above seems to be shrinking, the tools necessary to do a better and speedier analysis seem to be expanding. So how fine a point do we put on it? Within the confines of this workshop panel discussion, we have about 20 minutes to solve this problem.

SUMMARY OF SESSION 2: PART 1 - LEVEE PROJECT EVALUATION AND PERFORMANCE

Overview

This session included three papers and a panel on levee project studies that illustrated current levee freeboard policy, design, and related issues. Participants included representatives from seven different district offices.

Paper Presentations

Paper 4. Brian Tracy, Chief, Hydraulics Section, H&H Branch, Los Angeles District presented a paper entitled, "Lower Santa Ana River Levee and Freeboard Design." The Lower Santa Ana River project includes a leveed channel between Prado Dam and the Pacific Ocean. The project area includes a reach of over 20 miles of intensely urbanized flood plain. This channel will convey the 0.53 percent chance (190-year) flood with a minimum freeboard of 2.5 feet of freeboard in the overtopping zones and a minimum of three feet for the remainder of the levee. Least hazardous overtopping areas were identified and levee freeboard was adjusted to insure that all discharges for greater than design events leave the channel at these locations in a non-catastrophic manner. The design makes use of existing flood control structures and rights of way, minimizes impacts to bridges and utilities, and provides for sediment transport and interior drainage. There was considerable interest in the paper presentation and there were several questions pertaining to the levee freeboard design.

Paper 5. Darryl Dolanski, Great Lakes Hydrology and Hydraulics Branch, Detroit District, presented a paper entitled, "Levee Freeboard Design at Fort Wayne, Indiana." The City of Fort Wayne, Indiana is located in northeastern Indiana in Allen County, and is part of the Maumee River basin. The city presently has about six miles of levee and floodwall. The proposed Corps flood control plan would upgrade the existing line of protection to provide relief from the one percent chance flood, with freeboard.

During initial feasibility studies, based on traditional Corps practice, a minimum required freeboard of three feet was assumed. Subsequent, more detailed levee freeboard design attempted to identify and quantify the factors that cause uncertainty in water surface profiles. Issues which were addressed included the availability of stage and flow data for HEC-2 model calibration, the sensitivity of the HEC-2 model to roughness coefficients and other input parameters, the stability of the channels being modelled, and the effects of other factors upon hydraulics such as ice, debris, and wave action.

Initial overtopping reaches were determined at least hazardous locations, and a minimum freeboard established. Freeboard design water surface profiles were developed based on the minimum freeboard. Levee superiority was incorporated for remaining reaches of levee and floodwall to ensure that initial overtopping of the line of protection would occur at the least hazardous locations. As a result of additional effort to design the freeboard, instead of assuming a minimum of three feet, a cost saving was realized and the project was more favorably received by the local sponsor.

Paper 6. Ronald E. Hilton, Chief, Hydrology and Hydraulics Branch, Jacksonville District, presented a paper entitled, "High Velocity Leveed Channels - Puerto Rico." The Island of Puerto Rico is a land of abundant rainfall, varied topographic characteristics, numerous flood problems, and even more numerous hydrologic and hydraulic challenges. Flat developable land is one of this islands most scarce resources. Runoff in the high central mountain ranges moves very quickly to the lower foothills and disperses widely in the coastal alluvial flood plains. An incomplete and relatively short 20-years of stream gaging records do not provide an adequate period for design hydrology without sensitivity analyses and comparative studies. The hydraulic analysis included alternatives both above and below critical depth and transitions within individual reaches. Hydrologic and hydraulic variables and criteria for overtopping analysis of high velocity leveed channels at several sites in Puerto Rico were discussed during the paper presentation.

Ron emphasized the concern that any new guidance on levee freeboard be clear and specific so that misinterpretation that causes study delays during the review process can be avoided as much as possible.

Panel 2: Levee Project Evaluation and Performance

Four panel members made short presentations based on their individual experiences that highlighted specific issues related to current levee freeboard policy and application.

A. Joel James, Chief Hydraulics Section, Savannah District, submitted a paper entitled, "Floodplain Encroachment and Its Effects on Levee Overtopping Design." Joel was unable to attend the workshop. Her paper described the effects of future condition hydrology and flood plain encroachment on the design of overtopping in least hazardous locations. The analysis of an existing levee at Macon, Georgia, using current guidance on overtopping of levees, would result in a requirement of from 5 to 12 feet of freeboard above the one percent chance (100-year) flood design water surface elevation. The conclusion was to design for the one percent chance flood level of protection with a minimum of three feet of freeboard.

B. Jeffrey McClenathan, Hydraulics Section, Hydrologic Engineering Branch, Omaha District, presented a panel discussion that summarized the proposed freeboard designs for the Scribner, Nebraska, and Sioux Falls, South Dakota, flood control projects. The summaries included brief descriptions of how the freeboard was determined for each project while considering various design conditions. The Scribner, Nebraska project is a proposed levee project while the Sioux Falls, South Dakota project would raise an existing levee. The freeboard design of the Scribner project consisted of evaluating factors such as ice and debris blockage to determine if three feet of freeboard was adequate.

The Sioux Falls project was originally designed and built with two feet of freeboard. The project is being restudied using a revised one percent chance (100-year) discharge which is approximately double the design discharge used in the original project. An upstream diversion dam and the resulting split flow conditions along with increased development in the area are complicating factors in the restudy analysis. Discussion included some the issues associated with the freeboard design and the lack of guidance for risk based assessment for project engineering and planning.

C. James L. Lencioni, Seattle District, presented a panel discussion entitled, "Unique Freeboard Situations in Seattle District." Jim discussed the Green River Project, located in Western Washington State and the South Aberdeen project located along the south bank of the Chehalis River in the City of Aberdeen, Washington. The project evaluations revealed specific instances where the "standard" three feet of freeboard has been shown to be technically not warranted. Both projects were subjected to unique hydrologic/hydraulic conditions which supported the use of a lower freeboard.

The Green River project involved an upgrade design of a non-federal levee located downstream from a Corps of Engineers flood control storage dam. Approximately 25 years of flood control operational experience at the dam in conjunction with water surface profile observations made at the dam's flood control design discharge lend a high degree of certainty to computed water surface profiles in the study reach. The resulted in the adoption of a minimum freeboard of two feet.

The South Aberdeen project involved the design of a federal flood control levee located in a tidally-influenced reach of a natural river. In this situation, design condition water surface profiles were computed with a numerical unsteady flow model having a downstream boundary simulated by the tide hydrograph. The tidal frequency curve has a 95 percent confidence limit of less than about 0.4 feet which, in combination with the tide's strong backwater effect, results in a high degree of confidence in the computed water surface profiles. A 1.5 foot minimum freeboard was adopted for this project.

Jim's key point was that freeboard should be "designed" on a case-by-case bases and not assumed to be a "standard" minimum of three feet.

D. Gary Dyhouse, Chief, Hydrologic Engineering Section, Engineering Division, St. Louis District, presented a panel discussion entitled, "Levee Freeboard Issues Based on St. Louis District Experiences." He emphasized that minimum freeboard for a levee project, either existing or new, is a major decision affecting the economic viability of the analysis. Three feet minimum freeboard greatly increases the cost of a new project, while giving overly conservative estimates of level of protection for an existing project. Minimum freeboard could be determined by more analytical and defensible methods, including designing the top of levee for a specific frequency or discharge, allowing the sponsor to chose freeboard based on economic consequences, or on site-specific characteristics of the protected area.

Gary indicated that freeboard seems to be viewed as "gold plating" by many outside the H&H technical area. He would like to do away with freeboard entirely and design for the protection needed. The most significant issue is that there is no definitive method to design freeboard. He noted that this seems especially difficult in raising existing levees where a determination of benefits to be assigned to the old levees must be made. The St. Louis District usually uses two feet for old levees and three feet for new levees. He concluded that three feet of freeboard may be overkill in some areas. Perhaps construction of a project with less than three feet freeboard but which is economically feasible using Corps criteria would be better than no project at all.

Much discussion about determining benefits of existing levees and freeboard followed. Earl Eiker stated that data should exist which would allow capacity of existing systems to be determined. The benefits would be attributable to the capacity of the system which has be experienced by the project based on historical records. A flood which has reached within a few inches of the top of the levee and was contained by the levee has essentially established the system capacity. It was also pointed out that consistent treatment should be given to the capabilities of existing and new levees. This would require that some freeboard be included in the analysis of the existing levee. No consensus was reached during the remainder of the Panel 2 discussions as to how freeboard should be analyzed for existing levees.

LOWER SANTA ANA RIVER
LEVEE AND FREEBOARD DESIGN

by

Brian Tracy[1]

INTRODUCTION

Purpose. This paper presents a summary of the efforts that went into developing the channel and freeboard design for the Santa Ana River between Prado Dam and the Pacific Ocean in Orange County, California.

Key Issues. The work involved refining a channel design which had been developed and selected in a previous study phase. The previous design had not fully addressed the issues of 1) incorporating existing flood control structures into the design; 2) bridge, utility and right-of-way locations; 3) sediment transport conditions; 4) interior drainage; and 5) the design of channel freeboard.

PHYSICAL SETTING AND AVAILABLE DATA

Description of Watershed. The project is located at the downstream end of the 2450-square mile Santa Ana River watershed. This watershed is divided into upper and lower basins by the Santa Ana Mountains and the Chino Hills. Refer to Figure 1 for a schematic of the drainage area.

1) The upper basin has a drainage area of 2250 square miles and is ringed by the Chino Hills and the San Gabriel, San Bernardino, San Jacinto, and Santa Ana mountains. Approximately 37 percent of the drainage area lies within these mountains. Alluvial fans, valleys and plains fill the rest of the upper basin bordered by the mountain sediment material sources. The headwaters of the Santa Ana River itself are located in the San Bernardino Mountains. The river flows from a narrow canyon out onto a broad alluvial wash. It meanders over this wash for about 9 miles until it reaches the City of San Bernardino. At this point the river is collected into a well defined channel and is conveyed 30 miles to Prado Dam. The river has a dimensionless slope of about 0.045 in the mountains flattening out to about 0.0038 near Prado Dam. The entire upper basin drains to and is controlled by the existing Prado Dam and reservoir. Prado Dam is located at the boundary between the upper and lower basins. The northern half of the upper basin lies in San Bernardino County and the southern half lies in Riverside County. The Lower Santa Ana River project lies entirely downstream of the upper basin.

2) The lower basin has a drainage area of about 200 square miles. The Chino Hills and the Santa Ana mountains comprise much of the drainage area of the lower basin. The Lower Santa Ana River and Santiago Creek are the major watercourses draining these steep areas. The Lower Santa Ana River flows from

[1]Chief, Hydraulics Section, Los Angeles District, U.S. Army Corps of Engineers

Prado Dam at the upstream end of the lower basin for 7.4 miles through the lower Santa Ana Canyon to emerge onto the coastal floodplain. From the canyon mouth the river flows 12.9 miles to the confluence with Santiago Creek and from there it flows 10 miles further to the Pacific Ocean. Downstream from the canyon mouth, the entire drainage area and floodplain are intensely urbanized. Downstream of the Santiago Creek confluence the drainage area is very narrow. It is interesting to note that in this reach the river floods far beyond the drainage area boundaries. The upstream 2.5 miles of the Lower Santa Ana River lie within Riverside County, and the remainder of the river downstream lies within Orange County. The Lower Santa Ana River project extends for the entire length of the river in the lower basin.

3) Runoff is also diverted into the lower basin watershed from the adjacent Carbon Canyon Creek to the north. Discharges on Carbon Canyon Creek are controlled by the Corps' Carbon Canyon Dam. Flows are diverted to the Santa Ana River by means of a diversion channel downstream of the dam. This diversion channel has a capacity of 2800 cfs and enters the Lower Santa Ana River about 5.5 miles upstream of the Santiago Creek confluence.

Existing Channel. The 30 miles of the Lower Santa Ana River were divided into three reaches for hydraulic design purposes. Refer to Table 1 below for the definitions of these reaches.

Table 1.
Lower Santa Ana River Reach Definitions

Reach Name	Limits	Stations	Length (mi.)
Canyon Reach	Prado Dam Outlet Channel to Weir Canyon Road	1607+50 to 1216+47	7.4
Drop Structure Reach	Weir Canyon Road to d/s end of River View Golf Course	1216+47 to 535+30	12.9
Trapezoidal Ocean Reach	D/S End of River View Golf Course to Pacific Ocean	535+30 to 7+60	10.0

1) The Canyon Reach begins at the Prado Dam outlet channel drop structure and extends downstream to a point approximately 700 feet upstream of the Weir Canyon Road bridge. Within the Canyon Reach the channel varies in width from 200 to over 2000 feet. The channel is braided in some locations and follows a winding course through the Santa Ana Canyon. The average invert slope is about 0.0032. Most of the existing channel is in a natural, densely vegetated condition except for a 3500-foot long section of earthen low flow channel along the southern edge of the Green River Golf Course between stations 1514+10 and 1479+10. Several existing levees and bank revetment measures protect about half of the bankline in this reach to one degree or another.

2) The Drop Structure Reach begins at the downstream end of the Canyon Reach and extends to the downstream end of the River View Golf Course. In this reach the river channel has been improved by local interests. The river has a trapezoidal section with a soft bottom and revetted levee sideslopes. Base widths vary from 320 to 240 feet, and depths vary from 12 to 18.5 feet. The revetment is mainly stone with some reaches of concrete. Sideslopes vary from as steep as 1.5 horizontal to 1 vertical to as flat as 3 horizontal to 1 vertical. Eleven vertical drop structures are located in this reach, with heights varying from 4.5 to 9.8 feet. The average dimensionless invert slope in this reach is 0.0022. Orange County uses the earthen channel invert in this reach to recharge the groundwater aquifer.

3) The Trapezoidal Ocean Reach begins at the downstream end of the Drop Structure Reach and extends to the ocean outlet channel. The channel is improved in this reach with an invert that alternates between earth bottom, grouted stone, and concrete, and a base width that varies from 160 to 250 feet. The channel section varies from trapezoidal with sideslopes of 3 horizontal to 1 vertical to rectangular. Channel depths range from 13 to 17 feet, and the average invert slope is 0.0015.

4) Refer to Figure 2 for a diagram showing the reach locations.

Flood Problem. The present Lower Santa Ana River channel has a capacity of 30,000 cfs, which corresponds to the peak discharge of an 83-year flood. In addition, facilities within the channel itself such as bridges, drop structures, and channel revetment will sustain severe damage if subjected to long duration releases from Prado Dam in excess of 5000 cfs. A 190-year flood would break out of the channel at the downstream end of the canyon below Prado Dam and would inundate widespread areas in Orange County. This flood would inundate the cities of Westminister, Fountain Valley, Santa Ana, Stanton, Anaheim, Garden Grove, Los Alamitos, Fullerton, Placentia, Yorba Linda, Buena Park, La Palma, Cerritos, Hawaiian Gardens, Cypress, Costa Mesa, Huntington Beach and Seal Beach. This area is inhabited by more than 2 million people and will sustain about $8.4 billion in damage should this event occur without the proposed project.

The average annual inundation reduction benefits available to offset the cost of a flood control project equal $127,449,000 per year (October 1987 prices).

OVERVIEW OF THE SANTA ANA RIVER PROJECT

Solution to Flood Problem. The solution to the flood problem on the Lower Santa Ana River consists principally of three structural measures. From upstream to downstream these measures are: 1) Construct a new flood control dam in the upper Santa Ana River Canyon near the headwaters of the Santa Ana River; 2) improve the existing Prado Dam; and 3) improve the existing flood control channel on the Lower Santa Ana River below Prado Dam.

Seven Oaks Dam. Seven Oaks Dam is a 550-foot high rock and earth fill embankment dam currently under construction in the Upper Santa Ana Canyon approximately 40 miles upstream of Prado Dam. This dam will be a single-purpose flood control dam with a storage capacity at spillway crest of 145,600 acre-feet. This dam will be capable of controlling a 350-year flood from the watershed above it and will control 8% of the presently uncontrolled watershed contributing to Prado Dam. The purpose of this dam is to provide relief to Prado Dam. In a 190-year flood, Seven Oaks Dam will store nearly all of the runoff from the watershed above it until the peak reservoir water surface elevation at Prado Dam has passed.

Prado Dam. The existing Prado Dam will be improved to provide protection without spilling for events having a return interval of up to 190 years. The embankment will be raised 30 feet, and the spillway crest will be raised 20 feet. The capacity of the currently deficient existing spillway will be expanded through the use of a labyrinth weir and by increasing the design head. A new outlet works will be constructed to expand the discharge capacity of the dam. The storage capacity at spillway crest will increase from 217,000 to 362,000 acre-feet, the spillway capacity will increase from 181,000 to 578,000 cfs, and the outlet works capacity will increase from 10,000 to 30,000 cfs.

Lower Santa Ana River Channel. The Santa Ana River Channel downstream of Prado Dam will be improved to convey the with project 190-year releases from Prado Dam and the coincident contribution from the drainage area below the dam. The design of this channel is the subject of this paper.

Other Features of the Santa Ana River Project. There are a number of other features of the Santa Ana River Project which do not directly affect flooding conditions on the Lower Santa Ana River. These features are either independently justified, are provided to avoid future flood problems, are mitigation features for project induced impacts or were directly authorized by Congress. These other features are: 1) The upper Santa Ana River floodway, 2) the Mill Creek levee improvements, 3) the San Timoteo Creek channel and debris basins, 4) the Oak Street Drain channel, 5) the Santiago Creek bank stabilization, and 6) the Santa Ana River Marsh restoration. Refer to Figure 1 for the locations of the various Santa Ana River Project elements.

DESIGN OF THE LOWER SANTA ANA RIVER CHANNEL

General. Conceptually, the process by which the Lower Santa Ana River channel was designed may be summarized as follows: 1) A sediment transport analysis was conducted for the project based upon the hydraulic design developed in a previous study phase; 2) a physical model study was performed to determine the best way to adapt the existing drop structures in the river channel to the Corps design; 3) a hydraulic design including a freeboard design was performed incorporating the results of the sediment transport analysis and the physical model studies; 4) the design was adjusted to reflect civil design concerns such as rights-of-way, bridge and utility locations, bridge preservation, and balanced cut and fill; 5) the sediment analysis was adjusted to reflect all the design changes subsequent to the initial analysis; and 6) a detailed analysis of the residual interior flooding was performed.

Hydrology. The with-project 190-year flood design discharges for the Lower Santa Ana River Project are presented in Table 2 below.

Table 2.
Design Peak Discharges
190-year Flood With-Project

Location	Station	Design Discharge (cfs)
Prado Dam Outflow	1607+50	30,000
Downstream of Wardlow Canyon	1603+10	31,000
Downstream of Weir Canyon Road	1207+30	37,000
Downstream of Imperial Highway	1065+61	38,000
Downstream of Carbon Cyn Div Channel	846+25	40,000
Downstream of Santa Ana Freeway	625+39	42,000
Downstream of Santiago Creek	564+00	46,000
Downstream of Hamilton Avenue	72+90	47,000
Pacific Ocean	16+95	47,000

Sediment Transport Analysis. A detailed 3-phase sediment transport analysis of the proposed project was performed. The three phases consisted of 1) a qualitative analysis, 2) a quantitative fixed-bed analysis, and 3) a detailed moveable bed analysis using HEC-6. The objectives of the sediment transport analysis were to 1) identify reaches of aggradation and degradation and to adjust the channel design accordingly; 2) estimate operation and maintenance requirements with respect to sediment removal; and 3) evaluate the long term impact of the project upon sand delivery to the coast.

1) Qualitative Analysis. A qualitative analysis was performed to estimate where the proposed project would experience aggradation and degradation. This analysis was performed using Lane's Relation:

$$q_s \, D_{50} \propto q \, S$$

where:
- q_s = unit sediment discharge
- D_{50} = median sediment diameter
- q = unit water discharge
- S = channel invert slope

The results of this analysis predicted aggradation in the upstream part of the drop structure reach and at the downstream end of the trapezoidal ocean reach. A stable channel was predicted for the downstream end of the drop structure reach, and degradation was predicted within the upper part of the trapezoidal ocean reach.

2) Quantitative Fixed-Bed Analysis. This analysis was based upon the

equilibrium slopes method. A rating curve was developed from suspended sediment measurements that were later used as inflowing load at the upstream end of the drop structure reach. Then using Yang's Unit Streampower Equation, the bed slope required for each channel reach to transport the sediment inflow without aggradation or degradation was determined for a range of discharges. The results of this analysis generally confirmed the results of the qualitative analysis described above. As part of this analysis, the potential for the stream to armor itself was checked. Stream armoring was found to be unlikely.

3) Detailed Moveable Bed Analysis.

a) A moveable bed analysis was performed using the HEC-6 computer program. Geometric data for the model was developed from cross section surveys conducted prior to a flood in 1978. To estimate the inflowing load, a relationship for sediment discharge versus water discharge was developed using suspended sediment measurements, estimates of watershed sediment yield from several methods checked by observed data, estimates of bank erosion based upon aerial photographs, and by applying HEC-6 itself to generate the inflowing equal to the sediment transport capacity of the channel. Bed material gradations were based upon analysis of samples taken from the existing bed. The HEC-6 model was calibrated to reproduce the bed changes which occurred in the 1978 flood. The model was then verified by reproducing the bed changes which occurred in a flood in 1980. All of the sediment transport functions then available in HEC-6 were tried, and Yang's Unit Streampower equation calibrated the best.

b) The geometry of the HEC-6 model was adjusted to reflect with-project conditions using the channel design from the previous study phase. The design flood was analyzed with this revised HEC-6 model for two inflowing load conditions at the upstream end of the drop structure reach: 1) the inflowing sediment load estimated as described above, and 2) no inflowing sediment load. The first condition was run with 38 days of 5000 cfs antecedent flow followed by the design hydrograph. This condition was used to estimate the upper grade limit of sediment deposition for determining the design water surface elevation. The second condition was used to estimate maximum degradation for setting levee toe depths. Sensitivity analyses were conducted to check the effects of varying roughness coefficients, tailwater elevations, bed material grain size diameters, and antecedent flow conditions.

c) The incremental probability method was used to estimate the project's impact upon sand delivery to the coast and to estimate the operation and maintenance requirements for sediment removal from the channel. To estimate the impact upon sand deliveries to the coast, the with- and without-project HEC-6 models were run for the 10-, 25-, 50-, 100-, and 170-year balanced flood hydrographs. The average annual deliveries of sand sized material were then estimated for with- and without-project conditions using the following equation that weights the sand outflow for each event by the incremental probability for that event.

$$Qs_{annual}=.006\,(V_{170})+.004\frac{(V_{170}+V_{100})}{2}+.01\frac{(V_{100}+V_{50})}{2}+.02\frac{(V_{50}+V_{25})}{2}+.06\frac{(V_{25}+V_{10})}{2}$$

where Qs_{annual} = Average annual sand volume delivered to the coast

V_n = Sand volume from n-year event delivered to the coast

To estimate the average annual sediment removal for operation and maintenance, the same equation was used except that only the with-project condition was used and the meaning of the variables changed as follows:

Qs_{annual} = Average annual sediment deposition within the project reach

V_n = Sediment volume from n-year event deposited within the project reach

The results of the analyses indicated that, on an average annual basis, sand deliveries to the coast would increase by about 44%. The principal reason for the increase is that the project will prevent the larger events from breaking out of the channel. Material which formerly deposited in the floodplain will reach the coast with the project in place. The average annual sediment deposition within the project reach is estimated to be 31,000 cubic yards per year. With the proposed upper grade limit for the project invert, cleanouts would be required an average of once every 21 years. Virtually all of the deposition will occur at the downstream end of the project.

Physical Model Studies.

1) In an effort to minimize project costs, physical model studies were conducted at the Waterways Experiment Station to adapt the existing drop structures within the drop structure reach to with-project conditions. The objectives of the physical model study were to develop a design which would 1) provide good energy dissipation within the drop structure basin; 2) minimize downstream scour; 3) minimize the cost of modifications to the existing structures; and 4) perform well for a range of discharges. The maximum freeboard design discharge of 215 cfs/foot was used to design the drop structures. The freeboard design discharge was selected for use in designing the drop structures because in order for the freeboard design described below to function, the drop structures must not fail.

2) The existing drop structures have concrete vertical drops and end sills. Modifications to these structures were tested with both a sectional model in a flume and also with a full-width model.

3) The final design developed consists of adding a parabolic drop to the existing structures, adding two rows of baffle blocks and installing a sloping basin end sill further downstream. This design salvages all 11 of the existing drop structures.

General Hydraulic Design.

1) General. The channel design and water surface elevations were determined using the WASURO interactive channel design computer program. Bridges were assumed to have 2 feet of debris on each side of each pier. Revetment requirements were determined using an in-house riprap design program based upon the method described in EM 1110-2-1601. High and low invert roughness coefficients in the earth bottom reaches were determined by a sensitivity analysis of possible bed forms. The design incorporated the results of the sediment transport analysis for the minimum and maximum design inverts and the physical model study for the drop structure design.

2) Canyon Reach. Structural improvements within the Canyon Reach will be limited to a 2500-foot long levee to protect an existing mobile home park from the design discharge of 33,500 cfs at that location. The levee will have the nominal 3 feet of freeboard and 15-inch thick grouted stone revetment on a 2 horizontal to 1 vertical side slope extending 18 feet below the river invert.

3) Drop Structure Reach. The design in this reach will consist of an earth bottom trapezoidal section. The base width will vary from 270 to 330 feet conforming to the base width of the existing channel. The channel sideslopes will be protected by riprap and grouted stone at slopes of 2 horizontal to 1 vertical. Refer to Figure 3 for a diagram summarizing the proposed channel dimensions. In addition to modifying the eleven existing drop structures as described above, three new drop structures and twenty-one new stabilizers will be added to the reach. The need for the stabilizers was determined based upon the results of the zero inflowing load sediment transport analysis described above. The revetment toes on the levees will generally be 5 feet deep. On the downstream side of the drop structures however, the toe will be 15 feet deep. The 15-foot toe depth will extend for 100 feet downstream of each drop structure, then will slope upward to meet the general 5-foot toe depth. There are two significant confluences within the drop structure reach: 1) Carbon Canyon diversion (2000 cfs) on the right side at station 846+25; and 2) Santiago Creek (4000 cfs) on the left side at station 564+00. Flow at both confluences is subcritical on the tributaries and on the main stem; therefore, the streams were simply joined at the existing flow angles. Nineteen bridges are located within the drop structure reach. Virtually all bridges will require some modification, with the bridges at Katella Avenue and Orangewood Avenue to be replaced entirely.

3) Trapezoidal Ocean Reach. The design in this reach will consist of two distinct sections. The upstream section will have a concrete invert and sideslopes. The downstream section will have riprap sideslopes and an earthen invert. Base widths will vary from 180 to 450 feet. The channel section will vary from rectangular to trapezoidal with sideslopes of 2 horizontal to 1 vertical. Refer to Figure 3 for a diagram summarizing the proposed channel dimensions. The concrete invert was incorporated into the design for the reach where the sediment transport analysis predicted bed degradation. The concrete channel reach flows in the supercritical regime until it reaches a point where the backwater from the ocean and the downstream sediment deposit force a hydraulic jump. The location of this jump varies depending on the size of the sediment deposit. The channel wall heights were determined

assuming the maximum sediment deposition and the downstream limit of the concrete invert was determined based upon the minimum sediment deposition. Most of the bridges in the trapezoidal ocean reach provide at least 2.0 feet of freeboard and will therefore not require modification. One major confluence is located in the trapezoidal ocean reach. The Greenville-Banning channel will enter from the left side between stations 76+40 and 72+90.

4) Greenville-Banning Channel. The Greenville-Banning channel will be improved as part of the Corps project for a distance of 16,800 feet upstream from its confluence with the Santa Ana River. Under pre-project conditions this channel flows parallel to the Santa Ana River and enters the ocean separately. In order to reduce water surface elevations on the Santa Ana River, the channel will be brought in as a tributary, and the Santa Ana River channel will be widened to make use of the land formerly used by the Greenville-Banning Channel. The Greenville-Banning channel and the confluence with the Santa Ana River were designed for the worst case of peak and contemporaneous discharges in the Santa Ana River and in the Greenville-Banning Channel. The Greenville-Banning Channel will be a rectangular concrete channel with base a width of 60 feet.

Freeboard Design.

1) General. The channel depths and freeboard allowances in the Drop Structure and Trapezoidal Ocean Reaches are based upon a somewhat complex relationship between the nominal design discharge and the "freeboard design discharge". Factors such as the overbank topography, locations of least hazardous overtopping zones, locations and capacities of bridges, degree of channel entrenchment, and engineering judgement all played major roles in the determination of the freeboard design.

2) Canyon Reach. Within the Canyon Reach the proposed project structural improvements are minimal, resulting in a simplified freeboard design. The freeboard allowance for the 2500-foot long levee within the Canyon Reach was a nominal 3 feet.

3) Drop Structure Reach. The drop structure reach is where the primary freeboard design features are located. A step-by-step summary of the freeboard design in this reach is as follows:

a) Based upon inspection of maps and field reconnaissance, the areas most suitable for initial overtopping were identified. These areas are summarized in Table 3 below. The most critical least hazardous overtopping zone is the right overbank between stations 1202+50 and 1031+70. The overtopping capacity of this reach is so large that for virtually any event greater than the design event, the discharge remaining in the channel in this reach will not exceed the channel capacity. Furthermore, the topography of the right overbank slopes away from the channel so that flows overtopping on the right side of the channel will not return.

b) Water surface profiles were computed to determine the capacity of the proposed channel with a nominal 3.0 feet of freeboard used for conveyance. This "freeboard design" profile was virtually parallel to the design water surface profile. The parallel profiles result in virtually simultaneous overtopping of the channel throughout the drop structure reach for greater-than-design events. Simultaneous overtopping is fortuitous in that relatively little "superiority" of freeboard is needed to force water out of the channel at desired locations.

Table 3.
Least Hazardous Overtopping Locations

Location	Stations	Overbank
Riverside Freeway Right of Way	1202+50 to 1031+70	left
Yorba Regional Park	1202+50 to 1031+70	right
County Groundwater Recharge Facilities	1000+00 to 986+00	right
County Groundwater Recharge Facilities	941+00 to 928+00	right
County Groundwater Recharge Facilities	844+00 to 822+00	left and right
County Groundwater Recharge Facilities	733+00 to 710+00	left and right
Anaheim Stadium Parking Lot	682+00 to 670+00	right

c) At the least hazardous overtopping locations identified above the freeboard was reduced to 2.5 feet. The rationale for reducing the freeboard is that at these locations the levees are very localized and the channel is mostly entrenched.

d) At the upstream-most least hazardous overtopping zone mentioned above, all but 56,000 cfs will escape the channel for virtually any event larger than the design event. This discharge was computed assuming that the 2.5 feet of freeboard provided is available for conveyance.

e) Downstream contributions to the freeboard design discharge were based upon the bankfull and/or maximum capacities of the tributaries and interior drainage facilities.

f) For each of the subsequent least hazardous overtopping locations downstream, the length of the overtopping section was adjusted to force enough of the flow in the channel to leave such that the 3.0 feet of freeboard generally provided is enough to contain the flows remaining in the channel downstream of the overtopping zone.

g) The overtopping zones were analyzed as side overflow weirs with discharge coefficients of 2.65 (broad crested weir). The backsides of the levees were armored with a 12-inch thick layer of grouted stone at each of the overtopping locations to prevent failure of the levee. Table 4 below summarizes the design of the overtopping areas.

Table 4
Summary of Overtopping Area Design

Station		Side of Overflow Levee	Upstream Channel Capacity (cfs)	Discharge over Side Weir (cfs)	Discharge Remaining Downstream (cfs)
Upstream	Downstream				
1202+50	1031+70	both			56,000
1000+00	986+00	right	57,700	700	57,000
941+00	928+00	right	57,700	1,700	56,000
844+00	822+00	both	63,000	3,000	60,000
733+00	710+00	both	60,000	3,800	56,200
682+00	670+00	right	60,000	1,500	58,500

4) Trapezoidal Ocean Reach. The dominant factor in the freeboard design for this reach is the lack of least hazardous overtopping zones. The overbanks adjacent to the channel for the entire reach contain residential, commercial, and industrial development with potential for high damage and/or loss of life. The freeboard design in this reach therefore had to assure that the channel would never overtop. The step-by-step procedure for this is as follows:

a) Initially, the minimum nominal freeboard was provided for the entire Trapezoidal Ocean Reach. The value for this freeboard was 2.5 feet for entrenched sections and 3.0 feet for leveed sections. The capacity of the channel to the top of this nominal freeboard was determined assuming that the freeboard was available for conveyance.

b) The maximum possible discharge that could be delivered to the upstream end of the Trapezoidal Ocean Reach at station 535+30 was determined to be 65,000 cfs. This value is based upon the maximum capacities of the upstream project plus all the upstream tributaries and interior drainage facilities.

c) Contributions to the freeboard design discharge within the Trapezoidal Ocean Reach were based upon the bankfull and/or maximum capacities of the tributaries and interior drainage facilities.

d) It was determined that the channel in the Trapezoidal Ocean Reach between stations 535+00 and 290+00 could convey the maximum discharge that could reach the channel within the nominal freeboard.

e) Downstream of station 290+00 the freeboard was increased above the nominal such that the maximum discharge that could reach the channel could be conveyed in the freeboard. This increase provides as much as 8.0 feet of freeboard in some locations.

f) A summary of the freeboard design discharges is provided in Table 5 below.

Table 5.
Freeboard Design Discharges in the Trapezoidal Ocean Reach

Location	Station	Discharge (cfs)
Inlet	535+30	65,000
End of Nominal Freeboard	290+00	65,000
Adams Avenue Bridge	171+80	65,500
Hamilton-Victoria Bridge	90+40	65,500
Greenville-Banning Confluence	75+00	71,300
Pacific Ocean	7+60	71,300

5) Greenville-Banning Channel. Freeboard was provided on the Greenville-Banning channel such that it could carry the design discharge of 5800 cfs coincident with the freeboard design discharge in the main stem Santa Ana River channel. The resulting freeboard varied from about 5.0 feet at the confluence with the Santa Ana River, decreasing gradually to 3.0 feet at the upstream end of the project reach for the Greenville-Banning Channel.

Residual Interior Flooding. A detailed analysis of the interior drainage conditions was conducted as part of a special Feature Design Memorandum. It was determined that for no combination of coincident general and local events would with-project flooding exceed without-project flooding at any location. Recommended improvements to the existing interior drainage facilities were therefore limited to upgrading the sizes of existing pipes through the levee where the local sponsor indicated interior improvements were planned. The remainder of the work on the FDM consisted of delineating 25-, 50-, 100-, and 190-year local residual flood boundaries for each of the 200 drains.

CONCLUSION

The hydraulic design of the Lower Santa Ana River project provides a channel, which in combination with the other elements of the Santa Ana River project, provides a nominal 190-year level of protection from flooding from the Santa Ana River downstream of Prado Dam. In addition, the freeboard design of the project precludes a catastrophic failure of the channel levees from any event greater than the 190-year event, and assures initial overtopping of the channel in locations where the hazard is minimized. The hydraulic design of the project incorporates a detailed sediment transport and interior drainage analysis.

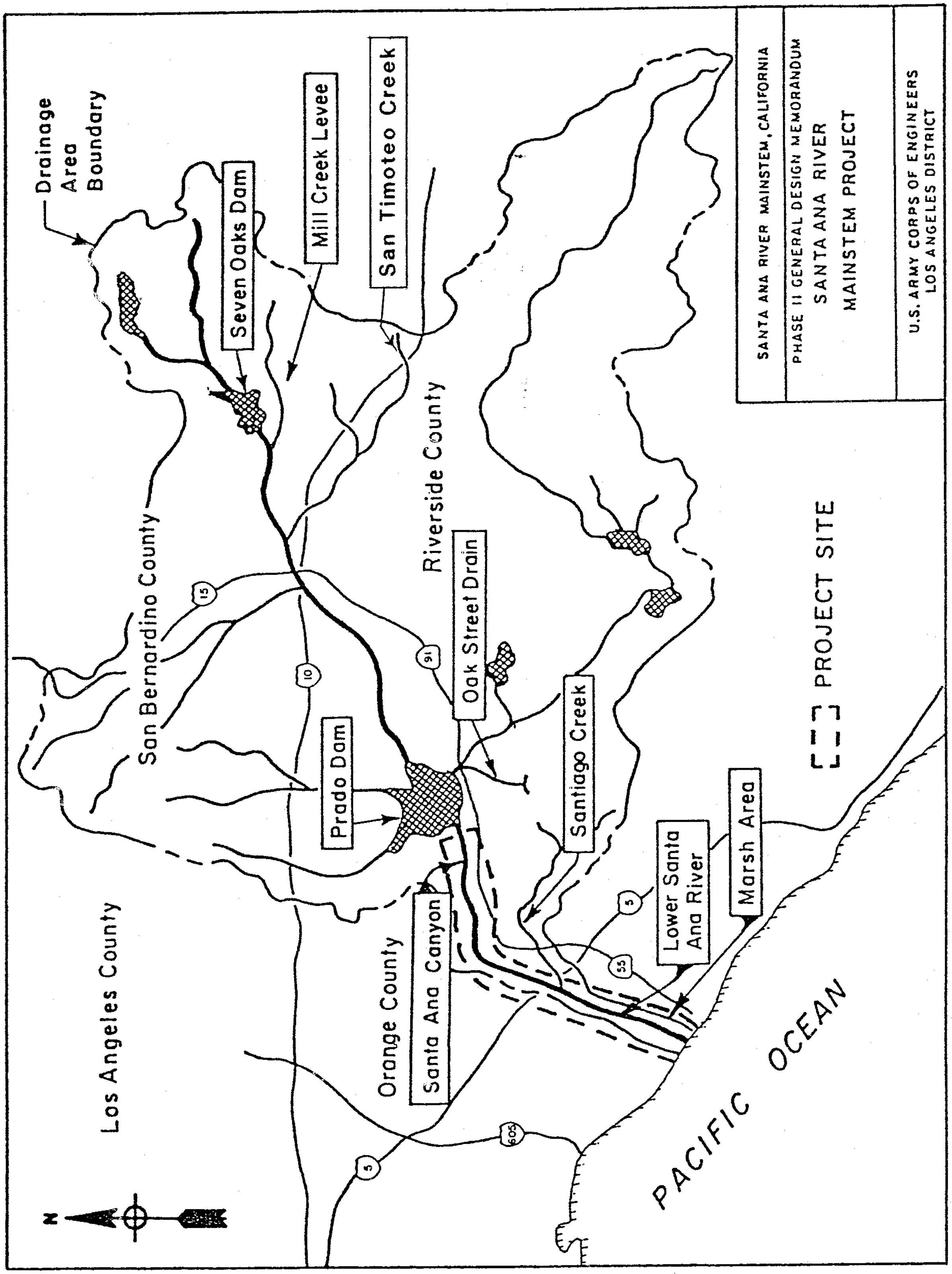
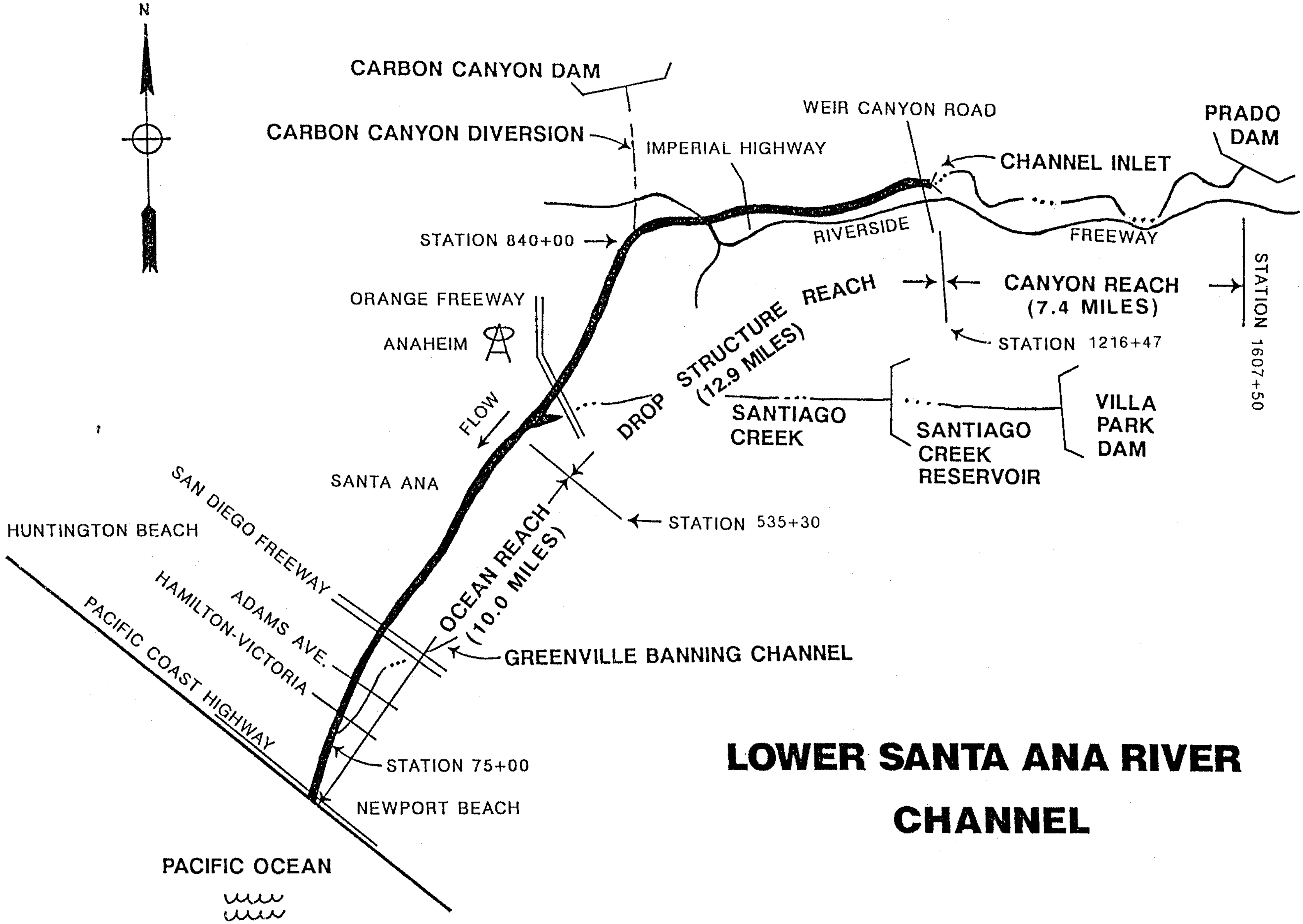
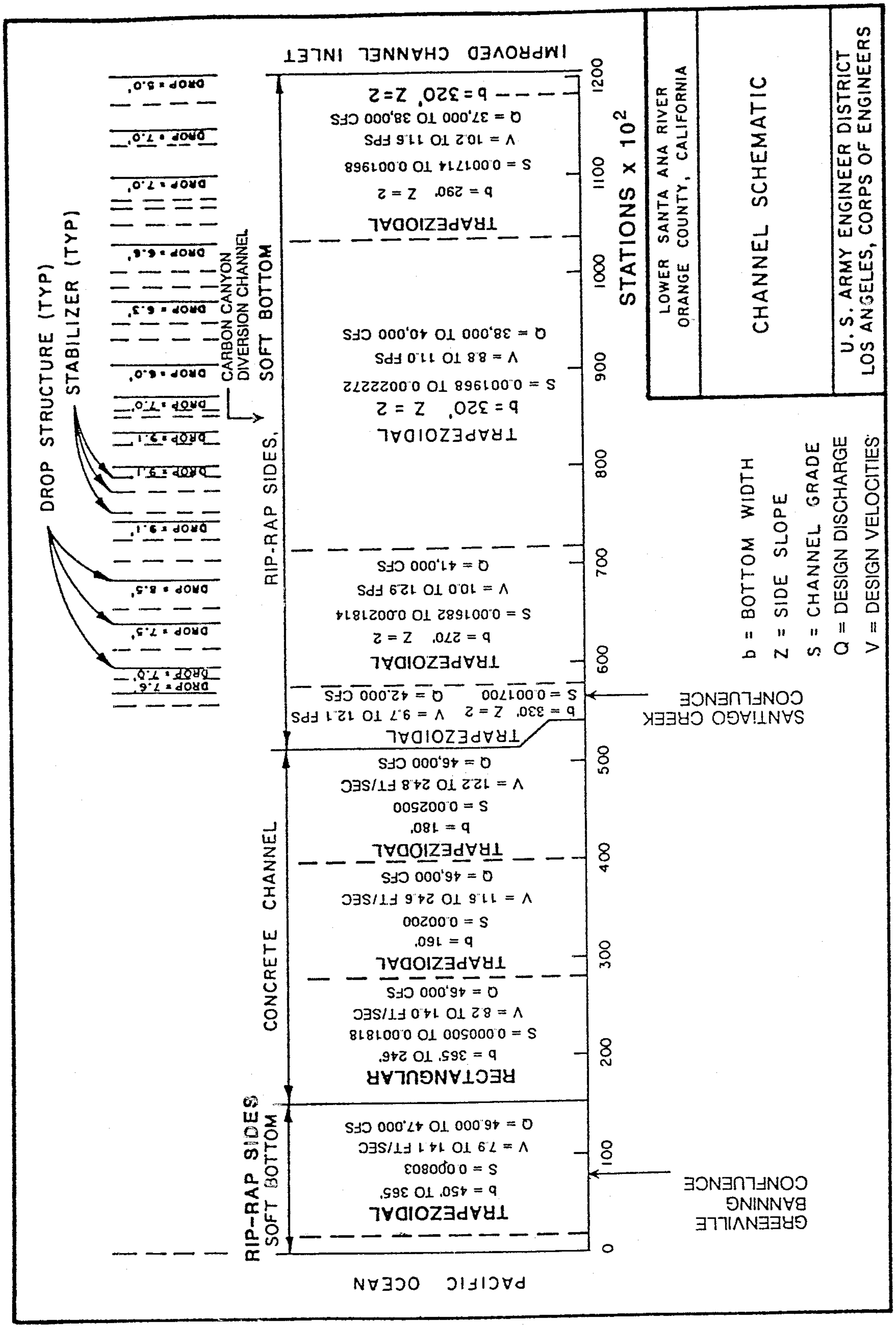

Drainage Area Boundary
Seven Oaks Dam
Mill Creek Levee
San Timoteo Creek
San Bernardino County
Riverside County
Los Angeles County
Orange County
Prado Dam
Oak Street Drain
Santiago Creek
Santa Ana Canyon
Lower Santa Ana River
Marsh Area
PROJECT SITE
PACIFIC OCEAN
N
15
10
91
5
55
605
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM
SANTA ANA RIVER
MAINSTEM PROJECT
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE 1



LOWER SANTA ANA RIVER CHANNEL

FIGURE 2



U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
CHANNEL SCHEMATIC
LOWER SANTA ANA RIVER
ORANGE COUNTY, CALIFORNIA
STATIONS x 10²
IMPROVED CHANNEL INLET
PACIFIC OCEAN
DROP STRUCTURE (TYP)
STABILIZER (TYP)
CARBON CANYON DIVERSION CHANNEL
RIP-RAP SIDES, SOFT BOTTOM
CONCRETE CHANNEL
RIP-RAP SIDES SOFT BOTTOM
SANTIAGO CREEK CONFLUENCE
GREENVILLE BANNING CONFLUENCE
b = BOTTOM WIDTH
Z = SIDE SLOPE
S = CHANNEL GRADE
Q = DESIGN DISCHARGE
V = DESIGN VELOCITIES

FIGURE 3

LEVEE FREEBOARD DESIGN AT FORT WAYNE, INDIANA

by

Darryl Dolanski[1]

INTRODUCTION

This paper describes the levee freeboard design which was performed by the Detroit District as part of its Fort Wayne, Indiana Flood Control Study. The freeboard analysis identified the factors important to the estimation of the water surface elevations at Fort Wayne, and attempted to quantify the uncertainty in the water surface elevations due to those factors. The objective of the freeboard design analysis was to ensure, as much as possible, a safe, functional flood control project at Fort Wayne.

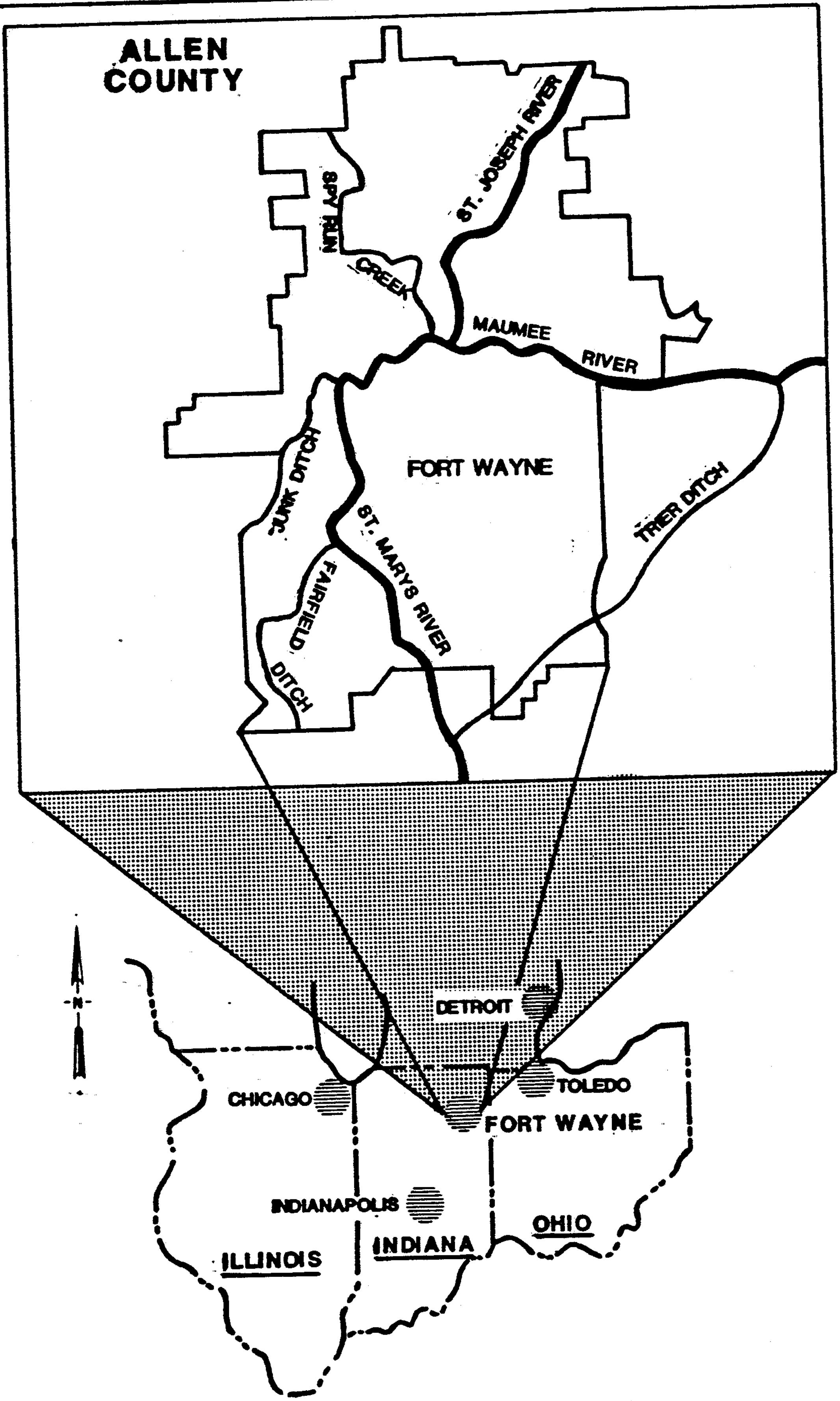
PHYSICAL SETTING AND AVAILABLE DATA

Geography. The City of Fort Wayne lies within the Maumee River basin, which covers an area of about 6,600 square miles in northwestern Ohio, northeastern Indiana and southeastern Michigan. This basin is one of the largest basins of the Great Lakes - St. Lawrence River system. The four main tributary streams are the St. Joseph, St. Marys, Tiffin and Auglaize Rivers. In Fort Wayne, the St. Marys and the St. Joseph Rivers join to form the headwaters of the Maumee River. The study area is shown in Figure 1.

The flood problem at Fort Wayne is the result of bank overflow of the St. Marys, St. Joseph and Maumee Rivers. The St. Marys and St. Joseph Rivers run along the extreme western edge of the watershed and each have a length of approximately 100 miles. The St. Marys River rises near the town of St. Marys, Ohio, and flows northwest to Fort Wayne, Indiana, draining an area of 840 square miles. The headwaters of the St. Joseph River originate in southern Michigan, and this stream flows generally in a southwesterly direction to Fort Wayne, where it joins with the St. Marys River to form the Maumee River. The drainage area of the St. Joseph River is 1,090 square miles. The Maumee River flows northeast through the central part of the Maumee watershed, and empties into Lake Erie at Toledo, Ohio. A basin map showing the location of each of these streams in the Maumee River watershed is presented in Figure 2.

Geology. Fort Wayne is located in a broad shallow valley marked by topography typical of the entire Maumee River basin, ranging from flat plains to rolling hills of low relief, with an elevation variance of 75 feet. Continental glaciation, extending as far south as the Ohio River, deposited mixed gravels, sands, clays and silts in terminal moraines that define the area's topography (Reference 1).

[1]Hydraulic Engineer, Detroit District, U.S. Army Corps of Engineers



FORT WAYNE/ALLEN COUNTY

FIGURE 1

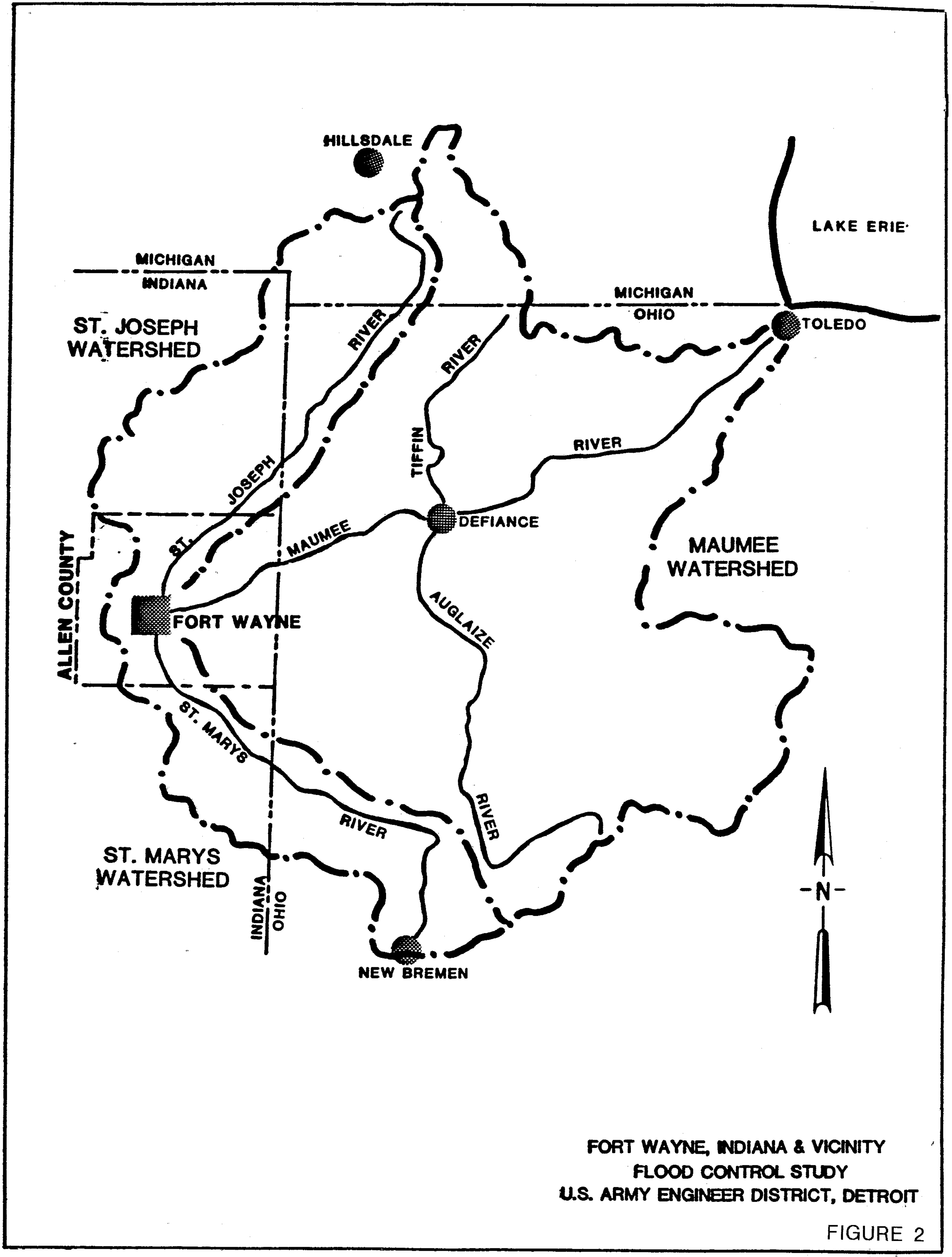

HILLSDALE
LAKE ERIE
MICHIGAN
INDIANA
MICHIGAN
OHIO
TOLEDO
ST. JOSEPH
WATERSHED
RIVER
RIVER
TIFFIN
RIVER
JOSEPH
ST.
MAUMEE
DEFIANCE
MAUMEE
WATERSHED
ALLEN COUNTY
FORT WAYNE
AUGLAIZE
ST. MARYS
RIVER
RIVER
ST. MARYS
WATERSHED
INDIANA
OHIO
NEW BREMEN
N
FORT WAYNE, INDIANA & VICINITY
FLOOD CONTROL STUDY
U.S. ARMY ENGINEER DISTRICT, DETROIT

FIGURE 2

Flood Potential. The history of Fort Wayne has been marked by periodic flooding problems. The worst flood occurred in March 1913 when the Maumee River crested at 26.1 feet, 11.1 feet above flood stage. Significant flooding also occurred in 1930, 1943, and 1950. A combination of rain and melting snow created the 1978 flood. All three rivers rose well above flood stage. The Maumee River reached 23.6 feet, 8.6 feet above flood stage before cresting. The March 1982 flood was caused by a combination of rainfall, snowmelt, and frozen ground conditions. Rainfall amounts from one to 1.5 inches accompanied the passage of a warm front through the Maumee River basin above Fort Wayne, which caused rapid melting of the snowpack on the basin. The Maumee River crested at 25.9 feet, only slightly below the record crest of 26.1 feet which occurred in 1913. Major flooding again occurred in February 1985. A combination of heavy snow and rain followed by warm temperatures and more rain resulted in a crest of 24.9 feet on the Maumee River. The most recent major flooding occurred in early January 1991, when a combination of rainfall and snowmelt caused the Maumee River to rise to a crest of 24.0 feet at the National Weather Service gage site at Anthony Boulevard.

Available Data. The U.S. Geological Survey (USGS) and U.S. National Weather Service (NWS) maintain and publish records for four stream gaging stations on the Maumee, St. Marys and St. Joseph Rivers in the vicinity of Fort Wayne, Indiana. Table 1 is a summary of the data recorded at these gaging stations.

TABLE 1
SUMMARY OF FLOW RECORDS FOR THE
MAUMEE RIVER BASIN AT FORT WAYNE, INDIANA AND VICINITY

Stream and Station Location	Drainage Area (Square Miles)	Period of Record	Peak Flow		Minimum Flow		Average Annual Flow (cfs)
			Date	Q(cfs)	Date	Q(cfs)	
Maumee River							
Landin Road (USGS)	1967	Dec 1946 to date	Mar 1982	26600	Oct 1963	48	1680
Anthony Blvd (NWS)	1926	May 1923 to date	Mar 1982	26150**	----	--	----
St. Marys River							
Near Fort Wayne (USGS)	762	Nov 1930 to date	Feb 1959	13600	Oct 1934	3	580
St. Joseph River*							
Near Fort Wayne (USGS)	1060	Jul 1941 to 1955, Oct 1983 to date	Mar 1982	17000**	Aug 1941	27	1000

* Gage was moved 4.5 miles upstream and maintained from Oct. 1955 to 1981.
** Estimated by running backwater computations to reproduce the observed March 1982 high water profile.

Existing Flood Measures. Throughout its history, the City of Fort Wayne has undertaken numerous flood control and flood damage reduction measures. The existing levee/floodwall system built by the city extends over approximately 35,000 feet, and has been raised and repaired as needed periodically following large floods. Other completed efforts have included additional gaging stations, the installation of flap gates on over 200 sewer outfalls, floodproofing of various structures, home and land acquisition, installation of a flood alert system, and development of a flood emergency response plan. In addition, the county is currently widening a four-mile reach of the Maumee River immediately downstream of Fort Wayne. Widening is being accomplished by excavating from the north overbank of the river, and is scheduled for completion in 1992. Since the Maumee River widening is currently under construction, it was incorporated into the existing conditions for purposes of the District's flood control study.

Recommended Flood Control Plan. As part of the Feasibility study phase, several flood control and flood damage reduction alternatives were investigated. The structural alternatives investigated included levees and floodwalls, channel modifications, and diversion of the St. Marys River flood flows via Trier Ditch. The nonstructural measures included floodplain evacuation, floodproofing, flood insurance and floodplain regulation.

Under the recommended plan, the existing line of protection would be increased in height to protect floodprone areas up to the design flood. New levees would be constructed only where necessary to tie the heightened existing protection into high ground after allowing for freeboard. Where space requirements preclude levees, concrete floodwalls or I-walls would be built. The total length of the proposed line of protection is approximately 53,000 feet. A map showing the location of the existing and proposed line of protection under the recommended plan is presented in Figure 3.

FREEBOARD DESIGN - STUDY APPROACH

Background. Levee freeboard was originally designed during the Feasibility Phase of the flood control study in accordance with current Corps guidance (Reference 2). The design was based on a minimum freeboard of three feet, which has been the traditional Corps of Engineers practice in leveed urban areas. Additional freeboard was added to the minimum three foot amount based on the superiority concept discussed in the guidance.

Based on discussions between the District, the Division, and OCE, the freeboard analysis, including the determination of the minimum freeboard amount, was reevaluated and subsequently revised for the current Design study phase.

Freeboard is included in the design of levees to allow for uncertainty factors in the assumptions used to develop water surface profiles, and to allow for small or intractable factors that are otherwise not specifically accounted for in the project design water surface profiles. These factors were evaluated to determine what uncertainty, if any, they would cause in the project design water surface profile, and therefore what amount of freeboard would be needed to account for them (Reference 3).

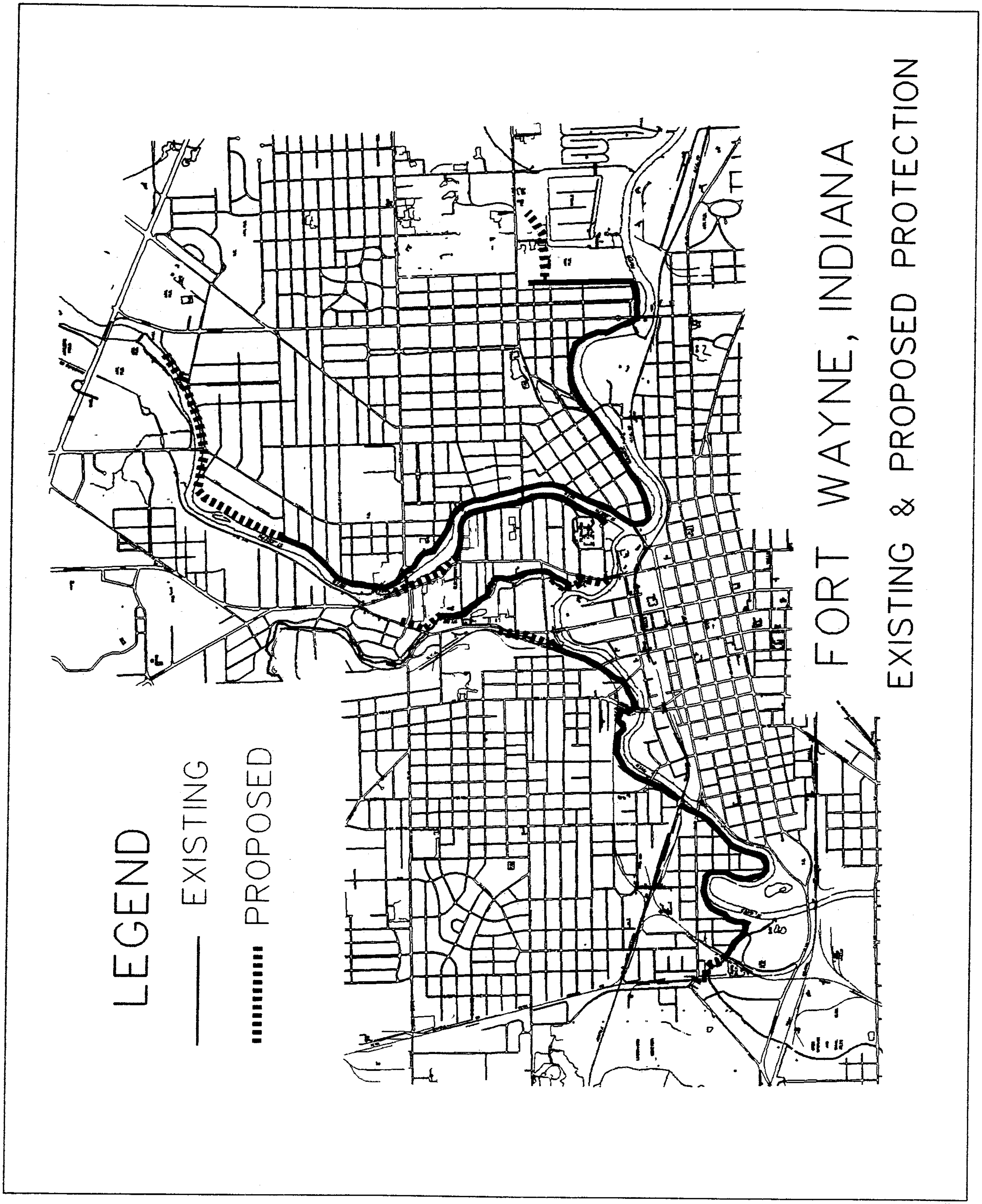

FORT WAYNE, INDIANA
EXISTING & PROPOSED PROTECTION
LEGEND
EXISTING
PROPOSED

FIGURE 3

Factors Which Influence Freeboard.

1. Data Availability. A considerable amount of stage and discharge data exists for the study streams in Fort Wayne. The Maumee, St. Marys and St. Joseph Rivers all have United States Geological Survey (USGS) stream gages near the project limits with significant periods of record. Additionally, the National Weather Service (NWS) has a gage on the Maumee River within Fort Wayne. Statistical analyses of the discharge records from these gages were performed to determine the project design flows.

Additional data in the form of surveyed high water marks were also available for use in the study. Approximately 100 high water marks were taken during the 1982 flood along the three rivers. The 1982 flood is the flood of record at the Maumee and St. Joseph River USGS gages, and is the third highest flood discharge measured at the St. Marys River gage. Another 80 high water marks were obtained during the 1978 flood, one of the largest on record. The high water marks from these two flood events were used by the Indiana Department of Natural Resources (IDNR) to calibrate the HEC-2 water surface profile models used in the study.

It is worth noting that the USGS obtained discharge measurements during the 1978 and 1982 flood events, which formed the basis of the HEC-2 calibrations. These measurements were taken at or very near the peak of each event.

2. Accuracy of Manning's "n" . A major factor impacting the determination of water surface elevations is the uncertainty associated with determining channel and overbank roughness coefficients (i.e. Manning's "n" values). In the flood control study, Manning's "n" coefficients were determined by the HEC-2 calibrations previously discussed. Due to the amount of data available, the calibrations, and therefore the Manning "n" coefficients which resulted, are considered to have a high degree of accuracy.

3. Channel Stability. Discussions with local representatives of the NWS and USGS, as well as a Fort Wayne official with twenty years of experience, indicate that river channels at Fort Wayne have historically remained stable. According to a USGS official, no long-term progressive trends have occurred in the rating curves of stream gages in the area. It was indicated that short-term scour and fill occurs in the rivers, as a result of sand bars that historically migrate progressively downstream. This phenomenon can affect the lower end of rating curves to cause minor upward or downward shifts of 0.5 to 1 foot. However, flood studies are more concerned with the upper end of rating curves, which are mainly affected by factors such as levees or bridges. The field measurements made by the USGS during the 1978 and 1982 flood events are generally in agreement with the most recent rating curves, further indicating the long-term stability of the stage-discharge relationships for these rivers.

4. Ice and Debris. The accumulation of ice or debris at a bridge or other constriction in a river can cause an increase in the water surface profile immediately upstream of the blockage. Discussions with the aforementioned city, NWS and USGS officials indicate that ice/debris jams have not occurred within the Fort Wayne study area during floods. Ice and debris jams sometimes occur, usually during normal flow, upstream of Fort Wayne, especially on the St. Joseph River. One such log jam occurred upstream of Fort

Wayne during the 1982 flood. Fort Wayne bridges are continually monitored for debris accumulation; the state and county have a formal debris removal program, which consists of clearing debris three to four times a year, including late fall and winter if accumulation is present. Flood watchers are also assigned to look for debris build-up at bridges in Fort Wayne during floods.

5. Other Traditional Factors. Other factors which were considered for their possible effects on project water surface elevations included waves, abrupt expansion or contraction of flow, and superelevation. Wind-generated waves do not occur on area rivers due to a lack of fetch. Since boat traffic only consists of canoes, man-made waves have a negligible effect. Flow expansions and contractions are gradual due in part to the non-constrictive nature of the bridges. Superelevation effects are negligible due to the gradual bends and tranquil flow in the rivers.

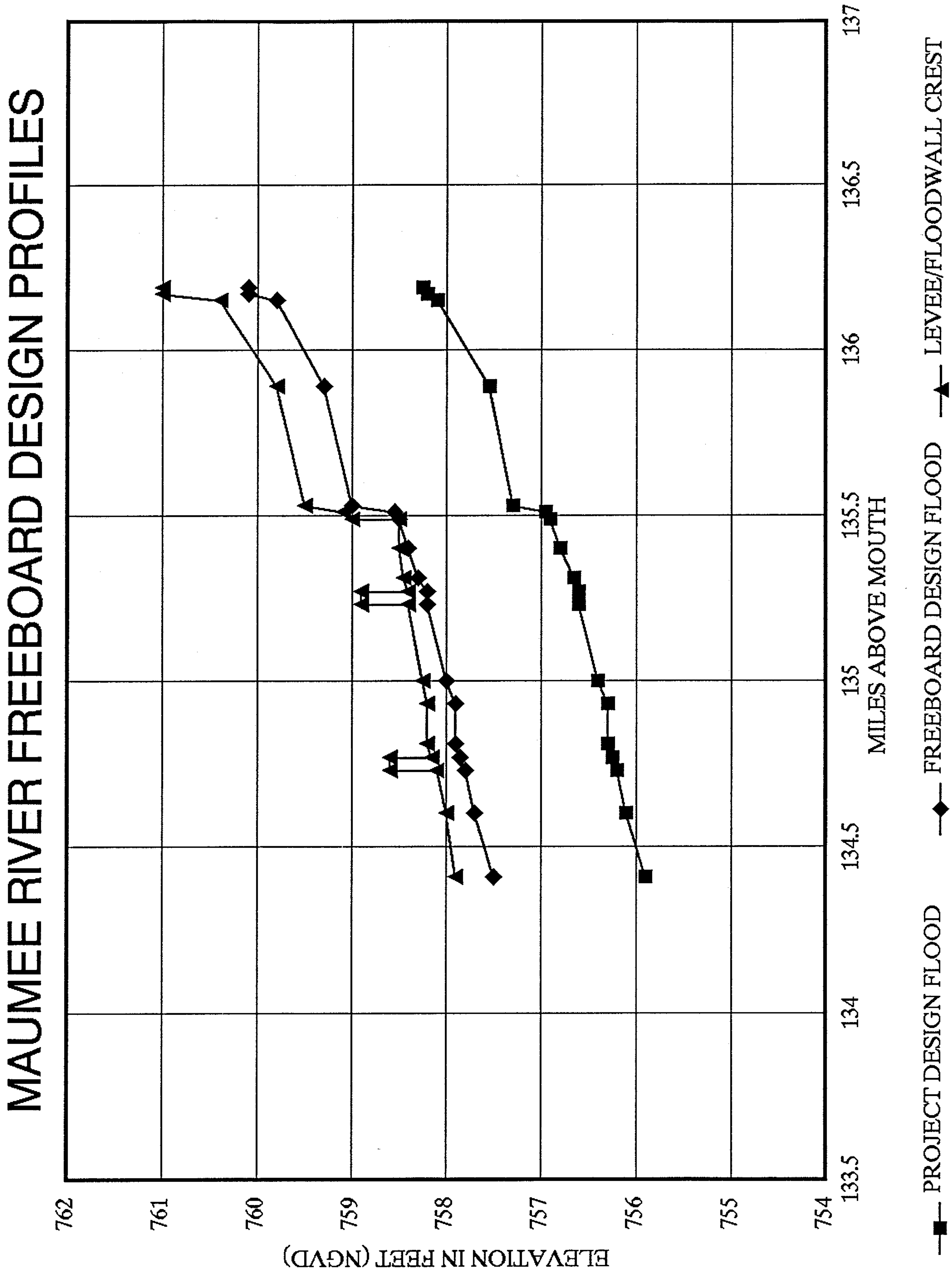
6. Natural Diversions. Two natural diversions, Junk Ditch and Paul Trier Ditch, exist on the St. Marys River. Junk Ditch is located at the upstream flood control project limit of the St. Marys River, while Paul Trier Ditch is located about nine miles further upstream, outside of Fort Wayne. These ditches flow into the St. Marys River when it is at low stage, and away from the St. Marys when it is at high stage, thereby becoming natural diversions. Junk Ditch diverts a portion of the St. Marys River flood flow over a low drainage basin divide southwest of Fort Wayne and into the Little River of the Wabash River drainage basin. Paul Trier Ditch diverts a smaller percentage of the St. Marys River flood discharge over a low divide and into the Trier Ditch, from which it flows into the Maumee River, approximately five miles downstream of the project limit. The effect of these natural diversions is to reduce peak flood stages downstream of the diversions.

FREEBOARD DESIGN - STUDY RESULTS

A strict interpretation of the impacts of the aforementioned factors on the water surface profile indicates that levee freeboard would not be required. Since this is unacceptable, uncertainty was introduced into some of the factors in order to arrive at the freeboard design. A specific discussion for each river follows.

Maumee River. Flood control protection is proposed along approximately two miles of the north bank of the Maumee River downstream from its confluence with the St. Marys and St. Joseph Rivers. The flood control project design water surface profile was developed based upon the HEC-2 model calibrated to the 1978 and 1982 floods, and is shown in Figure 4. This profile represents a 100-year discharge without the Maumee widening project. With the increased channel capacity due to the local widening project, this profile now represents a discharge in excess of a 150-year flow.

An initial levee overtopping location was selected approximately 300 feet downstream of Tecumseh Street, adjacent to Lakeside Park, based on its least damaging location and local sponsor preference.



MAUMEE RIVER FREEBOARD DESIGN PROFILES
ELEVATION IN FEET (NGVD)
762
761
760
759
758
757
756
755
754
MILES ABOVE MOUTH
133.5
134
134.5
135
135.5
136
136.5
137
PROJECT DESIGN FLOOD
FREEBOARD DESIGN FLOOD
LEVEE/FLOODWALL CREST

FIGURE 4

An alternative design water surface profile was developed for the Maumee River for the purpose of establishing the minimum levee grade at the initial overtopping location. The HEC-2 model was revised based on the assumption that energy losses could be greater than estimated, thereby increasing the design water surface profile. Specifically, Manning roughness coefficients were increased by 25 percent, expansion/contraction coefficients were increased at bridges up to 0.5 to 0.7 for expansion and 0.3 to 0.5 for contraction, and the Hosey Dam downstream of the initial levee overtopping location was re-modelled to have its gates closed with three feet of debris atop the closed gates.

Rating curves were developed for the initial overtopping location based on both the normal (i.e. project design) and high loss HEC-2 models, and are shown in Figure 5. Based on the difference in rating curves, the minimum or nominal freeboard would be 1.6 feet, and the freeboard design flood (FDF) would be 34,600 cfs, which is approximately the 500-year flood. The FDF represents the discharge that would be passed by the levee at its minimum grade assuming the project design HEC-2 modelling is accurate. This discharge was used to determine the freeboard design profile, which is shown in Figure 4.

Freeboard would be gradually increased by 0.4 foot downstream from the initial overtopping location, resulting in a total freeboard of two feet above the project design water surface elevation at the downstream project limit. This design would ensure that levee overtopping began at Lakeside Park and increased gradually. Upstream of Tecumseh Street, freeboard would be increased by 0.5 foot, with an additional increase of 0.4 foot for superiority upstream of Columbia Street. This would further ensure against overtopping in these highly-developed reaches. As a result, total freeboard would be 2.2 feet between Tecumseh and Columbia Streets, and 2.8 feet upstream of Columbia Street. Freeboard would be increased an additional 0.5 foot for superiority along 200 feet of the line of protection at the existing Morton Street pumping station (see Figure 4).

<u>St. Joseph River</u>. Flood control protection is proposed along approximately 2.8 miles of the east bank and 1.2 miles of the west bank upstream from the mouth of the river. The project design water surface profile represents a 100-year discharge without the Maumee widening project, and in excess of a 100-year discharge with the widening project considered.

The initial overtopping location was selected to be the west side of the river at the mouth, based on its downstream location, relative development and property values between the east and west sides of the river, and local sponsor preference. The minimum freeboard was determined as described previously for the Maumee River, and would be 1.8 feet. Freeboard would incrementally increase in an upstream direction based on superiority, reaching a maximum of 3.2 feet at the upstream project limit. The freeboard design for the St. Joseph River is summarized in Figure 6.

<u>St. Marys River</u>. Flood control protection is proposed along the west bank of the St. Marys River from its mouth to Junk Ditch, a distance of approximately 2.8 miles. The project design water surface profile represents a 100-year discharge without the Maumee widening, and approximately a 150-year discharge with widening considered.

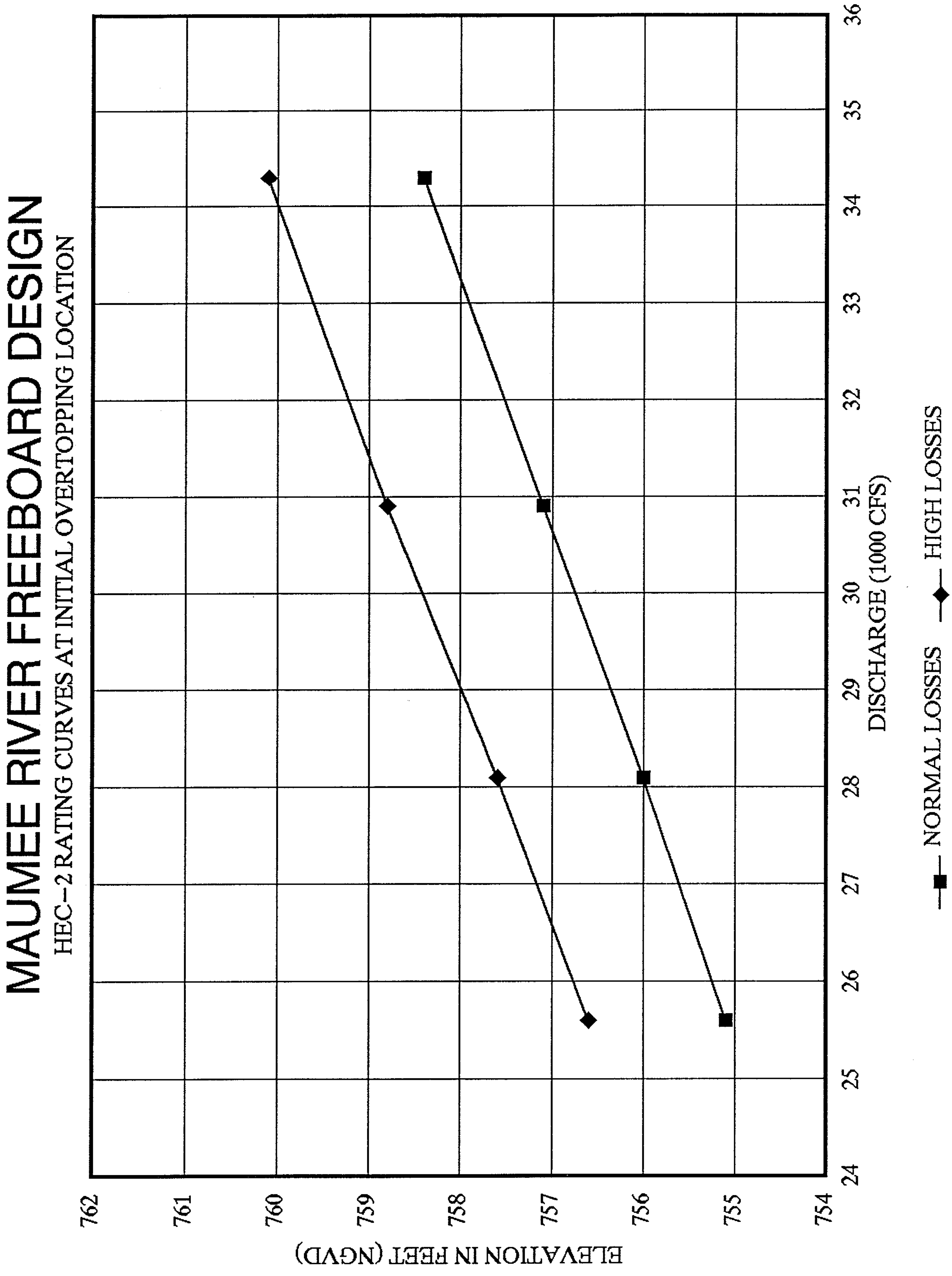

MAUMEE RIVER FREEBOARD DESIGN
HEC-2 RATING CURVES AT INITIAL OVERTOPPING LOCATION
ELEVATION IN FEET (NGVD)
762
761
760
759
758
757
756
755
754
24
25
26
27
28
29
30
31
32
33
34
35
36
DISCHARGE (1000 CFS)
NORMAL LOSSES
HIGH LOSSES

FIGURE 5

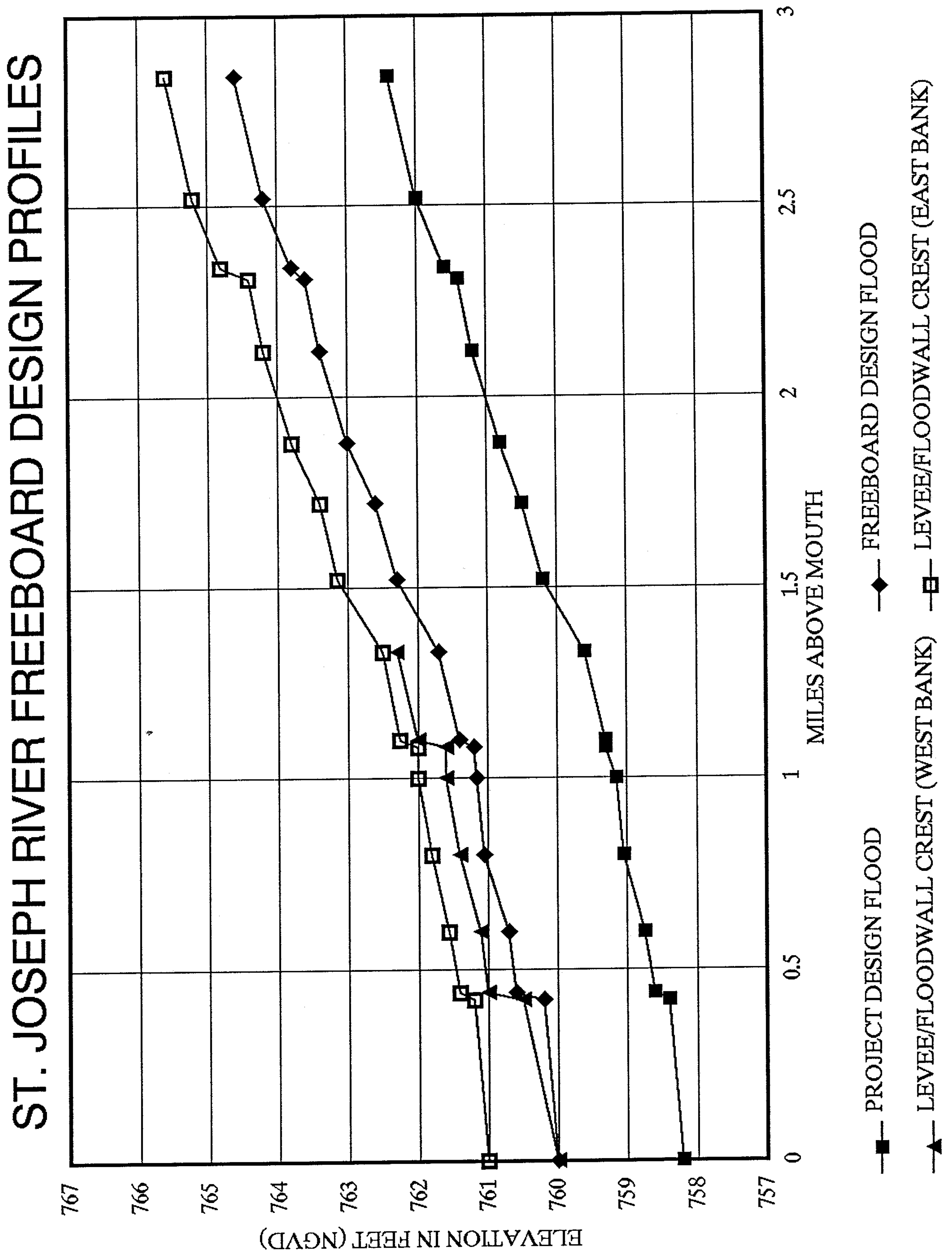

ST. JOSEPH RIVER FREEBOARD DESIGN PROFILES
ELEVATION IN FEET (NGVD)
767
766
765
764
763
762
761
760
759
758
757
MILES ABOVE MOUTH
0
0.5
1
1.5
2
2.5
3
PROJECT DESIGN FLOOD
FREEBOARD DESIGN FLOOD
LEVEE/FLOODWALL CREST (WEST BANK)
LEVEE/FLOODWALL CREST (EAST BANK)

FIGURE 6

The initial overtopping location was selected to be immediately upstream of Clinton Street, based on its relative downstream location, sparser development, and local sponsor preference. The minimum freeboard at this location would be 1.8 feet. Freeboard would incrementally increase in an upstream direction based on superiority, reaching a maximum of 2.5 feet at the upstream project limit at Junk Ditch. The freeboard design for the St. Marys River is summarized in Figure 7.

Spy Run Creek. Flood control protection is proposed along approximately one mile of the east bank of Spy Run Creek upstream from its mouth. The project design water surface elevation for the St. Marys River at Spy Run Creek exceeds the Spy Run Creek 500-year flood water surface profile throughout the protected reach. Therefore, the Spy Run Creek protection would function as a tie-back levee for the St. Marys River.

Based on the freeboard design analysis for the St. Marys River, the freeboard for the Spy Run Creek line of protection at its downstream limit would be 2.1 feet. Based on an assumption of complete non-coincidence of flow between Spy Run Creek and the St. Marys River, the line of protection could be extended horizontally to the upstream project limit on Spy Run Creek and still provide protection for the 500-year Spy Run Creek flood with two feet of freeboard. The assumption of complete non-coincidence of flow between Spy Run Creek and the St. Marys River is reasonable considering that the drainage areas of these two streams at their confluence are 15 and 824 square miles, respectively. However, if local runoff occurred in Spy Run Creek coincident with a high stage in the St. Marys River, it would cause a water surface gradient which could overtop the Spy Run Creek protection at its upstream end.

A sensitivity analysis was performed to design the upstream freeboard on Spy Run Creek. Water surface profiles were developed for Spy Run Creek for various discharge frequencies using a common starting water surface elevation equivalent to the proposed top of protection at the downstream limit. Based on this analysis, the 5-year discharge was selected to develop the freeboard design profile.

Freeboard would gradually increase to 2.8 feet at the upstream project limit to minimize overtopping potential at that end of the line of protection. The freeboard design for Spy Run Creek is summarized in Figure 8.

The overall freeboard design is summarized in Table 2. The freeboard as it was originally designed in the earlier Feasibility study is also shown for comparison.

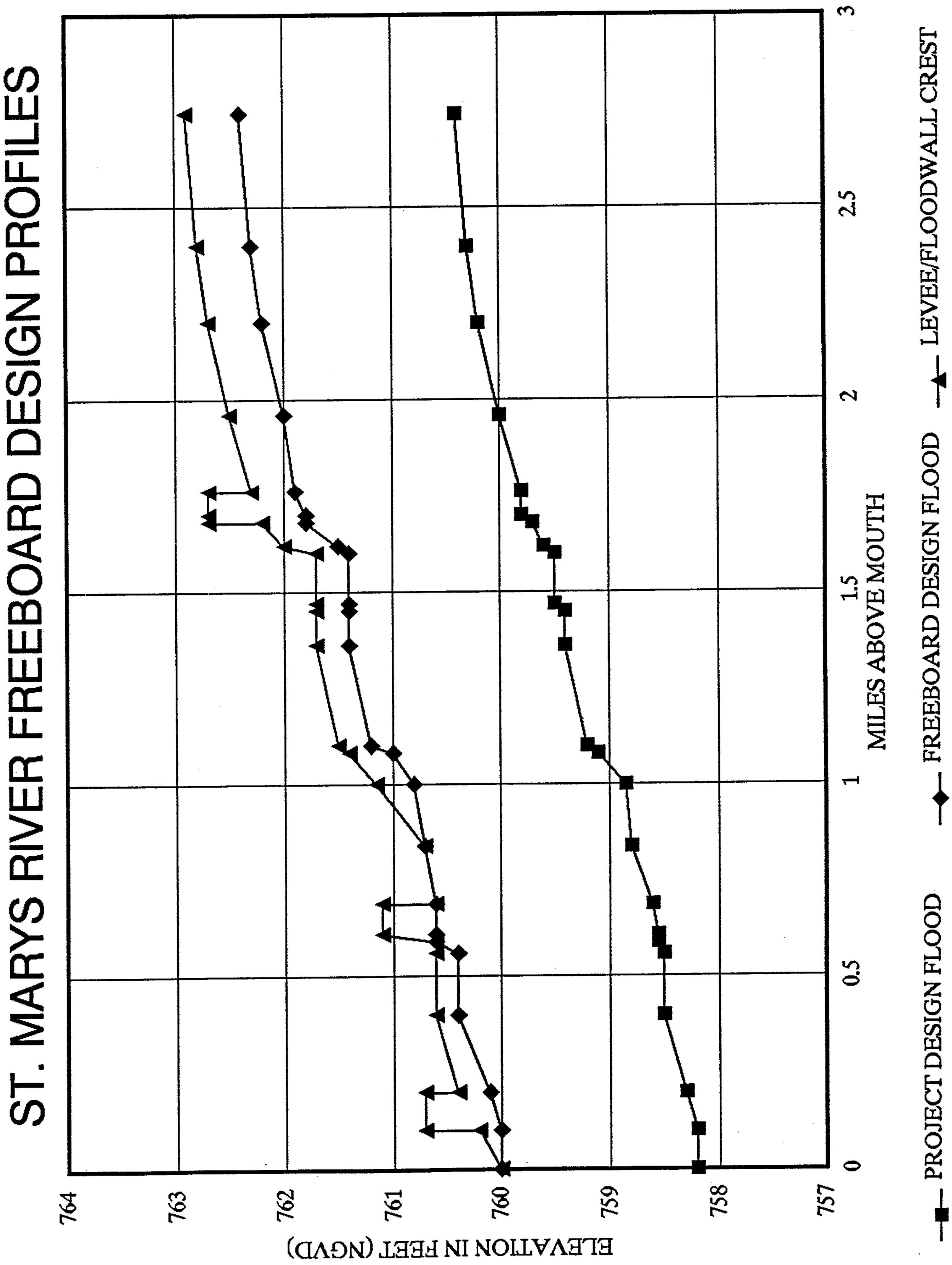

ST. MARYS RIVER FREEBOARD DESIGN PROFILES
ELEVATION IN FEET (NGVD)
764
763
762
761
760
759
758
757
0
0.5
1
1.5
2
2.5
3
MILES ABOVE MOUTH
PROJECT DESIGN FLOOD
FREEBOARD DESIGN FLOOD
LEVEE/FLOODWALL CREST

FIGURE 7

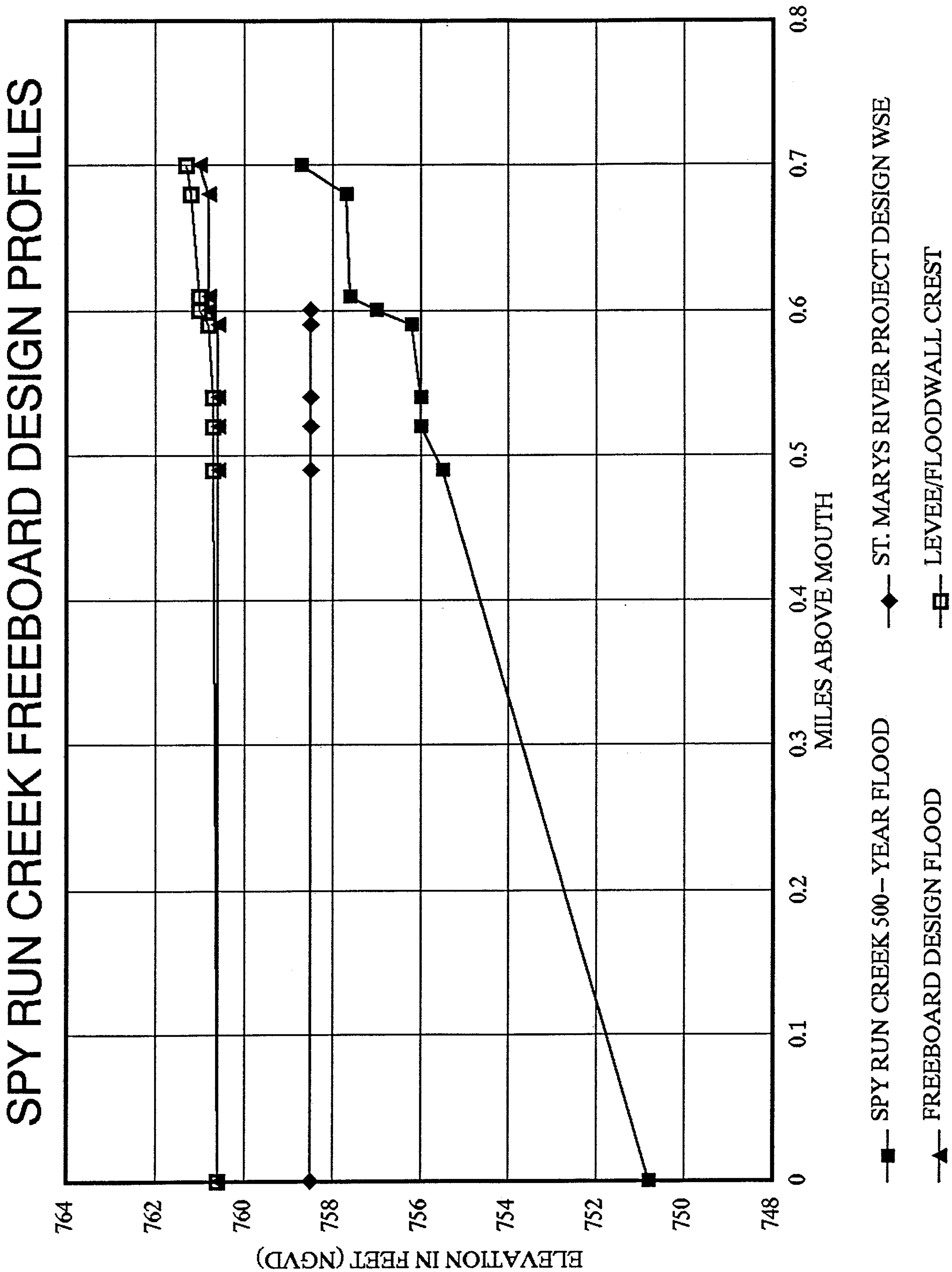

SPY RUN CREEK FREEBOARD DESIGN PROFILES
ELEVATION IN FEET (NGVD)
764
762
760
758
756
754
752
750
748
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
MILES ABOVE MOUTH
SPY RUN CREEK 500- YEAR FLOOD
ST. MARYS RIVER PROJECT DESIGN WSE
FREEBOARD DESIGN FLOOD
LEVEE/FLOODWALL CREST

FIGURE 8

TABLE 2
LEVEE/FLOODWALL FREEBOARD

Stream	Reach	Redesigned Freeboard[1]	Original Freeboard
MAUMEE RIVER			
	Downstream project limit to Tecumseh St	2.0 to 1.6	3.0
	Tecumseh St to Columbia St	2.2	3.5
	Upstream of Columbia St	2.8	3.8
ST. JOSEPH RIVER			
	Mouth to Tennessee St	1.8 to 2.1[W];2.8[E]	3.0[W];3.8[E]
	Tennessee St to State St	2.4[W];2.8[E]	3.5[W];3.8[E]
	State St to Parnell St[2]	2.7[W];3.0[E]	4.0
	Upstream of Parnell St	3.2	4.0
ST. MARYS RIVER			
	Mouth to Spy Run Creek	1.8 to 2.1	3.0
	Clinton St to Harrison St	1.8	3.0
	Harrison St to Wells St	1.9 to 2.3	3.0
	Wells St to Norfolk & Western RR	2.3	3.5
	Upstream of Norfolk & Western RR	2.5	4.0
SPY RUN CREEK			
	Mouth to Elizabeth St	2.1 to 2.3	3.0
	Elizabeth St to Railroad Spur	2.3 to 2.5	3.5
	Railroad Spur to Clinton St	2.5 to 2.8	3.5

[1] Feet above project design water surface elevation. An additional 0.5 foot of freeboard would be added within 100 feet of pump stations.
[2] West side protection terminates 500 feet north of State St.
[E] East side of river.
[W] West side of river.

CONCLUSIONS

Freeboard is an important component of the overall levee/floodwall design, and as such, it should be thoughtfully designed rather than just assumed. This was only partially accomplished during the Feasibility phase of the Fort Wayne Flood Control Study, since the minimum freeboard was assumed to be three feet based on traditional Corps practice, instead of being designed. Levee freeboard was reevaluated during the current project design phase, based upon the current Corps guidance, engineering judgment, and the collective experience of local and Federal officials knowledgable of the system. The revised freeboard design resulted in a cost savings to the Federal project, and was more favorably received by the local sponsor, which had desired lower levee/floodwall heights.

REFERENCES

1. "Final Feasibility Report, Fort Wayne and Vicinity, Indiana Flood Control Study," U.S. Army Corps of Engineers, Detroit District, September 1987.

2. ETL 1110-2-299, "Overtopping of Flood Control Levees and Floodwalls," by Lewis A. Smith and Thomas E. Muncey, U.S. Army Corps of Engineers, Washington, D.C., 22 August 1986.

3. Civil Works Engineer Bulletin 54-14, "Improvements in Design and Construction Practices in Civil Works," U.S. Army Corps of Engineers, Washington, D.C., 23 April 1954.

HIGH VELOCITY LEVEED CHANNELS - PUERTO RICO

by

Ronald E. Hilton[1], Eric P. Holand[2], Michael L. Choate[3]

INTRODUCTION

Puerto Rico is a small, beautiful, fascinating island in the Caribbean Sea. Puerto Ricans have a four-century heritage of Spanish language and culture. Christopher Columbus reached Puerto Rico on November 19, 1493. They have been a part of the United States since 1898, United States citizens since 1917, and a Commonwealth of the United States since 1952.

Puerto Rico lies approximately 1,000 miles southeast of Miami, Florida. It lies on the northeast periphery of the Caribbean Sea, part of an elongated cluster of 7,000 tropical islands of varying shapes and sizes, which are called the West Indies. The West Indies are divided into three main groups: the Bahamas, the Greater Antilles and the Lesser Antilles. Puerto Rico is the easternmost and smallest of the Greater Antilles, which also include Cuba, Jamaica, and Hispaniola.

Shaped somewhat like a parallelogram, Puerto Rico measures 111 miles east-west and 36 miles north-south. Together with its smaller offshore islands, as well as several keys and islets, Puerto Rico's land area is 3,435 square miles, roughly the size of Connecticut.

There is an astonishing variety of landscapes and geological formations on the island of Puerto Rico. They include flat coastal plains, tightly knotted mountains, strange limestone "haystacks" and "sinkholes", sun-bleached desert land (replete with cacti), and shady, rain-soaked tropical forests in the Cordellera, the island's central mountain range. Cerro de Punta (4,389 feet) is the tallest mountain. The best known is El Yunque, which rises 3,494 feet above the Rain Forest, and is almost always shrouded in gray mist.

Puerto Rico has more than 1,000 waterways, but only 50 of them are large enough to be classified as rivers. All rivers are short, none navigable by large vessels. The longest is Rio de la Plata (46 miles).

Puerto Rico is summery all year. Though the island is in the tropics, the steady tradewinds from the northeast keep temperatures at about 80 degrees in summer and in the 70's during the winter. Average islandwide temperature

---

[1]Chief, Hydrology and Hydraulics Branch, Engineering Division, Jacksonville District, U.S. Army Corps of Engineers

[2]Chief, Hydraulic Data and Design Section, Hydrology and Hydraulics Branch, Jacksonville District, U.S. Army Corps of Engineers

[3]Chief, Hydrologic Investigations Section, Hydrology and Hydraulics Branch, Jacksonville District, U.S. Army Corps of Engineers

is 77 degrees. Although some 400 billion cubic feet of rain fall on Puerto Rico each year, a completely sunless day is rare (only five days a year can be expected without sunshine). Rainfall varies by region; from a mere 29 inches yearly in the parched south to 108 inches in the highland. The Rain Forest truly lives up to its name: 1600 showers yearly, more than four per day, drop 180 inches of rain, dousing the slopes, where the ferns grow to heights of 30 feet, orchids bloom profusely, and birds dart through the foliage of 200 different types of trees.

Although the entire Caribbean area is in the zone of an active fault movement, there have been no serious earthquake shocks near Puerto Rico since 1918. October 11, 1918 was the worst recorded earthquake for Puerto Rico. Vibration and tidal waves took 116 lives and caused millions of dollars in damage along the northwest coast.

Puerto Rico's hurricane season is June through November. Since the year 1508, when record-keeping commenced, Puerto Rico has been hit by 74 hurricanes. In this century, eight hurricanes have been recorded. The most recent hurricane to affect the island was Hugo in 1989.

Division of the island by cardinal points yields the North with 54 percent of the area and 59 percent of the annual runoff; the South with 23 percent of the area and eight percent of the runoff; the East with six percent of the area and 10 percent of the runoff and the West with 17 percent of the contributing drainage area and 23 percent of the runoff.

The unique topography and climate of Puerto Rico plays an important role in the design of freeboard for leveed high velocity channels. The hydrology affects the storm hydrograph characteristics and wind and wave setup. While the hydraulics are impacted by overtopping sites, erosive velocities and critical depths. This paper will touch on some of these factors and share with you our approach in dealing with them.

HYDROLOGY

General. Even though hydrologic characteristics may not directly contribute to freeboard estimates, the runoff response time, storm volume and routing characteristics influence the methods used to estimate freeboard.

Watersheds. Study areas and contributing watersheds usually vary between 20 and 200 square miles. The orographic effect of the central mountain range produces high rainfall amounts with common characteristics of fast runoff response times. For example, the Cerrillos and Bucana Rivers with a 17.5 square mile watershed, peaks in about 2.5 hours with very steep runoff hydrograph limbs. The Cerrillos and Bucana Rivers were recently impounded by the completion of the Cerrillos Dam, a 323 feet high by 1555 feet long rock and earthfill dam. In 1975, about the time we were initiating the design hydrology for the Cerrillos Dam, we were fortunate (although the people downstream may not have felt that way) to have a period-of-record high peak discharge of 22,000 cfs. Also, to show how peaked the hydrographs are, the river had an average daily flow of 4500 cfs. The 1975 storm was widely accepted as a 100-year event by the U.S. Geological Survey. Our hydrology for the dam indicated that the 1975 flood was about an 8-year event. Another flood in 1985 produced a maximum discharge of about 24,000 cfs. Therefore, during the 20 years we have gaged the basin, the locals had the perception that two "100-year events" had occurred. As the record has been extended, our original design based on long term rainfall has proven reliable. Figure 1 shows a typical runoff hydrograph. The steep ascent of the leading edge of the hydrograph does not provide adequate time to warn local officials. By the time rising flood waters indicate an overtopping situation is imminent, flood warning is not practical.

From the description above, one can imagine these very peaked hydrographs with limited volumes forming gravity waves and coming out of the mountains with very high velocities. Our studies have shown that the storm volume is more than sufficient to inundate and fill the protected leveed areas. Levee projects in Puerto Rico are designed to protect only those areas that can be incrementally justified. It is not uncommon for the tieback portion of a levee to be moved upstream, closer to the high value developments, because the added protected lands could not carry the cost of levee extensions.

Storm Types. Design hydrology in Puerto Rico is heavily influenced by the type of storms creating runoff. The passage of frontal storms produce intense short duration rainfall and peaked runoff hydrographs. However, the duration of these events are usually short and early rainfall is infiltrated and intercepted. These storms do not generally produce the design storms. In contrast, rainfall associated with tropical disturbances and hurricanes produce most of the significant flood events with durations over 6-hours. Rainfall can precede hurricane passage by five days and deplete soil storage, water supply dam storage and karstic capacity. A wet antecedent condition is used for all design storms with exceedance probabilities of 0.04 or less.

FIGURE 1

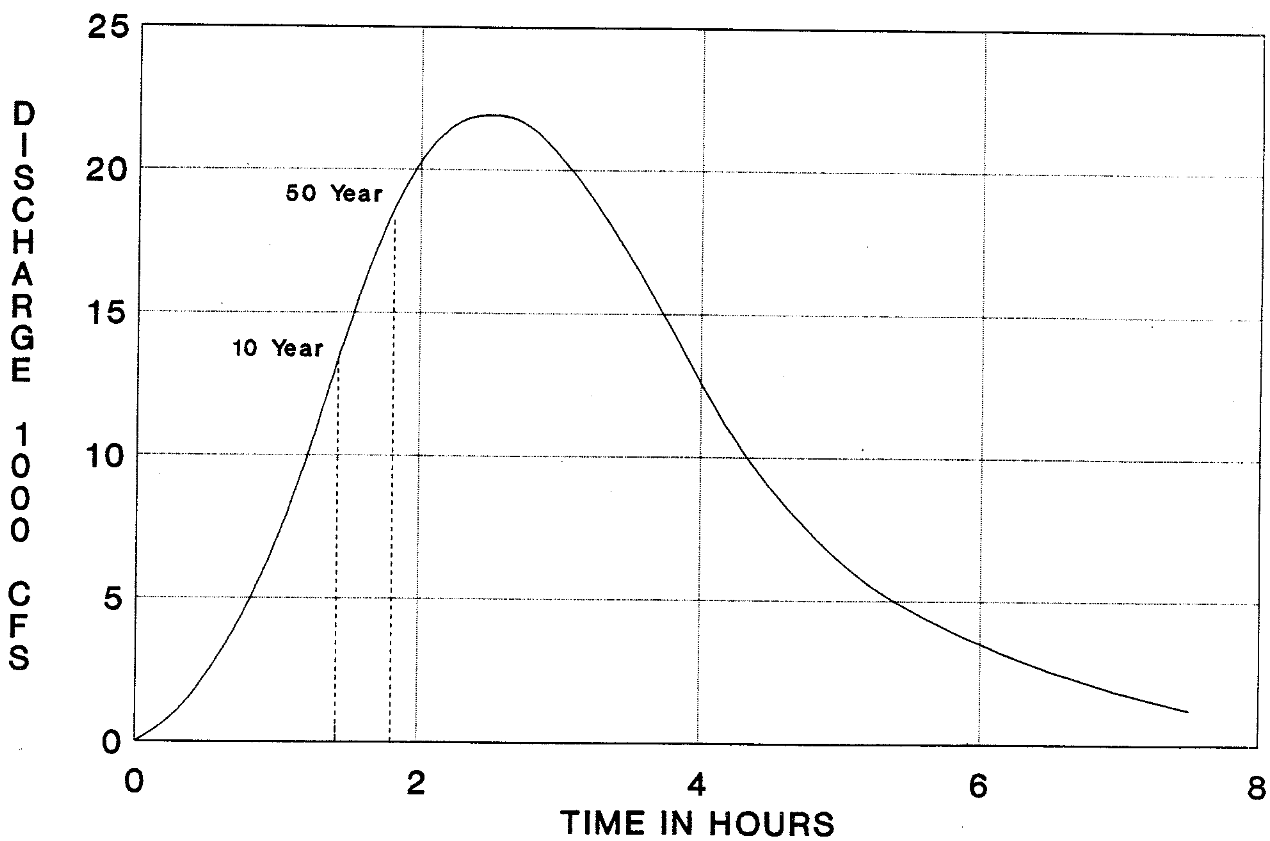
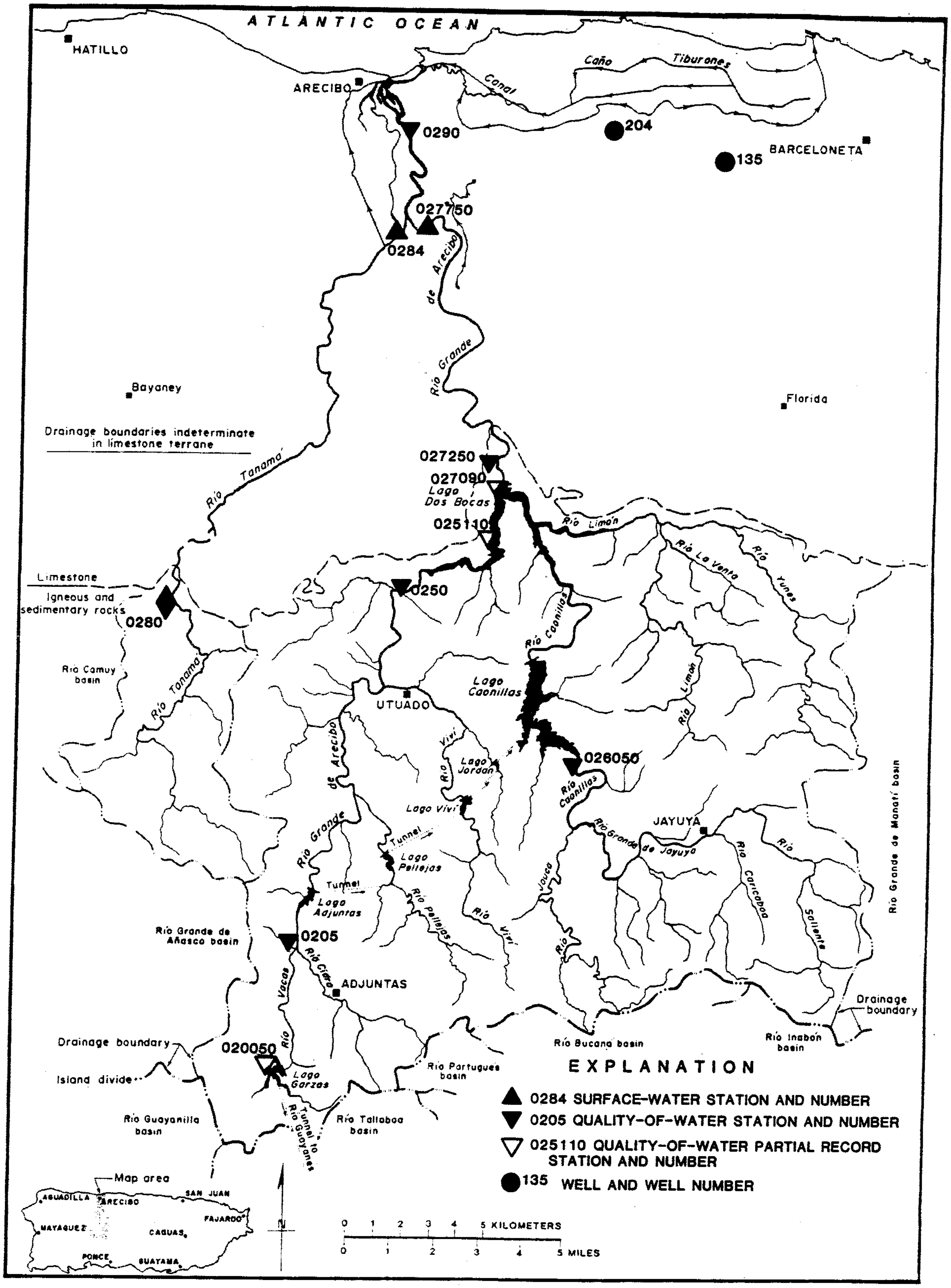
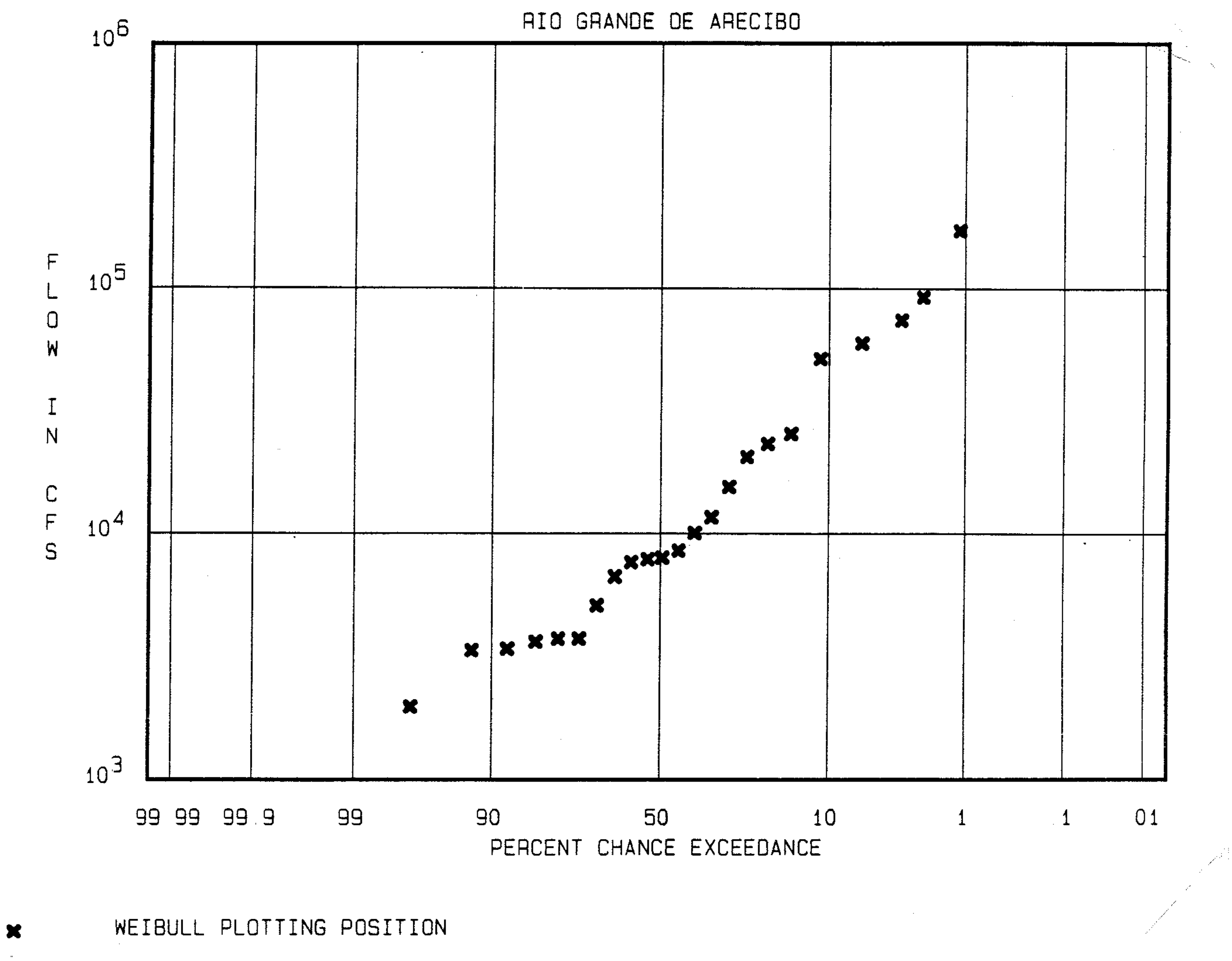


FIGURE 2


ATLANTIC OCEAN
HATILLO
ARECIBO
Caño Tiburones
Canal
0290
204
135
BARCELONETA
027750
0284
Río Grande de Arecibo
Bayaney
Florida
Drainage boundaries indeterminate in limestone terrane
Río Tanamá
027250
027090
Lago Dos Bocas
025110
Río Limón
Río La Venta
Río Yunes
Limestone
Igneous and sedimentary rocks
0280
0250
Río Caonillas
Río Camuy basin
Río Tanamá
Lago Caonillas
UTUADO
Río Limón
Río Viví
Lago Jordan
026050
Río Caonillas
Lago Viví
JAYUYA
Río Grande de Jayuya
Río Caricaboa
Río
Río Grande de Manatí basin
Tunnel
Lago Pellejas
Tunnel
Lago Adjuntas
Río Pellejas
Río Viví
Río Jauca
Río
Saliente
Río Grande de Añasco basin
0205
Río Vacas
Río Cidra
ADJUNTAS
Drainage boundary
Río Bucaná basin
Río Inabón basin
Drainage boundary
020050
Island divide
Lago Garzas
Río Portugues basin
EXPLANATION
0284 SURFACE-WATER STATION AND NUMBER
0205 QUALITY-OF-WATER STATION AND NUMBER
025110 QUALITY-OF-WATER PARTIAL RECORD STATION AND NUMBER
135 WELL AND WELL NUMBER
Río Guayanilla basin
Tunnel to Río Guayanes
Río Tallaboa basin
Map area
AGUADILLA
ARECIBO
SAN JUAN
FAJARDO
MAYAGUEZ
CAGUAS
PONCE
GUAYAMA
N
0 1 2 3 4 5 KILOMETERS
0 1 2 3 4 5 MILES

Gage Records. Well documented stream gage records in Puerto Rico date back to the mid-1960's. Prior to that time, the record is historical and incomplete. However, rainfall records are available from the turn of the century and offer the most reliable statistical data source. Stream gage records do provide insight into the physical characteristics of the watersheds. For example, Figure 2 is a reprint of the Rio Grande de Arecibo watershed map from the U.S. Geological Survey, Water Resources Data - Puerto Rico and the U.S. Virgin Islands. Of the 250 square mile drainage area, approximately 60 square miles is considered non-contributing karstic areas. The runoff contribution of this area is a function of the karstic infiltration capacity. The karstic capacity is the infiltration capacity of exposed limestone in portions of the contributing watersheds. Solution cavities in the limestone accept high infiltration rates. However, after the infiltration capacity is reached the area begins to produce runoff. This phenomenon is evident (Figure 3) in historic steam gage records and must be considered when using rainfall records on ungaged basins.

FIGURE 3



OVERTOPPING CONCERNS

General. Project formulation in Puerto Rico poses certain unique characteristics. Remote sites and quick response times prevent any realistic warning times. Island-wide remote sensing is being implemented. However, the physical characteristics of the basins indicate the warning times will be short. Maintenance of plant equipment will also be difficult or nonexistent because of the climate and the lack of skilled technicians at small towns and remote sites.

Failure Location. Current procedures call for design of levee grades by studying sensitivity of design water surface elevations to roughness changes in the channel or floodway. Changes in roughness values occur due to vegetation or development of bedforms in the flow-way. Bridges in Puerto Rico are subject to buildup of floating debris which can become lodged against bridge piers during storm events.

Once a calibrated existing conditions computer model has been adjusted to reflect the impact of the levee on the stages and velocities in the floodway, the levee grade is designed by increasing roughness values to a level which reflects the maximum expected changes to the channel or floodway. Existing bridges are modeled by increasing the thickness of each bridge pier by an additional 4 feet to simulate debris buildup. The resulting water surface profile will identify the locations which could be designated to "fail" or be overtopped before other locations. Results of this analyses can reveal where levee superiority would be needed to eliminate flood damage due to overtopping. Most bridges in Puerto Rico are designed by the Jacksonville District with no piers in the channel. However, if piers are required, they are designed with pier extensions to minimize or eliminate debris buildup.

Duration of Overflow. Duration of overflow is determined by modeling a series of discharges to determine the discharge that would overtop the levee. The overtopping discharge is then plotted on the discharge axis of the hydrographs of various frequency storms. The volume of the hydrograph above that discharge is the volume which could be expected to pass over the levee. Armoring the levee at the point of overflow may be prudent to decrease the erosion potential. The duration of runoff hydrographs in Puerto Rico usually produce short expected overflow times. Properly grassed and maintained levee tops offer resistance of discharges up to 8 fps. This limits the erosion potential.

Depth of Overflow. Depth of overflow is determined by modeling lower frequency storm events in the design of the floodway. The results of an overtopping analyses should show the station and discharge at which the levee would be overtopped. Once the overtopping discharge is identified, the next lower storm frequencies should be reviewed to determine the volume (acre-feet) of water which would be able to leave the channel or floodway. The top of levee is considered a long weir. The peak stage in the floodway minus the top of the levee grade is the depth of water over the weir crest. The peak discharge is then computed from the weir equation.

Overtopping. The location of overtopping is usually designed to pass into an area which has been designated as a storage or ponding area for collection of upland runoff. Those areas are generally drained by a project structure such as a flapgated culvert. Areas which have high material value or possible loss of life are avoided as overtopping locations. Flooding of benefit areas can be prevented by construction of lateral tieback levee segments which connect the main flood control levee with high ground. Levee heights for tieback levees are set by designing freeboard based on criteria outlined in EM-1110-2-1601.

HYDRAULIC METHODS

Sensitivity to Roughness. The slopes of floodways in Puerto Rico produce velocities that are very high and stages that are sensitive to changes in hydraulic radius and roughness. Consequently, most channel analyses and designs in subcritical or supercritical flow regimes have high velocities. Stage differences of several feet can result if roughness values are not chosen correctly and project channels and levees are long. Formulation and subsequent calibration of existing condition hydraulic models are a very important step in determining pre-project roughness values which are used in overbank discharge conveyance.

Subcritical Flow Regime. Determination of Manning's "n" roughness values depend on the condition of the flow-way being analyzed and on the type of flow regime. Guidance available for determining roughness values can be found in various publications. The hydraulic designer should be sensitive to maintaining as much of the existing flow regime as possible by maintaining a "low flow channel". The resulting subcritical project design often calls for a combination of features incorporating a project channel and a levee system.

Assigning roughness to specific areas within a floodplain should be done after a careful inspection of current and expected future land use and density of vegetation in the area. Roughness values as high as $n=0.18$ are often used in calibration of an existing condition hydraulic model. Computed Manning's roughness values chosen for project channels in Puerto Rico without vegetation range from 0.03 to 0.044.

If the project levee is long, differences in roughness values can produce differences in stages of several feet. Analyses of roughness values on channel and levee design for Rio de La Plata produced stage differences up to 6 feet within 9 miles with a 25% change in roughness. Consequently, roughness value changes should be taken into consideration when designing levee systems.

Super-elevation at curves in channel alignments should be computed for design velocities and added to computed water surface elevations. The Jacksonville District uses the Cowen method as presented in Chow (1959) to determine super-elevation at curves. The Cowen method includes bend losses in determination of roughness values at curves in the channel. The authors of this paper do not recommend a single roughness value which includes unchannelized and or unmaintained sections of a floodway. We use the roughness value(s) commensurate with the type of vegetation or project feature in the floodway.

Supercritical Flow Regime. Supercritical flow regime channel designs offer the benefits of decreased land requirements. However, it requires construction of concrete channels to prevent erosion. The designs are analyzed with friction losses determined from criteria and procedures outlined in EM-1110-2-1601. Design water surfaces are designed with an equivalent roughness coefficient of k=0.007. Maximum velocities are determined by recomputing roughness values with k=0.002. Super-elevation heights for channel curves are computed for both roughness coefficients. Channel walls are chosen from the analyses which produce the highest stages. Under most circumstances, flow depths with super-elevation computed using k=0.007 will exceed flow depths with super-elevation computed using k=0.002. That was the case for the entire high velocity channel reaches for the Rio Puerto Nuevo. Velocities in curves for that project ranged from 26 to 30 fps.

Confluences. Physical model studies of the Rio Puerto Nuevo high velocity channels showed that highly unstable flow patterns would be generated unless the design water surface elevations at the confluence of the two channels were designed to be at the same stage. Since peak discharges for both channels rarely happen at the same time, the final design must consider coincident discharges for peak flow in the main channel and the tributary separately.

Confluence design is always based on momentum analyses. A centerline offset type configuration was shown for Rio Puerto Nuevo to provide the least turbulent flow conditions. Stepped bottom elevations are usually required to achieve proper momentum balance. The angle between the tributary and main channel is ideally set as close to zero as possible. This approach has been found to minimize uncertainties in stages attributable to turbulence and standing wave effects which would require levee superiority.

Normal Depth. Water surface elevations of flood profiles should be plotted upstream for a distance long enough to show that the effects of the project will not be detrimental. Ideally, post-project water surface elevations should be no higher than pre-project conditions at all storm frequencies. The Jacksonville District is initiating the levee design procedures outlined in ETL 1110-2-299. Most levee design for Puerto Rico results in provision of the 1- in 100-year level of protection. Care must be taken in the design of long levee systems since establishment of normal depth may encourage the use of several overtopping points.

Sediment Considerations. A Sediment Assessment should be performed during the initial phases of the study to determine the impact sediment could have on project designs. Sediments deposited into a project channel may produce bedforms that would result in higher roughness values. A visual inspection of the existing channel, soil types and velocity/flow characteristics of the project design and existing deposition points can be helpful in determining the potential impacts on roughness values. Hydrologic Engineering Center Program HEC-6 is used to determine sediment movement in project channels. High peak flows and short durations for design storm events require very short time steps between individual discharge events.

Debris basins are designed to capture sediment grain sizes greater than very fine sand. Silt and clay size materials account for up to 65% of the sediment load in Puerto Rico. However, velocities in project channels are usually high and base flows are only a fraction of design discharges. Settlement of clay and silt in project channels is usually minor and is not considered in debris basin design. Tieback levee heights are determined by computing water surface elevations through the debris basin by considering the basins to be filled with debris to the top of the outlet weir.

PROJECTS

General. High velocity leveed channels are common in Puerto Rico. Freeboard design has played an important role in several notable example that are briefly discussed below.

Portugues and Bucana Channels. The Cerrillos Dam on the Cerrillos and Bucana Rivers is complete. After completion of the Portugues Dam in 2002, the Portugues and Bucana (P&B) channels will provide Standard Project Flood protection to the town of Ponce, in southern Puerto Rico. The P&B channels have been under construction since 1975 and, prior to completion of the Cerrillos Dam, provided about 35-year protection to the urban areas of Ponce. The concrete-lined channels were designed with three feet of freeboard. During the 1985 flood, flows measured at upstream drop structures approximated the channel design discharges. Flood high water marks showed that the design discharge utilized all of the available freeboard. Post storm event surveys indicated sediment deposition over two feet deep with vegetation and trees firmly rooted. Freeboard compensated for lack of maintenance and the resultant change of "n" value during the 1985 flood event.

Rio Puerto Nuevo. Transiting through the heart of urban San Juan, Puerto Rico, the Rio Puerto Nuevo channel will have 9.5 miles of high velocity concrete channel. Leveed sections will be minimal except in areas of low topographic features. Freeboard averages three feet for the leveed sections. Channel designs were modeled at the Waterways Experiment Station at a cost of $637,000. Project savings, using the physical model channel designs with pilot channels and bridge pier extensions are estimated to range from $4 to $6 million. Physical modeling enabled the hydraulic engineers to identify areas of unstable flow, standing waves and the expected perturbations in the water surface, which are not evident in numerical modeling.

Rio Guanajibo. The Rio Guanajibo passes through the center of the town of San German. This small town in eastern Puerto Rico experiences frequent flooding from high velocity flows. The flood protection plan which provides the highest net annual benefits has been found to be unacceptable, in part, because of the leveed high velocity channel. The urban area of San German closely lines both sides of the channel, making channel improvements and levees the most efficient plan. The optimum level of protection is about 25 years with an expected sixty percent chance of being overtopped twice during the project life. The false sense of security created by levees combined with the low level of protection was not considered acceptable. A new plan is underway that will consider an excavated channel with no levees and freeboard.

CONCLUSION

The Jacksonville District has followed, to the extent possible, the current guidelines for the design of freeboard. This paper has outlined not only that freeboard design criteria, but also the many other variables inherent in the hydrology and hydraulic disciplines that will influence freeboard design. It is the design engineers responsibility to recognize the criteria that may represent high variability and possible risk. It is also their responsibility to determine the sensitivity of that criteria and design a levee supplement commensurate with the risk. Levee height adjustments do not have to be additive and they do not have to be 100 percent effective against overtopping. However, the total levee supplement should be designed with as many of the known variables considered. The presence of freeboard in the design of leveed channels has proven to be an important, functioning part of the total design. The freeboard standards outlined in EM-1110-2-1601, which provides for three feet of freeboard for earth leveed channels, has proven to be beneficial both in performance and safety. EM-1110-2-1601 also provides for additional freeboard for design purposes. If these criteria are not satisfactory, additional guidance that clearly states the criteria should be published by the Office of the Chief of Engineers. Problems arise in the field when design guidance is non-specific. The Corps credibility with local sponsors and flood impacted communities are severely strained when delays increase study costs and time schedules. Oftentimes, these delays are a direct result of the District trying to comply with non-specific guidance. Our experience has been that higher authority ultimately accepts and supports the professional judgments of our design engineers; however, the delays encountered in explaining and justifying these decisions has caused severe budget problems. Technical Review Conferences have helped, but the problem still exists.

FLOODPLAIN ENCROACHMENT AND

ITS EFFECTS ON LEVEE OVERTOPPING DESIGN

BY

JOEL W. JAMES [1]

Description. The southeast quadrant of the city of Macon, Georgia is protected from flooding on the Ocmulgee River by the Macon levee. The original levee was built to a height of 12 feet from the Profile of Proposed Alignment by the city and extended down the river to the Southern railroad bridge. Below that point the levee was built using Bibb County funds with the assistance of several of the nearby plant owners. Due to severe flooding over the years, the present levee was authorized as a Federal project, and construction was initiated 29 August 1949 and completed 7 November 1950. The levee is maintained by the city of Macon and Bibb County through a local cooperation agreement.

The Federal project is located on the west bank of the Ocmulgee River. (See figure 1). It begins on high ground at the Fifth Street Bridge and joins the original levee some 1,160 feet to the south. The original city levee was enlarged for 13,600 feet and a new levee crosses the floodplain downstream of Macon and ties in with the Southern Railway embankment. The levee consists of 115 linear feet of concrete flood wall and 28,000 linear feet of earthen dike with a crown width of 10 feet. The upstream portion is capped with 1450 feet of concrete constructed to raise this portion of the levee. Figure 2 shows the location and typical sections of the flood wall and the earth embankment. The levee gently slopes from an elevation of 303.3 feet mean sea level at the Fifth Street Bridge to 281.5 feet at Stratton Street with the height of the embankment averaging 14 feet. The area protected by the levee is flat and poorly drained.

Ponding behind the levee is controlled by culverts designed for the 250-year storm and fitted with automatic flap gates. The levee currently provides flood protection for events with less than a 63-year recurrence interval.

Interstate 16, which connects Macon to Savannah was constructed in 1969. This road intercepts the floodplain of the Ocmulgee River and for some distance parallels the levee on the opposite side of the river. The construction of this major thoroughfare has increased flood stages on the river by as much as two feet.

[1] Chief, Hydraulics Section, Savannah District, U.S. Army Corps of Engineers

Hydrology and Hydraulics. Discharge-frequency data for the 62 years of gage record on the Ocmulgee River at Macon was used to establish existing condition peak flows. Equations developed for the metro Atlanta areas, which increase peak flows based on the change in the impervious area, were used to adjust the frequency curves for future conditions. The adjusted peak flows were used to determine the top of levee heights.

An analysis of the Macon levee was conducted using the Corps of Engineers HEC-2 Water Surface Profile Program. The events analyzed include the mean annual, 10-year, 25-year, 50-year, 100-year, and the 500-year storms. Using the existing conditions model, it was determined that the 100- and 500-year events would overtop the levee or encroach upon the freeboard. The levee was completely removed from the model and the flood elevations were recalculated for those two frequencies. Cross sections were surveyed inside the levee and added to the model in order to determine the extent of flooding.

The water surface profile for the 100-year discharge for future conditions (94,500 cfs) was computed, a minimum freeboard of 3 feet above this water surface was determined to be adequate for this design. Civil Works Bulletin (CWB) 54-14 and EM1110-2-1601 were used to determine the freeboard allowance.

Overtopping Analysis. After establishing a freeboard profile for the 100-year levee crest, we began to look at our levee area for overtopping design. The downstream area is a brick yard and would act as a natural storage area and would be susceptible to the least damage if flooded. With this in mind, we began to look at discharges above the design discharge and their impact on the levee freeboard. The water surface profile at the downstream end was 1.5 feet above the design discharge and at the upstream end, because of the encroachments the profile increases from 5-6 feet above the design profile.

Several profiles first touched the freeboard profile at the middle of the levee. This is not the most desirable overtopping area and in order to keep overtopping from first occurring at the upstream end, the levee would need approximately eight feet of freeboard. The process was continued until the profile crossed the freeboard profile at the desired location. (See Figure 3)

The resultant profile based on the overtopping analysis outlined in ETL 1110-2-299, ranged from 5-12 feet above the design profile. To correct the resultant damage caused by floodplain encroachment proved too costly for the local assurer, also tying in to existing ground would also be a problem at the higher elevations.

A notch in the freeboard to cause overtopping flow in the desired area was also considered and ruled out since this would not meet the FEMA freeboard criteria for 100-year protection. FEMA requires at least three feet of freeboard above the 100-year flood, with an additional foot within 100 feet either side of structures within the levee or where ever the flow is constricted, such as at bridges. An additional 1/2 feet above this minimum is also required at the upstream end tapering to the minimum at the downstream end of the levee. Any compromise of this requirement changes the flood zones for insurances purpose. In general, FEMA will not recognize human intervention for the purpose of increasing a levee's design level. Therefore, flashboards and sand bagging were ruled out as methods to achieve overtopping in the downstream levee.

<u>Conclusion</u>. As of this writing the project has been designed for the 100-year flood with three feet of freeboard. This provides protection from floods up to the 200-year flood. While overtopping would occur in the upstream portion of the levee, we feel it would not do so with catastrophic consequences.

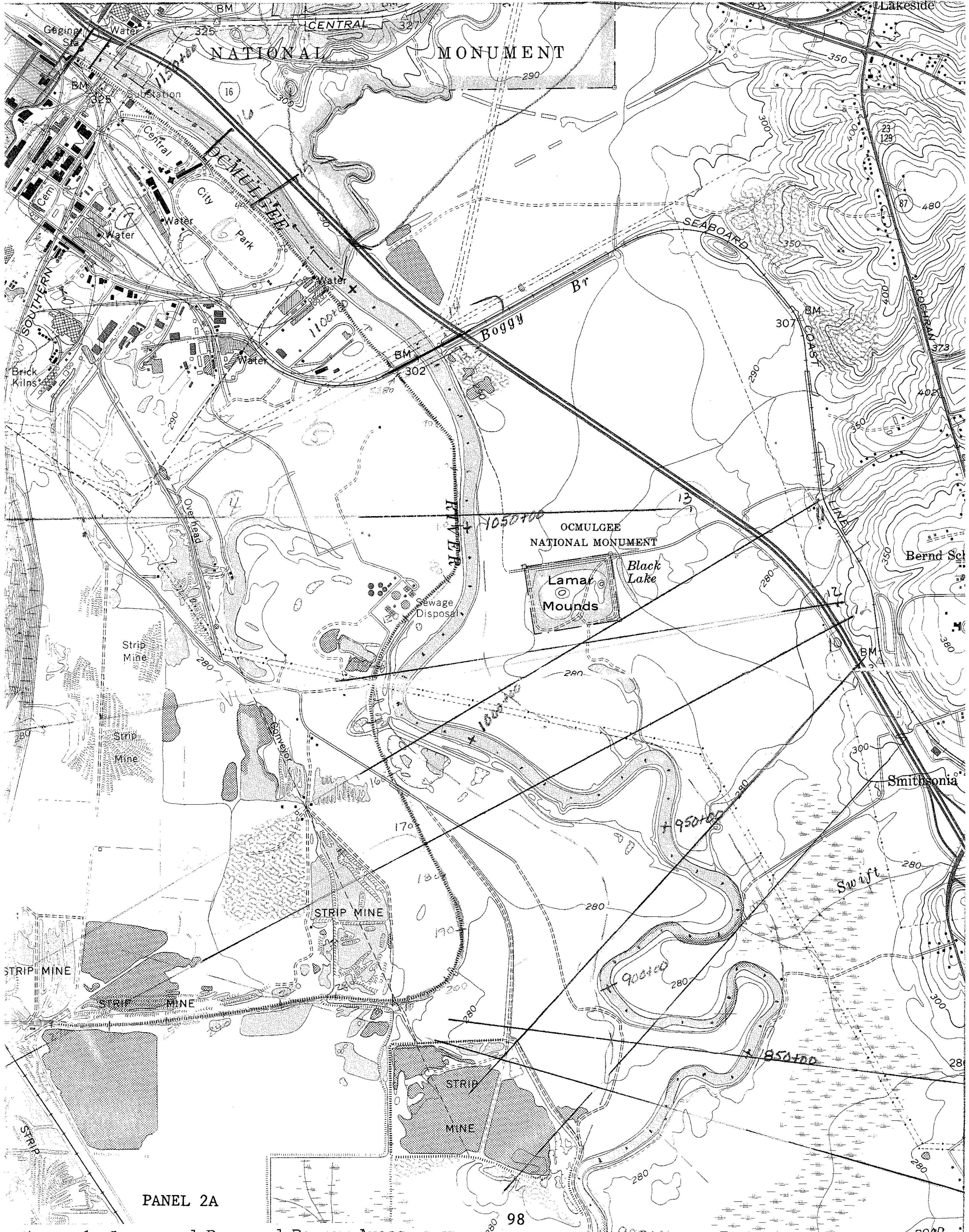
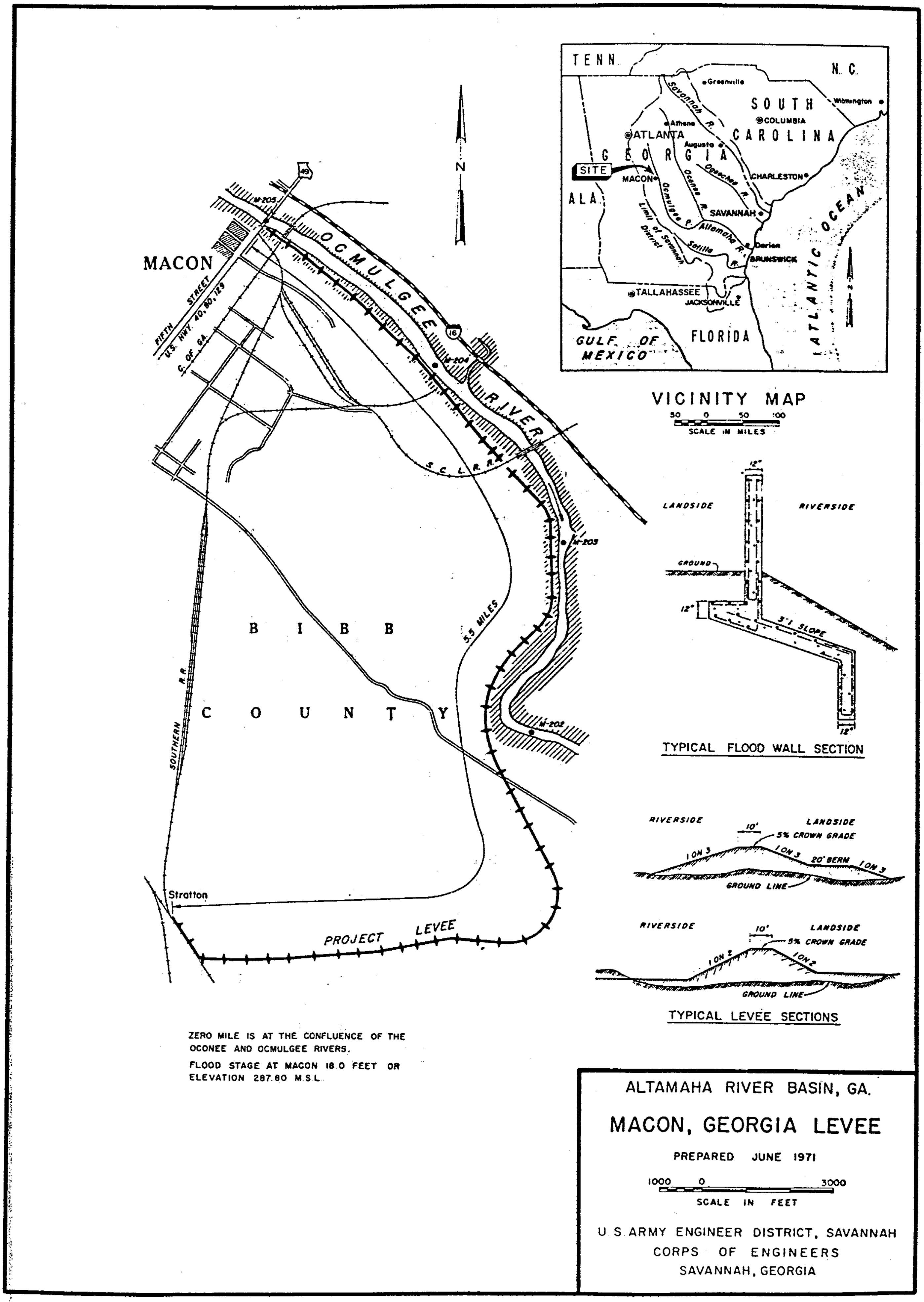


Figure 1. Levee and Proposed Borrow Areas


MACON
FIFTH STREET
U.S. HWY. 40, 80, 129
C. OF GA.
OCMULGEE RIVER
S. C. L. R. R.
5.5 MILES
BIBB COUNTY
SOUTHERN R.R.
Stratton
PROJECT LEVEE
N
TENN.
N. C.
SOUTH CAROLINA
GEORGIA
ALA.
FLORIDA
GULF OF MEXICO
ATLANTIC OCEAN
SITE
ATLANTA
MACON
COLUMBIA
CHARLESTON
SAVANNAH
BRUNSWICK
TALLAHASSEE
JACKSONVILLE
VICINITY MAP
LANDSIDE
RIVERSIDE
GROUND
3:1 SLOPE
TYPICAL FLOOD WALL SECTION
RIVERSIDE
LANDSIDE
10'
5% CROWN GRADE
1 ON 3
20' BERM
GROUND LINE
1 ON 2
TYPICAL LEVEE SECTIONS
ZERO MILE IS AT THE CONFLUENCE OF THE OCONEE AND OCMULGEE RIVERS.
FLOOD STAGE AT MACON 18.0 FEET OR ELEVATION 287.80 M.S.L.
ALTAMAHA RIVER BASIN, GA.
MACON, GEORGIA LEVEE
PREPARED JUNE 1971
U.S. ARMY ENGINEER DISTRICT, SAVANNAH
CORPS OF ENGINEERS
SAVANNAH, GEORGIA

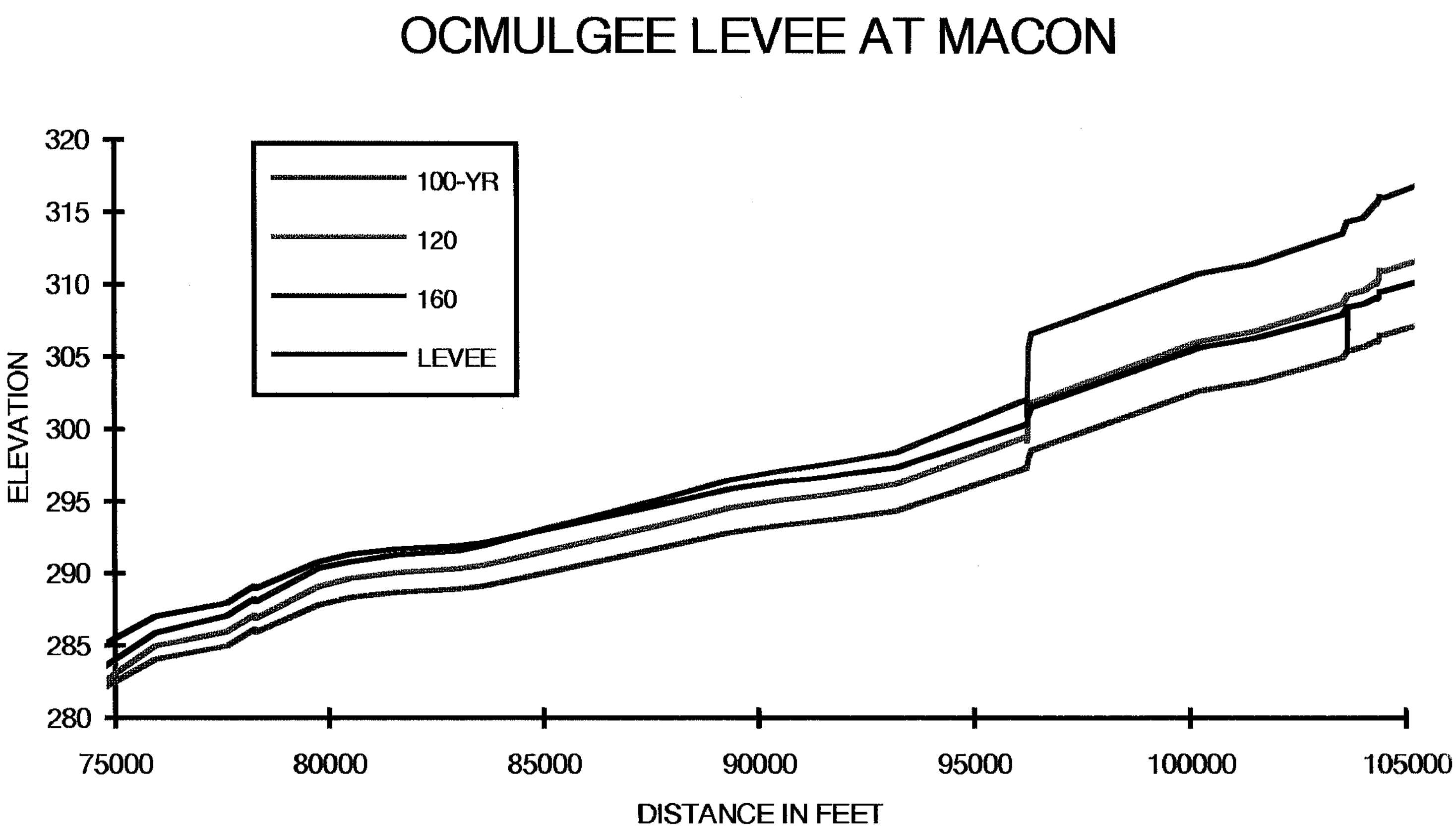

OCMULGEE LEVEE AT MACON
100-YR
120
160
LEVEE
ELEVATION
320
315
310
305
300
295
290
285
280
75000
80000
85000
90000
95000
100000
105000
DISTANCE IN FEET

Figure 3

LEVEE FREEBOARD ISSUES IN OMAHA DISTRICT

by

Jeffrey McClenathan, P.E.[1]

Two projects will be briefly presented here to show the wide range of project designs, freeboard conditions, and freeboard issues confronting designers in the Omaha District. The projects are a proposed levee project at Scribner, Nebraska and a proposed improvement of an existing project at Sioux Falls, South Dakota.

SCRIBNER, NEBRASKA

The project determined to be the National Economic Development Plan (NED plan) was a 100-year levee protecting the City of Scribner from Elkhorn River flooding. A variety of conditions such as ice and debris blockage greatly affect the water surface profiles, therefore, the design levee top profile was initially assumed to the 100-year open water surface profile with 3 feet of freeboard added. Various ice and debris blockage conditions were then evaluated to ensure that the resulting water surface profile was within the design freeboard. Five conditions were evaluated which included: ice-affected flow based upon a stage frequency curve at West Point (upstream), ice-affected flow conditions based upon snowmelt-season discharge with channel flow blocked, ice- or debris-affected flow conditions assuming the Elkhorn River bridge is blocked, ice-affected flow conditions assuming the flow is affected by an embankment located downstream from Scribner, and the 500-year all-seasons conditions water surface profile.

The evaluation showed that the design capacity was sufficient to convey flows in each instance with between 2.3 and 0.3 feet of freeboard remaining. The 500-year all-seasons condition most nearly overtopped the levee with freeboard. Based upon these findings, 3 feet of freeboard was determined adequate and the project is awaiting authorization.

SIOUX FALLS, SOUTH DAKOTA

In 1965, a Corps project was completed on the Big Sioux River at Sioux Falls, South Dakota that included a combination of levees, channel improvements, and a new diversion channel with levees, a diversion spillway, and stilling basin. The design hydrology was based upon a limited period of gaging data (13 years) for the 5,550 square mile basin. The original design discharge of 17,100 c.f.s. corresponded to a 100-year flood event. The diversion channel design capacity was 17,100 c.f.s., the diversion dam design capacity was approximately 7,300 c.f.s. The diversion channel freeboard of 4 feet was above the 17,100 c.f.s. design channel capacity enabling the channel to convey 24,400 c.f.s. should the diversion dam be inoperable. The diversion channel spillway chute and stilling basin design capacity was 24,400 c.f.s. An overflow section upstream of the diversion dam was also designed to flow beginning 2 feet below the normal levee top elevation. The overflow, with 2 feet of head, would pass approximately 4,100 c.f.s.

---

[1]Hydraulic Engineer, Omaha District, U.S. Army Corps of Engineers

In 1969 the project was tested with a flood event estimated at 40,000 c.f.s. Approximately 30,000 c.f.s. was conveyed through the diversion channel and spillway while approximately 10,000 c.f.s. was conveyed through the diversion dam and down the improved Big Sioux channel. Because of the size of the Big Sioux drainage basin, sufficient time was available to allow the Corps to assist the City of Sioux Falls in a flood fight. The flood fight consisted of raising the diversion spillway chute and some levees with flashboards, raising other levees with sand bags or soil, and filling in the levee overflow weir upstream of the diversion dam.

The Omaha District is currently restudying the project and is proposing to increase the project design to the current 100-year discharge of 35,000 c.f.s.; a discharge double the estimated 100-year capacity of the original project. Aggradation in the reach downstream of the diversion dam has lowered the channel capacity from 7,300 c.f.s. with 2 feet of freeboard to 7,000 c.f.s. without freeboard. In addition, development of the land area downstream of the overflow section has occurred.

During the NED plan selection, all alternative plans were based on 2 feet of freeboard above the design water surface profile assuming the diversion dam was inoperable. For the selected NED plan (100-year) the design top of levee was refined based on HEC-2 models using split flow for the overflow section and diversion dam with various conditions of dam blockage. The diversion channel design top of levee profile was selected to confine the 500-year event assuming 5,600 c.f.s. passing through the diversion dam or partial blockage. The design levee top profile was approximately 2.5 to 3.5 feet above the design water surface profile of 35,000 c.f.s. (0.5 to 1.5 feet above the preliminary freeboard of 2 feet). The water surface profile for the 500-year event with diversion dam inoperable was 1 to 1.5 feet higher than that selected for the design top of levee but was assumed to be too conservative since the 500-year event was not the selected NED plan. The levee overtopping section was designed to be equal to the design top of levee, but was to be built in such a manner as to allow the section to be easily breached and eroded to a control section for events exceeding the design top of levee. The rationale for the design was that sufficient warning was available for snow melt floods that a flood fight would be waged similar to that waged in 1969. The first action taken in 1969 was to fill the overflow section and raise the existing levees.

DISCUSSION

The difficulties associated with the current design of the Sioux Falls project include the combination of variables such as accounting for future channel roughness, ice effects, and debris effects in conjunction with the possibility that the diversion dam may be inoperable. The new development downstream of the levee overflow section further complicates the design by placing the first and third highest damage areas at risk from flooding from the predetermined failure point. The second highest damage area is the left bank of the diversion channel. If these are considered together, how much freeboard is enough? The conservatism used in the 1965 construction should be applauded when it is considered how useful it was during the 1969 flood, but how would the economic minded public receive it today? Should the freeboard be increased to reduce the probability of catastrophic damage to the city? It is not the NED plan to build a 200-year or 500-year level of protection, but won't we approach or surpass them if too much freeboard is added?

For Sioux Falls the designer must include all of these concerns in the project, but there are no existing methods to calculate probabilities for intangibles like debris blockage or sudden structural failure of a protection component to justify the added cost should the design prove too expensive for the benefit cost ratio. For a chute and stilling basin the design guidance is to size them for a conservative estimate of the upstream channel capacity. Shouldn't the risk and consequences associated with such a failure also be examined? At Sioux Falls designing a chute and stilling basin for the upstream capacity, depending on the freeboard selected, could cost an additional 2 to 4 million dollars. Is this expense justified?

For the project at Scribner, did the conditions examined consider all combinations of freeboard conditions? The freeboard conditions checked were conservative but could they happen in conjunction with lower discharge resulting in a higher stage? Were the conditions considered too conservative and the freeboard too high?

The current trend is to select the NED design based upon economics and then incorporate a higher level of protection into the design using freeboard and conservative design assumptions. The need to account for discharges and events larger than the designed level of protection is clear and real, however, the economics does not have a methodology to account for these design elements. Freeboard design should consider the consequences of overtopping or failure of the levees by taking into account high velocities, depths, duration, damage to structures, loss of critical services (hospitals, police stations, etc.), and potential loss of life (SMITH, 1984). Yet the mechanisms to justify the additional costs for structural designs to reduce risk are not in place. Instead it is left up to the designer to "sneak" these requirements into the project without ruining the benefit cost ratio.

SUMMARY

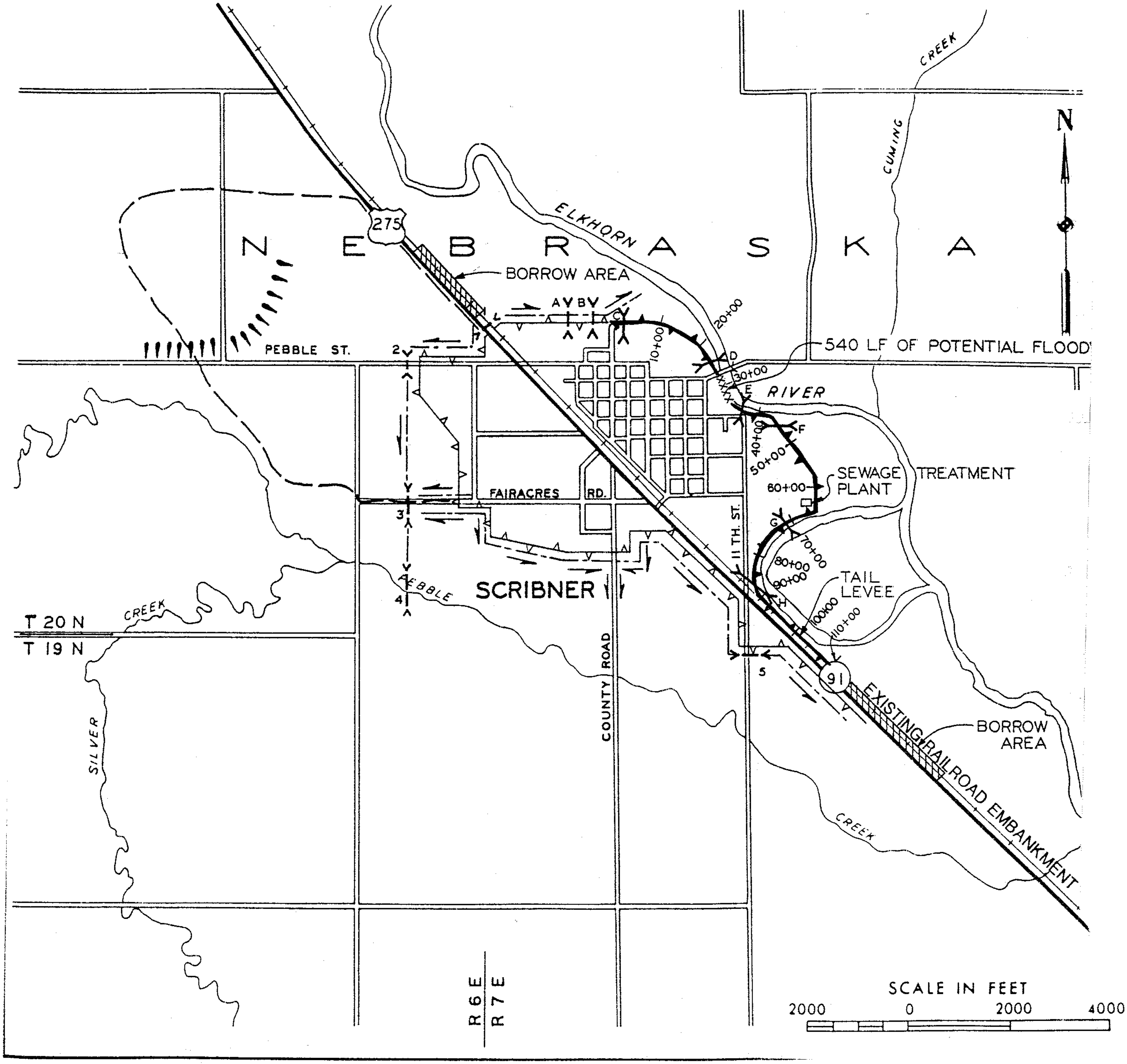
The engineering and design side of the Corps has taken great strides in the advancement of determining project freeboard, the function of a project during exceedance flood events, and other uncertainties. Guidance is still needed however, on how to balance a complex project with multiple variables to determine the proper amount of freeboard for a project. Guidance is also needed to advance the economics that control the project's direction by including these "uncertainties" also. Just because there is not specific probability for loss of life or debris blockage does not mean there is not an economic advantage to designing the project to reduce the likelihood of their occurrence. In projects like Sioux Falls and Scribner, perhaps more time should be taken to make risk based decisions instead of purely economic decisions, to allow all participants, engineers, planners, and the locals input into the final decision and design.

REFERENCES

Supplement to Detailed Project Report, Pebble Creek and Elkhorn River at Scribner, Nebraska, Corps of Engineers, Omaha District, January 1991.

Design Memorandum No. MS-5, Spillway Chute, Big Sioux River, Sioux Falls Flood Protection, Corps of Engineers, Omaha District, October 1956.

Engineering Technical Letter No. 1110-2-299, Overtopping of Flood Control Levees and Floodwalls, Corps of Engineers, 14 May 1986.



PEBBLE CREEK AND ELKHORN RIVER

SCRIBNER, NEBRASKA

FLOOD PROTECTION PROJECT

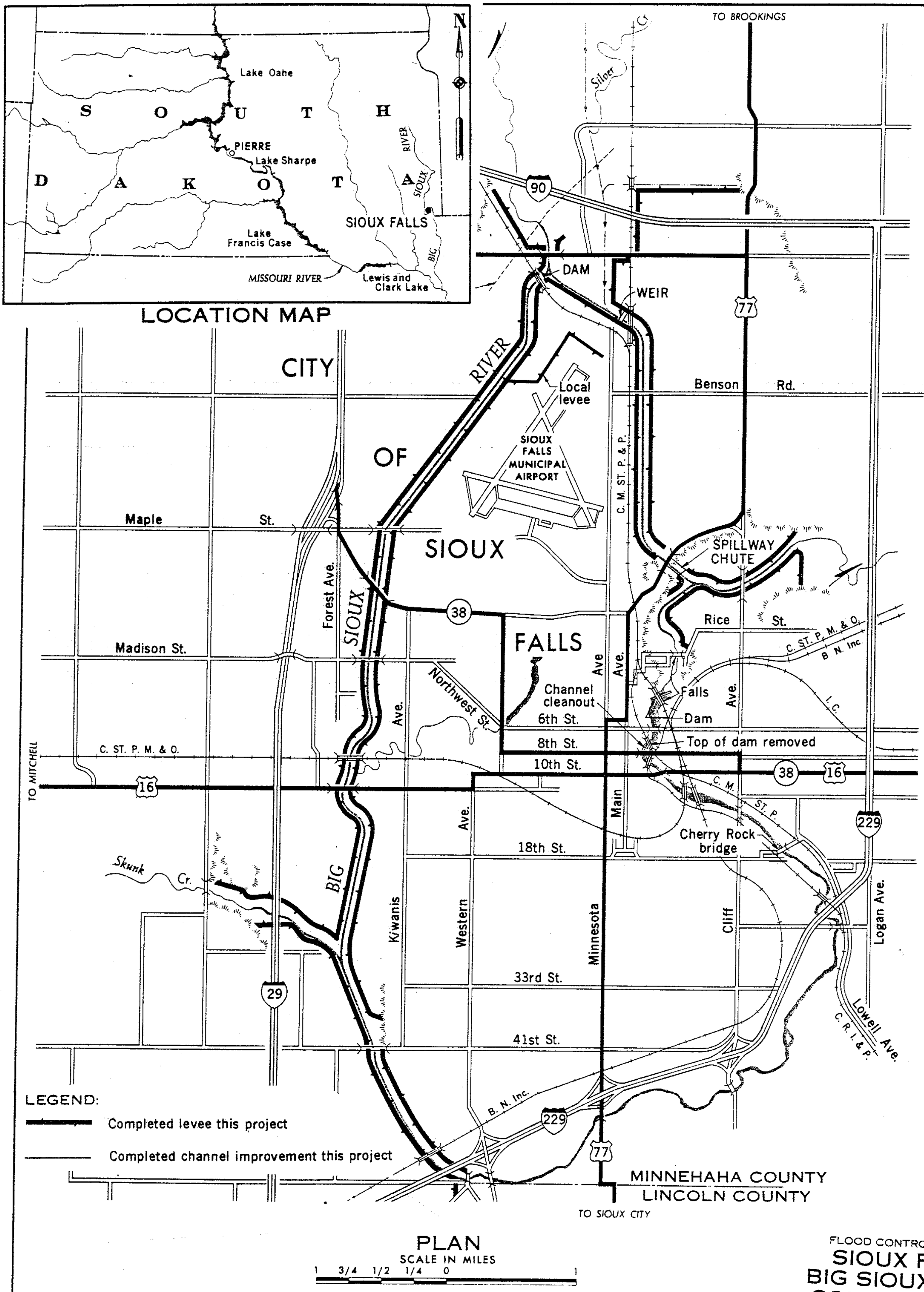
ELKHORN RIVER

ALIGNMENT 1

U.S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS OMAHA, NEBRASKA

SEPTEMBER 1990

PANEL 23



FLOOD CONTROL PROJECT
SIOUX FALLS
BIG SIOUX RIVER
SOUTH DAKOTA
U S ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

UNIQUE FREEBOARD SITUATIONS IN SEATTLE DISTRICT

by

James L. Lencioni, P.E.[1]

INTRODUCTION

Over the past few years the Seattle District has designed two projects having less than the 3 feet of freeboard which has become more or less construed, by many outside agencies as well as some segments within the Corps itself, to be a Corps of Engineers design "standard." These two projects provided the Seattle District's hydrology and hydraulics staff the opportunity to address the freeboard issue in a manner which resulted in reasonably prudent designs having decreased project costs while meeting the objectives of the projects. However, due to the misconstrued perception of a 3-feet-of-freeboard requirement, the hydraulics staff was required to spend countless hours defending the freeboard evaluation process on these two projects.

PHYSICAL DESCRIPTION OF PROJECTS

Case 1, Green River Project. The Green River is located in western Washington State and drains a 483-square-mile basin. The proposed project involved an upgrade to existing non-Federal levees along an approximately 30-mile reach of river. Howard A. Hanson Dam (HAH), a Corps of Engineers flood control project, is located about 30 miles upstream of the city of Auburn, which is HAH's control point as well as the upstream terminus of the levee system. HAH, which has been in operation since the early 1960's, is sized to contain the standard project flood (SPF) inflow of the Green River while controlling the flow at Auburn to 12,000 cfs. However, local controlled and uncontrolled inflow to the Green River downstream from Auburn substantially reduce the level of protection through the highly urbanized and industrialized area protected by the non-Federal levees.

Case 2, South Aberdeen Project. The Corps of Engineers' proposed project is an approximately 3-mile-long levee located along the south bank of the Chehalis River near the river's mouth through the city of Aberdeen, Washington. The Chehalis River has a 2,114-square-mile drainage basin which terminates at the Pacific Ocean; water surface elevations influencing the project's design are therefore highly tidal-dependent. The proposed project would provide SPF level of protection to an area encompassing a population of about 3,500 people.

STUDY APPROACH

General. Technical studies for both projects included detailed evaluations of the unique site-specific physical conditions for which freeboard is provided in order to ensure the intended level of protection. These studies included sedimentation analyses, sensitivity evaluation of hydraulic

---

[1]Hydraulic Engineer, Seattle District, U.S. Army Corps of Engineers

computation coefficients, assessment of future development, evaluation of river discharge and/or elevation controls, bridge swellhead effects, and bend superelevation.

Computational Methods. Unsteady flow numerical computational models were utilized to compute basic water surface profiles in both cases. For the Green River project, the "Simulated Open Channel Multiple Junction" (SOCHMJ) model developed by WES was used, while the National Weather Service's DWOPER model was used for the South Aberdeen project. Unsteady flow computation methodology was considered warranted in both cases due to the unsteady hydrologic phenomena of upstream, downstream, and interior boundaries for both projects.

STUDY RESULTS

Green River Project.

1) Hydraulic analyses revealed that the 10- and 100-year condition water surface elevations in the project reach were essentially equal and that the SPF water surface profile was only 0.5 to 1.0 foot higher as a result of the control provided by HAH. Technical studies accomplished led to the conclusion that a freeboard value of 1 foot was acceptable; however, a value of 2 feet was adopted for design purposes due to the highly urbanized character of the flood plain and to provide maximum flexibility for future operation of HAH.

2) Technical studies of the uncertainties associated with development of a design water surface profile which led to this conclusion were:

   a) The numerical model used to compute water surface profiles was calibrated and verified to historic, near design discharge data. Knowledge of maintenance accomplished on the existing levees was considered sufficient evidence to conclude that the existing conveyance would be maintained in the future.

   b) Channel cross sections originally surveyed in 1961 were resurveyed. Comparison of these cross sections in conjunction with computation of a reach-by-reach mass volume change in area showed that a net degradation had occurred over the 20-year period. Evaluation with a steady state numerical flow model revealed that the 1980 cross sections resulted in a water surface profile about 0.5 foot lower than the 1961 condition profile at a comparable discharge. The sedimentation processes which had occurred during the 20-year period were considered indicative of those expected in the future due to discharge control provided by HAH.

   c) Numerical model computations indicated that conveyance reductions of 25 percent at bridges caused bridge losses of 0.5 foot maximum. Based on historic observations and due to the debris-producing characteristics upstream of the project, such debris blockage conditions were considered to reflect a maximum condition.

d) Superelevation of the water surface at curves was computed to be a maximum of 0.2 foot.

e) Visual inspection of the levees by experienced hydraulic and geotechnical engineers suggested that the existing non-Federal levees had been well maintained and appeared to be structurally sound.

f) The effect on project water surface profiles resulting from inaccurate operation of HAH to meet the 12,000 cfs maximum controlled flow at Auburn was evaluated using the unsteady numerical model. The past 20 years of operation revealed that in-place monitoring and gaging facilities ensured that the dam could be operated such that discharges at Auburn in excess of 12,000 cfs would be no greater than 1,000 cfs and would exist for no longer than 12 hours. Simulated discharge hydrographs of 13,000 and 14,000 cfs for 12 hours superimposed on the levee project design hydrograph resulted in increases of approximately 0.5 and 0.8 foot, respectively, over the design condition's water surface profile.

3) Economic analysis revealed that only portions of the proposed levee were economically justified. Because the project would therefore have required portions of a Federal levee to be tied into a non-Federal levee, the Seattle District concluded that a "whole and complete" Federal project was not feasible and the project was not recommended for construction.

4) Shortly after completion of the District's studies, FEMA published a draft flood insurance study which would have required the flood plain inhabitants to purchase flood insurance because the existing levees failed to provide the 3 feet of freeboard which FEMA requires and has oftentimes been referred to as the Corps of Engineers "standard." Through several iterations of discussion and coordination with FEMA representatives, the Corps of Engineers studies for the nonfavorable levee project were accepted and FEMA relaxed their freeboard criteria to 2 feet along the lower Green River. This change resulted in sparing thousands of inhabitants from purchasing flood insurance.

<u>South Aberdeen Project</u>.

1) Hydraulic analyses revealed that tidal elevations were the predominant water surface elevation control through the study reach. The difference between the 100-year frequency flood and the SPF is only 0.5 foot and the 95 percent confidence limit in the tidal frequency curve elevations for the 100-year through SPF tides is 0.25 to 0.35 foot. Based on the relative insensitivity of roughness coefficients on water surface elevations and the strong tidal

influence, the Seattle District concluded that a maximum freeboard value of 1.5 feet was warranted. An evaluation of the effects of roughness and tidal elevation uncertainties on water surface elevations for various event frequencies also suggested that the amount of freeboard required varied with the frequency of the event being considered.

2) Technical studies accomplished for the GDM:

a) Unsteady flow numerical modeling revealed that the combination of design frequency tidal conditions with coincident river hydrologic conditions resulted in water surface elevations generally 2.5 feet higher than the combination of design frequency river hydrologic conditions with a coincident tidal hydrograph.

b) The numerical model was calibrated to data derived from a previous physical model study of the Grays Harbor estuary (mouth of the Chehalis River) accomplished at WES in the mid-60's. The calibrated model was then verified to within about 0.2 foot on a separate data set of water surface elevations and river discharges obtained from a flood which occurred in January 1972.

c) Results of the numerical model were tested for sensitivity to roughness coefficients by varying the Mannings value between 0.025 and 0.055. Such variations resulted in water surface elevation changes of only about 0.2 foot.

d) The tidal frequency curve is quite flat--having an elevation difference of only about 1.5 feet between the 5-year frequency and the SPF frequency. The 95 percent confidence limits of the tidal frequency relationship were determined to vary from 0.1 to 0.35 foot for that same frequency range.

e) Sedimentation effects are considered minimal in the project reach due to an ongoing dredging program accomplished for navigation interests. Boat traffic was considered to result in approximately 0.5 foot of wave action on the levee. The two bridges located in the project area either have considerable high clearances or are capable of opening to accommodate vessels; therefore, the effect of debris accumulations on bridges was considered to be insignificant.

f) Considering all the uncertainties discussed above, a freeboard amount of 1.5 feet above the computed SPF water surface profile was proposed for levee design purposes. Of the uncertainties, those associated with tidal elevations and roughness effects on computed water surface elevations were considered to be the most dominant. Because of the strong tidal elevation influence on the water surface profile in the project reach, that uncertainty was assumed to be weighted twice that of the roughness. Using this assumption, freeboard for various frequency events was computed as a function of the tidal confidence limit plus the water surface slope through the project reach for the respective frequency event being evaluated. This approach resulted in freeboard amounts varying between 0.7 and 1.5 feet for the 5-year frequency to SPF events.

3) Formulation analysis indicated that the project design maximized at the SPF condition. Following approval of the GDM and immediately prior to anticipated initiation of P&S, the Washington Level Review Center formally questioned, without coordination with knowledgeable staff at headquarters level, the rationale for deviation from the "standard" 3 feet of freeboard and the concept of a varying freeboard amount. Although the questions have evidently been resolved, technical staff at the District, Division and Headquarters level have been required to devote an inordinate, and needless, amount of effort to address an issue surfacing from misinformed staff members within the Corps.

CONCLUSIONS

Use of a "standard" freeboard amount is not technically supportable. Hydrology and hydraulics staff must continue to ensure and reinforce the incorrect perception existing even among in-house personnel that the Corps of Engineers designs to a standard 3 feet of freeboard. Freeboard is a separate part of each project which must be evaluated, and stand on its own merits, considering the at-site conditions of the project.

LEVEE FREEBOARD ISSUES BASED ON ST. LOUIS DISTRICT EXPERIENCES

by

Gary R. Dyhouse[1]

FREEBOARD ISSUES

Freeboard determination has been a source of continuing discussions with St. Louis District (SLD) planning and project management personnel. They often feel that the freeboard amount included in a levee project is too conservative, causes great increases in project cost, and puts an unnecessary burden on the local sponsor. Improved, and more defensible, freeboard selection processes are greatly needed for both the evaluation of existing levees and for the design of new ones.

Levees no longer serve their purpose when floodwaters enter the protected area (not including seepage). This situation can happen by piping and failure prior to levee exceedance, or by levee overtopping. In the St. Louis District over at least the past 25 years, all levee failures but one have occurred by overtopping. Failed levees have been of non-Federal construction in almost every case, with the quality of both the fill material and construction largely unknown. Maintenance has ranged from good to non-existent, but failure has still been almost exclusively by overtopping. Thus, geotechnical considerations have had little input into the minimum freeboard call, making the hydraulic analysis even more important for level of protection determinations. If geotechnical concerns are not significant for freeboard, what minimum freeboard level for an existing levee would be selected? Presumably the same freeboard as will be used for a levee raise or new levee project, but this may not be known at the time of the determination. Should this be two feet, three feet, include a variable freeboard, etc.? If variable freeboard is the norm for new levee projects, how should this be addressed for existing levees that often have not been "designed" at all? Current guidance does not give specifics for the minimum freeboard problem for an existing levee and suggests that three feet is the minimum freeboard for new projects.

LEVEL OF PROTECTION ESTIMATES FOR EXISTING PROJECTS

SLD Applications. Two feet of freeboard has been used for most level of protection determinations in the District. This amount is consistent with the majority of the District's Federally-constructed levees, designed and built in the 1950's and '60's with a uniform two feet of freeboard. For a level of protection evaluation, water surface profiles for each of several frequencies have been plotted against the existing top of levee profile and low reaches (having a distance of 500 feet or more--considered to be the maximum distance to provide emergency sandbag protection) along the levee noted. Stage-frequency curves at each of these points are plotted and the minimum frequency corresponding to two feet below the average levee crown has generally been the level of protection assumed for an existing levee. This level of protection

[1]Chief, Hydrologic Engineering Section, St. Louis District

is compared against the levee's past performance for consistency; i.e., if the level of protection is 25 years and the levee has not failed more than once in the past 20-30 years, the level of protection would seem appropriate. However, the application of higher or variable freeboard requirements to an existing levee poses a problem.

Higher/Variable Freeboard. Arguments by planning/project management personnel state that freeboard greater than two feet adds significant cost to a project with little increase in benefits--often a valid position. To compensate, they want to include a similar freeboard evaluation to the existing levee protection call. The problem with this approach experienced in the SLD is it often results in a level of protection deemed unreasonably low by H&H personnel. In one instance, assuming a uniform three feet of freeboard on an existing levee resulted in a level of protection of about a 20-year recurrence interval rather than a 30- to 35-year recurrence interval obtained with two feet of freeboard. The 20-year call gives much more damages for existing conditions, yet the levee did not fail during nearly 50 years of record, yielding an obvious inconsistency. The difference between a 20- and 35-year level of protection call for the existing levee was the difference between a benefit/cost ratio more or less than one for a proposed levee raise. Although this situation has fortunately not arisen often, level of protection estimates inconsistent with actual data and with hydrologic/hydraulic engineering analysis have resulted when freeboard levels exceed two feet. A more equitable way of handling the increased costs resulting from higher and/or variable freeboard are needed without resorting to unrealistic estimates of existing level of protection.

MINIMUM FREEBOARD FOR NEW PROJECTS

Current guidance states that a minimum freeboard of three feet is generally appropriate. However, many past agricultural and urban protection projects have been built for two feet of freeboard. Although this freeboard could be thought of as inadequate under today's standards, the SLD still feels it is generally satisfactory for the level of protection claimed. The addition of a single foot of freeboard to a high levee system can easily cost several million dollars and only give an uncertain amount of additional safety. The benefit-cost ratio of such a raise would almost certainly be less than unity. Rather than debate whether the minimum freeboard should be two feet, three feet or some other figure, more rational and defensible methods need to be developed. Some possibilities could include the following:

Alternate Frequency Profiles for Levee Grade. Most Corps levees are designed for a specific return period flood; i.e., the 50-year or 100-year recurrence interval flood. Rather than adding X-feet of freeboard, why not use a higher design frequency, or an estimate of the design discharge corresponding to the 5% confidence level, to establish the minimum levee crown?

1) Design for a higher discharge. For example, if the level of protection is to be a 50-year recurrence interval flood, establish the minimum levee crown as a 100-year recurrence interval flood profile, with variable freeboard then incorporated, as needed. Doubling the frequency should be an adequate factor of safety for determining a minimum freeboard. Where long term gage data are available, the value of the design discharge corresponding to the 5% confidence level could be used for a top of levee design. The

procedure giving the greatest freeboard could be adopted for the project. Designing the minimum levee crown with either procedure would seem more defensible, as well as more palatable to the local sponsor, than a rather arbitrary three feet of additional height. If the difference between the design water surface and top of levee profiles are small (less than a foot), additional freeboard may be necessary, especially for instances where unstable stage-discharge relationships exist, where poor levee and channel maintenance practices may be expected, etc.

2) Design for an X% risk of exceedance over project life. Determine the frequency corresponding to a levee crown design that has a specific risk of exceedance during the project economic life. A standard value of risk could be 50% during the 50-100 year economic life. Using the binomial distribution, a 50% risk of exceedance over a 50-year life would require a 75-year recurrence interval flood to be used for the net levee grade profile. The same criteria for a 100-year project life would result in about a 150-year recurrence interval flood. This type of analysis would be most applicable for levee designs of 50-100 year frequency. Ten to 25-year recurrence interval designs would probably have too great a freeboard under this method. Similarly, small risks of exceedance would usually result in too rare a flood, with no acceptable means of computing the appropriate discharge. For example, if a 10% risk of exceedance during the economic life of 50 years is chosen, the levee crown would be based on a 475-year event, while a 100-year life with a 10% risk would require a 950-year design. Neither of these rare design floods could likely be developed with a significant level of confidence in their accuracy.

Sponsor selection, based on economic consequences. As freeboard increases, the probability of overtopping and subsequent levee damages become less. The Federal Government (80%) and the levee district (20%) currently cost-share, through the PL-99 program, the expenses associated with levee exceedance and repair. Sponsor-selected freeboard requirements could be reflected in the amount of future levee repair the sponsor would pay. An example could be: one foot freeboard-100% repair by sponsor, 2 feet freeboard-50%, 3 feet freeboard-20%, greater than 3 feet-100% Federal. FEMA could also provide a sliding flood insurance rate for protected structures to reflect the lower risk levels resulting from progressively higher freeboard amounts, further encouraging the sponsor towards a higher freeboard.

Local considerations. Site-specific situations should also have a bearing on the appropriate freeboard, including:

1) Source of flooding/overflow time. The time that a flood would exceed the top of levee and the associated inflowing volume should have a bearing on freeboard. Small streams exceeding the top of levee for a few hours or less could result in only a small freeboard being necessary. Flank or back levees conveying flood flows from small hillside watersheds to the main river often fall in this category. These levees could have minimal (one foot) freeboard where backwater from the main river is not dominant, as the potential for a catastrophe would be remote.

2) Low levees. Three to five feet high levees could have zero freeboard, if the area protected is small and adequate means of escape during overtopping situations exist. Urban situations have shown the economic justification for these type levees protecting a cluster (5-10) of buildings,

where levee overtopping would result in flooding of walk-out basements only, or when occupants could easily wade to safety. The incorporation of one or more feet of freeboard to these low levee situations seems somewhat ludicrous.

3) Uniform freeboard. Although uniform freeboard is generally not considered appropriate any more, a short levee protecting a relatively small area may not gain a significant benefit from variable freeboard and controlled overflow at the downstream end. Unless the forecast calls for the levee to be exceeded by several feet, the levee district will usually begin sandbagging operations at the initial overflow point. Whether or not the initial overflow point is operational, insufficient time may be available for the interior to fill prior to the upstream portions of the levee overtopping, especially for steeper rivers that can easily rise at one foot per hour or more. If the interior distance from the upstream to downstream levee limits is short, the interior water surface will be essentially the same, no matter where the initial overtopping occurs. The need for variable freeboard to allow the water to "back in" and provide a cushion to slow interior velocities may not be critical, thus allowing a uniform freeboard to be acceptable.

CONCLUSIONS

Variable freeboard and controlled overtopping requirements have been emphasized in recent years, but the minimum freeboard appropriate for any given situation should also be carefully examined, whether for the evaluation of existing levees or for the design of new ones. Additional guidance is needed to establish a minimum freeboard in an logical and defensible manner, rather than simply applying a minimum of three feet.

SUMMARY OF SESSION 3: PART 2 - LEVEE PROJECT EVALUATION AND PERFORMANCE

Overview

This session was a continuation of Session 2 and included three papers and a panel. The focus was to present levee project studies which illustrated current levee freeboard design applications and related issues. Participants included representatives from seven different district offices.

Paper 7. Dennis Seibel, Chief, Hydrology-Hydraulics Section, Baltimore District, presented a paper entitled, "Wyoming Valley Levee Freeboard Design." In response to a directive from the Office of the Assistant Secretary of the Army for Civil Works "to analyze the appropriate amount of freeboard for the project," a methodology was developed to determine the amount of freeboard required for the Wyoming Valley project to assure that existing and proposed levees will not be overtopped during their design discharge event. The methodology assessed the impact of various parameters on the determination of water surface profiles and, therefore, the amount of freeboard required.

Factors that were assessed in the design of the freeboard included the accuracy of discharge estimates, the accuracy of the water surface profile computations, expansion and contraction coefficients, the potential for debris blockage at bridges, ice jams, superelevation, waves and sediment deposition. The effect of ice jams, superelevation at bends, waves, and sediment deposition were all determined to have no effect on the amount of freeboard required for the reach of the Susquehanna River in the Wyoming Valley of Pennsylvania. It was determined, however, that the uncertainties associated with the remaining four factors could have a significant impact on the required freeboard.

A technique was developed to quantify the effect of the uncertainties in the accuracy of discharge estimates, the accuracy of the water surface profile computations, expansion and contraction coefficients, and debris blockage at bridges. The combined effect of the four factors was utilized to determine the minimum freeboard required. The methodology described was utilized to determine the level of protection provided by the four existing levee projects in the Wyoming Valley, in addition to the top of protection profile required to increase the level of protection for the communities in the Wyoming Valley.

Paper 8. Lester Cunningham, Hydrology Branch, Planning Division, Walla Walla District, presented a paper entitled, "Levee Freeboard Design in the Walla Walla District." Experiences with several existing levee projects within the Walla Walla District were discussed as examples of certain risks and uncertainties which must be considered when selecting an adequate levee freeboard. Two levee systems, one on the Columbia River near the Tri-Cities, Washington, and the other on the Snake River at Lewiston, Idaho, present a contrast in political and engineering considerations associated with levee height and freeboard requirements. Additional hydrological data, and changes in sediment transport and upstream flood control since the projects were completed, have affected the height and freeboard requirements for both projects. Local city officials and developers have urged the Corps to lower and landscape the Tri-Cities levees, while, at Lewiston, the extensively-landscaped and developed levees may need to be raised.

Les emphasized that water surface profiles should be based on the best estimate of quantifiable factors and uncertainties and consideration of risk should be assigned to freeboard. He feels that a minimum freeboard will always be required but that the freeboard design should take into consideration the consequence of failure.

Paper 9. Patrick Foley, Chief, Hydraulics Section, St Paul District, presented a paper entitled, "St. Paul District Experience with Credit to Existing Levees." His paper describes some of the St. Paul District's experience in determining flood damage reduction benefits for existing levees. His main point is that Corps' guidance for evaluating the economic benefit of existing levees differs significantly with the guidance for evaluating new levees. A new levee must meet the high Corps standards and conservative design criteria while an existing levee can be given partial economic credit even if it does not meet Corps' standards. Giving credit to existing levees with unknown reliability can infer Corps' "certification" up to that level. There is also the problem of a mixed message being sent to the public when the Corps gives flood damage reduction benefits to an existing levee and FEMA will not allow any reduction in flood insurance rates or flood plain zoning because the levee does not meet their criteria. Criteria which is mainly based on the Corps' standards for new levee design.

Panel 3: Levee Project Evaluation and Performance (Continued)

Four panel members made short presentations based on their individual experiences that highlighted specific issues related to levee freeboard.

A. Ken Halstead, Chief, Hydrology Section, H&H Branch, Huntington District, gave a panel presentation on, "Levee Freeboard Design for West Columbus, Ohio LPP." During the Reevaluation study of the West Columbus, Ohio, the local protection project, the construction cost estimate was found to have increased significantly over the estimate provided in the Feasibility Report. An intense study was conducted to determine all minimum project requirements that could be implemented in order to reduce the project cost. As a part of this effort, a detailed study was conducted to determine the minimum freeboard design that would be required to provide the proposed level of protection. A list of factors that could affect the design water surface profile was generated. Minimum freeboard was developed by an appraisal of those factors for which the effects could not be estimated with certainty or accurately defined for the occurrence of the design flood. After careful consideration, the location of the initial overtopping section was established. Ken's main conclusions were that the recognition of general and project specific factors that could affect the design water surface profile and freeboard are of equal or greater importance than the actual computations, and different combinations of the location of the initial overtopping section and the selection of the design flood can be significant factor in determining the amount of freeboard required.

B. Robert Fitzgerald, Chief, Hydrologic Engineering Section, Hydraulics Branch, Vicksburg District, presented a panel discussion on, "Levee Freeboard Issues in the Vicksburg District." Recent guidance within the Corps has brought about a higher level of interest in the design of levee freeboard. Local cost sharing has brought about increased participation by the local sponsor. Along with this increased participation by the local sponsors has come questions regarding the amount of freeboard included in projects and the possible cost savings if the freeboard requirements were reduced. This has been especially sensitive in areas where natural ground is almost but not quite high enough. The question of whether to build freeboard only levees in these areas has been raised. Several solutions to the above issues have been suggested and are described. The freeboard was reduced to as low as one foot where natural ground is at or above the design water surface elevation.

C. John R. D'Antonio, Jr., Hydraulic Design Section, Albuquerque District, presented a panel discussion on, "Levee Freeboard Issues Rio Grande at Alamosa, Colorado." The Rio Grande at Alamosa project present several freeboard design issues. The study was complicated by complex flow conditions resulting from severe channel meandering at the upstream end of the project. Freeboard considerations at the upstream end differ for the north and south levees. The south levee has a tie-back section that does not allow the water to outflank it upstream of the project, thus preventing flow back into town. The north levee does not have a suitable tieback location within reasonable proximity to the Rio Grande and must tie back into Six Mike Road which will function as a natural levee. The presentation discussed freeboard recommendations for the levees and Six Mile Road taking into consideration the existing flood warning system, inter-agency coordinated forecasting, levee superiority, long-duration snowmelt hydrograph and other factors. The results of freeboard design investigations allowed for 1.5 feet of freeboard for the north levee and one foot raising Six Mile road one foot at the levee tie-in location.

D. Robert Engelstad, Chief, Hydrology Section, St. Paul District, presented a panel discussion entitled, "Effect of Channel Erosion on Freeboard at Houston, Minnesota." Bob described several problems regarding the levee and freeboard design for the Root River at Houston, Minnesota project. Problems in selecting a plan were related to policies of several state agencies involved. Minnesota State law requires that levees must provide protection from the one percent chance exceedance flood and the levee freeboard should be able to contain the Standard Project Flood. Other problems described included channel instability and constrictions due to bridges.

WYOMING VALLEY LEVEE FREEBOARD DESIGN

by

Lori K. Bank[1], James J. Curtin[1], Barry M. Flickinger[1], Khaled M. Masoud[1], and Dennis C. Seibel[2]

1. INTRODUCTION

In a memorandum dated 27 December 1989, the Assistant Secretary of the Army for Civil Works (OASA(CW)) directed the Chief of Engineers: "In proceeding with engineering and design using 1990 appropriated funds, the District is to analyze the appropriate amount of freeboard for the project."

The purpose of this report is to document the methodology developed for the Wyoming Valley Levee Raising Project in response to that directive and to display the results achieved. In summary, the carrying capacities (design discharges) of the existing levee projects were determined by making allowances for minimum freeboard, computed by this methodology. The methodology was also used to compute the top of levee profile for proposed conditions for the design discharges of 318,500 cfs (reoccurrence of Tropical Storm Agnes Flood) and 290,000 cfs. Proposed conditions involves the raising of existing levees and floodwalls, as well as the construction of new levees and floodwalls, to provide additional flood protection to the Wyoming Valley area.

2. METHODOLOGY FOR FREEBOARD DETERMINATION

Freeboard is the increment between the design discharge profile and the top of protection profile. Freeboard is provided to account for the uncertainties involved in estimating the values of the various factors used in computing water surface profiles. These factors include:

historic record of discharges
accuracy of computed water surface profiles
expansion and contraction coefficients
debris
ice jams
superelevation at bends
waves
sediment

The effects of ice jams, superelevation at bends, waves and sediment were all determined to have no effect on the amount of freeboard required for the reach of the Susquehanna River in the Wyoming Valley of Pennsylvania. From the experience of past floods in the Wyoming Valley, the floods of record

---

[1]Hydraulic Engineer, Hydrology-Hydraulics Section, U.S. Army Corps of Engineers, Baltimore District

[2]Chief, Hydrology-Hydraulics Section, U.S. Army Corps of Engineers, Baltimore District

have been the result of large storm events, which created high flows in the river, rather than ice jams. Superelevation was investigated at the most severe bends within the project area (river stations 372.00 and 541.95) and was determined to have no effect on the amount of required freeboard. Wave heights can be significant for sheltered bodies of water such as reservoirs and lakes. For the Susquehanna River at the Wyoming Valley, the depth and width (fetch) of the river are insufficient to form significant wave heights. A sediment impact assessment study was performed in 1988. The study determined that the Susquehanna River, in the project area, does not experience problems with scour or deposition. Deposition during flood events has not been known to raise water surface profiles in the study reach.

It was determined, however, that the uncertainties related to selection of the values for the other four factors could have a significant impact on water surface elevations. The previously developed computer model, utilizing computer program HEC-2 (Water Surface Profiles), was used to determine the potential changes in water surface elevations that could be associated with each of these factors. This was accomplished by revising the input to this model to take into account the uncertainty of the historic record of discharges, expansion and contraction coefficients, debris, and accuracy of the computed water surface profiles. These revised water surface profiles were used in determining the minimum freeboard for the existing and proposed conditions.

A. <u>Accuracy of Discharge Estimates</u>

When determining the discharge associated with a given river stage, there are two potential errors. Discharges are estimated using a stage verses discharge function developed by making a number of discharge measurements for a range of stages, plotting the stage-discharge values and drawing a smooth curve. The curve will not go through all of the points and, as more values are measured, the curve will change. The first error is that associated with the scatter of measured values and how far they can vary from the adopted rating curve. The second error would be the error associated with the individual field measurements used to compute the rating curve.

The discharge rating curve for the U.S. Geological Survey (USGS) gaging station on the Susquehanna River at Wilkes-Barre, PA (station number 01536500) has been revised 21 times by USGS since 1950. The latest revision was in 1987. This revised rating curve was used to estimate discharges for the observed stages of the annual peak floods since 1950. The discharges, so obtained, were compared to the published discharges for those events. The comparison was used to assess the potential uncertainty in the estimation of discharges. This comparison is shown on Table 1. Comparing the discharges estimated from the revised curve, with the published discharges, indicates the difference in discharge ranges from -5.67% to +5.35%. To also account for the uncertainty in flow measurement, the factor utilized to assess the accuracy of discharge estimates was rounded to 6 percent. For comparison, USGS generally acknowledges that their gaging accuracy is 5-10% of the flow. Assuming that for any given flood peak the discharge rating curve is accurate within plus or minus 6 percent, the freeboard should account for the potential higher water surface of a flow 6 percent greater than the design flow.

B. Debris Blockage

The methodology for assessing the effect of debris blockage at bridges was determined after a literature search and by interviewing local officials and engineers. In the original calculations that were reviewed by the AAA Audit team, the potential for debris blocking parapet walls and truss work during high river events was taken into account at all the bridges that cross the Susquehanna River in the project area by blocking out the area between the low steel and top of parapet walls in the HEC-2 computer model. But, some bridges in the Wyoming Valley also possess the potential to collect debris at piers. These bridges were identified by researching the records of past flood events, from interviews with local people and authorities who observed the bridges during Tropical Storm Agnes and from photographs of the bridges depicting both normal and high river events. Also cross-sectional plots of the bridges were used in conjunction with the photographs and verbal information to determine which of the 11 bridges have pier width and spacing that is conducive to the collection of debris during high river events. It was concluded that five bridges have debris blockage problems. These bridges are: Plymouth-Breslau, Carey Avenue, Market Street, Pierce Street, and Water Street. The location of these bridges is shown on Plate 1.

The debris blockage at the piers of these bridges was addressed in the HEC-2 computer model by decreasing the total flow area under each bridge. The amount of decrease in flow area was determined by studying the photographs and interpreting the interviews. A diagram of the typical form of debris accumulation was developed. It was determined to be a triangular shaped mound of debris at the upstream face of the piers and abutments, with dimensions as shown in Figure 1. The total area of debris at the piers and abutments of the five bridges was determined using this configuration. This area was then compared to the total flow area under the bridge to determine the percentage of flow area reduction caused by debris accumulation at the piers. Table 2 presents the percentages used to decrease the flow area of the upstream bridge cross section in the HEC-2 model.

C. Contraction and Expansion Coefficients

Contraction and expansion coefficients are utilized in the HEC-2 model to compute the energy losses that occur when there is a pronounced variation of the river width. An example of the contraction and expansion of a river at a bridge is shown in Figure 2. Contraction coefficients (Cc) of 0.1 and expansion coefficients (Ce) of 0.3 were used in the reaches of the Susquehanna River at the Wyoming Valley that are between the bridges. These values were not changed when developing the minimum freeboard model since there is a high degree of certainty that they are appropriate for the Susquehanna River at Wyoming Valley, since there are no sharp changes in width.

At the bridges, a Cc of 0.3 and Ce of 0.5 were originally used. These values were increased to Cc of 0.4 and Ce of 0.6 when developing the minimum freeboard model. These increases in coefficients were based on conclusions reached through a literature search. Chow[3] states that for abrupt transitions, the value of both Cc and Ce is about 0.5. All the bridges in the

[3]Open-Channel Hydraulics, Ven Te Chow, Ph.D., 1959.

Wyoming Valley have abrupt contractions and expansions, so values of 0.4 for Cc and 0.6 for Ce are considered appropriate.

D. Accuracy of Water Surface Profile Computation

The Hydrologic Engineering Center's (HEC) HEC-2 computer program computes water surface profiles using the standard-step method for gradually varied, steady flow. The method is based on solving the steady flow equations using a cross section to cross section, step by step procedure. The potential for errors in computing water surface profiles using the standard step method can be associated with the following factors[4]:

1. Roughness factor (Manning's n-value) estimation
2. Accuracy of field surveying
3. Rounding-off computation
4. Numerical solution
5. Application of the standard-step method

The errors associated with all the factors except the roughness factor are considered negligible due to the use of modern equipment and computers as well as appropriately spaced cross sections. The roughness factors used in our original computations were determined through a calibration procedure that used field observations and judgement. To account for the potential error in determining n-values, a further calibration process was performed. The original HEC-2 model calibration was considered satisfactory when the computed water surface profiles for three flood events were found to be within 0.5 feet of the majority of the observed high water marks. The calibration did not meet all the high water marks, but rather produced profiles that gave a best fit through the range of high water marks. In fact, the calibration resulted in water surface profiles that were 0.5 feet or more lower than some of the observed high water marks. To account for the uncertainty in the accuracy of the computed water surface profiles, a trial and error process was conducted by increasing all the Manning's n-values in the HEC-2 model until all reliable high water marks for the three independent historical floods were met or exceeded. It was found that a 13% increase in the Manning's n-values achieved this.

The next step in the process involved increasing the Manning's n-values in the HEC-2 model. The difference between the profile adjusted for higher n-values and the unadjusted profile stabilized at a constant value. This value represents the uncertainty associated with the determination of the Manning's n-values. This procedure was followed for several discharges in the existing conditions HEC-2 model and for 290,000 cfs and 318,500 cfs in the improved conditions HEC-2 model. The differences varied from a low of 1.6 feet for a discharge of 192,000 cfs, to a high of 1.9 feet for a discharge of 318,500 cfs.

A literature search was conducted to determine if the increase in n-value of 13% that was used was reasonable. Several references presented a table containing a range of Manning's n-values for different stream conditions, with one or more stream conditions being applicable to the

---

[4]Accuracy of Computed Water Surface Profiles, Hydrologic Engineering Center, December 1986.

Susquehanna River at Wyoming Valley. The tables were reviewed to determine the percent variation in the range of n-values. The published Manning's n-value ranges were found to vary from 52% to 167%. The range of n-values for all of these references, as well as the percent variation, is shown in Table 3. All of the values for percent variation are significantly higher than the value of 13% that was used to meet the high water marks. This confirms that the 13% value that was determined using the field data is reasonable, since the percent variations given in these references would be for a wide range of conditions, whereas, the field data is site specific.

A further literature search was conducted to determine if any additional information was available. A publication by the HEC that deals with the accuracy of computed water surface profiles[5] was reviewed. Based on consultation with HEC and the Office of the Chief of Engineers (OCE) staff, it was determined that the results of this study are not applicable to the Wyoming Valley. In this publication, the computed error in the water surface computation is based on a regression equation developed for various data sets, representing a wide range of stream characteristics, varying from small to large streams, with mild to steep slopes. These characteristics are quite different from those of the Susquehanna River, which include a large available flow area and a mild gradient.

E. Application of Methodology

The total minimum freeboard was determined in two steps. First, the HEC-2 model was revised, incorporating the uncertainties in the accuracy of the discharge estimates, debris blockage at bridges, and contraction and expansion coefficients at bridges. The results of the revised HEC-2 model analyses were then combined with the values derived for the uncertainties due to the estimation of the roughness factors. The resultant profile represented the minimum desirable freeboard.

3. RESULTS

A. Existing Conditions

The existing conditions evaluation involved the assessment of the level of flood protection provided by the existing levees. Although the existing levees in the Wyoming Valley are one system, for economic analysis purposes they were considered as the following four separate systems:

1. Wilkes-Barre/Hanover Township
2. Plymouth
3. Kingston-Edwardsville
4. Swoyersville-Forty Fort

The top of protection profiles for the four existing projects were determined from recent field surveys performed in 1988 through 1990. The surveyed top of protection was closely reviewed to assure that it accurately depicted the topographic features in the Wyoming Valley.

---

[5]Accuracy of Computed Water Surface Profiles, Hydrologic Engineering Center, December 1986.

A design discharge (the discharge that the levee is capable of passing with appropriate freeboard) was determined for each of the four projects. This was accomplished through a trial and error process using the above described procedures for determining minimum freeboard allowances. The highest discharge that was completely contained within the levee system with appropriate freeboard was defined as the design discharge for that project area.

Similarly, an overtopping discharge was determined for each of the four project areas using the unadjusted model (no freeboard allowances). The lowest discharge that cannot be contained by the levee system was defined as the overtopping discharge for that project area. It was found that the location of the point which defined the design and overtopping discharges were the same.

The minimum freeboard for the existing projects is defined as the vertical distance between the computed profiles of the design and overtopping discharges at the location where the levee would be first overtopped.

Table 4 presents the design and overtopping discharges for the four existing projects. Plates 2 through 5 show the plotted design profiles. Each plate shows the following profiles: (1.) existing top of protection, (2.) design discharge, (3.) minimum freeboard, and (4.) overtopping discharge.

B. Proposed Conditions

Proposed conditions involves the raising of the existing levees to contain larger floods. To establish the proposed top of protection profile two design discharges were considered; a discharge of 318,500 cfs (recurrence of the Tropical Storm Agnes Flood) and a discharge of 290,000 cfs. For each of these discharges, two profiles were computed. The first profile representing the design flow was determined using the unadjusted model. For the second profile, the model was revised to reflect the previously described uncertainties. The difference between these two profiles constitutes the minimum freeboard. An overtopping analysis was then performed in accordance with ETL 1110-2-299.[6] Overtopping discharges of 387,500 cfs and 354,500 cfs were determined for the 318,500 cfs and 290,000 cfs design discharges, respectively.

To arrive at the final top of protection, levee superiority was added to the overtopping discharge profile. The superiority is the increment that is added to the overtopping discharge profile to assure overtopping would occur at the least damaging location, which is the downstream end of this project. For the two design discharges, superiority ranged from 0.0 feet at the downstream tie-out of Wilkes-Barre to 0.6 feet at the upstream tie-out of Swoyersville. It was then kept constant at 0.6 feet to the upstream end of the project. The overtopping discharge profile with superiority was compared to the minimum freeboard profile. The higher elevation of the two profiles determined the top of protection elevation. In most cases, the overtopping discharge profile plus superiority controlled the selection of the top of protection profile.

---

[6]U.S. Army Corps of Engineers, Engineer Technical Letter (ETL) 1110-2-299, Overtopping of Flood Control Levees and Floodwalls, 22 August 1986.

Plates 6 and 7 show plotted design profiles for the 318,500 cfs and 290,000 cfs design discharges, respectively. Each plate shows the following profiles: (1.) proposed top of protection, (2.) design discharge, (3.) minimum freeboard, and (4.) overtopping discharge.

4. SUMMARY

The Wyoming Valley freeboard analysis involved the analytical determination of the appropriate amount of freeboard for both the existing and proposed projects. The same methodology was used for both existing and proposed conditions. The methodology involved the assessment of the impacts of uncertainties in various factors, such as the accuracy of discharge estimates, debris blockage at bridges, contraction and expansion coefficients, and the accuracy of the estimation of the roughness factors. The minimum freeboard profile was determined by combining the impacts of the various factors. For existing conditions the minimum freeboard profile was utilized to define the design discharge for each of the four existing levee projects. These results are presented in Table 4. For the proposed conditions, the minimum freeboard profile and the results of an overtopping analysis were used to establish the top of levee profile. The higher of the minimum freeboard profile and the overtopping profile plus superiority defined the proposed conditions top of protection profile. The freeboard at the downstream end of the project was determined to be 2.7 feet for the 318,500 cfs design discharge and 2.5 feet for the 290,000 cfs design discharge. For comparison, a freeboard of 3.0 feet was previously utilized.

TABLE 1

WYOMING VALLEY, PA.
COMPARISON OF PUBLISHED PEAK DISCHARGE TO THE
DISCHARGE FROM THE LATEST RATING CURVE
FOR THE OBSERVED PEAK STAGE

WATER YEAR	DATE	OBSERVED STAGE (FEET)	DISCHARGE FROM RATING CURVE (FEET)	PUBLISHED DISCHARGE (CFS)	PERCENT DIFFER-ENCE
1950	3-30-50	27.04	170200	172000	-1.06
1951	4-01-51	22.72	133920	128000	4.42
1952	3-13-52	22.39	130290	124000	4.83
1953	12-12-52	19.43	100440	98000	2.43
1954	5-05-54	16.85	81980	78900	3.76
1955	3-03-55	17.80	88600	85900	3.05
1956	3-09-56	28.17	176020	186000	-5.67
1957	4-07-57	20.48	109800	107000	2.55
1958	4-08-58	26.80	169000	170000	-0.59
1959	1-23-59	21.14	116540	113000	3.04
1960	4-02-60	29.60	184000	184000	0.00
1961	2-27-61	26.20	165700	163000	1.63
1962	4-02-62	22.84	135240	128000	5.35
1963	3-28-63	22.26	128860	131000	-1.66
1964	3-10-64	30.15	186900	188000	-0.59
1965	2-14-65	11.10		44600	
1966	2-15-66	18.25	91750	93500	-1.91
1967	3-29-67	17.16	84120	84800	-0.81
1968	3-24-68	19.19	98520	101000	-2.52
1969	4-07-69	16.57	80076	80500	-0.53
1970	4-04-70	20.92	114200	115000	-0.70
1971	3-17-71	20.28	107800	110000	-2.04
1972	6-24-72	40.91		345000	
1973	4-06-73	18.04	90280	91800	-1.68
1974	12-28-73	18.24	91680	93400	-1.88
1975	9-27-75	35.06	228720	228000	0.31
1976	2-19-76	21.34	118740	118000	0.62
1977	9-26-77	21.62	121820	121000	0.67
1978	1-27-78	21.08	115880	116000	-0.10
1979	3-07-79	31.02	192120	192000	0.06
1980	3-23-80	19.50	101000	104000	-2.97
1981	2-22-81	19.57	101560	104000	-2.40
1982	10-29-81	17.24	84680	86400	-2.03
1983	4-16-83	23.86	145600	138000	5.22
1984	12-14-83	29.76	184800	185000	-0.11
1985	3-14-85	13.04	54712	55800	-1.99
1986	3-16-86	27.36	171800	172000	-0.12
1987	4-05-87	19.22	98760	98500	0.26

TABLE 2

WYOMING VALLEY, PA
DEBRIS BLOCKAGE AT BRIDGES

BRIDGE NAME	RIVER STATION	PERCENT OF FLOW AREA BLOCKED
CONRAIL RAILROAD*	130.80	0.0
PLYMOUTH-BRESLAU	218.90	3.5
CAREY AVENUE	253.20	2.5
D&H RAILROAD*	299.10	0.0
WILKES-BARRE CONNECTING RAILROAD	342.60	0.0
MARKET STREET	400.25	7.0
PIERCE STREET	421.55	2.5
WILKES-BARRE CONNECTING RAILROAD	459.80	0.0
CROSS VALLEY EXPRESSWAY	486.10	0.0
EIGHTH STREET	679.80	0.0
WATER STREET	805.65	2.0
FORT JENKINS	815.30	0.0
CONRAIL RAILROAD	853.35	0.0

* NOTE: BRIDGES TO BE REMOVED WITH IMPROVED CONDITIONS

TABLE 3

WYOMING VALLEY, PA.
VARIATION IN MANNING'S N-VALUES FROM LITERATURE SEARCH

REFERENCE	MANNING'S N-VALUE RANGE	PERCENT VARIATION
CHOW 1/	0.03 - 0.08	167
KING 2/	0.042 - 0.064	52
ABBETT 3/	0.04 - 0.10	150
HENDERSON 4/	0.044 - 0.073	66

1/ Open-Channel Hydraulics, Ven Te Chow, Ph.D., 1959.

2/ Handbook of Hydraulics, Horace W. King, 4th Edition, 1954.

3/ American Civil Engineering Practice, Volume II, Robert W. Abbett, 1956.

4/ Open Channel Flow, F. M. Henderson, 1966.

TABLE 4

WYOMING VALLEY, PA
DESIGN AND OVERTOPPING DISCHARGES
FOR THE EXISTING PROJECTS

EXISTING PROJECT	DESIGN DISCHARGE (CFS)	OVERTOPPING DISCHARGE (CFS)
WILKES-BARRE/HANOVER	229,000	272,000
PLYMOUTH	239,000	292,000
KINGSTON-EDWARDSVILLE	224,000	267,000
SWOYERSVILLE-FORTY FORT	244,000	289,000

WYOMING VALLEY, PA.

DEBRIS BLOCKAGE

CONFIGURATION AT PIERS
FOR PERCENT AREA
REDUCTION CALCULATION

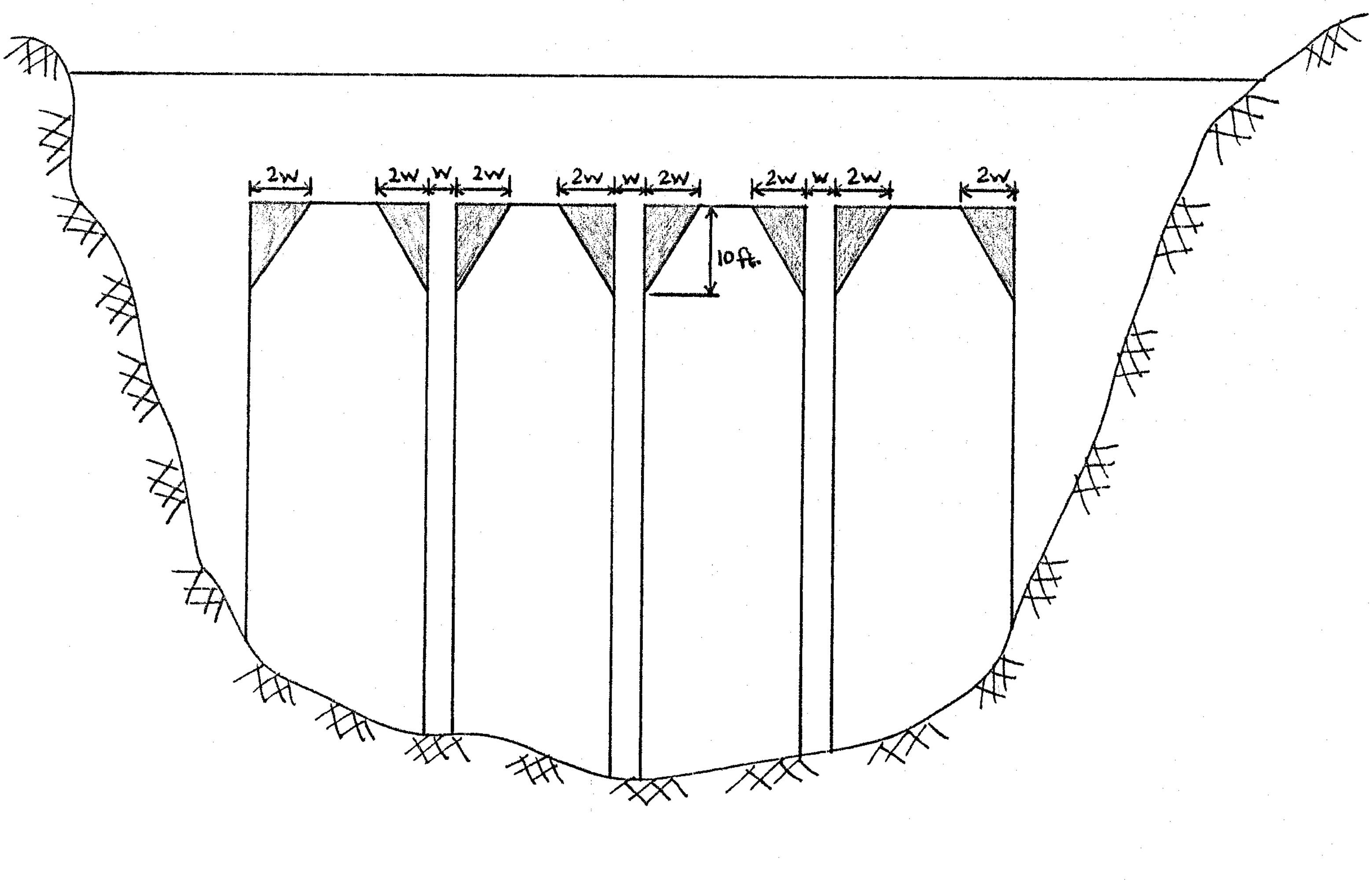


FIGURE 1

WYOMING VALLEY, PA.

SKETCH DISPLAYING
EXPANSION AND CONTRACTION
AT A BRIDGE

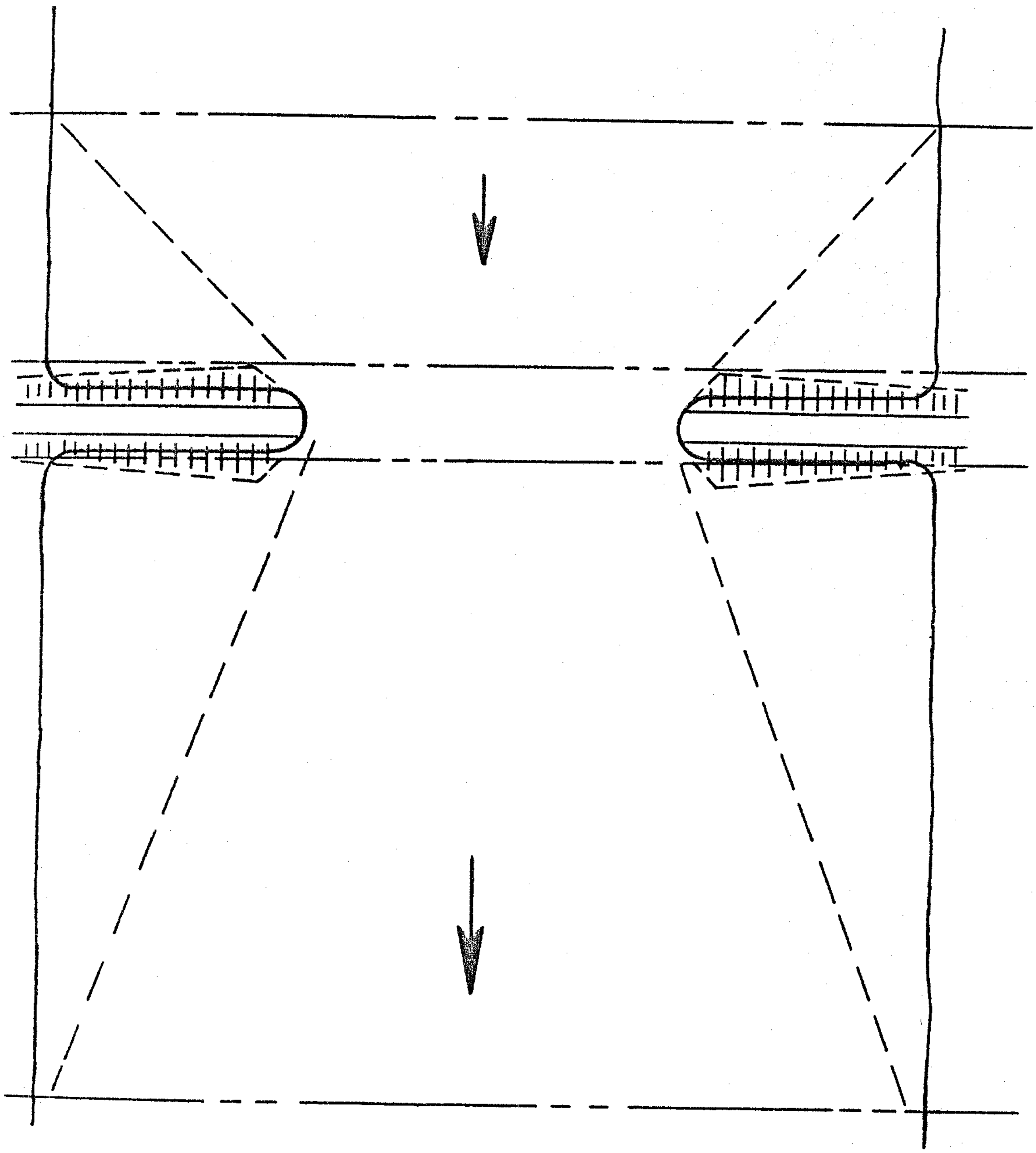


FIGURE 2

PAPER 7

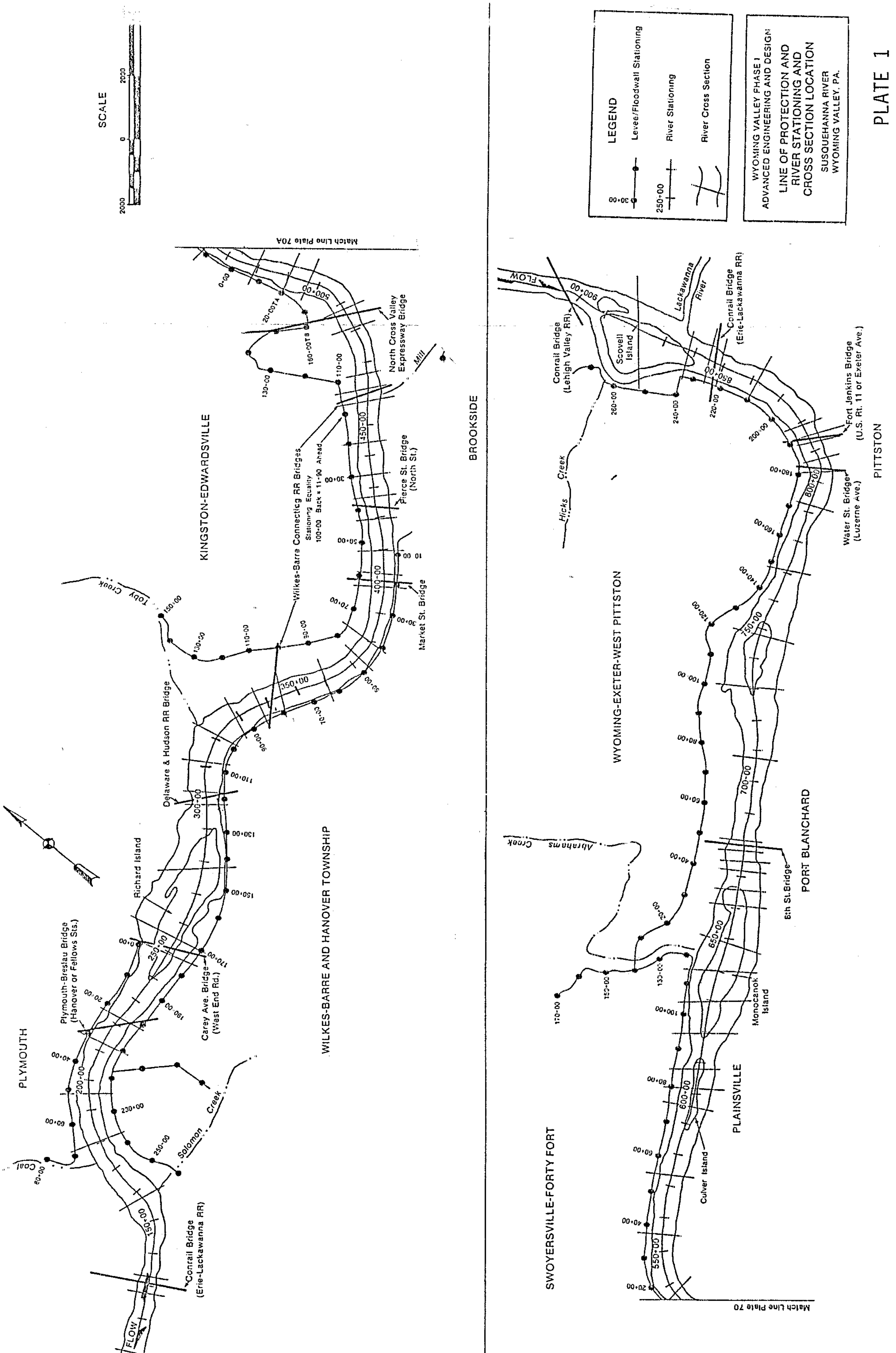


PLATE 1
WYOMING VALLEY PHASE I
ADVANCED ENGINEERING AND DESIGN
LINE OF PROTECTION AND
RIVER STATIONING AND
CROSS SECTION LOCATION
SUSQUEHANNA RIVER
WYOMING VALLEY, PA.
LEGEND
Levee/Floodwall Stationing
River Stationing
River Cross Section
SCALE
Match Line Plate 70A
Match Line Plate 70
KINGSTON-EDWARDSVILLE
BROOKSIDE
WILKES-BARRE AND HANOVER TOWNSHIP
PLYMOUTH
WYOMING-EXETER-WEST PITTSTON
PITTSTON
PORT BLANCHARD
PLAINSVILLE
SWOYERSVILLE-FORTY FORT
North Cross Valley Expressway Bridge
Mill
Wilkes-Barre Connecting RR Bridges
Pierce St. Bridge (North St.)
Market St. Bridge
Toby Creek
Delaware & Hudson RR Bridge
Richard Island
Plymouth-Breslau Bridge (Hanover or Fellows Sts.)
Carey Ave. Bridge (West End Rd.)
Solomon Creek
Coal Creek
Conrail Bridge (Erie-Lackawanna RR)
FLOW
Conrail Bridge (Lehigh Valley RR)
Scovell Island
Lackawanna River
Fort Jenkins Bridge (U.S. Rt. 11 or Exeter Ave.)
Water St. Bridge (Luzerne Ave.)
Hicks Creek
Abrahams Creek
8th St. Bridge
Monocanok Island
Culver Island

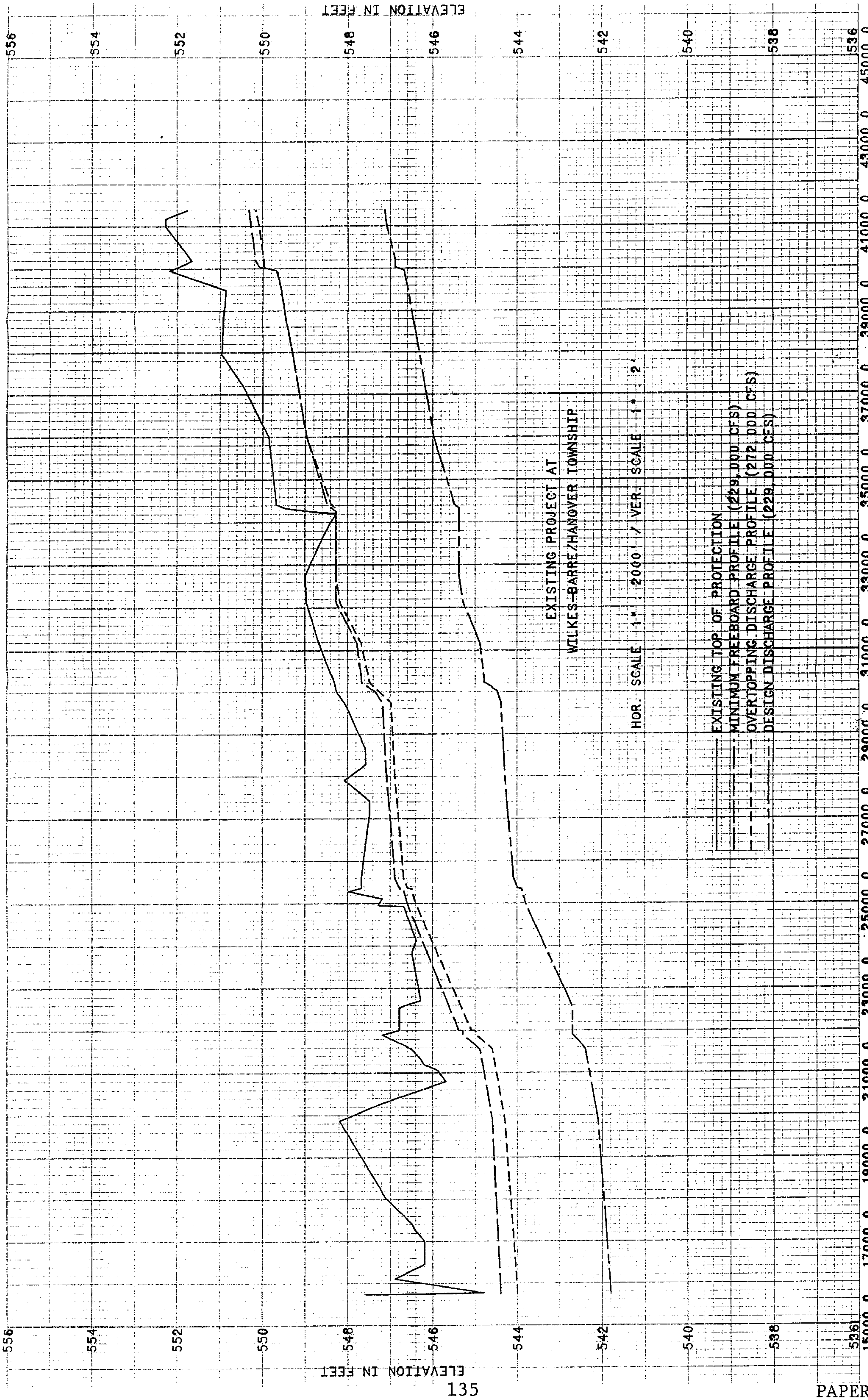

EXISTING PROJECT AT
WILKES-BARRE/HANOVER TOWNSHIP
HOR. SCALE 1" : 2000' / VER. SCALE 1" : 2'
EXISTING TOP OF PROTECTION
MINIMUM FREEBOARD PROFILE (229,000 CFS)
OVERTOPPING DISCHARGE PROFILE (272,000 CFS)
DESIGN DISCHARGE PROFILE (229,000 CFS)
ELEVATION IN FEET
RIVER STATION IN FEET
15000.0
17000.0
19000.0
21000.0
23000.0
25000.0
27000.0
29000.0
31000.0
33000.0
35000.0
37000.0
39000.0
41000.0
43000.0
45000.0
536
538
540
542
544
546
548
550
552
554
556

PLATE 2

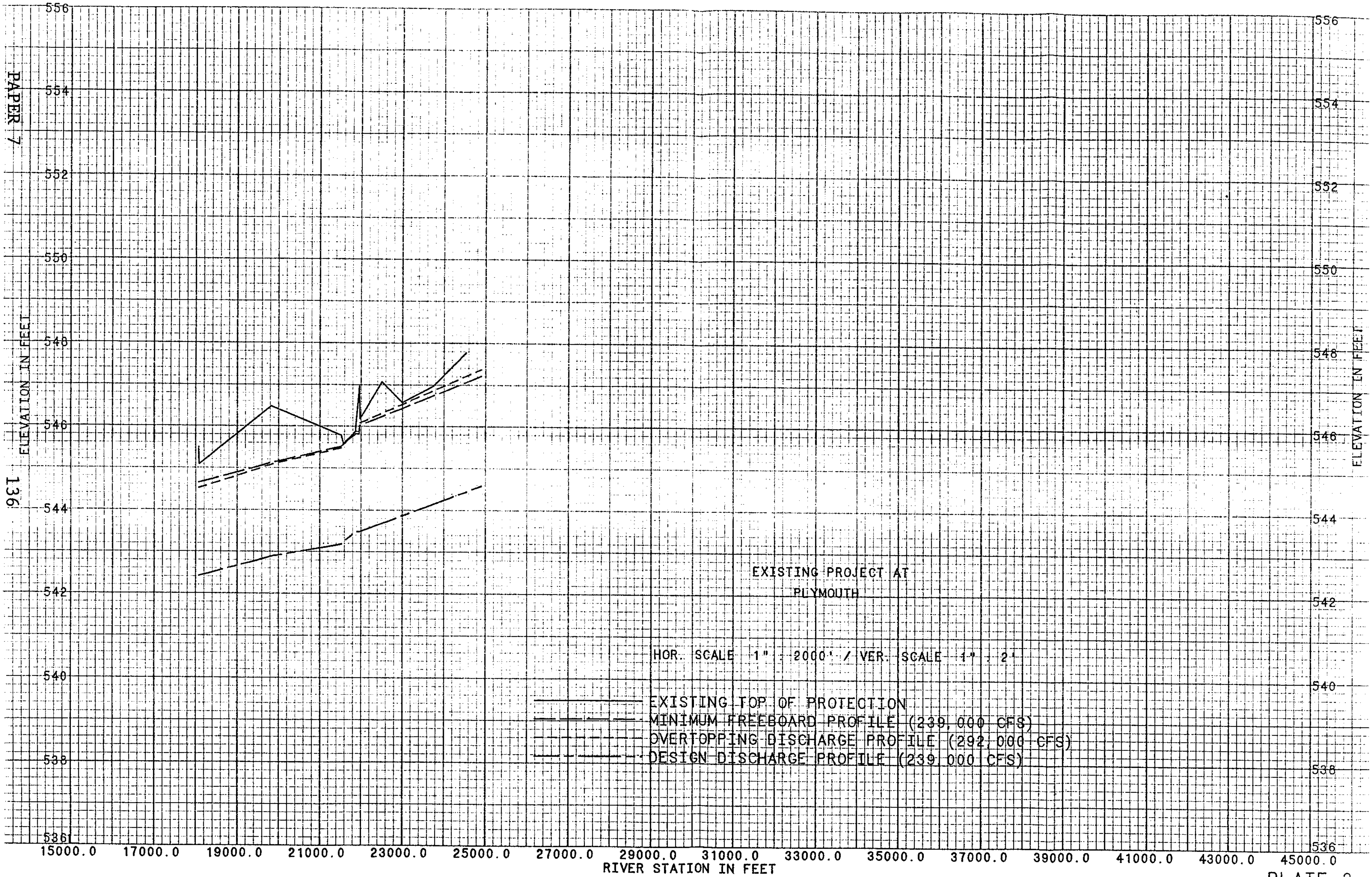

ELEVATION IN FEET
556
554
552
550
548
546
544
542
540
538
536
EXISTING PROJECT AT
PLYMOUTH
HOR. SCALE 1" : 2000' / VER. SCALE 1" : 2'
EXISTING TOP OF PROTECTION
MINIMUM FREEBOARD PROFILE (239,000 CFS)
OVERTOPPING DISCHARGE PROFILE (292,000 CFS)
DESIGN DISCHARGE PROFILE (239,000 CFS)
15000.0
17000.0
19000.0
21000.0
23000.0
25000.0
27000.0
29000.0
31000.0
33000.0
35000.0
37000.0
39000.0
41000.0
43000.0
45000.0
RIVER STATION IN FEET

PLATE 3

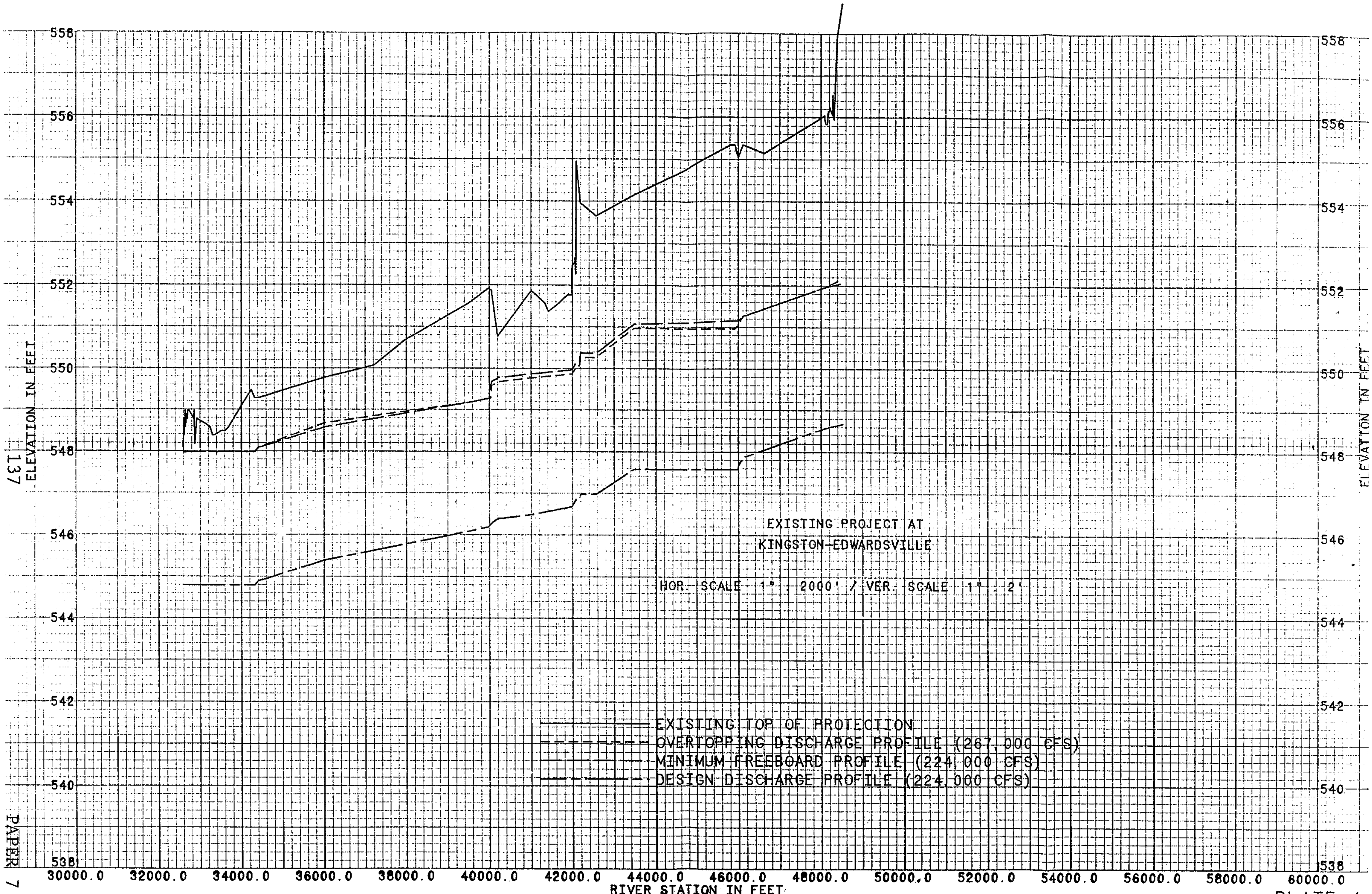

EXISTING PROJECT AT
KINGSTON-EDWARDSVILLE
HOR. SCALE 1" : 2000' / VER. SCALE 1" : 2'
EXISTING TOP OF PROTECTION
OVERTOPPING DISCHARGE PROFILE (267,000 CFS)
MINIMUM FREEBOARD PROFILE (224,000 CFS)
DESIGN DISCHARGE PROFILE (224,000 CFS)
ELEVATION IN FEET
558
556
554
552
550
548
546
544
542
540
538
RIVER STATION IN FEET
30000.0
32000.0
34000.0
36000.0
38000.0
40000.0
42000.0
44000.0
46000.0
48000.0
50000.0
52000.0
54000.0
56000.0
58000.0
60000.0

PLATE 4

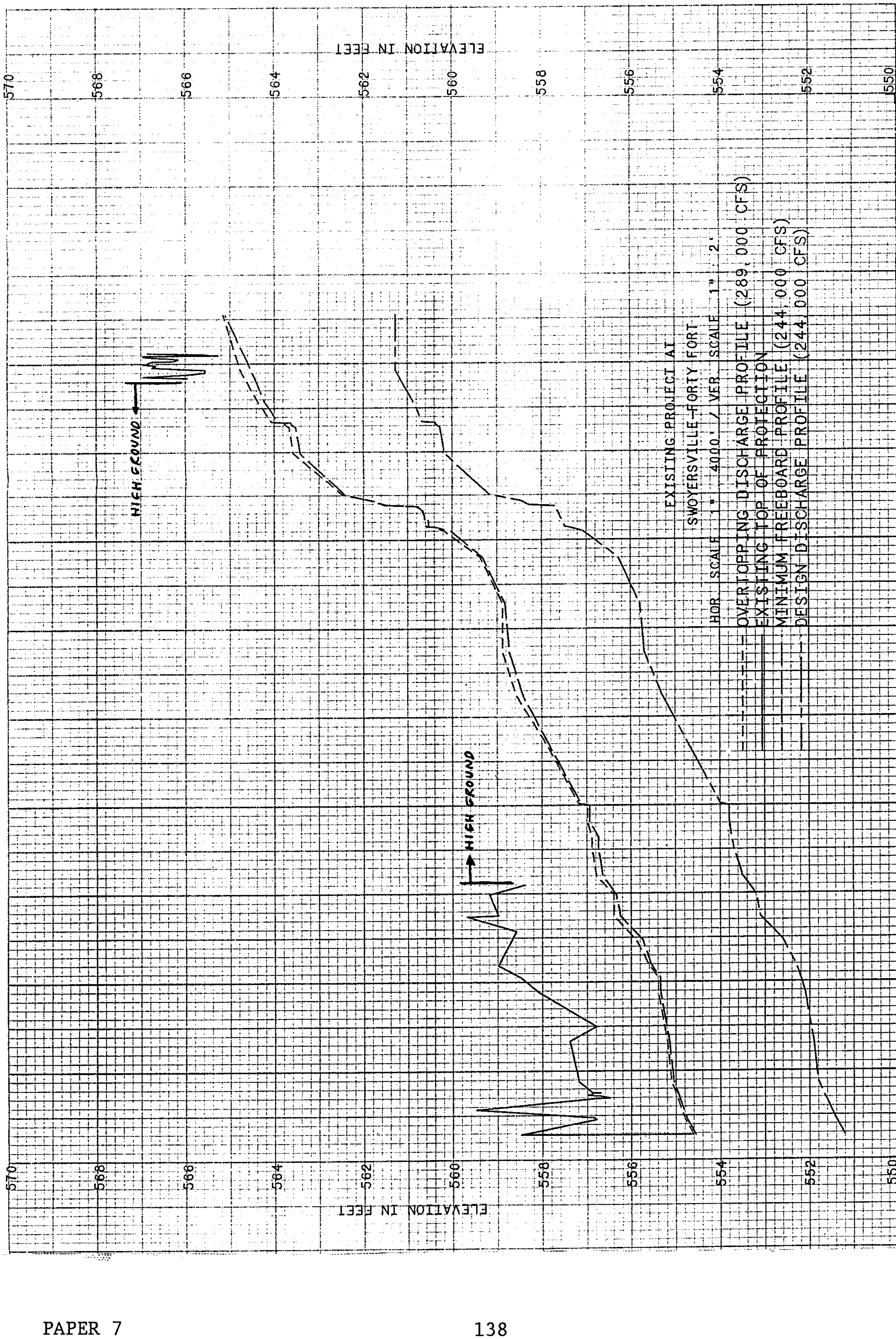

EXISTING PROJECT AI
SWOYERSVILLE-FORTY FORT
HOR. SCALE 1" = 4000' / VER. SCALE 1" = 2'
OVERTOPPING DISCHARGE PROFILE (289,000 CFS)
EXISTING TOP OF PROTECTION
MINIMUM FREEBOARD PROFILE (244,000 CFS)
DESIGN DISCHARGE PROFILE (244,000 CFS)
HIGH GROUND
HIGH GROUND
ELEVATION IN FEET
570
568
566
564
562
560
558
556
554
552
550
RIVER STATION IN FEET
52000.0
56000.0
60000.0
64000.0
68000.0
72000.0
76000.0
80000.0
84000.0
88000.0
92000.0
96000.0
100000.0

PLATE 5

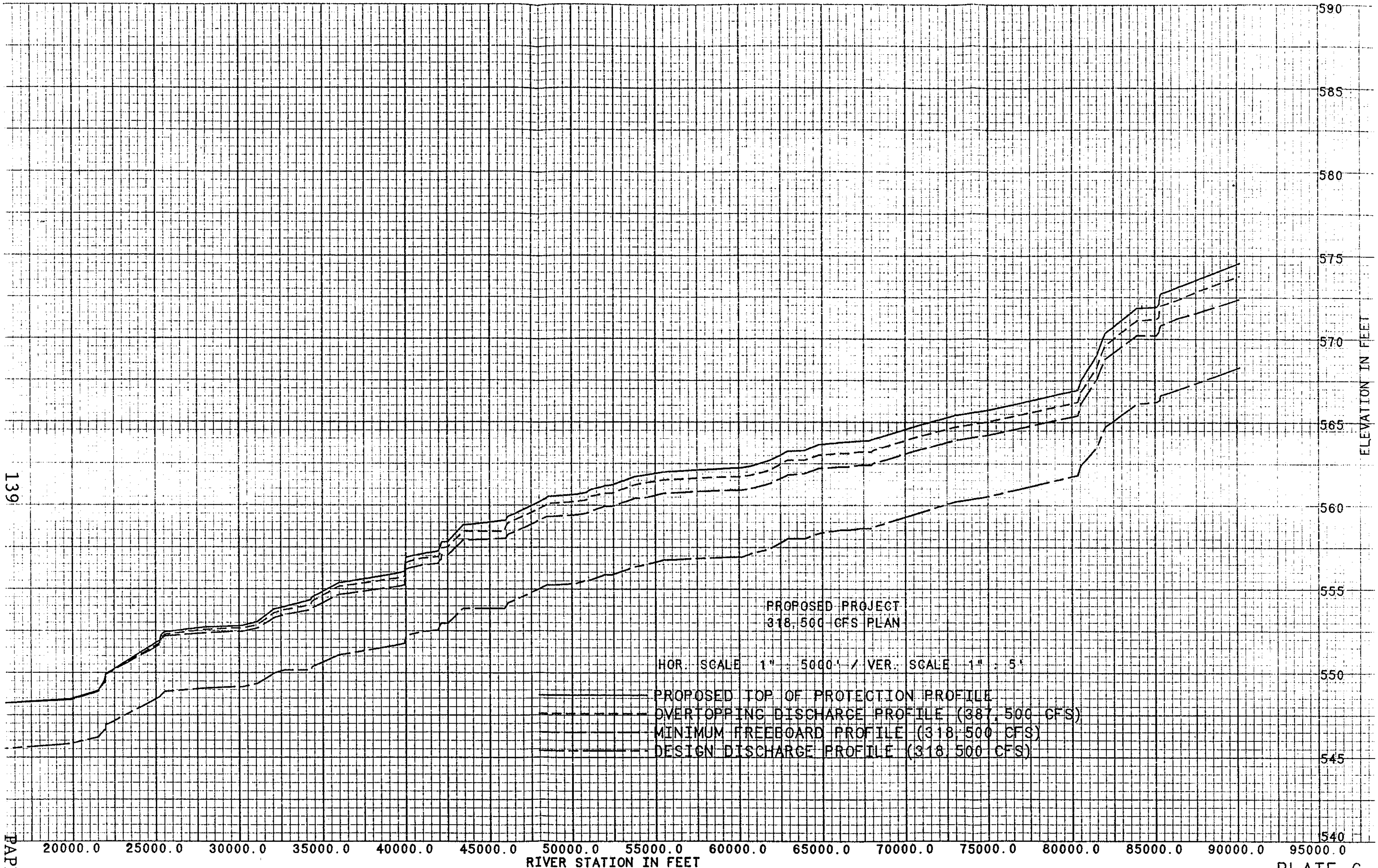

590
585
580
575
570
565
560
555
550
545
540
ELEVATION IN FEET
20000.0
25000.0
30000.0
35000.0
40000.0
45000.0
50000.0
55000.0
60000.0
65000.0
70000.0
75000.0
80000.0
85000.0
90000.0
95000.0
RIVER STATION IN FEET
PROPOSED PROJECT
318,500 CFS PLAN
HOR. SCALE 1" : 5000' / VER. SCALE 1" : 5'
PROPOSED TOP OF PROTECTION PROFILE
OVERTOPPING DISCHARGE PROFILE (387,500 CFS)
MINIMUM FREEBOARD PROFILE (318,500 CFS)
DESIGN DISCHARGE PROFILE (318,500 CFS)

PLATE 6

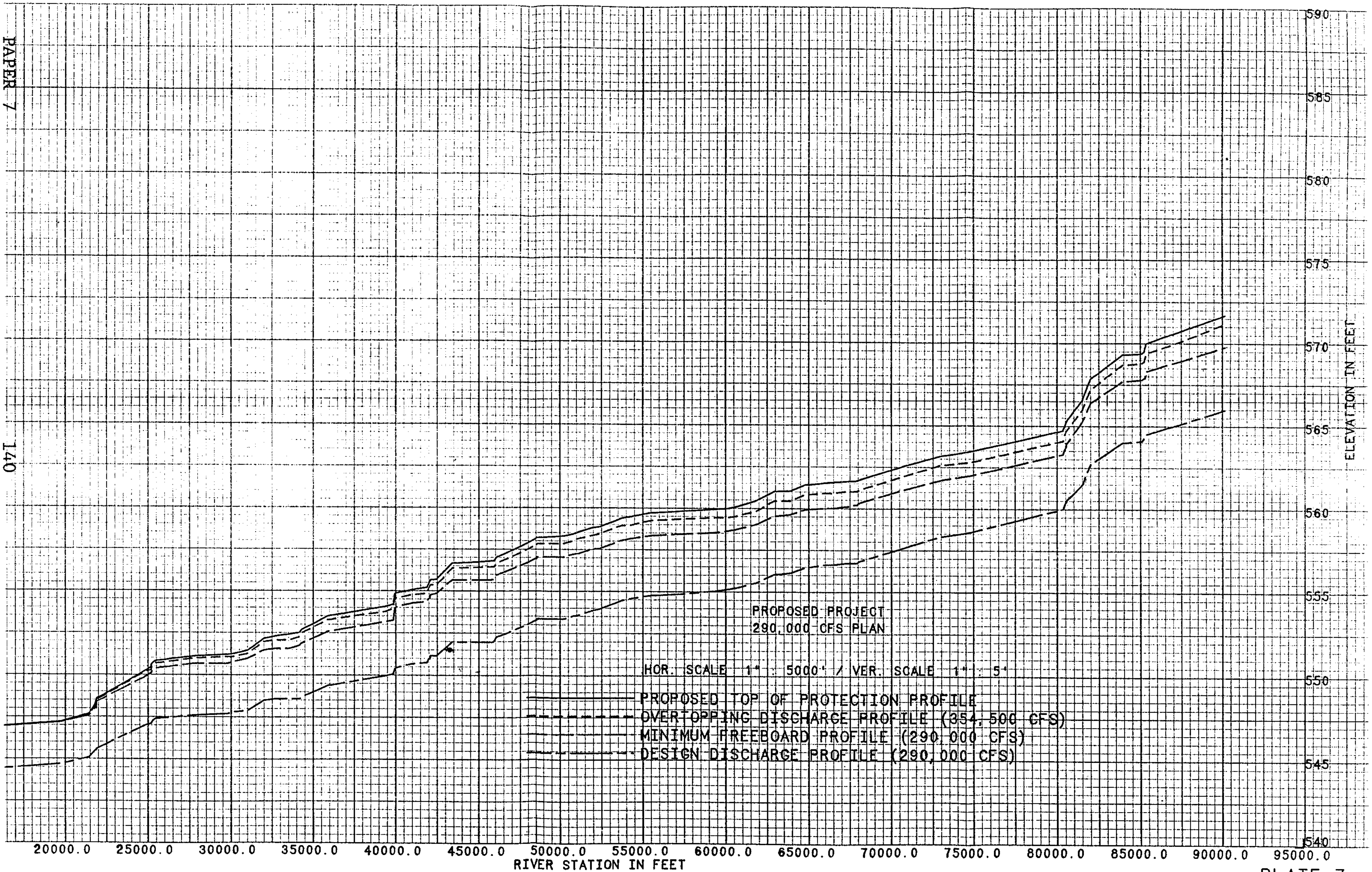

590
585
580
575
570
565
560
555
550
545
540
ELEVATION IN FEET
20000.0
25000.0
30000.0
35000.0
40000.0
45000.0
50000.0
55000.0
60000.0
65000.0
70000.0
75000.0
80000.0
85000.0
90000.0
95000.0
RIVER STATION IN FEET
PROPOSED PROJECT
290,000 CFS PLAN
HOR. SCALE 1" : 5000' / VER. SCALE 1" : 5'
PROPOSED TOP OF PROTECTION PROFILE
OVERTOPPING DISCHARGE PROFILE (354,500 CFS)
MINIMUM FREEBOARD PROFILE (290,000 CFS)
DESIGN DISCHARGE PROFILE (290,000 CFS)

PLATE 7

LEVEE FREEBOARD DESIGN IN THE WALLA WALLA DISTRICT

By

Lester L. Cunningham[1]

INTRODUCTION

Key Issues. A review of past levee studies indicates that freeboard design often has not been characterized by the rigorous, complete analysis that generally characterizes other aspects of project design. Perhaps this should not be surprising since freeboard, by its very nature, is designed to accommodate - and mitigate - for risks associated with factors which are unknown or uncertain at the time of project design. Where sufficient data is lacking minimum freeboards of 2 ft. for agricultural, and 3 ft. for residential areas seem to have gained widespread acceptance. In cases where a high degree of uncertainty exists the engineer may specify a higher freeboard allowance or part of the freeboard may be effectively hidden within an overly-conservative water-surface profile calculation.

Tighter budgets in recent years have dictated a closer look at all aspects of a project which may reduce costs. Higher level reviewers have recently begun to demand that any freeboard beyond the established minimums be justified by an evaluation of identifiable risks and uncertainties. The amount of freeboard required as well as the appropriate level of protection has come under close scrutiny in several completed projects including the Lewiston and Tri-Cities Levees. Attempts to resolve these questions has been a challenge and has raised some policy questions which have not been entirely resolved.

Criteria for Freeboard Design. The Walla Walla District has generally relied on EM 1110-2-1601 ("The Design of Flood Control Channels"), and Bulletin 54-14 for guidance in establishing the minimum freeboard allowance for riverine levees.

REEVALUATION OF EXISTING PROJECTS

General Description. Two existing levee projects: The Lewiston Levees on the Snake River in Idaho and the Tri-Cities Levees on the Columbia River in Washington have been reevaluated recently. Both projects were constructed in lieu of purchasing land which would have been inundated or exposed to a higher risk of flooding by downstream dams. The levees are about 150 miles apart allowing local government officials to compare and contrast the projects in their respective areas of jurisdiction. See Figure 2. Both levees are at, or near the upper end of man-made reservoirs which have the characteristics of a lake at normal flows. However, during a Standard Project Flood (SPF) the leveed reaches take on many of the characteristics of a river. Average SPF flow velocities range up to 12 fps, there is a distinctive slope to the water surface, and, in the case of the Lewiston Levees reach, the downstream dam has very limited influence on the SPF flood stage.

---

[1]Hydraulic Engineer, Walla Walla District, U.S. Army Corps of Engineers

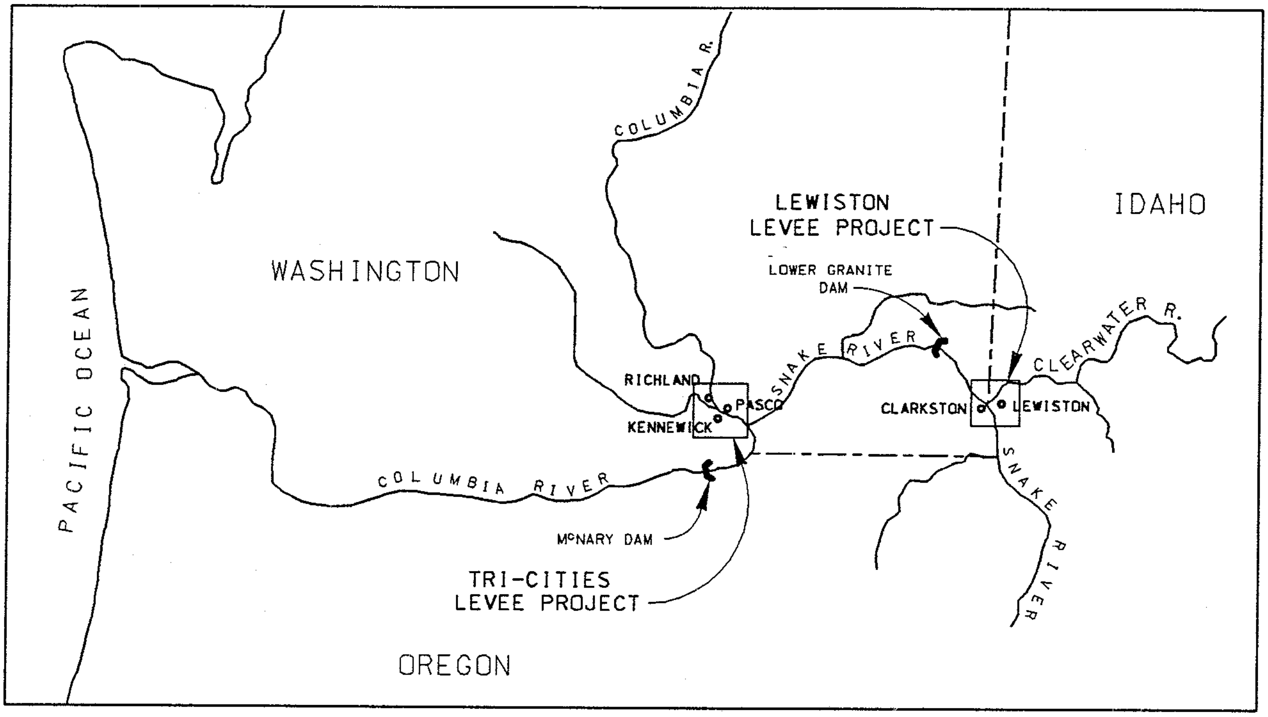


Figure 1. Vicinity map indicating locations of two recently-reevaluated levee projects.

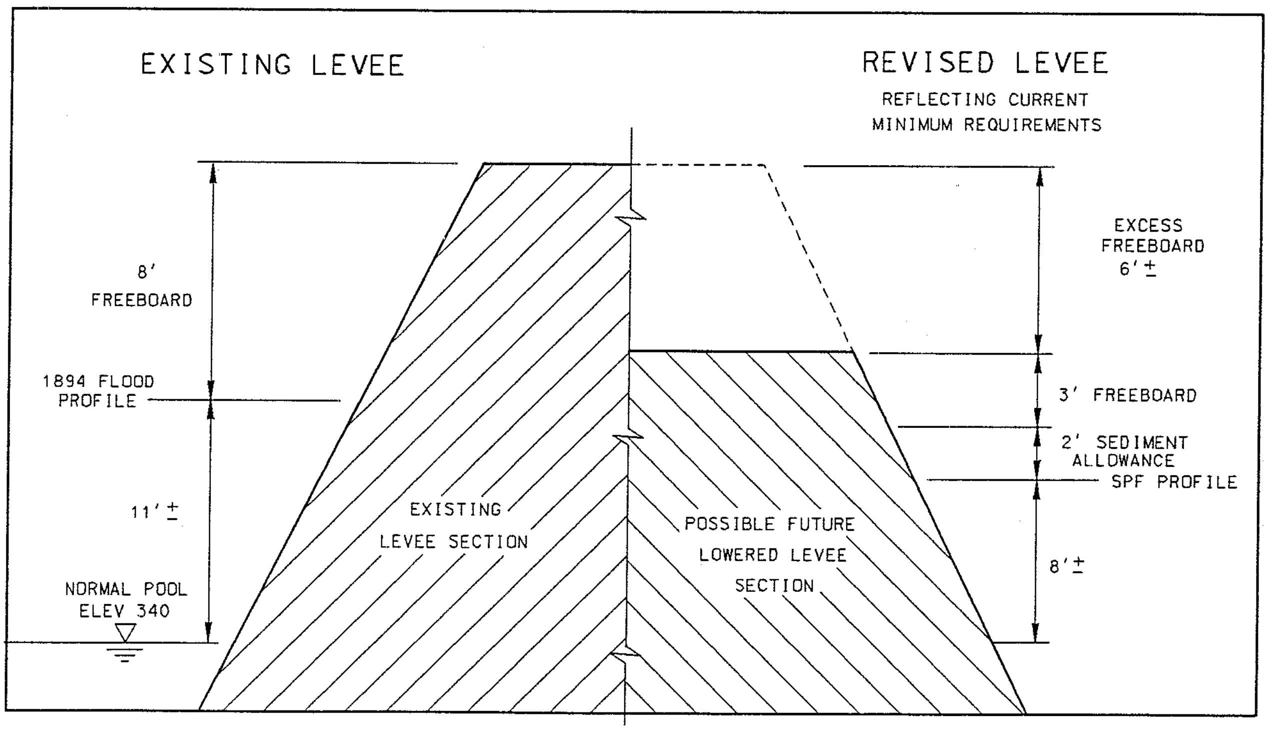


Figure 2. Original levee design compared with current minimum requirements. Dimensions apply to Cable Bridge location.

The Tri-Cities Levees.

1) Background. McNary Dam located at River-Mile(RM) 292 on the Snake River creates a slack-water reach which extends upstream some 60 miles through the Cities of Richland, Pasco, and Kennewick, Washington. The tri-cities levees, which are a part of this project, were constructed in the early 1950's when esthetic and environmental concerns were not a high priority.

During the planning process it was decided that the largest known flood event (1894) provided the most reliable basis for project design. Some uncertainty existed regarding the magnitude of possible floods and the degree of control which could be expected from future upstream flood control projects. In order to assure a measure of safety, the unregulated flood peak was used for the design profile. In addition, eight feet of freeboard was provided for urban, and 5 feet for agricultural areas. Weed killer is used to keep down unwanted vegetation, leaving the levees an ugly black or brown. The riprap-covered levees rising stark and bare some 20 feet above the maximum operating pool creating a visual obstruction to landward developments.

In the years since project completion several dams have been constructed upstream, significantly reducing the flood hazard. In an effort to revitalize the economy the locals have developed a plan which envisions a landscaped levee system forming a green belt along the banks of the Columbia River. Local developers would like to see the levee lowered by about 10 feet. Lowered and widened levees (using borrow from the tops of the levees) could become sites for new businesses with a view overlooking the Columbia River.

In contrast, the nearby Lewiston Levees, built in the 1970's, are heavily landscaped, with grass, potted trees, bike trails, interpretive centers, ponds and parks. However, sediment depositing in the pool has reduced the available freeboard to only three feet. These levees may eventually have to be raised.

2) Original Freeboard Design. The eight feet for urban and five feet for agricultural areas were chosen to ensure that the levees would not be overtopped by a Standard Project level discharge. The fact that a levee failure would result in extreme danger to human life and heavy property damage in the highly developed urban areas was a major consideration. The following uncertainties were also considered:

a) Calculated vs actual water surface profile.

b) Adequacy of the selected design discharge.

c) Local effects of blockage and debris.

d) Possible channel aggradation.

e) Wind-wave effects on the water surface profiles.

3) Current Freeboard Requirements. In the current study, the above factors were reevaluated, taking into consideration the increased regulation provided by many new projects on the Columbia and Snake Rivers. Since project completion in late 1953, the sediment-contributing basin area had decreased from 62,200 mi^2 to 4,300 mi^2. The project was originally designed to pass the

unregulated 1894 flood (1,200,000 cfs). The present revised Standard Project Flood is 839,000 cfs.

In the years leading up to the reevaluation, 3 ft was included in the freeboard allowance to cover the effects of future channel aggradation. The present allowance, although still a rough estimate, has been reduced to 2 ft., which has been added to the design water surface profile rather than the freeboard since it represents a predictable process. A rise of 1.6 ft was based on a linear extrapolation of the calculated SPF rise during the first 14 years of the project life adjusted for the estimated effects of changes in the contributing basin area. This was later rounded up to 2.0 feet to cover uncertainties in the method. It could be argued that the additional 0.6 ft was an uncertainty which should have been added to the freeboard.

Civil Works Engineer Bulletin 54-14 suggests a minimum freeboard allowance of two feet for agricultural, and three feet for urban areas to account for unexpected variations from design profiles. Since most of the levees in the Tri-Cities area protect urban or high density development, three feet was selected for this uncertainty.

According to EC 1110-2-27, "Policies and Procedures Pertaining to Determination of Spillway Capacities and Freeboard Allowances for Dams," the larger of the two values, the minimum freeboard to account for variations in design water surface profile, described in the preceding paragraph, and the wind-wave action freeboard allowance specified above, should be used as the freeboard requirement. After studying wind wave requirements, it was determined that a 3-ft allowance would be adequate. The final minimum top-of-levee consisted of the design water surface profile, a 2-ft sediment allowance, and 3 ft of freeboard. Figure 2 compares the original levee design with minimum levee height requirements after the reevaluation.

4) Risk Associated with Levee Height Reductions. Although the above section describes the process used to establish the minimum required freeboard based on present Corps' guidelines, it would not necessarily be beneficial to reduce the freeboard to this minimum. The existing project provides a margin of safety against major accidents or rare and unexpected flood events at no additional cost to the government. Any action taken to reduce the levee height would increase the risk of flooding to some degree. The following are some examples:

a) Flows exceeding the calculated SPF from rare or unexpected hydrometeorological events unaccounted for in the design.

b) Unanticipated sediment buildup in the channel.

c) An error in operation of the upstream reservoir system causing high flows, or failure of gate lowering equipment in the downstream project resulting in an unexpectedly-high hydraulic control.

d) Extremely high flows resulting from a dam failure.

The Lewiston Levees.

1) Background. The Cities of Lewiston and Clarkston are located at the confluence of the Snake and Clearwater rivers at the point where the Snake River bends southward forming the east-west boundary between the states of Washington and Idaho. Lower Granite Dam, located on the Snake River about 32 miles downstream of the City of Lewiston, Idaho, creates a slack-water pool which extends up to, and several miles beyond, the Cities of Lewiston and Clarkston. A series of levees (called Lewiston Levees) form an integral part of the Lower Granite Project, starting in Lewiston and extending upstream along both the Snake and Clearwater rivers. The levees were designed to safely pass the Standard Project Flood with a minimum of 5 ft of freeboard remaining above the calculated water surface profile.

Without the levees, a large portion of the downtown business and hotel district of Lewiston, valued at over one billion dollars, would be inundated during normal project operation. Depths of flooding could approach 18 feet in some areas of the city if failure occurred during a major flood. With the relatively great depth of inundation and rapid flooding once failure had been initiated, it is likely that some lives could be lost.

Although the project design called for a 5-ft freeboard allowance, 6 ft of freeboard was available with all spillway gates open when the project was completed. Since Lower Granite Pool was first raised in February 1975, over 40 million cubic yards of sediment has deposited in the reservoir. The most recent analysis, based on the fall, 1989 sediment range resurvey, indicates that only 3 ft of freeboard would remain at the peak of the Standard Project Flood. If sediment continues to enter the reservoir at the same rate the SPF water surface could rise an additional 10 feet during the remaining 84 years of the project life.

The only options that appear viable now are rising the levees and dredging the river channel. Both are very expensive, and require environmental tradeoffs. This has led to calls for a reevaluation of the Standard Project flood to see if it could be reduced; possible reduction of the level of protection provided by the levees, and a reevaluation of the required levee freeboard.

2) Freeboard Analysis. With regard to freeboard, Lower Granite DM #3, paragraph 6-4 states that "five feet of elevation above the standard project flood backwater profile would generally serve project requirements as far as inundation and normal wave action are concerned."

Although 5-ft freeboard above the standard project flood has been the adopted level of protection throughout most of the design process, the original computations summarizing all factors involved in this determination have not been found to date.

EM 1110-2-1601, Paragraph 12.a defines freeboard as the vertical distance above the design water surface needed to insure that the desired degree of protection will not be reduced by unaccounted factors. A list of these factors includes erratic hydrologic phenomena, accumulation of silt, trash, and debris, and variation in resistance from the assumed values. Some of the more important known risks and associated levels of uncertainty are listed in Table 1.

TABLE 1

Risk Factor	Best Estimate feet	Probable Range feet
Wind-Wave Runup	2.8	+1.7 - +3.1
SPF Sediment Inflow	0.4	0.0 - +1.0
Model Calibration	1.0	-2.0 - +2.0
SPF Flow Distribution	0.0	-1.0 - +0.0
Project Operation Error	1.7	-0.0 - +6.0
Bridge Obstruction	1.5	0.0 - +2.0

a) Sedimentation allowance. Sedimentation effects can be broken into classes: Long-term progressive buildup, and the sediment brought in by a single major flood. Based on sediment transport modeling using program HEC-6 it was estimated that sedimentation might raise the SPF profile as much as 14 ft during the 100-year project life.

The model predicted a freeboard loss of up to 0.4 ft during the Standard Project Flood event. Figure 3 indicates the spread of individual instantaneous sediment inflow measurements collected by the USGS during the years 1972 to 1979. (Jones, 1980). Model limitations inherent in the program adds additional uncertainly.

b) Roughness Calibration. Figure 4 indicates the effect of changes in roughness on the water surface profile at the confluence. An error in estimating the Manning's coefficient by 0.002 results in a 1 ft change in the calculated water surface. A coefficient of 0.024 would provide a minimum of 5 ft of freeboard during the SPF. However, the SPF would overtop the levee if the actual coefficient was 0.034.

A roughness coefficient of 0.028 downstream and 0.030 upstream of the Clearwater appeared to provide the best fit for major flood profiles measured between 1948 and 1956. See Figure 5. Note that the calculated profile differs from the measured profile by as much as 2 feet. Levee construction and other channel changes have added to the uncertainty in estimating the roughness coefficient.

c) Flow Distribution. In the development of the SPF a slight lag in the hydrograph peaks for the Snake and the Clearwater Rivers was assumed. Coincidence of the peaks would raise the SPF by 15,000 cfs.

d) Project Operation Error. Operator or forecasting errors can result in higher flows than would normally be expected. As an example: Following normal operating procedures Dworshak Reservoir was allowed to refill at the end of the flood season. Unexpected warm weather suddenly melted the remaining snow causing a flood which peaked after the reservoir had filled. The peak flow that resulted was within 20% of the Standard Project Flood for that stream. A similar loss of control during actual Standard Project Flood conditions could raise the profile by 1.7 feet. Although, unlikely, loss of control of all upstream projects could convert a 20 year flood into a 500-year event.

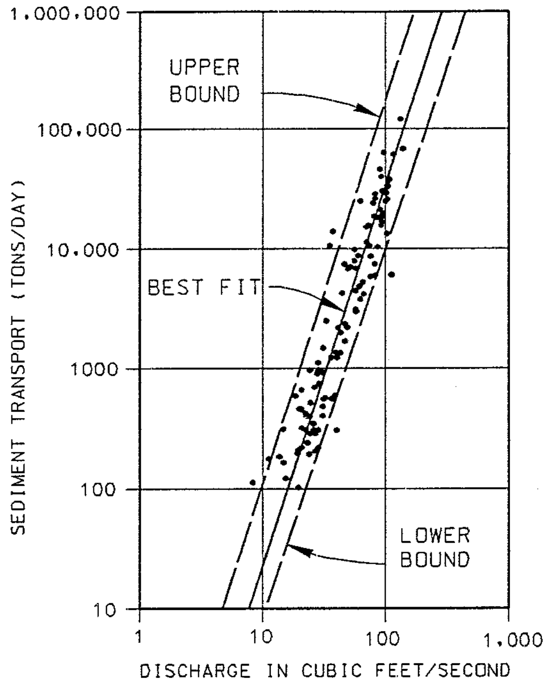


Figure 3. Suspended sediment transport. Snake River near Anatone, 1972-1979.

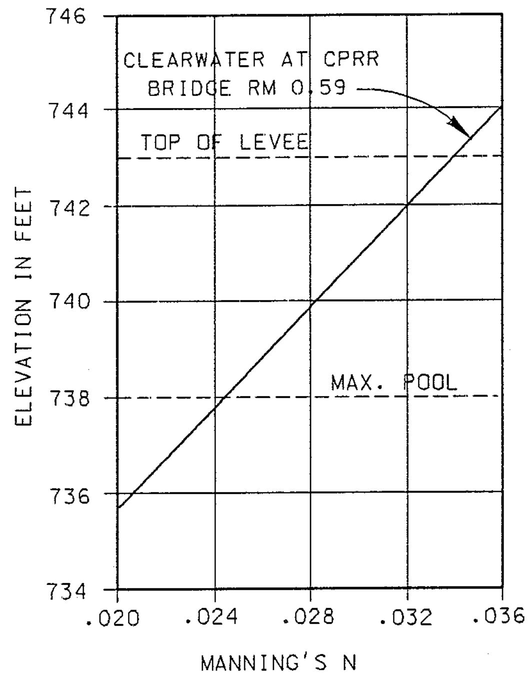


Figure 4. Effect of varying Manning's n on calculated SPF Flood stage.

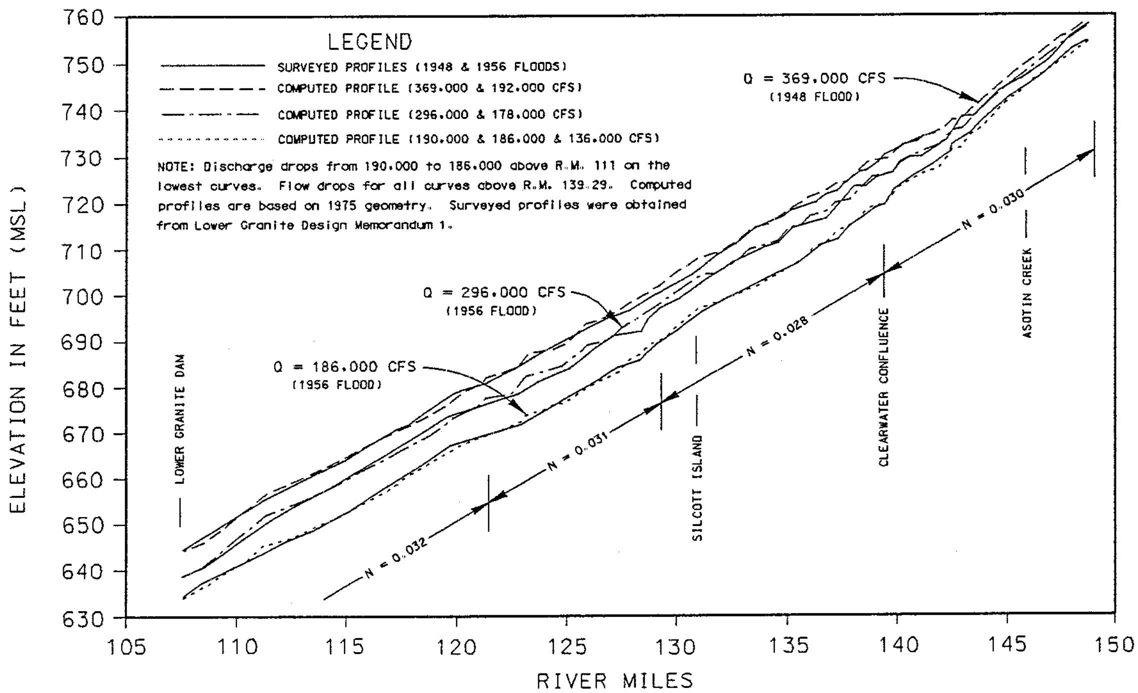


Figure 5. Comparison of calculated vs observed water surface profiles on the Snake River.

e) Debris Obstruction. A large quantity of floating debris (trees, logs, parts of buildings, etc.) could be expected during a SPF. However, a very serious situation could develop if a barge were to drift downstream, become lodged against the CPRR Bridge piers, and sink. An average-sized barge at this narrow point in the river could block about 30% of the flow area and raise the SPF water surface about 1.5 ft. See Figure 6. Blockage by an entire barge tow would be even more serious.

3) Recommended Freeboard Allowance. In keeping with the provisions of Civil Works Engineering Bulletin 54-14, Section 3,c,(5) it would appear reasonable to include in the freeboard an allowance for estimated SPF sedimentation, wind-wave run-up, and an additional allowance for at least one of the several unknown or unexpected factors such as model calibration error, project operation error, or bridge debris blockage. The above combination would result in a freeboard requirement 4.2 to 4.9 ft. Although a final decision on the freeboard has not been made, the outcome may rest on an evaluation of the probability of several additive uncertainties occurring at the same time at the peak of the SPF.

DISCUSSION

The Importance of Prospective. Does the design represent a new project? The freeboard for most projects is justified during the planning stage. The cases described above involved a reevaluation of an existing design in which funds have already been expended for the existing freeboard. A physical modification which would reduce the margin of safety provided by the existing freeboard might be difficult to justify economically.

Will the proposed levee provide a degree of protection against existing flood hazards, or does the levee provide protection against an imposed risk by the downstream project? In many new levee projects any levee height will provide additional protection against a naturally occurring flood risk, and the cost of providing this protection must be balanced against the benefits. However, in previously described projects a downstream dam imposed a greater risk of flooding on an upstream community. The levees were built in lieu of land acquisition. Is there an ethical obligation to provide a greater margin of safety (more than the usual freeboard) to assure that the levee will not be overtopped by the design flood in this case? What role does the risk to human life play in the selection of an appropriate freeboard allowance? Should economics always be the controlling consideration?

Confidence Limits. It is often possible to establish confidence limits, upper and lower bounds, or a worst case scenario for factors which cannot be precisely determined. Examples are: flood frequencies, estimates of roughness coefficients, and sediment transport. See figures 3 and 6. A sensitivity analysis can then be performed to convert these limits to minimum and maximum water surface profiles. It may then be possible to establish an upper limit for each factor or combination of factors. The selected freeboard would depend on the required level of assurance of non-failure.

Levee Location or Configuration. Figure 7 illustrates several possible levee positions relative to the river. A river confined on both sides by levees may require greater freeboard than a channel with offset levees or a levee on only one side of the channel. The overbank area can often provide additional conveyance with considerably less rise in the water surface profile.

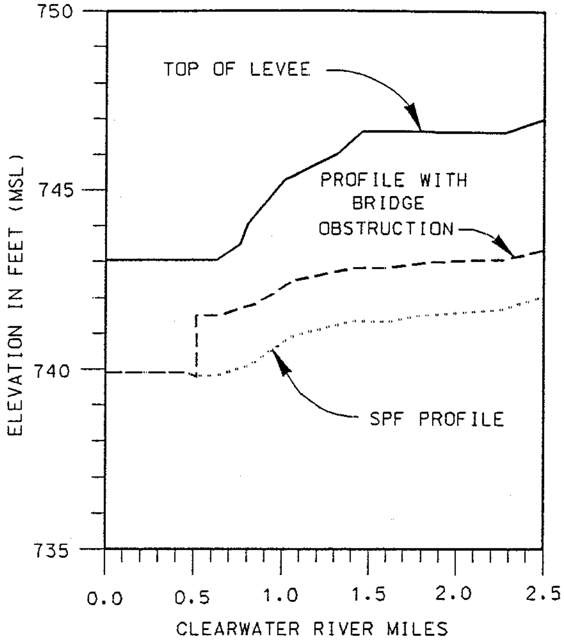


Figure 6. Effect of CPRR bridge blockage on SPF profile.

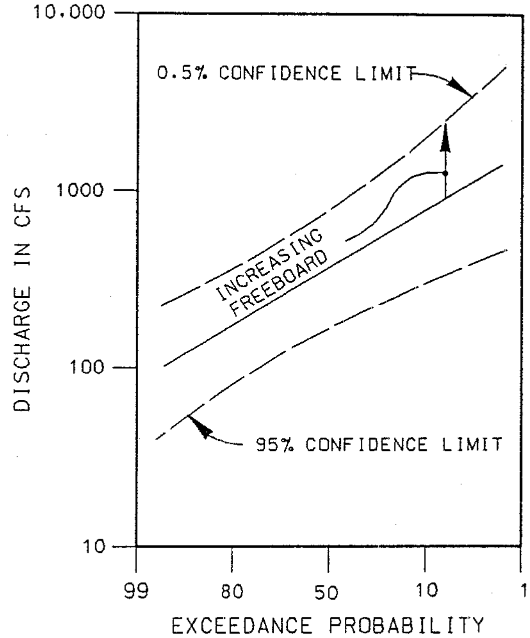


Figure 7. Typical frequency curve illustrating use of confidence limits to adjust freeboard allowance.

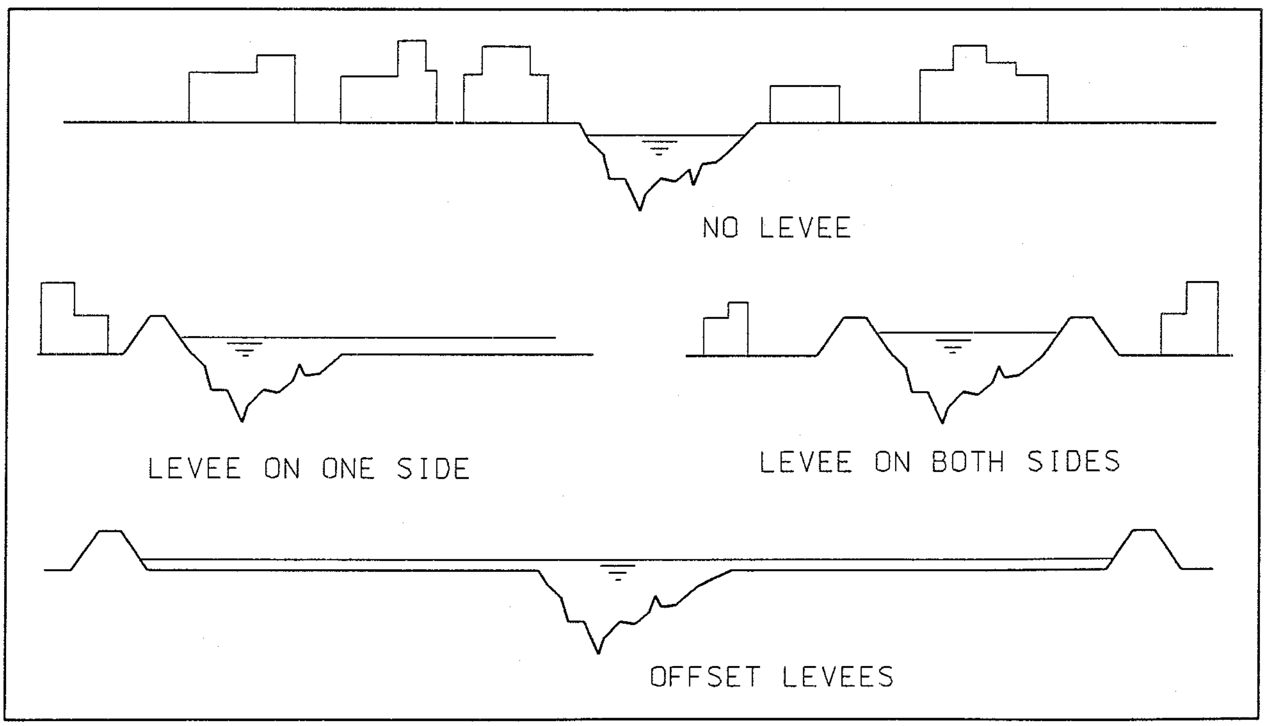


Figure 8. Several typical levee configurations.

The Case for No-Freeboard. In an existing channel with level overbanks, what freeboard should be assumed when calculating the level of protection provided? Some streams are barely three feet deep from top of bank to channel bed. If three feet of freeboard is recommended, what would be the design discharge for that channel? Would three-ft levees on both sides be required to assure that it would carry bank full discharge. From a purely economic point of view, on what basis is any freeboard needed? The problem becomes more apparent when considering a project which will increase the capacity of an existing channel without raising the channel banks. The zero-damage point for inundation damage on the improved channel is the bank-full discharge. The benefits derived from the channel improvement can be determined by maping the area that would have been inundated and comparing the average annual damage that would have occurred with and without the improved channel. The use of a freeboard allowance in the economic analysis would appear to introduce a bias (the improved channel is likely to be overtopped less frequently than assumed in the economic analysis).

Clearly Defining the Elements that Make Up Freeboard.

EM 110-2-1601, paragraph 12, defines freeboard as the vertical distance above the design water surface needed "to insure that the desired degree of protection will not be reduced by unaccounted factors."

How much of the height of a levee should be allocated to freeboard, and how much should be included in the design water-surface profile? The above guidelines suggest that the design water surface should include the effects of all factors which can be estimated with reasonable accuracy, including channel aggradation, anticipated increases in channel roughness, ice cover, and planned or anticipated channel obstructions.

In practice, questions have arisen regarding how much "conservativism" should be included in the calculation of the design water surface and when does this "conservativism" become a hidden increase in the effective freeboard. For example: The roughness will always be somewhat different in the completed project than it was in the natural river. Should the selected roughness be the best estimate, or the maximum possible value for the existing conditions? To be safe, engineers sometimes select a roughness factor which, with reasonable certainty, will never be exceeded. Freeboard is then added above the calculated profile to cover "other" unaccounted or unrecognized factors.

CONCLUSION

At one extreme all uncertainties and risk factors could be worked into the design water surface profile. A more reasonable alternative would be to base the design water surface profile on the best estimate of known and quantifiable hydraulic factors, with all unknowns, uncertainties and considerations of risk assigned to freeboard. A minimum freeboard will probably always be required in cases where assurance is needed that a stated level of protection will actually be provided. An engineer is seldom certain that all factors are known and have been properly evaluated. However, confidence limits or upper bounds can often be established for known uncertainties. A reasonable freeboard can then be established taking into consideration the probability of occurrence and the risk to human life or property that would accompany levee failure.

RECOMMENDATIONS

1) Levee freeboard policies and guidance should be consolidated into a single document which clearly defines the different requirements and factors which should be considered for riverine, lake, and coastal levees.

2) The concept of probability of failure as it relates to consequences of failure need to be looked at more closely. The required degree of assurance (provided by freeboard) that the levee will not be overtopped should relate directly to the consequences of failure.

3) The elements which make up the design height of the levee should be clearly defined. Height allowances which are influenced by considerations of risk, or are included to cover unknown or uncertainty should be credited to the freeboard allowance.

REFERENCES

Jones, Michael 1., and H. Seitz. 1980. Sediment Transport in the Snake and Clearwater Rivers in the Vicinity of Lewiston, Idaho. Open-File Report 80-690. U. S. Geological Survey.

U.S. Army Corps of Engineers. 1954. Improvements in Design and Construction Practices in Civil Works. Civil Works Bulletin 54-14. Office of the Chief of Engineers, Washington, D.C.

U.S. Army Corps of Engineers. 1970. Hydraulic Design of Flood control Channels. EM 1110-2-1601. Office of the Chief of Engineers, Washington, D.C.

U.S. Army Corps of Engineers. 1964. Design Memorandum No. 3: General Design Memorandum, Lower Granite Lock and Dam. Walla Walla District Corps of Engineers, Walla Walla, Washington.

U.S. Army Corps of Engineers. 1991. Lower Granite Sedimentation Study: Preliminary Evaluation and Economic Analysis. Unpublished Report. Walla Walla District Corps of Engineers, Walla Walla, Washington.

U.S. Army Corps of Engineers. 1966. Policies and Procedures Pertaining to Determination of Spillway Capacities and Freeboard Allowances for Dams. EC 1110-2-27. Office of the Chief of Engineers, Washington, D.C.

U.S. Army Corps of Engineers. 1991. Tri-Cities Levees Reconnaissance Report. Unpublished Report. Walla Walla District Corps of Engineers, Walla Walla, Washington.

ST. PAUL DISTRICT EXPERIENCE WITH CREDIT TO EXISTING LEVEES

by

Patrick M. Foley[1]

INTRODUCTION

This paper relates some of the St. Paul District's experience with determining flood damage reduction benefits for existing levees. The general procedures used are given and several case studies presented. The discussion concentrates on the hydraulic aspects of the evaluation and primarily focuses on levees of uncertain reliability installed during flood emergencies. However, there is some discussion of geotechnical considerations and of existing well built and maintained levees. The paper presents an organized method for evaluating the economic credit to be given existing levees that could be used for future evaluations.

The field has received different guidance for evaluating existing levees than it has for design of proposed levees. Guidance for evaluation of credit to existing levees has generally stated that we should give levees of uncertain reliability at least partial credit if they have withstood historic floods. Guidance for proposed levees stresses using conservative design criteria to insure reliability. In order to get any economic credit for a proposed levee it has to meet all of our conservative criteria. We can't build a levee that only meets part of our criteria and then take partial economic credit. These different guidances can cause a permanent levee project to be unfeasible and leave a community relying on an existing levee that could fail at any time. The position presented in this paper is that the same criteria should be applied to all levees. To get economic credit existing levees should meet the same criteria as proposed levees. The Corps of Engineers guidance should either be more conservative when evaluating existing levees or less conservative for design of new levees.

There are considerations other than economic that should be included in the evaluation of existing levees. The hydraulic engineer should consider whether giving economic credit to a levee of unknown reliability also gives implicit certification to the levee and causes the locals to think they can live with a potentially unsafe levee. In this case there may be moral and professional liability if the levee fails. On the other hand, locals may be encouraged to tear down what little protection now exists in order to improve the B/C ratio and have a better chance of eventually getting a permanent levee, leaving them with no protection in the interim. The consistency with federal and local policies for allowing credit to existing levees for floodplain zoning and flood insurance rates should also be considered. It would be difficult for our customers to understand why the Corps of Engineers said a permanent project was not feasible because of the protection provided by an existing levee and at the same time have the Federal Emergency Management Agency (FEMA) and the state not allow any reduction in flood insurance rates or floodplain zoning because the levee doesn't meet their criteria.

GENERAL PROCEDURE

The St. Paul District has analyzed numerous existing levees. There is no

[1]Chief, Hydraulics Section, U. S. Army Corps of Engineers, Saint Paul District

formalized, written procedure for the engineering analysis of them. The following procedure is based on items that have been considered in these evaluations. The basic philosophy for determining economic credit is to apply the same requirements and criteria to existing levees as are used for the design of new permanent levees. All factors are not evaluated for every levee. If the levee is so deficient in one area that no credit can be given, or if it is apparent the credit will be so low it won't jeopardize the B/C ratio, then no other factors need be examined. Based on past flood fight experience we often know what the critical factor for an existing levee and that is the only one that is analyzed. The analysis of credit to existing levees must also consider how the credit will change in the future. Poor maintenance, erosion potential, etc. may cause the engineer to give a short predicted life to the levee.

Steps in the Analysis. The following are the steps used in the engineering analysis of credit to existing levees. They are in rough order and can vary between projects.

<u>Geotechnical Analysis:</u>

1) The most difficult aspect of a geotechnical analysis is the probable absense of <u>any</u> subsurface information. If, based on general geology and prior experience in the regional area, poor foundation conditions are known to exist, a slope stability analysis can be completed. If the computed factors of safety do not meet the Corps of Engineers stability criteria, the levee is given no economic credit. For cases where the subsurface conditions can not be approximated, chances of failure due to seepage or thru-seepage are evaluated. Field trips to the site and records of past stability, seepage or piping problems are of vital importance. If, based on this information, it is evident that the levee or its foundation would probably be unstable during flood conditions, little or no economic credit would be given. (Since this is a very subjective analysis, field trips with Division and OCE reviewers can be very helpful).

2) In locations where no probable stability or seepage problems exist, a stable levee template is fit into the existing levee cross-section. The template represents a levee cross-section that would have stable side slopes and an adequate top width for the existing levee material. This gives the top of levee elevation to use in the hydraulic analysis. See Figure 1. Note that freeboard is subtracted from the template top-of-levee elevation, not necessarily from the top of the existing levee.

<u>Hydraulic Analysis:</u>

3) Determine if erosion could cause the levee to fail (with Geotech). If erosion is extremely bad (vertical side slopes, active erosion, etc.) then no credit is given. If there is the potential for erosion (computed high flood velocities, sharp bends, etc.) then the life of the levee may be shortened based on engineering judgment.

4) Determine the design flood profile and the flow that can be safely passed. Freeboard guidance for proposed levees is used including a requirement for overtopping at the least hazardous location. The

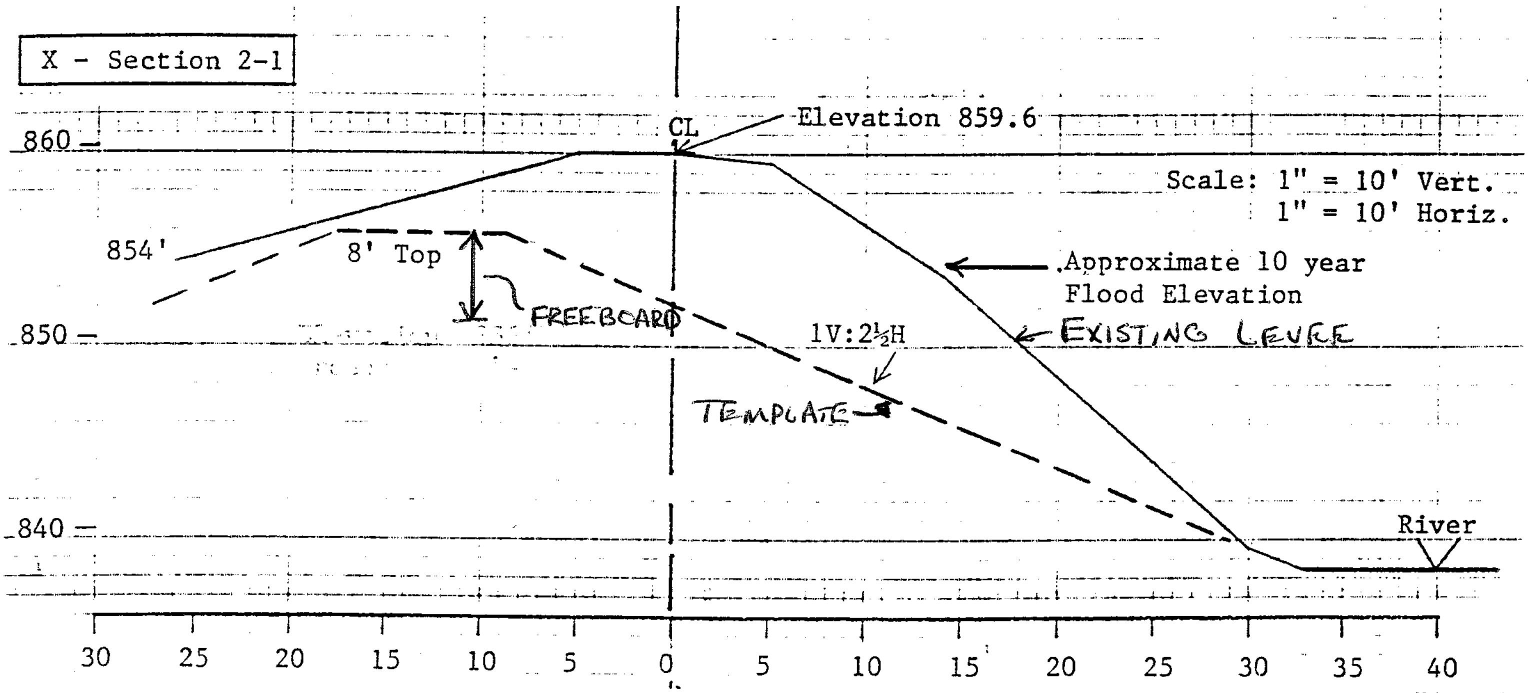


FIGURE 1
LEVEE TEMPLATE

freeboard is measured from the template elevation, not the actual top of levee if it is above the template. The same assumptions are used for computing the water surface profiles as are used for proposed conditions. These assumptions include bridge plugging, n values, future encroachments, etc. The freeboard for the existing levee could be given some economic credit if it is felt that the levee has a chance of performing for floods in that range. Water surface profiles are computed assuming the existing levees are the effective flow limits until they are significantly overtopped. The actual top of the existing levee rather than the template is used to determine the amount of overtopping and when the levees stop being effective flow limits. Note that the levees are considered effective flow limits even if they are given no economic credit. Usually only small portions of a levee fail and there is not effective flow through the protected area until the levee has been significantly overtopped. If there is significant storage within the protected area, an unsteady flow analysis of the impact of the levee failure on the stream hydrographs would be needed for floods that cause failure of the levee but don't overtop them.

5) For floods in excess of that which the levee can safely pass but less than that which would significantly overtop the whole system; determine when and where the levee will fail and determine how and to what elevations the interior would flood. A failure at the upper end could fill the interior to the elevation of the downstream levee height possibly resulting in higher elevations inside the levee than outside at some locations. Determine if failure of the levee would be catastrophic and if so determine the potential damages and include in the econ analysis.

6) Determine residual flooding from interior runoff, seepage, and flow through closures that aren't closed. If a closure doesn't meet guidance for proposed closures assume it won't be closed.

7) Examine maintenance practices and determine when the levee will be so deteriorated the credit will have to be reduced. To have the same life as new levees they must have as good O&M.

CASE STUDIES

Crookston - Geotechnical Evaluation. Crookston is on the Red Lake River in Northwestern Minnesota. A map is shown on Figure 2. A Reconnaissance Report was recently reviewed by North Central Division (NCD) and OCE. The proposed project calls for channel cutoffs downstream of the city and possibly a short levee for one portion of the city. Levees were first constructed by locals following a flood in 1950. In 1965 it was necessary to place 125,000 sand bags to avert a major disaster. In spite of these measures, the 1965 flood caused severe damage to portions of the city. In 1965 and 1966, a major levee improvement project was carried out locally in an effort to provide some degree of protection against the reoccurrence of the 1950 and 1965 flood stages. During the floods of 1969 and 1978, portions of the levee system were upgraded through the use of PL 84-99 funds provided by the Corps of Engineers. In 1969, extensive sandbagging and many other emergency measures were needed by numerous local, State, and Federal organizations to limit flood damages. Following the 1969 flood, another locally financed project was carried out

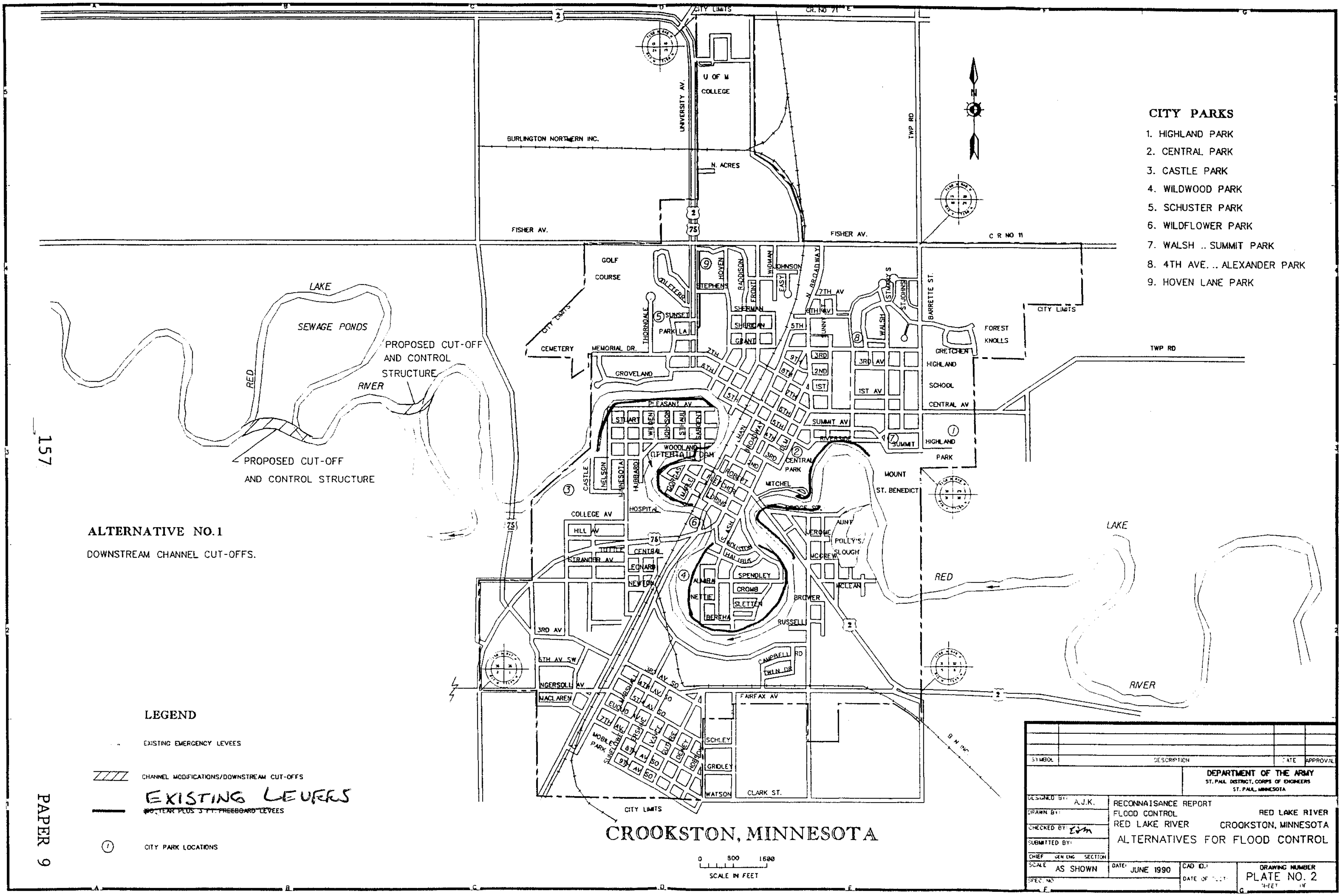


FIGURE 2
CROOKSTON, MN

extending, enlarging, and raising the height of the levee system.

In the Reconnaissance Report the St. Paul District proposed giving no credit to the existing levees in most areas. This was because of the uncertain nature of the conditions in which the levees were built, making their reliability questionable and because of the condition of the levees and the riverbank. The levees are typically built at the top of a steep riverbank and squeezed behind the backs of garages. Several garages are cut into the levee prism. The steep river banks are actively eroding in some areas. While the levee itself is maintained, the riverbanks are heavily wooded with mature trees. Several of these trees are within the riverward toe of the levee and could cause a major weak point should they become uprooted during a flood. Because the geotechnical analysis found that basically no credit could be given the levee, a hydraulic analysis was not performed.

Reviewer's comments indicated some credit should be given to the existing levees since they had withstood some historic floods. After several months of back and forth discussion, OCE, Division, and District geotechnical engineers and division and district hydraulic engineers made a field trip in May 1991, to examine the levees. After viewing the levees and talking to a local representative about past failures of the levee, everyone in attendance agreed no credit should be given the levees. This was primarily due to the fact that the levees had failed during past floods and that they were built on the top of a bank that could fail at anytime.

This same team also inspected existing levees on the Red River of the North at Grand Forks, North Dakota. The District's reconnaissance report for Grand Forks presented the benefit-cost ratio for both no credit to the existing emergency levees and also for full credit to them as if they were a permanent project. For both cases the B/C ratio was above 1.0 and the report said no engineering evaluation of the emergency levees was warranted. This saved significant study funds as well as disputes with higher Corps of Engineer authorities.

<u>Houston - Analysis of how the interior floods.</u> Houston is located in Southern Minnesota on the Root River. A levee project is proposed to protect the city. The General Design Memorandum (GDM) was sent to NCD this year, 1991. The economic analysis was performed for the 1989 General Reevaluation Report. The upstream portion of the existing levee was built by the Minnesota State Department of Transportation in the early 1960's in connection with a bridge replacement. This portion varies in height from 5 to 15 feet, with an average of 11 to 12 feet. The downstream portion was built by the Corps of Engineers during the 1969 flood emergency and has an average height of 5 to 6 feet. A map of the city is shown on Figure 3.

The geotechnical analysis determined a template with top widths and side slopes appropriate for the levee material present to define the maximum water levels that the existing levees could be subjected to without significant fear of failure due to through-seepage, under-seepage, or slope stability. Template top widths = 10 feet for both reaches, side slopes = 1V:3H riverside, and 1V:5H land side for the upstream portion and 1V:2.5H for both the riverside and land side for the downstream portion. The lowest top-of-template elevation for the downstream portion was determined to be 679.1 and for the upstream portion 684.5. Seepage and uplift computations indicated a factor of safety for uplift at the toe of the existing upstream

HOUSTON, MINNESOTA REEVALUATION STUDY
EXISTING CONDITIONS

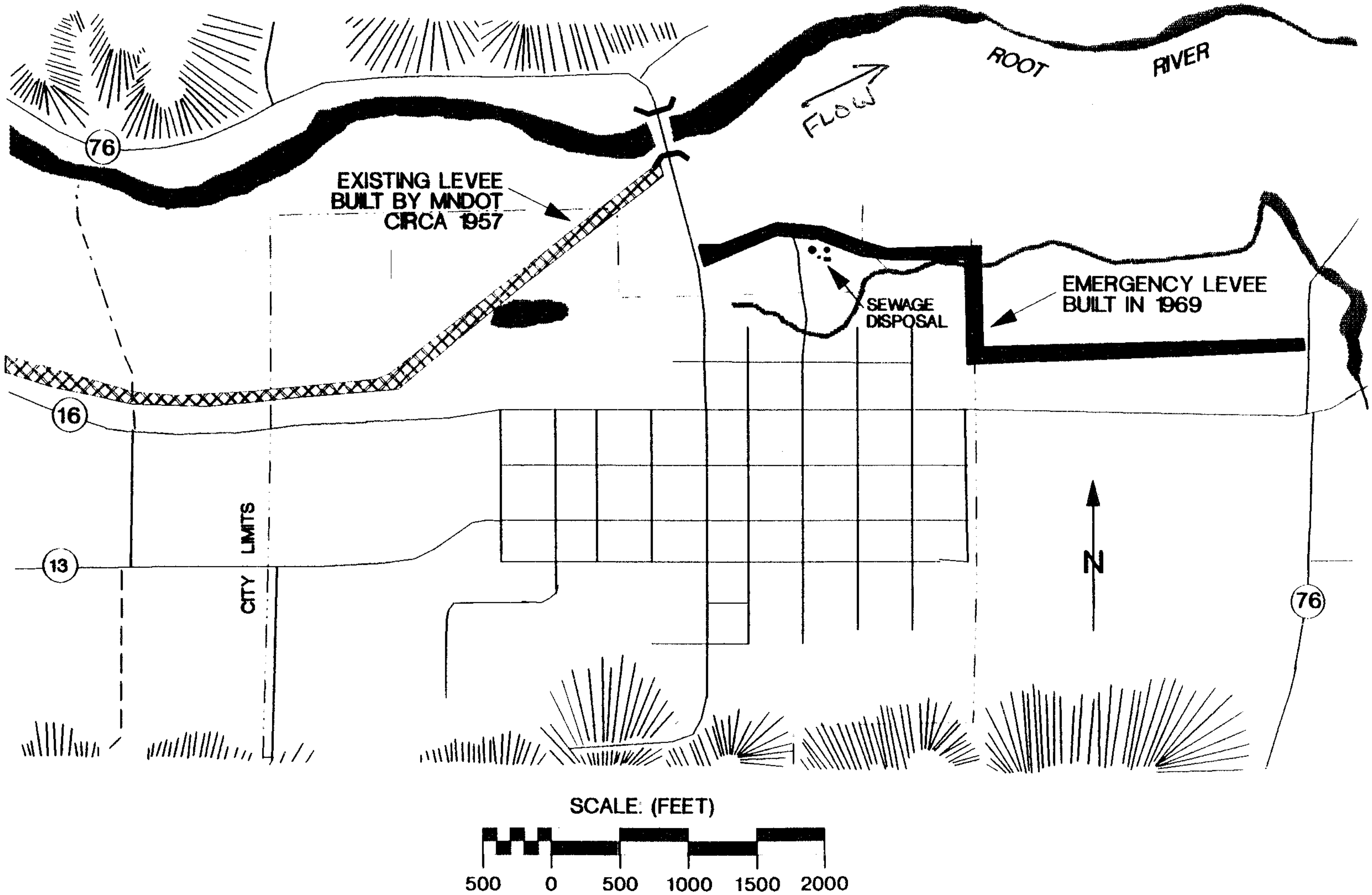


FIGURE 3
HOUSTON, MN

levee to be 1.2 with the river at 684.5.

The economic analysis was done assuming the existing levees did not exist and also assuming they were good up to the template elevations. The hydraulic analysis assumed 3.0 feet of freeboard at the downstream end and 3.5 feet at the upstream (design flood elevations of 676.1 and 681.0) to allow initial overtopping at the downstream end. It was computed that a 32-percent had an upstream elevation of 681.0, 3.5 feet below the template elevation of 684.5. Thus, the economic analysis assumes no existing conditions flooding for floods smaller than the 32-percent flood. This was a conservative assumption since it assumes all culverts through the levee will be closed when in fact several of them are ungated and require manual plugging, the city has no plan for plugging them and the warning time is quite short. Interior flooding for floods in excess of this flood is complicated. There is a road embankment through the city normal to the river that does not overtop until a 4-percent flood event. So between the 32- and 4-percent flood event, the water inside the levee and upstream of this road was assumed to pond to the elevation of the river at the breach location. For these same floods the interior water surface downstream of the highway was computed by analyzing flow through a 24-inch culvert under the highway embankment. The flow through this pipe had to be routed to the downstream levee and ponded until the levee was overtopped, either from the inside or the outside. This resulted in interior elevations higher than the river at some locations. Once a levee was assumed to fail an approximate breach analysis was used to compute the flow through the levee breaches. Because of the head loss through the breach, the interior elevations were lower than the river at some locations. Above the .07% flood it was assumed the levees would be significantly overtopped and the interior elevation were equal to the river.

The computed benefit cost ratios were 1.45 for no credit to the existing levees and 1.01 with the limited credit given to them. The hydraulic analysis of the existing levees for this project was more detailed than normal because the B/C ratio was so marginal. The initial analysis assumed the interior elevations were equal to the river elevations for all floods in excess of the 32% flood. However, due to the economic sensitivity the detailed analysis of interior stages was performed. Analysis such as this can be quite time and money consuming and do little or nothing to improve the design of the proposed project.

<u>Chaska - Flooding from interior runoff.</u> The flood control project at Chaska, Minnesota, is intended to protect the city from floods on the Minnesota River, Chaska Creek, and East Creek. A map of the city is shown on Figure 4. All large historic flooding has been caused by the Minnesota River. After a flood in 1953 the city built a levee along the Minnesota River. In 1965, the largest flood recorded at Chaska overtopped the city's levee by 5 feet. Following the flood the Corps of Engineers restored the levee under PL 84-99 and the city raised the levee an additional 4 feet.

Another large flood was predicted for the spring of 1969. The Corps of Engineers raised the levee about 2 feet under Operation Foresight. Pumps, sandbags, and technical assistance were also provided. The levee held with no major problems; however, seepage was evident and a substantial amount of pumping capacity was required. Another problem was the large number of sandbags required as each creek had to be sandbagged along both banks to tie the levee to high ground.

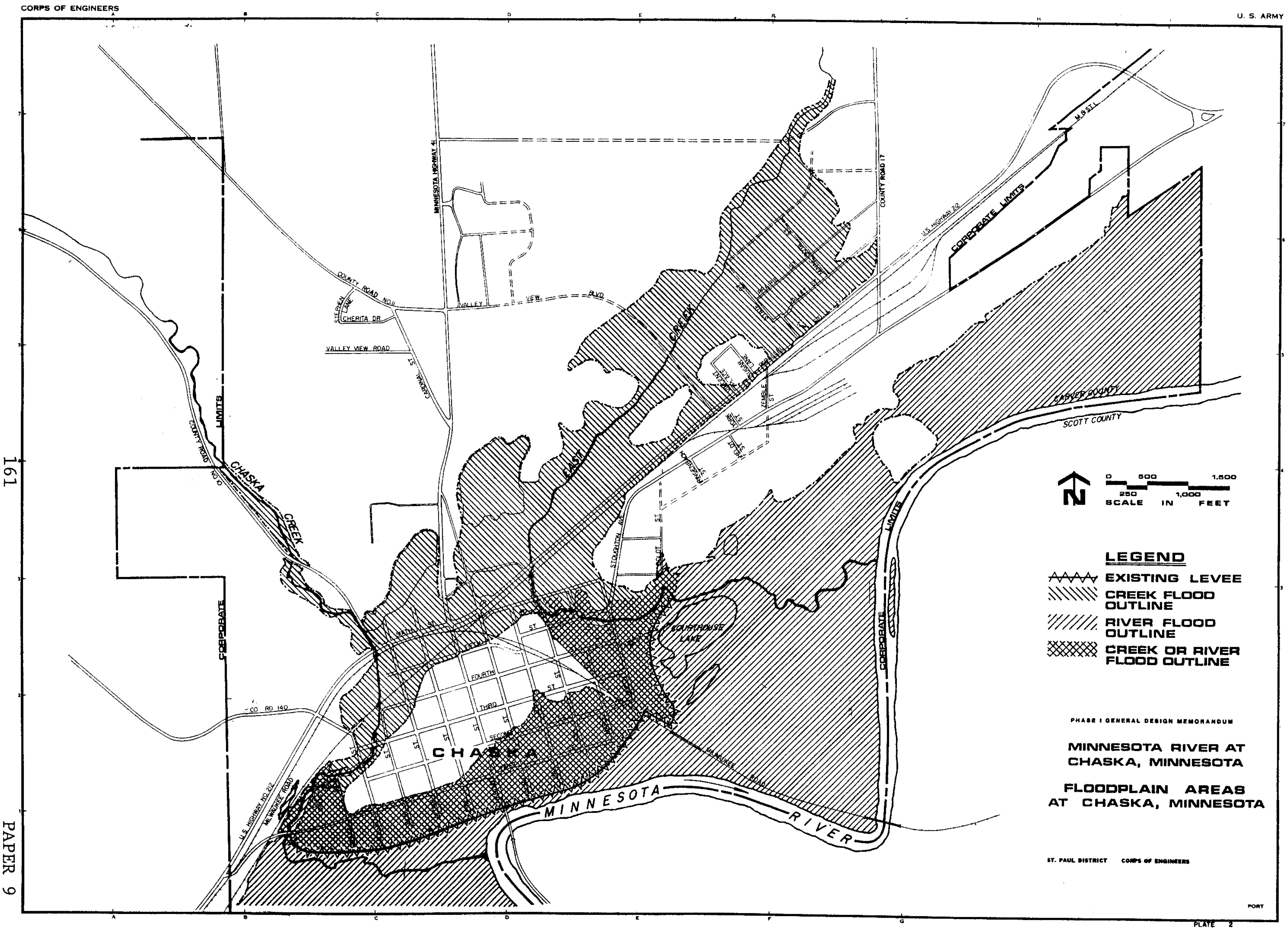


FIGURE 4
CHASKA, MN

The existing levees at Chaska have always been raised under emergency conditions, either during or following a flood. For this reason, there has been a tendency to raise the levees by adding more fill to the top than to the side slopes. As a result the levees are overly steep in the side slopes and do not meet the Corps of Engineers stability criteria. Levee deficiencies include slope stability, seepage, erosion, and maintenance problems attributable to steep slopes. Boring information indicates the material used in the construction of the emergency levees is basically suitable material, if place under controlled conditions. However, during emergency construction the material was placed in a random manner, with emphasis on placing as much material as possible in a short period of time and assuming that potential weaknesses in the structure could be handled as they developed.

The hydraulic analysis of credit to the existing levee was considered in the 1982 Limited Reevaluation Report (Phase I level of detail). The factor that played the largest role in the credit given to the levees was interior flooding caused by Chaska and East Creeks. Chaska Creek goes around the city to the west and enters the river adjacent to the upstream end of the levee. East Creek goes around the city to the east and enters the river adjacent to the downstream end of the levee. Floods on both creeks could overtop their banks and enter the protected area. The amount of flood flow leaving the creek and entering the protected area was computed for various frequency floods. The interior pond elevations were then computed based on the storage volume available. The computed interior pond elevations approached the elevations of equivalent frequency floods on the river. Figure 5 shows the stage-frequency curve for the ponding caused by Chaska Creek and some Minnesota River elevations.

The fact that interior stages caused by the creek flooding were about equal to the river showed that the existing levees basically provide no protection to the city and have little or no flood damage reduction benefits. In this case, it was not necessary to determine what protection the existing levee would provide from floods on the Minnesota River.

The proposed project is presently under construction. A supercritical channel has been built for Chaska Creek capable of containing the standard project flood. A diversion of East Creek is proposed and the levee along the Minnesota River is to be improved to meet the Corps of Engineers standards.

<u>St. Paul - permanent project, test-of-time, rating curve not stable, use overtopping criteria not old design criteria.</u> The City of St. Paul is on the Mississippi River. A map is shown on Figure 6. There is an existing permanent levee that was constructed by the Corps of Engineers in 1964. The project was designed to provide protection from a flow of 168,000 cfs, presently about a 0.6% or 166-year flood. The project was design to have 2.8 feet of freeboard over it's whole length. The proposed project calls for an approximate 4-foot raise of the barrier, bringing the levee of protection up to a 0.17 percent or 588-year flood, 210,000 cfs.

The hydraulic study of the proposed levee included a freeboard analysis. Factors used to determine the minimum freeboard included the length of the historical record, slope of the rating curve, the stability of the rating curve, and the calibration accuracy. The United States Geological Survey (U.S.G.S.) gage is located in the project limits and has 119 years of record. Large floods of 171,000 and 156,000 cfs occurred in 1965 and 1969,

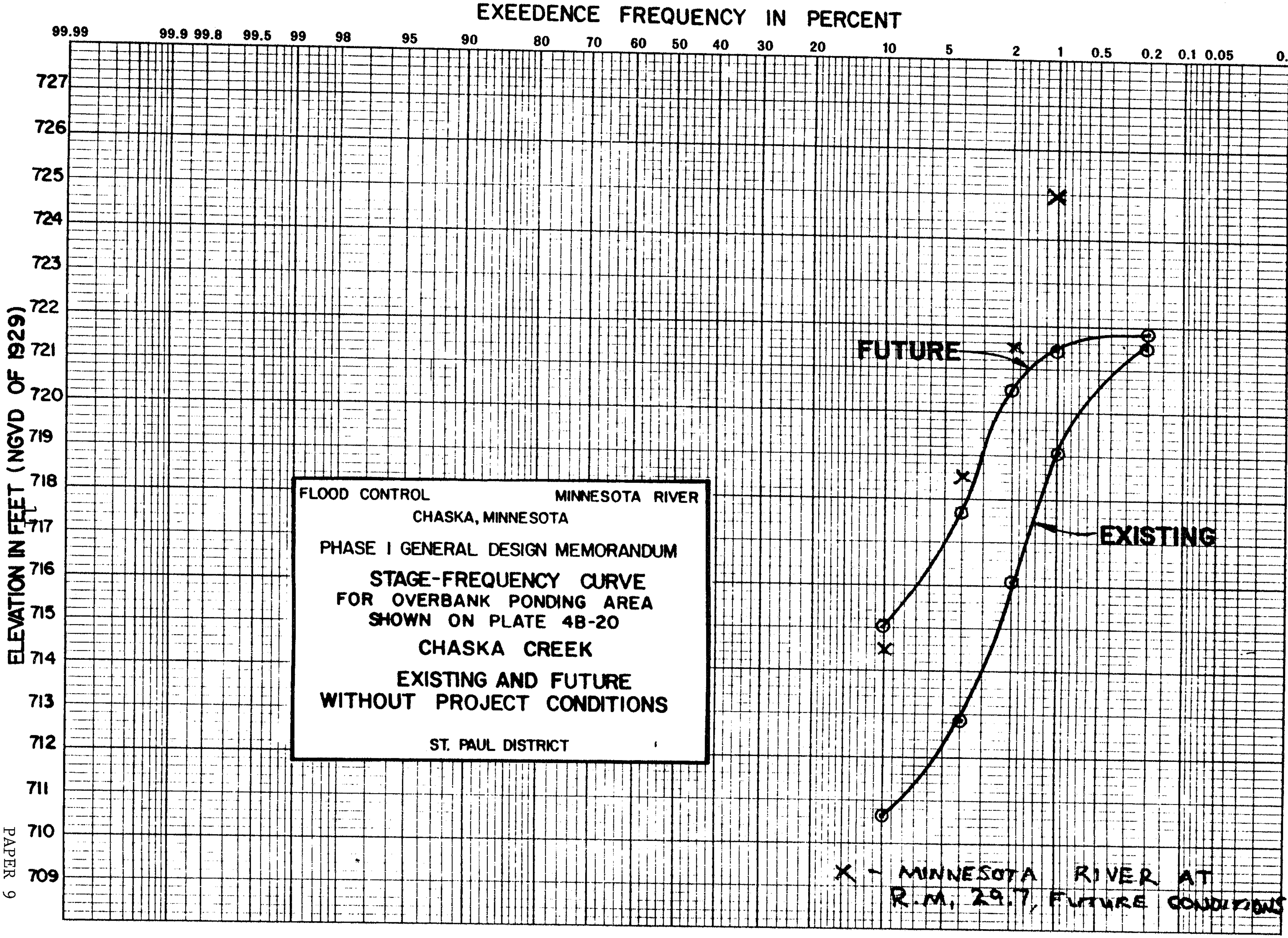


FIGURE 5

PAPER 9

PLATE 4B-31

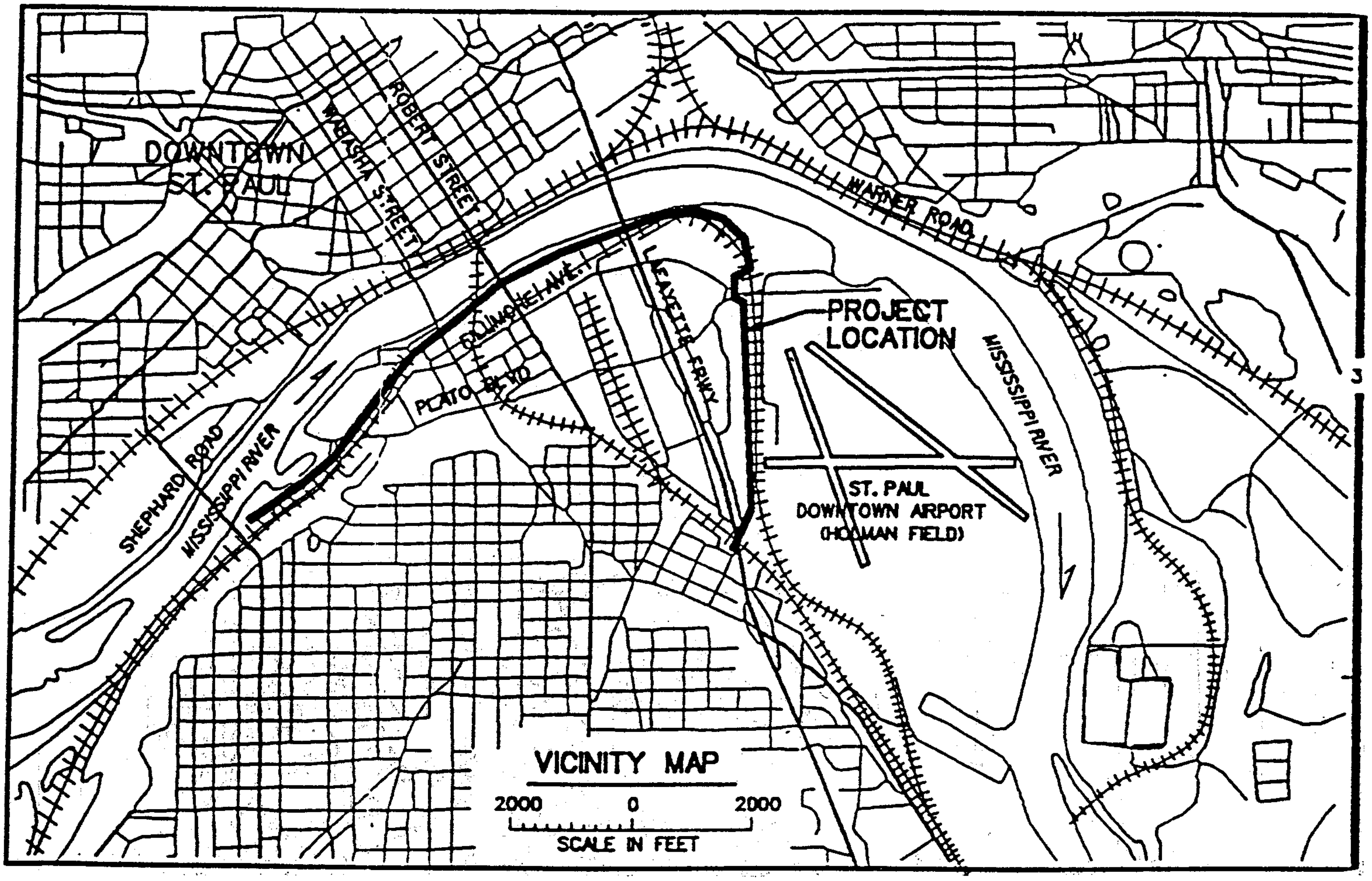


FIGURE 6
ST. PAUL, MN

respectively, allowing a high confidence in the model calibration. These two factors indicate a fairly low freeboard of 2 feet or less might be used, however, the rating curve is quite steep, 12,000 cfs/foot, and the rating curve for the U.S.G.S. gage has varied over time. It was decided to use 2.5 feet as the minimum freeboard at the downstream end of the project. The top-of-levee profile for the proposed levee was based on the discharge that had a profile 2.5 feet above the design flood at the downstream end of the project. This was a discharge of 250, 600 cfs, slightly greater than the standard project flood. Superiority was added to this profile to get the top of the levee elevations. Superiority was added in increments. Scour computations showed the existing bridges could fail possibly causing some blockage of the channel and because of this more superiority was used upstream of them than normal. The total superiority over the 250,600 cfs profile at the upstream end of the project was 1.5 feet. the 250,600 cfs profile was 3.1 feet over the 210,000 cfs design profile at the upstream end. Thus, the top of the levee at the upstream end was 1.5 + 3.1 = 4.6 feet over the design profile.

The evaluation of the economic credit for the existing levee used the same freeboard and superiority assumptions as those used for the proposed levee: 2.5 feet of freeboard at the downstream end and the same amounts of superiority. Required superiority was subtracted from the existing levee elevations and the largest flood that could be contained at all points found. The 2.5 feet of freeboard was subtracted from this profile at the downstream end of the project and the corresponding discharge was the flood that the existing levee could safely contain. It was found to be less than the 168,000 cfs that the levee was originally designed for. Flood and top of levee profiles are shown on Figure 7.

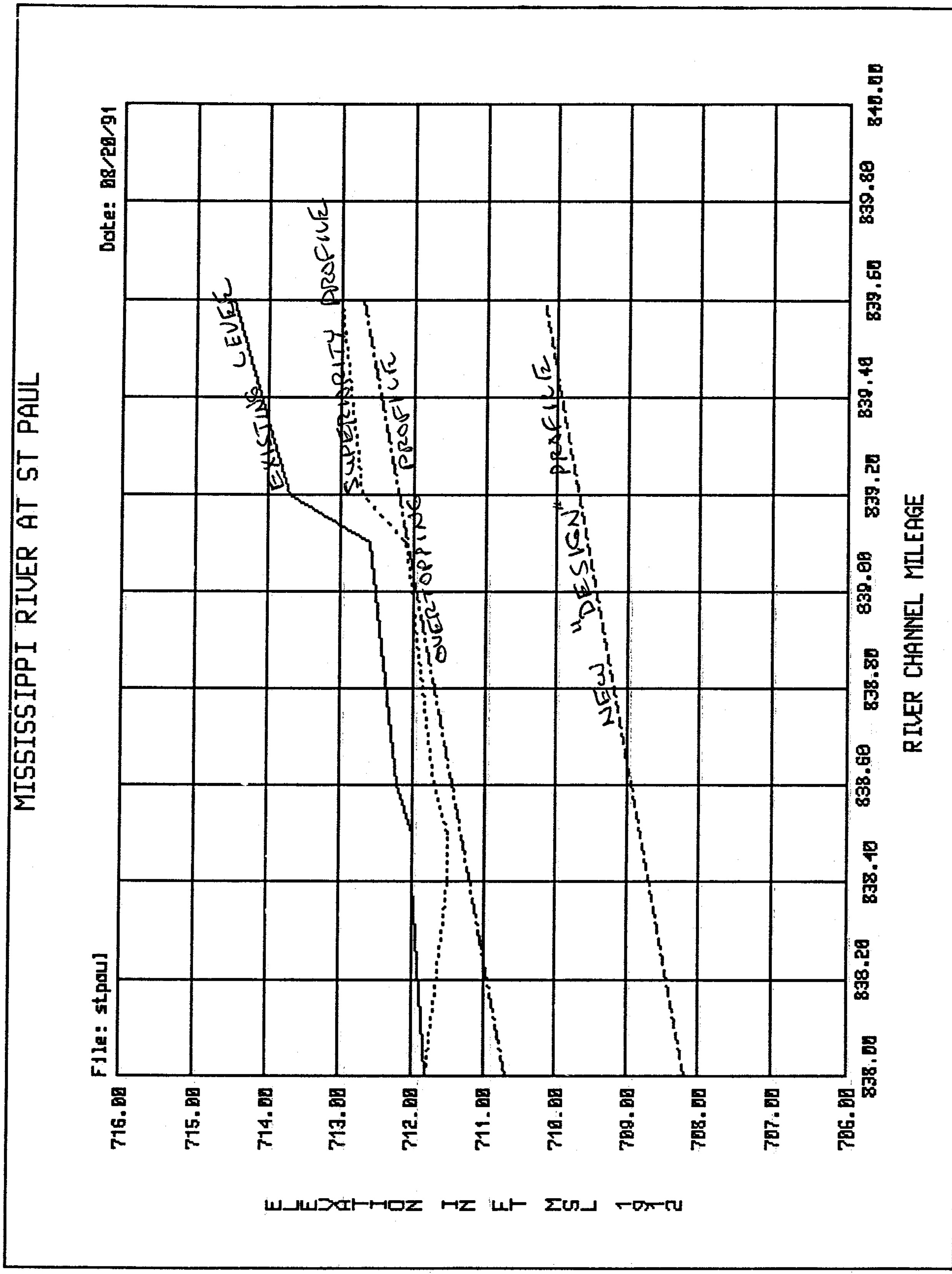
No economic credit was given to the freeboard of the existing levee. This was because in its 22 September 1982 report recommending the proposed project, the Board of Engineers for Rivers and Harbors stated "...the Board believes that benefits should not be attributed to freeboard for the existing project. However, design of the proposed project should eliminate past operational problems such that benefits accruing to freeboard can be attributed to the recommended plan."

OTHER IMPACTS THAT SHOULD BE CONSIDERED

The results of the analysis of credit to existing levees can have impacts other than on the economic feasibility of a new project. If we give an existing levee of unknown reliability credit for protecting a community to some flood level, there will be a tendency for those people to feel the Corps of Engineers has given that levee at least implicit certification up to that level. We have a professional responsibility to consider the possible false sense of security and damages we may induce by doing that. It is also possible some communities will realize the rules of the game and allow destruction of an existing levee in order to increase the computed damages and get a new project. If a permanent project has a B/C of 0.5 when credit is given to an existing emergency levee and 1.5 if the levee was considered to not be there; we could be encouraging the locals to tear down the emergency levee and lose what little protection they now have.

The United States Government through FEMA has said that no credit will be given to a levee for flood insurance purposes unless it meets a set of minimum

FIGURE 7



MISSISSIPPI RIVER AT ST PAUL
Date: 08/20/91
File: stpaul
716.00
715.00
714.00
713.00
712.00
711.00
710.00
709.00
708.00
707.00
706.00
838.00
838.20
838.40
838.60
838.80
839.00
839.20
839.40
839.60
839.80
840.00
RIVER CHANNEL MILEAGE
EXISTING LEVEE
SUPERIORITY PROFILE
OVERTOPPING PROFILE
NEW "DESIGN" PROFILE

criteria. These requirements are given in Section 6510 (b) (2) of the National Flood Insurance Program regulations (Title 44, Part 65, of the Code of Federal Regulations). The requirements are quite strict on several points. The levee must: provide protection from the 100-year flood with at least 3 feet of freeboard, have well designed closures, adequate erosion protection for the 100-year flood, meet good geotechnical requirements, consider interior drainage flooding, and have good operation and maintenance plans and implementations. If the Corps of Engineers evaluates an existing levee that does not come anywhere close to the FEMA requirements and says that it does provide flood damage reduction benefits, we will be sending very mixed signals to our customers. The FEMA policy is designed to discourage communities from relying on poorly built and maintained flood control projects, the Corps of Engineers should have this same goal and it should show up in our policies.

LEVEE FREEBOARD DESIGN FOR WEST COLUMBUS, OHIO LPP

by

Ken Halstead, P.E.[1]

1. INTRODUCTION

During the Reevaluation study of the West Columbus, Ohio LPP, the construction cost estimate was found to have increased significantly over the estimate provided in the Feasibility Report. An intense study was conducted to determine all minimum project requirements that could be implemented in order reduce the project cost. As a part of this effort, a detailed study was conducted to determine the minimum freeboard design that would be required to provide the proposed level of protection.

2. SETTING AND PERTINENT DATA

Columbus, the capital city of Ohio, is located in the central portion of the state. The Scioto River flows into the city from the northwest and exists the corporate boundary to the south. A major tributary, the Olentangy River, flows into the city from the north and joins the Scioto River in the middle reach of the proposed project area. Delaware Dam, a Corps of Engineers flood control project is located upstream on the Olentangy River.

The drainage area of the Scioto River at the USGS stream gaging station located downstream of the project area is 1629 square miles. Upstream of the confluence with the Olentangy River the drainage area is 1068 square miles, while Delaware Dam controls 381 of the 536 square miles drained by the Olentangy. The upstream watershed is primarily agricultural, with some wood land and other communities much smaller than Columbus.

The west side of the city is situated on low ground on the inside of a long meandering bend of the Scioto River, and adjacent to the highly developed downtown area on the east side of the river. The majority of the area that is subject to flooding is residential, with some commercial and light industrial areas, and public parks. The west side is offered some protection from low frequency floods by elevated road and railroad embankments, some existing high ground and an old levee of unknown design along Dry Run Creek, a small tributary of the Scioto River, at the upstream end of the proposed project area. However, the integrity of these features are highly questionable for floods of a twenty year frequency or greater.

The flood of record through the Columbus reach of the Scioto River is the 1913 flood. This flood, and the SPF with reductions by Delaware Dam, are approximately equivalent to a 460-year frequency flood event. The SPF, determined by economic optimization to be the design flood for the levee and floodwall portion of the project, would cause nearly $370,000,000 of damage if it were to occur under current conditions.

[1]Chief, Hydrology Section, Hunington District, U.S. Army Corps of Engineers

The last major flood occurred in 1959. This event was highlighted by the breach of a portion of the old Dry Run levee prior to the crest of the flood on the Scioto River. The 1959 flood was equivalent to a 70-year frequency event by current computations. Stream gaging information has been collected for the period of record beginning in 1921.

3. **DESCRIPTIONS OF STUDY AND RESULTS**

a. Pre-Freeboard Analysis. Prior to the beginning of the development of a minimum freeboard value, it was considered essential to establish a sound basis for the project design flood (PDF) profile. A calibration to a historical flood was attempted, a sensitivity of HEC-2 input variables was conducted, and judgement was used to determine reasonable values for Manning's "n", bridge pier loss coefficients and weir coefficients. Then, through close coordination and field investigations with CECW-EH and CEORD-PE-W, a mutual agreement was reached regarding the factors that were used in the backwater computations, and factors that should be considered in the determination of minimum freeboard.

b. General Freeboard Factors. A general list of items, that could conceivably affect minimum freeboard, was developed as follows:

- Extension of rating curve beyond known historical events
- Debris blockage at bridges
- Uncertainty in "n" values (due to insufficient or poor high water mark data)
- Superelevation at bends
- Sensitivity of weir loss coefficients
- Sedimentation
- Settlement
- Wind waves

These items were then examined in a manner similar to a qualitative analysis. The extension of the rating curve at the gage beyond historical events was ruled out, because previous studies downstream of the gage provided reasonable backwater effects for the starting water surface elevation of the PDF. Superelevation at bends was not considered to be a problem due the majority of the project being on the inside of the significant bends. The sensitivity of weir loss coefficients was eliminated from freeboard consideration because all weirs in the project reach are submerged by tailwater effects at the PDF level. Geotechnical personnel were of the opinion that settlement would not be a factor in view of core boring information. Wind waves were also dropped, because a significant fetch length does not appear to be available through the project length.

c. Project Specific Freeboard Factors.

1) General. Minimum freeboard was then developed by considering the reasonable upper limit of the remaining factors which cannot be estimated with certainty or accurately defined for the occurrence of the PDF. Through close coordination and consensus with CECW-EH and CEORD-PE-W, the factors that were used in the determination of minimum freeboard were established as follows:

- roughness
- debris
- sediment

2) Flow Resistance Coefficients. Manning's "n" value is the coefficient that is used to describe boundary friction in the HEC-2 model. In general, for any given hydraulic study, the accurate estimation of this coefficient is inhibited by the limited amount of documented field data that is available for use as guidance. The process used to select the Manning's "n" values for the base condition was by judgment and experience with reference to previous studies of downstream reaches of the Scioto River and streams in adjacent drainage basins with similar topography.

In order to determine a reasonable upper limit of Manning's "n", reference was made to "Accuracy of Computed Water Surface Profiles", a study prepared by the Hydrologic Engineering Center for the Federal Highway Administration. A computational procedure presented in this publication indicates that the estimate of an "n" value of 0.035, the value used through the project reach, could vary by as much as ± 25 percent. Therefore, a value of 0.044 through the project reach was adopted as the upper limit of variation for "n" values of all levels of protection that were considered in this study.

3) Debris Blockage. The city and the highway department were contacted in an attempt to characterize debris blockage by actual experience. It was reported that debris accumulation has not been a problem in recent years. However, consideration was given to the fact that large debris producing floods have not been experienced in recent history, particularly on the order of those being studied as possible levels of protection. Therefore, an additional 10 percent of the flow area of the Broad Street bridge was blocked as the upper limit of anticipated debris accumulation. This is in addition to the five percent reduction that was used in the development of the existing condition profiles, for a total blockage of 15 percent of the flow area.

4) Sedimentation. Historical field notes of USGS velocity measurements were obtained and reviewed in an attempt to determine if a trend has been developing toward aggregation or degradation for this reach of the river. This investigation revealed that measurements have not been taken at the same location over the years. A small group of the measurements has been taken within a 60 foot reach of the gage by wading during very low flow conditions. Elevations for the end stations were not provided. Therefore, the stage, as recorded at the gage during the measurement, was used as the vertical reference for plotting purposes. All sections in this group plotted within three feet of each other, which could be considered as a trend toward

stability. However, given that a continuous record at a common location does not exist, it cannot be considered as all conclusive.

Relative channel stability is indicated by other sediment related factors which were considered. These include: 1) the fact that the plan form of this reach of the Scioto River, has been retained in recent geologic history by the system of railroad and highway embankments and existing high ground along the streambanks, provides some indication of stability, 2) the heavy clay-type material that composes the banks, and 3) the maturity of the vegetation cover that exists on both banks along undeveloped reaches.

In some reaches of the project, sediment deposition is not expected to be a factor, particularly in the pools of two of the three weirs that exist in the study reach. The city routinely dredges the Main Street pool to provide recreational boating capability, and will maintain the major water supply intake at the upstream weir.

Therefore, based on the available information, sedimentation was used as justification to "round-up" values produced by the other factors and to account for the consideration of minimal deposition that could occur during the design flood in other reaches of the stream that are not as closely maintained by the city.

d. Computation of Minimum Freeboard. Water surface profiles were computed with the upper limits of Manning's "n" values and debris blockage for various levels of protection that were considered during the Reevaluation study. Increases in water surface profiles at the Veterans Memorial Center, the proposed overtopping location, are shown in the following table.

Minimum Freeboard Determination
at Veterans Memorial Center

Factor	100-yr	200-yr	SPF
Upper limit "n"	1.5	1.5	1.5
Debris Blockage (15% at Broad St. Bridge)	0.2	0.2	0.3
Sedimentation	0.3	0.3	0.2
Total minimum freeboard	2.0	2.0	2.0

With consideration added for sedimentation, this tabulation indicates that a minimum freeboard value of two feet is appropriate for the design of any level of protection from the 100-year through the SPF with the initial overtopping section located behind the Veterans Memorial Center. These values are applicable only for the proposed project with the initial overtopping section at the Veterans Memorial Center. The development of freeboard for the remaining reaches of the project is described in the following sections.

e. Selection of Initial Overtopping Location. During the study for the Feasibility Report and the early phase of the Reevaluation, Dodge Park was considered as a potential site for the initial overtopping section, because it is the most downstream location of the proposed project alignment that is immediately adjacent to the river. The required top of protection profile for the perimeter of the project was developed, by applying the minimum freeboard and using the procedures outlined in the following section of this report. The resulting freeboard for selected locations in the upstream reach of the project is shown on the attached exhibit. In view of the uniform freeboard of three feet was applied in the Feasibility study, the increased freeboard values were considered excessive.

In reconsidering the selection of Dodge Park as the initial overtopping location, it was noted that the existing levee elevation at this location is relatively high and would require excavation to obtain the desired overtopping elevation. Also, the removal of an existing overlook facility would be required. Residential properties are located across the street from the park and adjacent to the existing levee near the overlook facility. The city likely would not be receptive to the removal of the overlook and local residents likely would view a lowering of the levee as reducing their level of protection. Also, the results of a preliminary overtopping analysis indicated that due to the differences between Scioto River flood elevations at Dodge Park and the ground elevations in the upstream reach of the project, the inundation of the toe of the upstream reaches of the proposed levee cannot be achieved. These factors were incentives to consider another location for the initial overtopping section.

The existing wall behind the Veterans Memorial Center was then considered as a potential site for the initial overtopping section. This location already is the lowest reach along the perimeter of the proposed project alignment with respect to Scioto River flood heights and appears to be the least hazardous area for overtopping. The existing wall at this location appears to have been intentionally designed and built at a lower elevation to act as a spillway during higher flood events to prevent the downstream levees from being overtopped and damaged. However, no report on the original design has been found. Regardless of whether it was "intentionally designed" or not, the construction was at an elevation lower than adjacent reaches upstream and downstream of this location. The wall that is proposed for this location that will be of sufficient height to provide the minimum required freeboard. The paved parking lot should provide ample protection against erosion of the foundation of the wall in the event of an overtopping flood. It is noted that with the configuration of raised roads and railroad embankments that subdivide the interior area, the flow from overtopping at this location would be dampened to reasonably low velocities through the underpasses. It is also noted that no residential areas are located in the immediate vicinity of the initial overtopping location. This change also reduced the amount of freeboard that is required for the project in the reaches upstream of the initial overtopping section, as shown on the attached exhibit.

f. Standard Project Flood Top of Wall. The floodwall and levee freeboard was designed in accordance with criteria and procedures outlined in ETL 1110-2-299, entitled "Overtopping of Flood Control Levees and Floodwalls".

The top of protection was set at the SPF water surface profile plus freeboard, such that in the event of a flood greater than the SPF, overtopping would occur initially at the Veterans Memorial Center wall located just upstream of the Broad Street bridge. The SPF water surface profile with project conditions (project design profile) was used as the basis for determining the top of wall profile. As an aid in designing the project freeboard to assure initial overtopping at the least hazardous location and lessen impacts to the protected area, water surface profiles for flows in excess of the design SPF discharge were computed. Flood events corresponding to the SPF discharge increased 16.7% and 25.4% were analyzed for this purpose. The water surface profile for the SPF discharge increased 16.7% coincides with the minimum freeboard profile at the Veterans Memorial Center wall. This portion of the floodwall will be at the elevation of this water surface profile (2 feet of freeboard).

The sections of wall and levee downstream of Broad Street were also considered for a minimum freeboard of two feet. However, it is noted that the existing ground profile elevations downstream of the overtopping section are higher than the design freeboard which does not compromise the factor of safety for the wall. The remaining sections of wall and levee upstream of the Veterans Memorial Center wall will have a minimum freeboard of three feet. The SPF discharge increased by 25.4% produced a profile that is three feet above the SPF water surface profile at the initial overtopping section, and was used to establish freeboard superiority for the upstream wall and levee sections. With superiority added to the 25.4% profile, controlled initial overtopping at the Veterans Memorial Center wall is ensured. With the proposed top of protection profile, the Veterans Memorial Center wall will be overtopped by one foot, prior to the beginning of overtopping of other portions of the floodwall.

4. CONCLUSIONS

From the study and design of the West Columbus, Ohio Local Protection Project, the following conclusions were drawn with respect to freeboard:

a. The "qualitative" portion of the study, or the recognition of general and project specific factors that could affect the design water surface profile and freeboard, are of equal or greater importance than the actual computations.

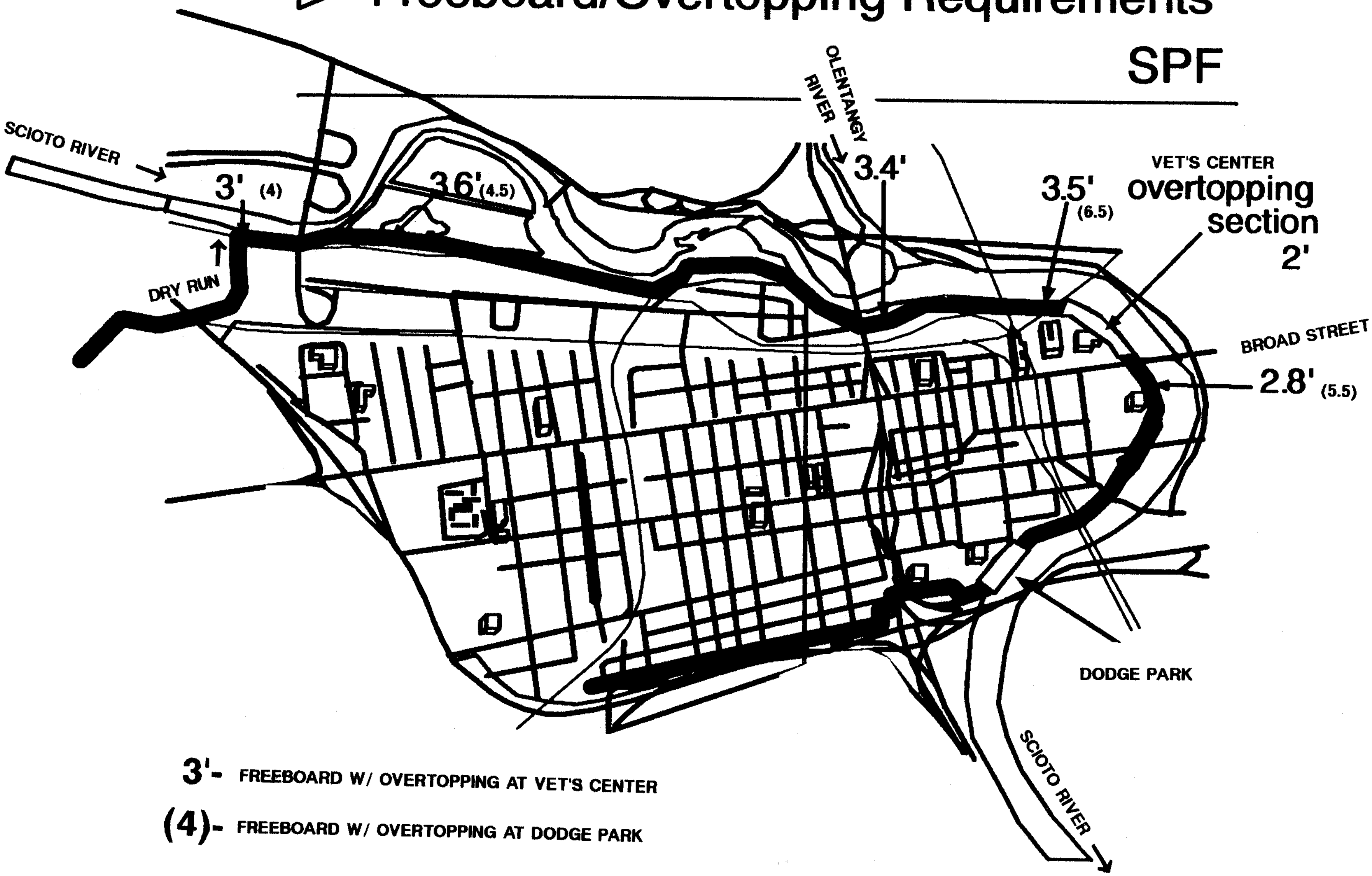
b. Different combinations of the location of the initial overtopping section and the selection of the design flood can be a significant factor in the amount of freeboard in other reaches of the project to provide the same level of protection. A thorough investigation of these items in the project formulation process is required in order to arrive at the most economical design.

WEST COLUMBUS, OHIO LPP

JUST112.IMG 04/17/90.CW

▷ Freeboard/Overtopping Requirements

SPF



LEVEE FREEBOARD ISSUES IN THE VICKSBURG DISTRICT

BY

Robert H. Fitzgerald[1] and William D. Shumate[2]

INTRODUCTION

Riverine levee freeboard requirements have become an issue on several projects within the Vicksburg District during recent times. The projects involved have included both urban and agricultural flood protection. A key issue pertaining to both types of projects is design of freeboard allowance in those areas where the design flowline is very near the natural ground elevation.

The current interest in freeboard design may have been influenced by several factors including changes in Corps guidance and a renewed interest in minimizing project costs. Increased participation by the project sponsors during all phases of project formulation and design appears to have also had an impact. Excessive levee freeboard may create problems for the sponsor in acquiring project rights of way in those areas where flooding is not a problem but where levee construction is required to meet the project level of protection. Land owner objections have been a problem on a number of occasions when right-of-way for levee freeboard only was requested.

PHYSICAL SETTING AND AVAILABLE DATA

<u>Urban Protection</u>. Much of the area contains residential subdivisions sited along a low bluff adjacent to the floodplain of the river. Flooding occurs only during extreme events and natural ground elevations are very near or slightly above the design flowline. The line of protection must pass through the area to provide the desired level of protection to more flood prone areas located further away from the river. The potential for damage to the more flood prone areas is quite high. However, the potential for loss of life is low due to the relatively slow rise of the river and the low velocities experienced. The computed required freeboard allowance considering wind and waves, flowline stability and other pertinent factors is 3 feet.

<u>Agricultural Protection</u>. Rivers often develop natural levees from deposition during flood events. As a result of this process the land adjacent to a river is usually somewhat higher than the natural ground farther away from the river. We tend to locate levees on this higher ground when designing riverine levee flood protection projects. This results in lower levee height and generally less disturbance of natural drainage. Unfortunately, this higher ground is the more productive farm land and is often taken out of production to provide flood protection to the more flood prone lands farther from the river. The potential for damage resulting from levee failure is generally lower than for urban areas and the potential for loss of life is

[1]Supervisory Hydraulic Engineer, Vicksburg District, US Army Corps of Engineers
[2]Hydraulic Engineer, Vicksburg District, US Army Corps of Engineers

generally lower than for urban areas and the potential for loss of life is usually quite low. The completed freeboard allowances are generally 2-3 feet depending upon the project.

STUDY APPROACH AND RESULTS

Urban Protection. Questions were raised by landowners along the proposed levee alignment of a particular project. The questions concerned whether a levee was needed along certain reaches of the line of protection. The area had recently experienced a flood of record and not all areas along the line of protection were inundated. The initial response to the landowners was that levees would not be constructed along those areas where existing natural ground elevation exceeds the design flowline. Subsequently, these criteria were changed such that the natural ground elevation must exceed the design flowline by 1 foot or levee construction would be required. This change resulted in a request to consider raising natural ground to 1 foot above the design flowline rather than building a traditional levee section with the required 3 feet of freeboard in those areas with only small elevation deficiencies. The non-typical levee section would be more aesthetically acceptable in the highly developed residential neighborhoods.

Extensive surveys were obtained and a line of protection developed to utilize as much "high" natural ground as possible. The freeboard design was also reviewed and the residual flooding conditions analyzed. Based on results of these analyses, the project was modified to include the "non-typical" or overbuilt levee section in those areas where total levee height would not exceed 3 feet. The adopted criteria contain allowance for engineering judgement in areas where levee height may exceed 3 feet for very short distances such as when crossing roadside ditches or other minor depressions. Figure 1 shows a cross-sectional view of the non-typical levee.

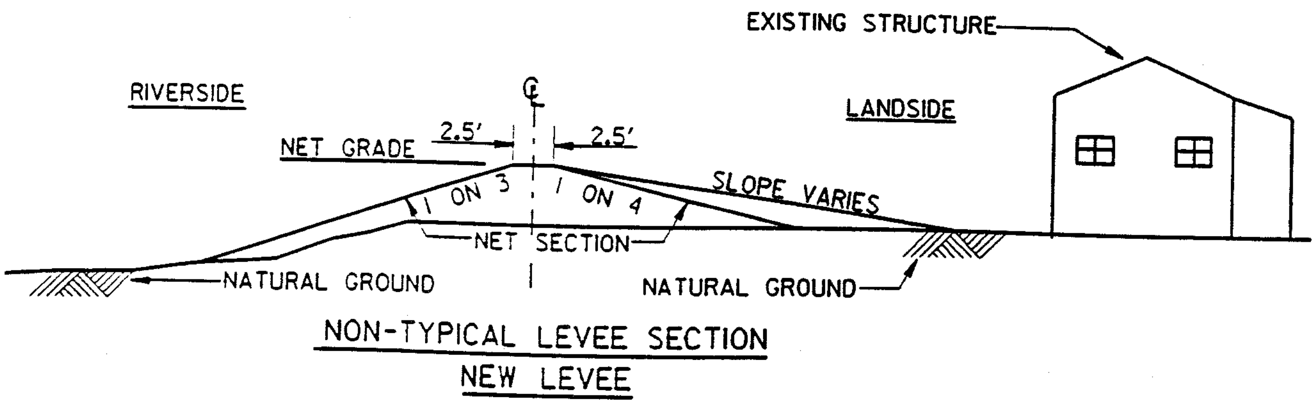


Figure 1. Non-typical levee section proposed for certain areas where total levee height does not exceed 3 feet.

Agricultural Protection. As with the urban areas, questions have been raised concerning the design of agricultural levees in those areas where the design flowline is near natural ground. Without freeboard only levees, slightly low areas would be raised to an elevation higher than the adjacent natural high ground with the addition of the required freeboard allowance. The result of the levee construction would then be reversal of the undulating or sawtooth pattern of the natural ground. This design would allow any overflow to occur on natural ground rather than overtop the levee. However, the results have been viewed as unacceptable by both Corps personnel and local sponsors/landowners.

A solution has been to review the freeboard design and to determine whether freeboard allowance can be reduced to 1 foot in those areas where the levee height is low and catastrophic failure is unlikely. The result is a levee which more smoothly transitions to natural ground. Project costs are reduced somewhat by reduced earth quantities and right of way requirements. These criteria are currently being applied to levees that are about 3 feet or less in height. Higher levees generally have a minimum freeboard allowance of 2-3 feet. Figure 2 shows an example of this concept for low levees.

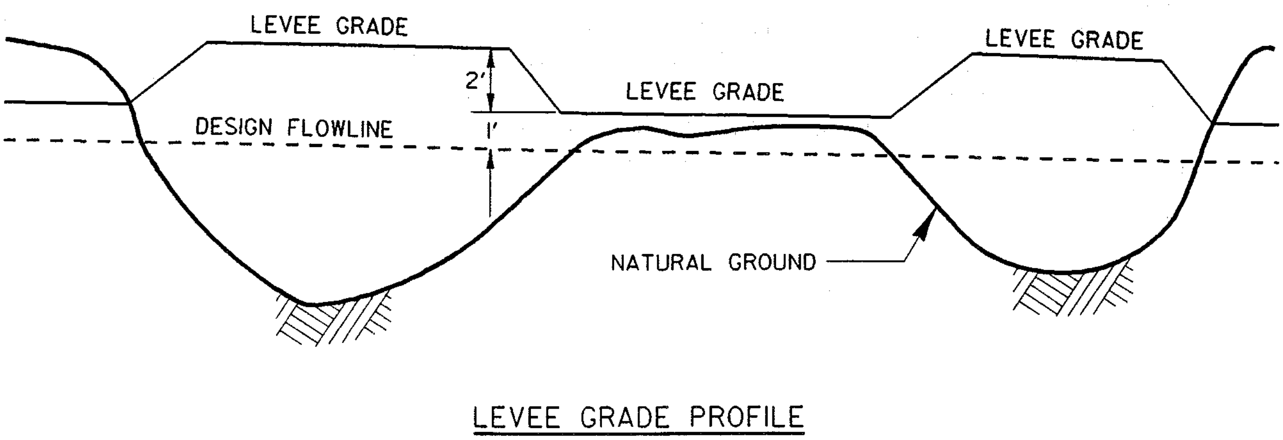


Figure 2. Typical levee grade profile with design flowline near natural ground elevation.

CONCLUSIONS

Current levee freeboard policy and guidance require that the hydraulic engineer design freeboard allowance for each project. The hydraulic engineer is the proper technical expert to determine the suitable freeboard requirements. Competition among projects for available funding and closer scrutiny by local sponsors requires that we provide sound engineering design in all aspects of our projects. Freeboard is no exception. However, we must not allow potential cost savings to compromise the safety of our projects.

LEVEE FREEBOARD ISSUES
RIO GRANDE AT ALAMOSA, COLORADO

BY

John R. D'Antonio Jr [1]

INTRODUCTION

The purpose of this paper is to discuss the issues of levee freeboard with respect to flood control levees on the Rio Grande in Alamosa County, Colorado. The key issues of this study include the complex flow conditions resulting from severe channel meandering and the absence of a suitable tie-back location for the north levee (left levee) within reasonable proximity to the Rio Grande. Levee freeboard policy requires that freeboard design incorporate sound engineering judgement and experience to reasonably assure that the project design flow will be contained, including any uncertainty factors, within the design levee height. If the levee height is exceeded, it should be at a location where damages and the threat of loss of life is minimized. (Levee Superiority)

PHYSICAL SETTING

The community of Alamosa is located in south-central Colorado in a broad, high mountain valley, the San Luis Valley. The headwaters of the Rio Grande are in the San Juan Mountains, which form the western boundary of the San Luis Valley. The Valley is almost level and has an elevation of approximately 7,500 feet, with the mountains ranging from 11,000 to 14,000 feet above sea level. Alamosa is drained by the Rio Grande and its tributaries. The Rio Grande is one of the major drainage areas in the United States and extends 1,800 miles from its headwaters in southern Colorado to the Gulf of Mexico.

AVAILABLE DATA

High flows on the Rio Grande through the study area are almost always a result of spring snowmelt runoff. Historically, floods have occurred from the melting of heavy snowpack augmented by rain from the middle of May until the end of June. The drainage area above the Del Norte stream gage is 1,320 sq. mi. ranging in elevation from 7,980 feet to nearly 14,000 feet. Between the source and the Alamosa stream gage, the drainage area is approximately 1,710 sq. mi. Vegetation ranges from pine and spruce, to pinon and juniper, and stands of sage brush. Trees lining stream banks are mostly cottonwoods and willows.

The potential for severe flooding exists in Alamosa and poses a threat to the citizens and improvements within the floodplain. It is currently estimated that the 500-year flood would cause damages in excess of $49 million within the study

---

[1]Hydraulic Design Section, U.S. Army Corps of Engineers, Albuquerque District

area: damage to residential structures would be the largest single category from the 500-year event. Damages start at approximately the 7-year event. Also, intangible damages such as a loss of life, disruption of community services, and increased health risks could occur as a result of flooding.

The Rio Grande Basin in Colorado, because of its relatively high elevation and sheltered position, is less subject to destructive floods than any other major basin in Colorado. The San Luis Valley itself receives only about 6 inches of precipitation annually.

STUDY APPROACH

The project plan consists of a north (left) levee and south (right) levee of approximately 5,000 feet and 11,100 feet, respectively. The north levee begins at the U.S. Highway 160 Bridge and follows a northwest alignment until termination at the intersection of Six Mile Road. The south levee begins where the Phase II construction ended, upstream of Highway 160 Bridge. The alignment bends away from the Rio Grande approximately 4,800 feet upstream of the USGS gage and runs southwest for some 3500 feet tying back to an elevation of 7551.0 feet.

Due to complex flow conditions and severe channel meandering at the upstream end of the project, consideration was given to using an unsteady flow model or two-dimensional flow model to compute water surface profiles through the project reach. After consultation with the Waterways Experiment Station (WES), the Hydrologic Engineering Center (HEC), reviewing a video tape of high flow conditions (5,100 cfs) and discussions of the extremely long duration snowmelt hydrograph there was a consensus that flow was, in-effect, steady-state and could be accurately modelled using HEC-2. Design water surface profiles were then computed for the 100-year design flow, 11,000 cfs, using HEC-2.

STUDY RESULTS - FREEBOARD CONSIDERATIONS

Freeboard Considerations	Incremental Freeboard
Instabilities in stage-discharge relationships resulting from flow retardance by debris and ice.	0.5 ft.
Instabilities due to severe meandering can increase the "n" value by as much as 30% when the majority of flow is confined to the stream channel. (Chow)	0.4 ft.
Variation in "n" value due to variations in density of Vegetation below the high water line. (30% increase)	0.4 ft.
Diff. in vel. head between channel and overbank locations	0.25 ft.
Energy losses due to changing flow area, e.g. constrictions, expansions and bends.	0.25 ft.
Allowance for uncertainty in assumptions.	0.5 ft.
Earthen Levee	0.5 ft.
	2.80 ft.

The above assumptions and allowances were made to determine an upper limit of acceptable freeboard for the design flow of 11,000 cfs and will be rounded up to 3.0 ft. The south levee (right levee) has a tie-back section that does not allow the water to outflank it upstream of the project, thus preventing flow into the town of Alamosa. The majority of the project benefits are derived from the protected areas of the right overbank which also contains the most potential for loss of life. From the above considerations, the recommended freeboard for the south levee is 3.0 ft.

The north levee (left levee) does not have a suitable tieback location within reasonable proximity to the Rio Grande and must tie into Six Mile Road at an elevation of approximately 7545.0. The computed water surface elevation at this location is approximately 7543.4 which corresponds to only 1.6 ft. of freeboard.

STUDY RESULTS - FREEBOARD REDUCTIONS AND KEY HYDROLOGIC ENGINEERING ANALYSIS

Issues that support a reduction in freeboard for the left levee are as follows:

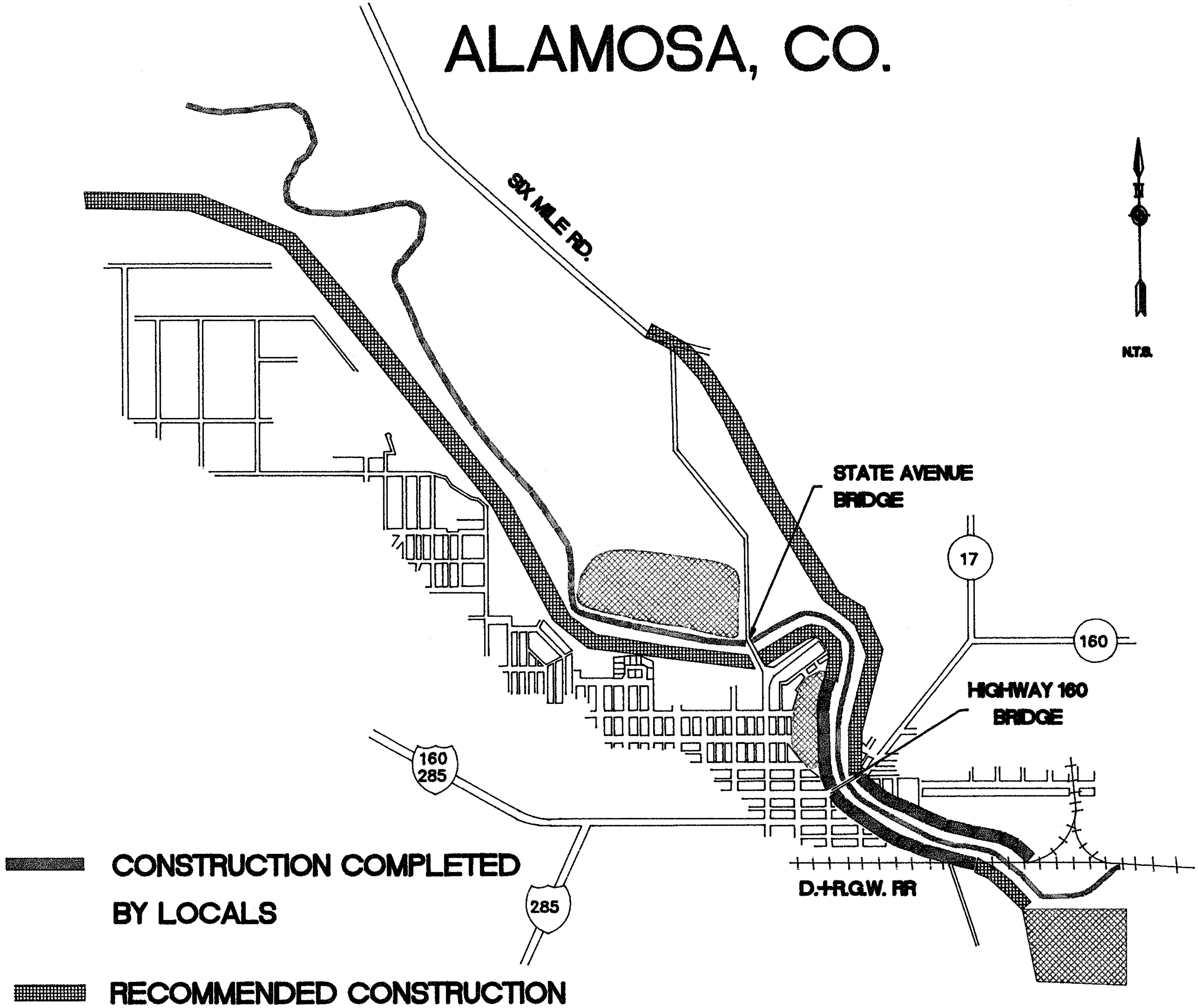
1. Since the left levee is protecting mainly agricultural land a value judgement should be made allowing overtopping of the agricultural before the urban area. (Under the present design this will happen anyway.) The urban area would get wet last and possibly attain a higher level of protection due to the volume of water going over Six Mile Road. (Levee Superiority)
2. Low risk of loss of life.
3. There currently exists a sufficient flood warning system.
4. Very predictable, long duration snowmelt hydrograph along with an interagency coordinated forecasting procedure will give ample warning time to sandbag and/or evacuate.

In some flood control projects, when checking water surface profiles above the design discharge, a small increase in elevation at the downstream end results in a much larger increase in elevation at the upstream end. However, the upstream end of the Alamosa Project is spread out over a much wider floodplain as compared to its downstream end. When the SPF discharges (18,000 cfs) are run through the HEC-2 model, water surface elevations increase 2 ft. to 3 ft. throughout the project with only a 1 ft. increase at the upstream end.

CONCLUSIONS

The Alamosa, Colorado Feasibility Report was approved by the Board of Engineers for Rivers and Harbors (BERH) on July 17, 1991. At that meeting an agreement was reached allowing 1.5 ft. of freeboard for the north levee (left levee) because of the lack of a suitable tie-back location and consideration of the issues that support a reduction in freeboard as discussed in the results section of this paper. This will require that Six Mile Road be raised approximately 1 ft. at the tie-in location. At this time PED studies are continuing with the project scheduled for authorization in the Water Resources and Development Act of 1992.

ALAMOSA, CO.



SIX MILE RD.
N
N.T.S.
STATE AVENUE
BRIDGE
17
160
HIGHWAY 160
BRIDGE
160
285
285
D.+R.G.W. RR
CONSTRUCTION COMPLETED
BY LOCALS
RECOMMENDED CONSTRUCTION

EFFECT OF CHANNEL EROSION ON FREEBOARD AT HOUSTON, MINNESOTA

by,

Robert G. Engelstad[1]

BACKGROUND INFORMATION

Houston Minnesota is located in southeastern Minnesota. The Root River has a drainage area of 1,270 square miles, and flows along the northern edge of the city. The Root River is characterized as a fast rising and also highly erodible river.

In the June 1975 Feasibility Report for Flood Control, both Standard Project Flood and 1% (100-yr) levels of protection were found to be economically feasible; however, the 100-yr level was chosen as the NED plan. Minnesota State Law requires where feasible that the Standard Project Flood be able to be contained within the freeboard of a 100-yr project design.

In viewing the entire Root River valley at Houston, it is evident that the Highway 76 bridge crossing is a severe channel constriction at this location. It was constructed in 1957, after a 1952 flood of record. Although this bridge hasn't experienced inundation from what the Corps would consider a rare flood event, apparently lessons from he 1952 event did not result in a project enlargement. A recent inspection by the Minnesota Department of Transportation has resulted in a recommendation that the bridge has at least a 30 year life remaining. Although removal/enlargement of the bridge is desirable from a Corps perspective, and it is a consideration in the project design, it is an event that won't occur for some time to come. The city of Houston has experienced several damaging flood events, and in 1969 built a stretch of flood control levee downstream of the Highway 76 bridge with PL-99 funding. This present levee system does not offer reliable flood protection, and in fact may instill a false sense of security.

CHANNEL EROSION CONSIDERATIONS

The erodible nature of the channel bottom has been suspected for some time in the District, but was not quantified until a moderate flood event of about 3600 cfs occurred in April of 1990. Flood reconnaissance streamflow measurements taken during one late evening, and the following day showed channel bottom changes that when correlated to the last available cross-section at the Highway 76 bridge showed a dramatic change. Comparison of the old channel, the 1990 measurement, and known bedrock elevations have dramatic effects on the upstream water surface profiles. We have learned from earlier field trips that the Minnesota Highway Department actually has an area downstream of the bridge where they have dug a trench to essentially "mine" the sand and gravel in the channel bedload, as a normal operation.

The proposed channel excavation is actually a shelf near the northern abutment which will go down only to about the waterline, for environmental

[1]Chief, Hydrology Section, St. Paul District, Corps of Engineers

reasons. Nevertheless, substantial benefit to the water surface profile will be evident.

PROPOSED CONDITIONS

Two plans were studied in detail in the Design Memorandum. The following table shows elevation for each of these plans.

Water Surface Profiles

Standard Project Flood (SPF, 167,000 cfs), Top-of-Levee (TOL, 92,000 cfs) and Design Flood (1%, 43,000 cfs)

	Channel			M3B1 (1) BRIDGE IN PLACE			M3B2 BRIDGE AND EMBANKMENT REMOVED		
Location	Cross Section	Dist. (ft.)	Min. Elev.	SPF CWSEL	TOL CWSEL	1% CWSEL	SPF CWSEL	TOL CWSEL	1-% CWSEL
U/S old Hwy 76	5.4	0	663.0	686.2	683.3	680.8	686.2	683.3	680.8
	6.0	520	663.0	686.6	683.6	680.9	686.6	683.6	680.9
Levee D/S End	7.0	680	664.8	686.9	684.0	681.0	686.9	684.0	681.0
	8.0	700	665.0	687.3	684.3	681.6	687.3	684.3	681.6
	9.0	1220	665.3	687.9	684.7	681.8	687.9	684.7	681.8
	10.0	700	666.6	688.4	685.2	682.2	688.4	685.2	682.2
	11.0	800	666.6	689.2	686.0	683.0	689.2	686.0	683.0
	12.0	1230	667.2	689.8	686.3	683.4	689.8	686.6	683.5
D/S Hwy. 76	13.0	550	668.0	690.5	686.7	683.9	690.3	686.8	683.7
U/S Hwy. 76	13.5	340	667.0	699.7	690.1	684.5			
	14.0	440	668.9	700.9	690.9	684.8	692.3	688.4	684.8
	15.0	670	668.5	702.0	692.1	685.8	693.9	689.6	685.4
	16.0	1190	667.2	702.7	693.1	686.8	696.3	691.2	686.5
Levee U/S End	17.0	600	668.6	702.7	693.2	686.9	696.4	691.4	686.7

(1) Elevations shown for plan M3B1 reflect modifications to the Hwy. 76 bridge channel and channel geometry as surveyed in April 1990.

Alternative M3B1 consists of amain levee along and set back from the river with the existing Highway 76 bridge in place. Alternative M3B2 is similar to the other, but considers moving the Highway 76 bridge and embankment upstream of Houston. The top-of-levee (TOL) profile was computed by running the HEC-2 model with the discharge of 92,000 cfs, which was selected by using a rating curve developed at the downstream tie-back levee. The discharge at the elevation three feet above the design (1-percent flood) elevation was the top-of-levee discharge. This is the same technique adopted by the St. Paul District some years ago on the Souris River at Velva, North Dakota which basically set the technique with OCE for freeboard evaluation.

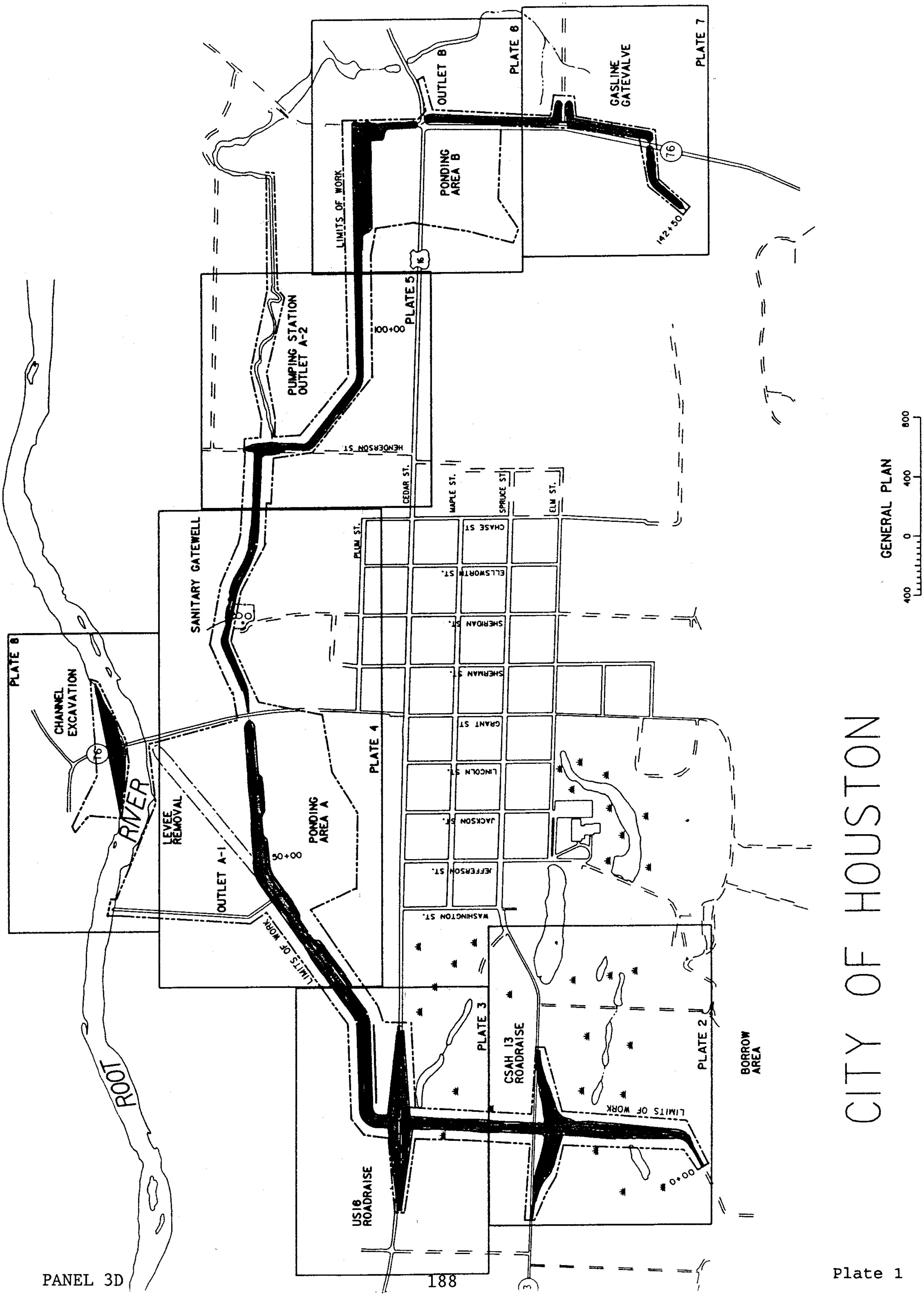
While this upstream bridge relocation has obvious advantages, the community currently understands and accepts the risks they are under in the interim. The best job the Corps can do in this particular situation, recognizing the flashy propose excavating key material under Highway 76 and take full advantage of the channel erosion during a flood at this critical location. The slope near the abutment will be riprapped.

DEGREE OF PROTECTION

The question of what degree of protection to design the levee for was complicated by the number of agencies and their various criteria with authority over this project. The three agencies most involved are the Minnesota Department of Transportation (MnDOT) , Minnesota Department of Natural Resources (MnDNR) , and the Corps. The MnDNR criteria states that the levee is required to protect the community from the 1-percent flood plus freeboard, and the SPF where practical be contained within this freeboard. The MnDOT criteria only allows replacement of a bridge if it is near its economic life. The Corps criteria states that only projects that are economically justified can be built. The MnDNR and the Corps criteria conflict because an SPF-without-freeboard level of protection levee is not economically justified with the Highway 76 bridge at its current location. If the bridge were moved upstream of Houston and the embankment removed by the MnDOT, the SPF-without-freeboard protection would be economically justified. Criteria of the MnDOT conflicts with the Corps in that the Highway 76 bridge still has about 30-years of economic life left.

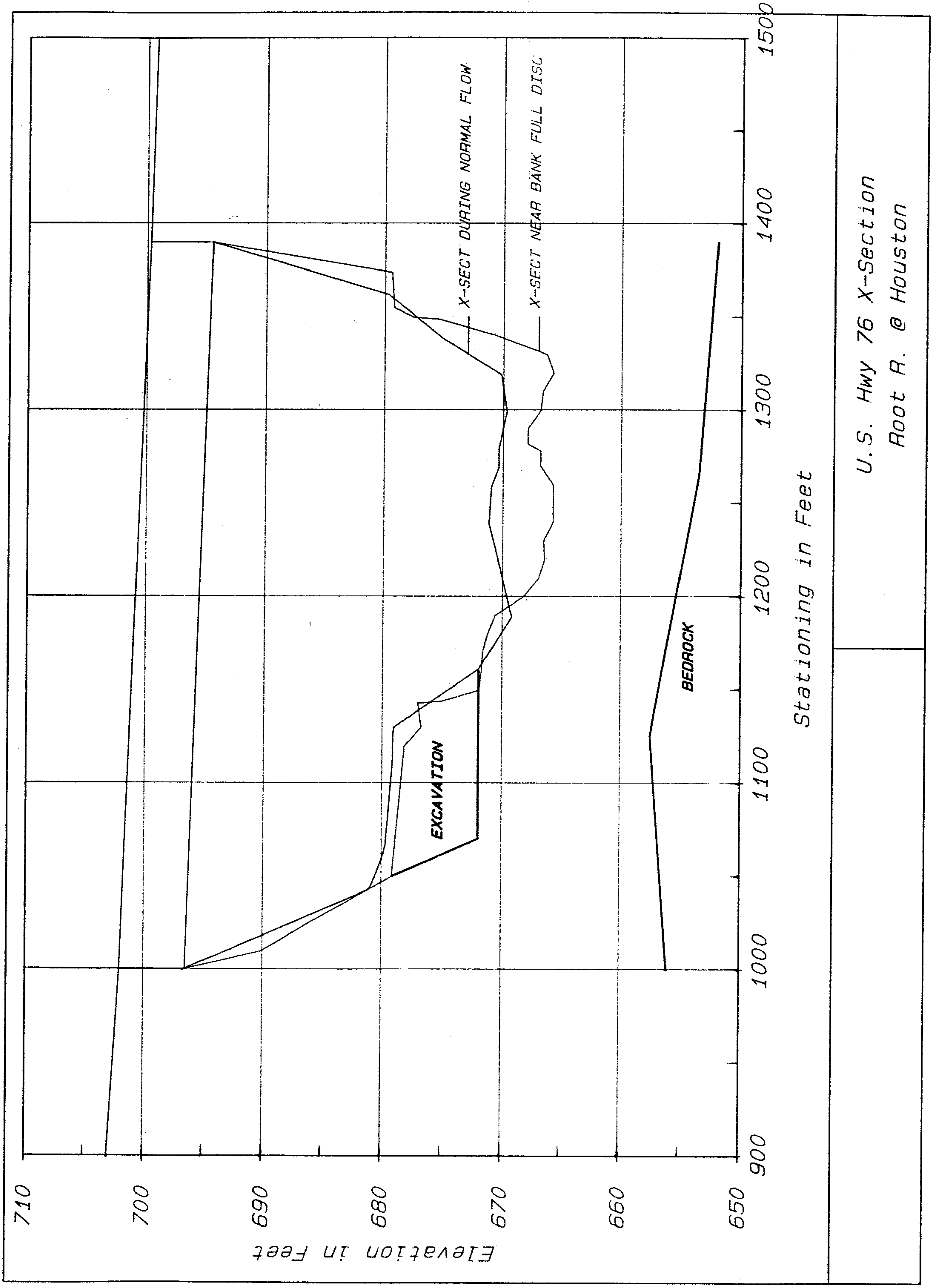
A solution to these conflicts is plan M3B1 modified to meet the requests of the MnDNR and the MnDOT. The selected plan is M3B1, with the top of levee elevations, with the exception of a portion of the downstream tie-back levee, constructed to contain the SPF-without-freeboard elevations of plan M3B2 assuming the MnDOT will remove the bridge after its useful life and remove the embankment. The top of the downstream tie-back levee from the northeast corner southward for a distance of approximately 450 feet to U.S. Highway 16 will remain at the Corps TOL design (92,000 cfs) which is the 1-percent exceedance frequency flood plus three feet of freeboard. The remainder of the downstream tie-back levee from U.S. Hwy. 16 southward will be constructed to the SPF-without-freeboard elevations. The top of that portion of the downstream tie-back levee must remain at the M3B1 level until the bridge can be removed. When the bridge is removed, that portion of the tie-back levee on the downstream end of the project would be raised to contain the SPF-without-freeboard as required by the local cooperation agreement. The portion of the levee upstream of Highway 76 will not provide SPF-without-freeboard protection until the Highway 76 bridge and embankment is removed. The MnDNR has agreed to allow a levee with a lesser degree of protection than SPF-without-freeboard protection at Houston if the MnDOT agrees to move the bridge upstream when it is scheduled to be replaced and the downstream tie-back levee is raised to the SPF-without-freeboard at that time.

Regardless of what the MnDNR requirements are, SPF-without-freeboard level of protection is the recommended plan due to the nature of the Root River. The Root River is an extremely flashy river with very limited warning time to evacuate or perform any emergency measures. In addition, the entire community is located within the floodplain. This level of protection will be provided once the existing Highway 76 bridge and embankment is removed and relocated upstream.



PANEL 3D

Plate 1



PANEL 3D
Plate 2

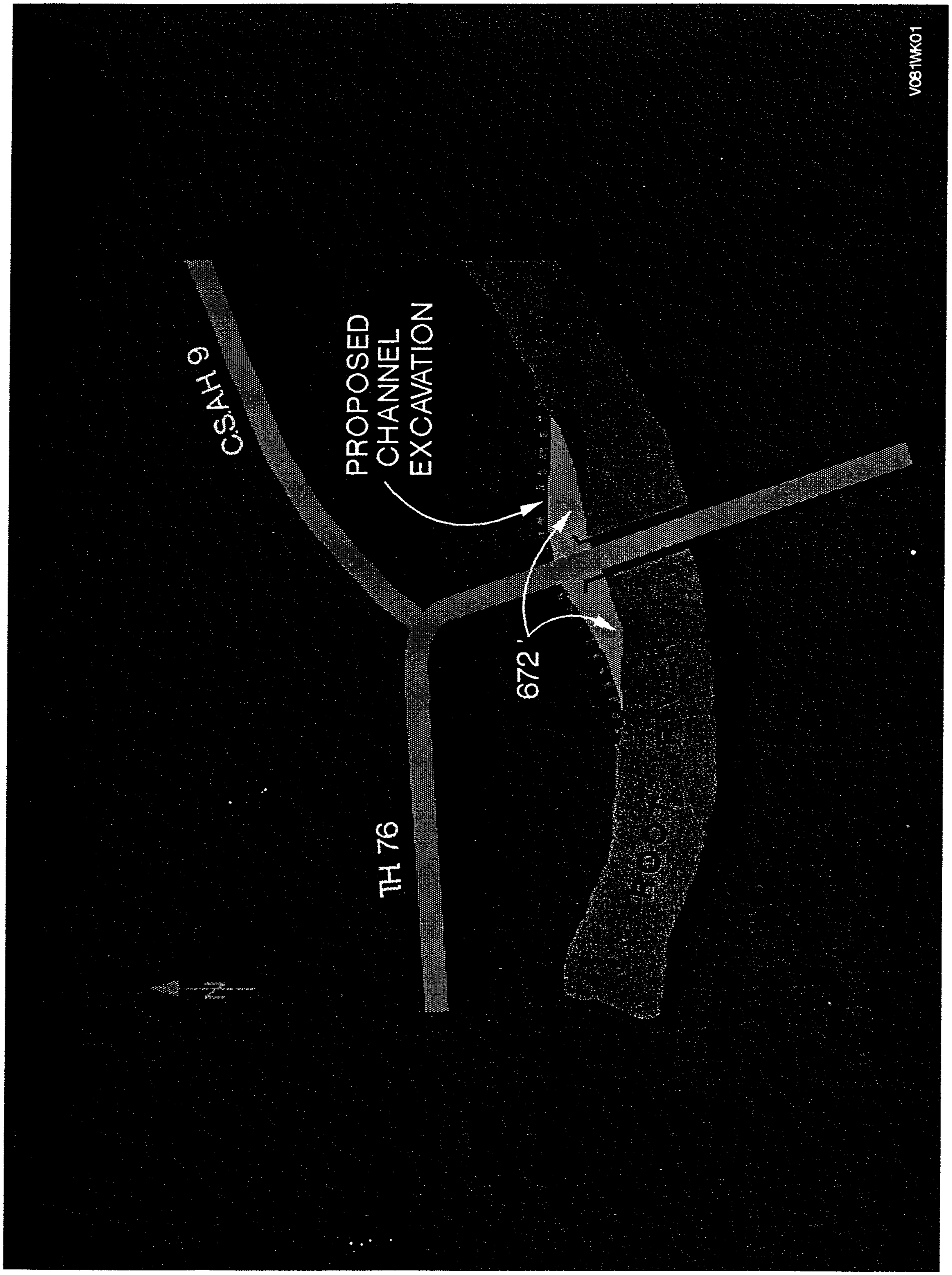
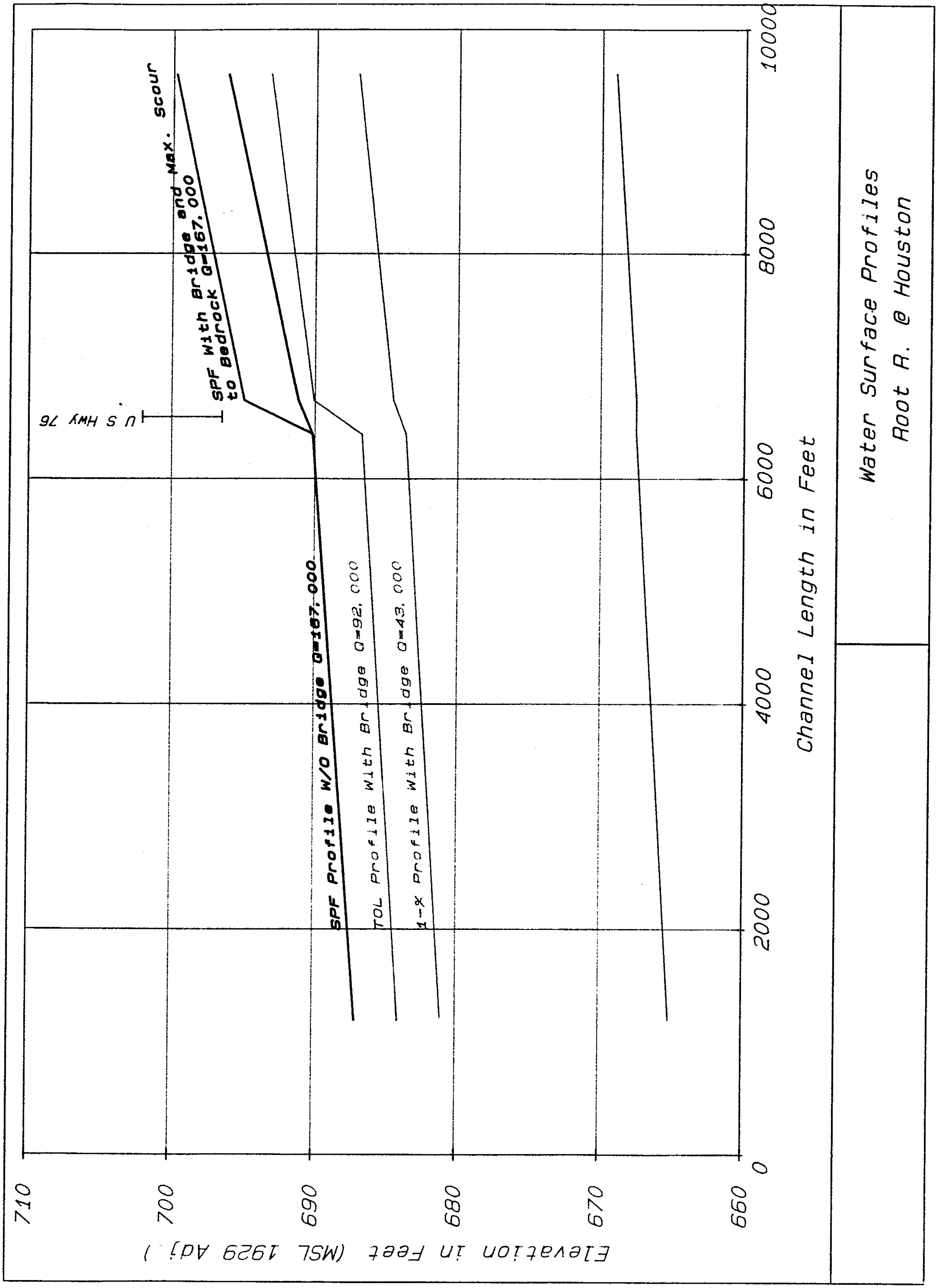
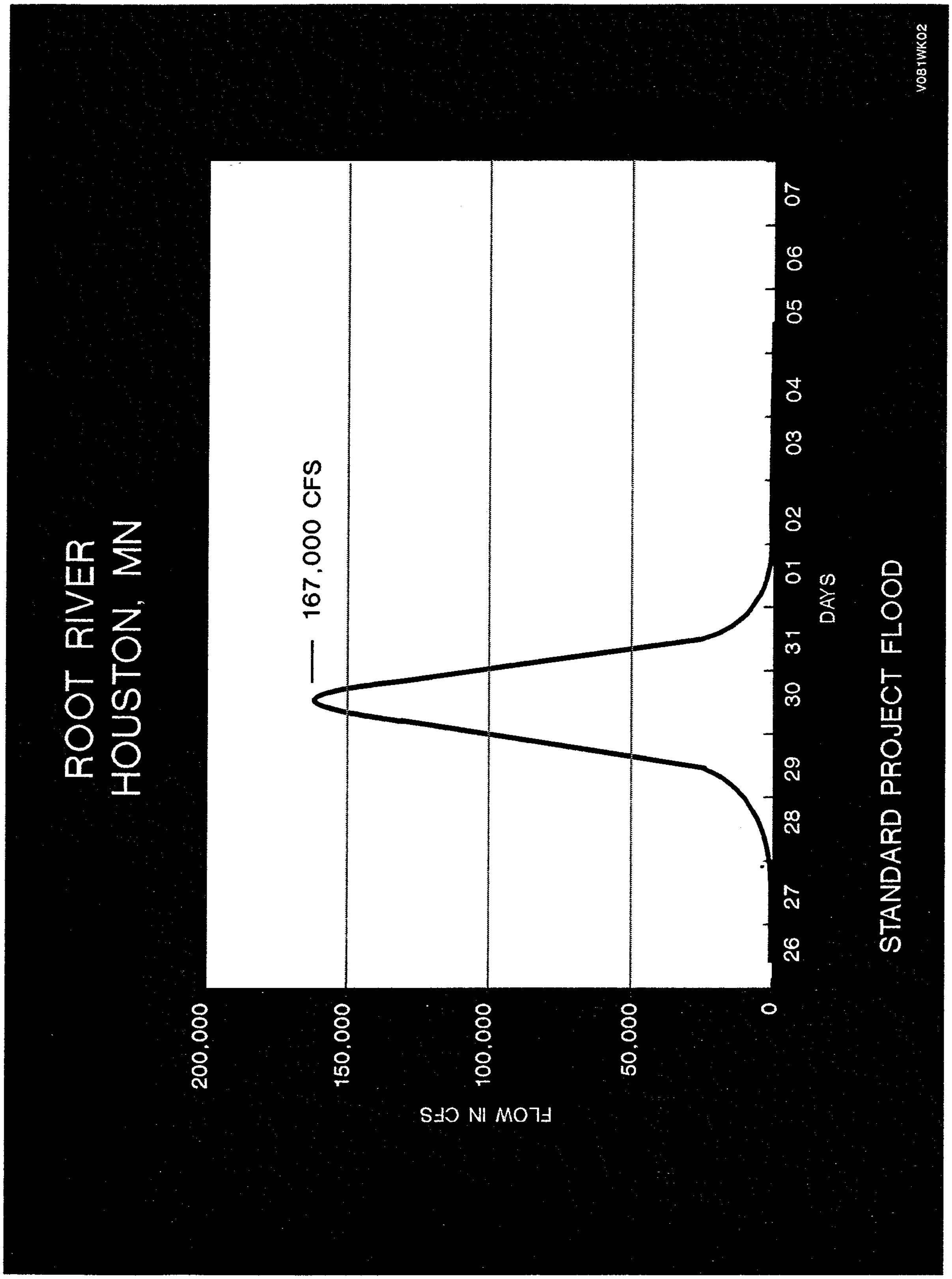


Plate 3


Water Surface Profiles
Root R. @ Houston
Channel Length in Feet
Elevation in Feet (MSL 1929 Adj.)
SPF With Bridge and Max. Scour to Bedrock Q=167,000
U S Hwy 76
SPF Profile W/O Bridge Q=167,000
TOL Profile With Bridge Q=92,000
1-% Profile With Bridge Q=43,000
0
2000
4000
6000
8000
10000
660
670
680
690
700
710



ROOT RIVER
HOUSTON, MN
167,000 CFS
FLOW IN CFS
200,000
150,000
100,000
50,000
0
26
27
28
29
30
31
01
02
03
04
05
06
07
DAYS
STANDARD PROJECT FLOOD
V081WK02

SUMMARY OF EVENING SESSION

Overview

The evening session included two paper presentations. The first paper described uncertainties in water surface profile computations and the second paper presented the results of an international survey of levee freeboard procedures.

Paper 10. William (Tony) Thomas, WES, presented a paper entitled, "Hydraulic Uncertainties in Water Surface Calculations." Tony described his paper as a collection of insights he has gained from a career of trying to calculate water surface profiles in open channels having both fixed and mobile boundaries. Uncertainties are grouped into categories: uncertainties embedded in theory, uncertainties embedded in the numerical solutions, uncertainties embedded in baseline data, uncertainties embedded in design coefficients and uncertainties embedded in forecasting the future.

Physical processes are 3-dimensional in space and unsteady in time. The current technology requires transforming these into 1-dimensional space and steady state in time. However, that is because of limited computer power - not because 1-dimensional is the best way to make the design. One of the fundamental uncertainties in water surface profile calculations comes from the "art" of making the 1-dimensional approximation. Computer codes like HEC-2 or HEC-6 or UNET do not help in that transformation. Consequently, the reliability of results depends on the insight and resourcefulness of the modeler. Cross section spacing, cross section alignment with the flow, flow paths, ineffective areas, hydraulic roughness values, the behavior of mobile boundary channels and the complexity of bridge hydraulics must all be given careful attention. In cases where flow diversions are part of the project design, a detailed hydraulic design is required. Only bits and pieces of the uncertainty in these several areas has been researched; so, it is the responsibility of the engineer to conduct sufficient studies to insure his/ her assumptions produce calculated results that are within the tolerance permitted by levee freeboard.

Paper 11. Robert MacArthur, Principal Civil Engineer, Water Engineering & Technology, Inc., Davis, CA, presented a paper entitled, "International Survey of Levee Freeboard Design Procedures." The paper summarized the result of an international survey which was conducted to determine the design and analytical practices of freeboard for stream channels and flood control levees. A detailed questionnaire was sent to more than 250 individuals, universities and water control agencies around the world. The questionnaire requested information regarding how the design flood and top elevation of flood control levees are determined, if freeboard is used, what computational methods are used to quantify these parameters, and if there are any environmental or other issues taken into consideration during levee design. A total of 205 questionnaires were sent to 45 countries. Over 50 responses were received from 18 countries. Of those responding, 35 provided information that was meaningful and that could be summarized in this paper.

The survey determined that freeboard is used in every country responding to the survey even though many different methods are used internationally to determine the amount of freeboard necessary for various levee designs. Several of the developing countries apply a simple fixed value for freeboard while others base freeboard design on local conditions. Responses from the international freeboard survey are summarized according to country and specific design criteria. Results are tabulated according to various design criteria. A summary of research needs and recommended future studies is also included in the paper.

Hydraulic Uncertainties in Water Surface Calculations

by

William A. Thomas[1]

1. Introduction. The material in this paper does not represent an exhaustive study of the topic nor is it the result of an extensive program of research and modeling which examined uncertainties in hydraulic factors in free surface flows. Rather, it is a collection of insights the author has gained from a career of trying to calculate water surface profiles in open channels having both fixed and mobile boundaries. Uncertainties are grouped into categories: uncertainties embedded in theory, uncertainties embedded in the numerical solutions, uncertainties embedded in baseline data, uncertainties embedded in design coefficients and uncertainties embedded in forecasting the future.

2. Summary and Conclusions: The Art of Computational Hydraulics. Physical processes are 3-Dimensional in space and unsteady in time. The current technology requires transforming these into 1-dimensional space and steady state in time. However, that is because of limited computer power - not because 1-D is the best way to make the design. One of the fundamental uncertainties in water surface profile calculations comes from the "art" of making the 1-D approximation. Computer codes like HEC-2 or HEC-6 or UNET do not help in that transformation. Consequently, the reliability of results depends on the insight and resourcefulness of the modeler. Cross section spacing, cross section alignment with the flow, flow paths, ineffective areas, hydraulic roughness values, the behavior of mobile boundary channels and the complexity of bridge hydraulics must all be given careful attention. In cases where flow diversions are part of the project design, a detailed hydraulic design is required. Only bits and pieces of the uncertainty in these several areas has been researched; so, it is the responsibility of the engineer to conduct sufficient studies to insure his/her assumptions produce calculated results that are within the tolerance permitted by levee freeboard.

3. Uncertainties Embedded in Theory.

a. Equations of Flow and Continuity. The equations for unsteady flow in mobile bed channels are: the equation of continuity for sediment, the

---

1. Research Hydraulic Engineer, U. S. Army Engineer Waterways Experiment Station, 3909 Halls Ferry Road, Vicksburg, MS

equation of continuity for water and the equation of motion for the water–sediment mixture. The continuity equations for unsteady flow are established by considering the conservation of mass in an infinitesimal space between two channel sections.

$$\frac{\partial \rho Q}{\partial t} + \frac{\partial \rho Q V}{\partial x} + \frac{g A \partial \rho y}{\partial x} = \rho g A(S_o - S_f + D_\ell) \qquad \text{(Motion)}$$

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} + \frac{\partial A_d}{\partial t} - q_w = 0 \qquad \text{(Continuity of Water)}$$

$$\frac{\partial Q_s}{\partial x} + (1-P)\frac{\partial A_d}{\partial t} + \frac{\partial A_s}{\partial t} - q_s = 0 \qquad \text{(Continuity of Sediment)}$$

where

A = end area of channel cross section
A_d = volume of sediment deposited on the bed per unit length of channel
A_s = volume of sediment suspended in the water column per unit length of channel
D_ℓ = momentum loss due to lateral inflow
g = acceleration of gravity
P = porosity of the bed deposit, (Volume of voids divided by the total volume of sample.)
Q = water discharge
Q_s = sediment discharge
q_s = lateral sediment inflow per unit length of channel, outflow (+), inflow (–)
q_w = lateral water inflow per unit length of channel, outflow (+), inflow (–)
S_f = friction slope
S_o = slope of channel bottom
t = time
x = horizontal distance along the channel
U = flow velocity
y = depth of flow
ρ = density of the water

b. The following assumptions were made in deriving these equations.

1. Open channel hydraulics is an extension of pipe hydraulics.

2. The channel is sufficiently straight and uniform in the reach so that the flow characteristics may be physically represented by a one–dimensional model.

3. The velocity is uniformly distributed over the cross section.

4. Hydrostatic pressure prevails at every point in the channel.

5. The water surface slope is small.

6. The density of the sediment-laden water is constant over the cross section.

7. The unsteady flow resistance coefficient is assumed to be the same as for steady flow in alluvial channels and is approximated from resistance equations applicable to alluvial channels or from field survey.

c. Cross section shape. In pipe hydraulics the hydraulic radius term has physical significance. In free surface flow in complex cross sections, there are a variety of ways to come up with a "hydraulic radius" value. Some, like HEC-2, subdivide the cross section into subsections having similar hydraulic properties. James and Brown compared the water depth calculated by subdividing the cross section into channel and flood plain subsections with the water depth observed in a physical model (James and Brown, 1977). They found the physical model depth to be more than the calculated depth, as the flow depth increased, figure 1. Moreover, as the total depth approached (1.4 * channel depth), the physical model depth approached the value calculated by not subdividing the cross section. In this test both the channel and flood plain were constructed from the same material. Therefore, the hydraulic roughness values should have been the same for both.

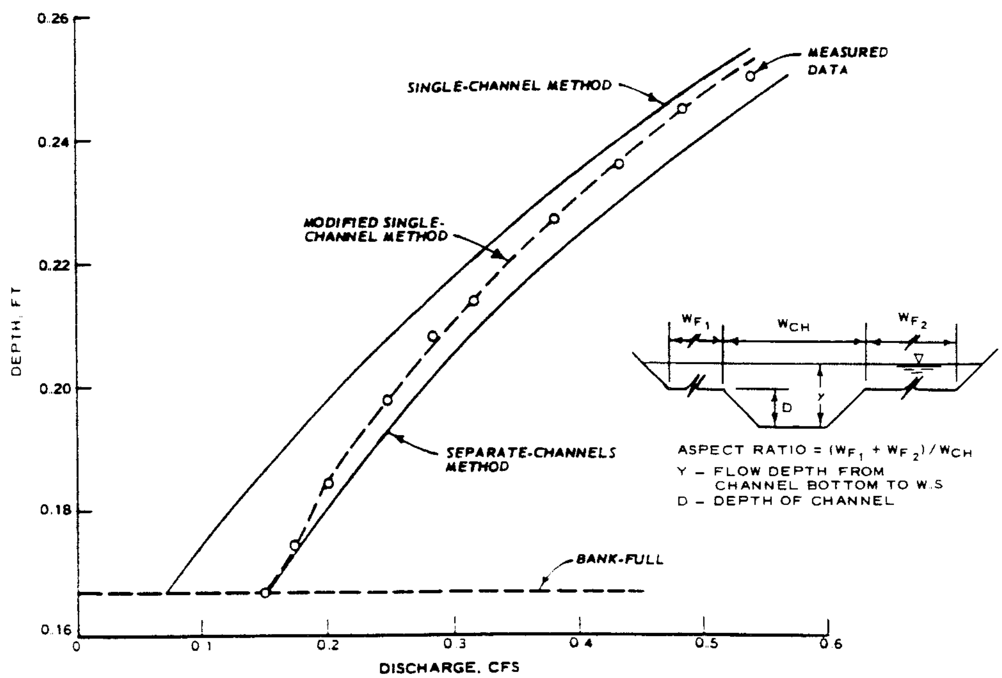


Figure 1. Comparison of Measured with Calculated Depth

Of course, when high water marks are available for confirming n-values, that deviation in water depth would be accounted. On the other hand, the n-values one uses are a function of the hydraulic theory used to calculate them and not purely a measure of flow resistance. This introduces a hydraulic uncertainty in the application of 1-Dimensional theory to complex cross sections that has not been investigated for the general case.

d. Bridge and Culvert Hydraulics. This is an area of considerable uncertainty in many cases because the flow is strongly 3-dimensional. Recent model studies of Brush Creek, Kansas City District, and Truckee River, Sacramento District, showed substantial differences between the physical model water surface and calculated values (verbal comments by John George, WES). In both cases the flow was supercritical or nearly so. Reports of that work are in preparation. Suffice it to say that in some cases there is so much uncertainty in calculated water surface profiles that even a freeboard correction may not be safe.

e. Surface Waves. The equations solved by HEC-2, HEC-6, DWOPER, and UNET are the long wave equations. Surface waves are depicted by short wave equations. These can be generated by wind, by changes in cross section shape or by channel bends (in high Froude Numbers) and by boats. In steep chutes roll waves can develop. These all result in increased water depth, but they can be considered theoretically and, therefore, should not be lumped into the uncertainty category.

4. Uncertainties Embedded in the Numerical Solutions. Thomas (1975) reported the sensitivity of water surface profile calculations to different formulations of the variables in the Manning Equation. HEC (1986) reported on the effect of cross section spacing on accuracy of water surface profiles. This is an area of uncertainty which can be minimized by the careful location and spacing of cross sections.

5. Uncertainties Embedded in Baseline Data. The water discharge is measured within a tolerance of ±5 to 10 percent at gages. However, on major alluvial rivers like the Mississippi and Arkansas, the scatter in stage - discharge measurements ranges up to 6 ft. Some of that is measurement error, and some is due to physical processes such as temperature effect, the dynamic loop effect and, I believe, losses into ground water. Consequently, two successive floods of the same probability can have significantly different water surface profiles.

In many streams tributary inflow makes it difficult to interpolate gaged discharges to the location of observed high water marks. Moreover, the channel bed can scour or deposit and change the location of controls from those present in the backwater deck - i.e., usually channel surveys are made during low water.

High water marks are usually measured on one side of the river and the water surface assumed to be horizontal across the section.

Flow distribution between left flood plain, channel and right flood plain are usually not measured. Such field data is not available to "confirm" the water surface profile model, and by its very nature the only question that it can answer is "What is the water surface profile for conditions that do not differ very much from those observed?"

Between the high water marks, the calculations are an approximation of the true water surface profile.

6. <u>Uncertainties Embedded in Design Coefficients.</u>

a. <u>Hydraulic Roughness by Handbook Methods.</u> Hydraulic roughness (i.e., n-values for the Manning Equation) is a major source of uncertainty in water surface profile calculations. Field data at each project are required to verify selected values. However, designers create situations for which there is no field data. The engineering solution is to use "calibrated photographs" and other subjective methods to associate hydraulic roughness values with the conditions they observe and anticipate in the project reach. These methods are documented in "handbooks," and, therefore, they will be referred to as "Handbook Methods" in this paper. References are presented in appendix A.

(1) <u>For Example.</u> Figure 2 is the proposed design for a levee project in an urban area in which the sponsor wants to include vegetation in the project. The hydraulic roughness values for this section are

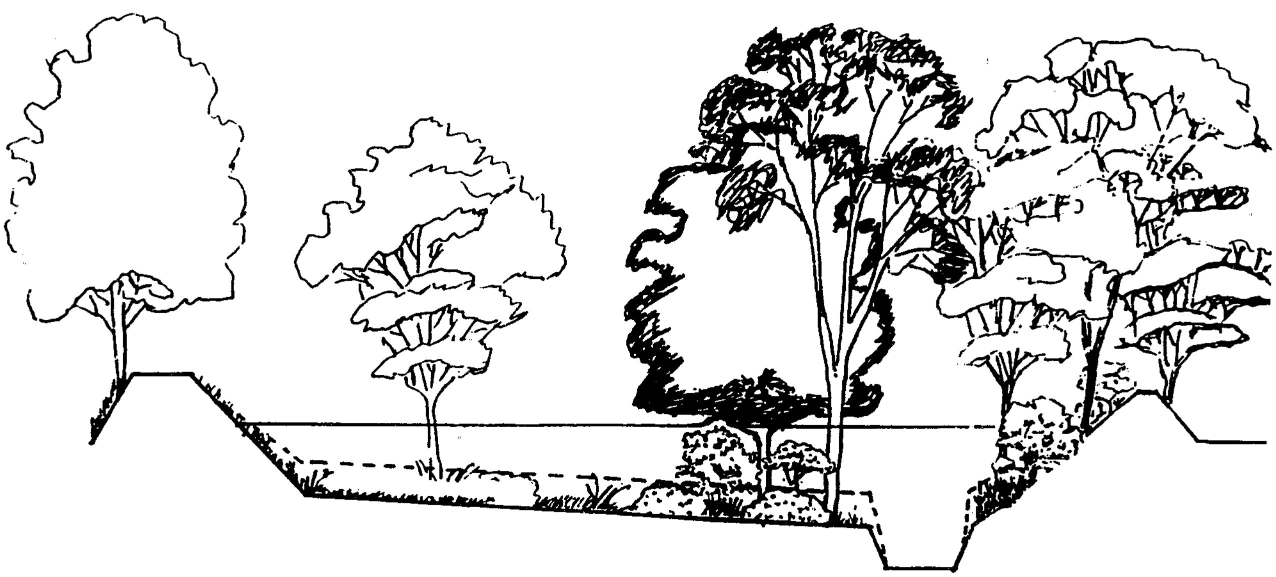


Figure 2. Design Cross Section

TABLE 1. Hydraulic Roughness, Channel Bed and Banks

Reference	m	n0	n1	n2	n3	n4	n(1)
Barnes, p 78	–	.037					.037
Chow, p109, Fine Gravel	1.0	.024	.005	.0	.0	.005	.034
Chow, p112, D-1a3	–	.040					.040
Chow, p120, #14	–	.030					.030
USDT, pp 4 & 7	1.0	.024	.002	.002	.001	.005	.034
Schwab, et al, Appendix C, #17	–	.035					.035
King & Brater, p7-17, Natural.2							(2)
mean							.035
standard deviation							.003

Notes:

(1) n = (n0 + n1 + n2 + n3 + n4)m

where

m = Ratio for meandering
n0 = Base n-value
n1 = addition for surface irregularities
n2 = addition for variation in channel cross section
n3 = addition for obstructions
n4 = addition for vegetation

(2) The n-values for this channel type seem to come from the same source as the Soil and Water Conservation Table. Therefore, they were not included in this calculation.

TABLE 2. Hydraulic Roughness, Flood Plain

Reference	n0	n1	n2	n3	n4	n(1)
Barnes, None Given						
Chow, p113, D-2c5	.100					.100
Chow, p123, #23	.125					.125
USDT, pp 4 & 9	.028	.010	–	.012	.050	.100
Schwab, et al, None Given						
King & Brater, None Given						
mean						.108
standard deviation						.012

Notes: (1) Same n-equation as channel bed and banks estimated from several different "handbook" sources in the Tables 1 and 2. Note that Handbooks divide n-values into two categories: (1) channel bed and bank and (2) flood plains.

(2) Sensitivity of Calculations to n-Values. The calculated water depth is shown in Table 3 using the mean values of both channel and overbank roughness. The mean values are considered to the best estimate, statistically.

TABLE 3. Sensitivity of Depth to n-value

Case	n-value Channel	Flood Plain	Water Surface
Mean	.035	.108	9.4
+1 Standard Deviation	.038	.120	10.1

Both n-values were increased by adding their standard deviation. The resulting water surface elevation increased about .7-ft, from 9.4-ft to 10.1-ft. This standard deviation in n-values is really quite small. However, it demonstrates how sensitive water depth is to n-value and, consequently, the need for freeboard.

b. Hydraulic Roughness by Analytical Methods. Investigators continue to explore physically based hydraulic roughness equations. The Moody-Type Diagram (USACE, EM -1601) and Strickler's equation (Chow, 1959) are classic examples proposed for fixed bed rivers. More recently, the Limerinos equation (Limerinos, 1970) and the Brownlie bed roughness predictor (Brownlie, 1983) have been published for use in mobile bed rivers.

(1) Surface Roughness by Panel. To use analytical methods, the engineer is faced with assigning physically based parameters, like surface roughness or material type, to each element (i.e., panel) in the cross section. Analytical methods convert these to n-value. The new Hydraulic Design Package (SAM), under development at WES, (WES, DRAFT), offers 10 methods for expressing n-values, Table 4.

(2) Compositing Hydraulic Properties. It is necessary to assemble these "panel" values into a "composite" n-value for representing the cross section. This is called "compositing." There are several procedures for compositing (Chow, 1959). HEC2 uses a "Conveyance Method" for the flood plain and the "Equal Velocity Method" for the main channel (HEC-2, 1990). The SAM package offers 4 Methods for Compositing, Table 4.

(3) Example. Table 5 illustrates the development of n-values for the cross section in figure 2, by the application of analytical equations.

TABLE 4. n–Value Equations and Compositing Methods in SAM

n–value Equations	Methods for Compositing(1)
MANNING–n	ALPHA METHOD
MOODY	EQUAL VELOCITY METHOD
STRICKLER	TOTAL FORCE METHOD
LIMERINOS	TOTAL DISCHARGE METHOD
BROWNLIE	
GRASS E	
GRASS D	
GRASS C	
GRASS B	
GRASS A	

(1) All methods for compositing are available to all equations.

TABLE 5. Hydraulic Roughness from Surface Properties

Panel No	Station	Elevation	n–Value	Ks–ft	Comment
	0.0	18.00			
1					Grass D: Bermuda grass cut to 2.5–in From Soil Conservation Service, (Chow, pp 179–184)
	50.0	5.50			
2			.100		Left Flood Plain,(USDT, Table 3) n = (n0+ n1 + n2 + n3 + n4) = (.028+.010 + +.012+.050)
	125.0	2.00			
3				1	Strickler Ks–ft; Assumed (Chow, p 206)
	129.0	0.00			
4					Brownlie Bed Roughness Equations (Brownlie, 1983) D84 = 6.5mm, D50 = 1.7mm, D16 = 0.4mm
	154.0	0.00			
5				1	Same as left Bank, (Panel 3)
	158.0	2.00			
6			.125		Right Flood Plain, (USDT, Table 3) n = (.028 + .010 + + .012 + .075)
	168.0	5.50			
7					Same as left levee, (panel 1)
	218.0	18.00			

The analytical methods in the Hydraulic Design Package, SAM, will be used to develop the n-value for each panel, to composite, and to make the water surface calculations. The cross section is coded as station and elevation starting at the levee on the left, panel 1. That panel is designed to be a mowed, grass surface. The n-value will depend on the flow depth and velocity over the panel. Panel 2 is the left flood plain.

The best source for n-values in large, woody vegetation is the USDT procedure, referenced in Table 2. Therefore, that n-value will be coded directly. Channel bank roughness will be calculated by estimating a surface irregularity, Ks of 1 foot, for the bank. Channel bed roughness will be calculated from the bed sediment gradation using the Brownlie Bed Roughness Equations. That method predicts whether the roughness is lower or upper regime. It uses the D84, D50 and D16 grain sizes of the bed surface. Vegetation on the right flood plain was judged to be more dense than on the left side, see panel 6. The right levee, panel 7, will be the same as the left levee.

(4) Calculated Results. Calculated results using the analytical method are shown in the Table 6. Note the column headed "n-value" in the Flow Distribution Table. The value for each panel is shown, and at the bottom of that column the composited value for the entire cross section is .0621. Table 6.c shows the equivalent n-value for the conveyance method to be 0.0506. It is important not to intermix n-values determined from different compositing methods.

c. Uncertainty due to Flow Distribution. The usual procedure to confirm the water surface profile model is to reconstitute observed high water marks from historical floods. However, in complex geometry and/or with several different roughness types across the section, an accurate reconstitution of the hydraulics requires flow distribution measurements, also.

That is, the levee grade depends on how much flow conveyance is eliminated by the levee structure. That calculation requires the correct distribution of flow across the section. Therefore water surface calculations are necessary but not sufficient to insure model confirmation for the design case.

For example, suppose a channel n-value was estimated as .03 and the flood plain n-value as .125. The water surface elevation was calculated, and subsequently it became known that the channel n-value was actually .038 and the overbank n-value .100. That results in a different flow distribution than the design case and the water depth would increase 4 percent, in the example cross section of figure 2.

d. Comment The examples in this paper are calculated for a single cross section, water discharge and Froude Number. They illustrate sensitivity and must not be used for an actual calculation. However, designs based entirely on handbook n-values or empirical formulas are not as reliable as those using n-values calculated from field data which was collected along the project reach. Therefore, a sensitivity analysis using a range of n-values will aid in determining how much uncertainty is present for a project.

TABLE 6. Water Surface Elevation Using the Brownlie Equation for Bed n-Value.

a. NORMAL DEPTH USING COMPOSITE PROPERTIES BY ALPHA METHOD.

**** N	Q CFS	WS ELEV FT	TOP WIDTH FT	COMPOSITE R FT	SLOPE ft/ft	COMPOSITE n-Value	VEL FPS	FROUDE NUMBER	SHEAR STRESS #/SF
**** 1	2300.	9.58	150.6	7.77	0.000800	0.0621	2.64	0.17	0.39

b. FLOW DISTRIBUTION BY ALPHA METHOD. Q = 2300.00

STATION	INC. %Q	area sqft	perm ft	r= a/p	Ks ft	n value	vel fps	tau #/sf	CMT
0.0									
	3.06	33.2	16.8	1.98	1.179	0.0312	2.11	0.10	RGD
50.0									
	25.74	437.0	75.1	5.82	624.9	0.1000	1.35	0.29	R S
125.0									
	7.10	34.3	4.5	7.67	1.000	0.0342	4.76	0.42	R S
129.0									
	51.31	239.4	25.0	9.58	4.563	0.0383	4.93	0.47	RLB
154.0									
	7.10	34.3	4.5	7.67	1.000	0.0342	4.76	0.42	R S
158.0									
	2.64	58.3	10.6	5.50	2384.	0.1250	1.04	0.29	R S
168.0									
	3.06	33.2	16.8	1.98	1.179	0.0312	2.11	0.10	RGD
218.0									
	100.00	869.9	153.2	7.77	18.59	0.0621	2.64	0.38	

c. EQUIVALENT HYDRAULIC PROPERTIES USING CONVEYANCE METHOD.

HYDRAULIC RADIUS ft	MANNING n-VALUE	.........SUBSECTION........... DISCHARGE cfs	AREA sqft	VELOCITY fps
5.68	0.0506	2300.00	869.86	2.64

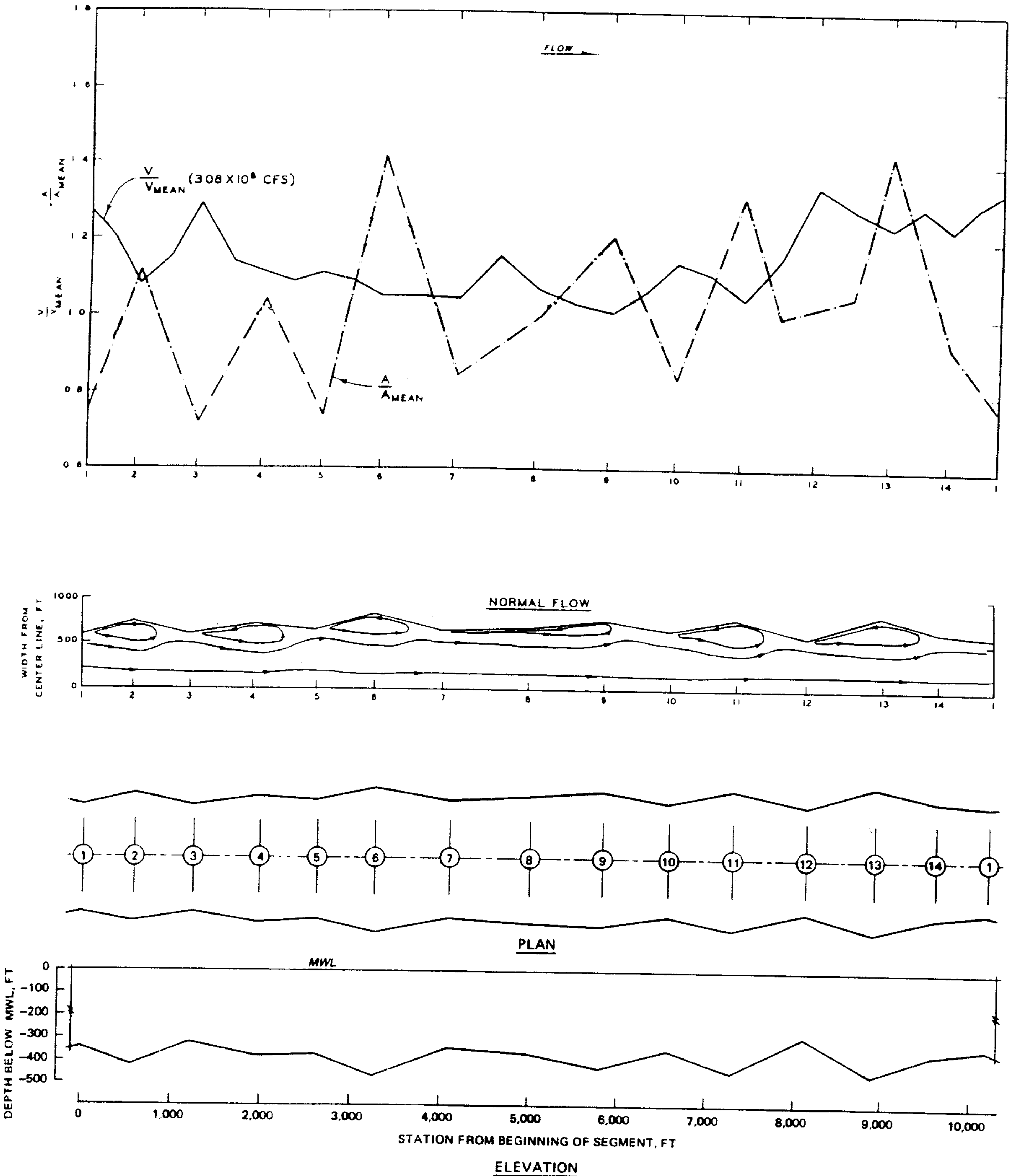


Figure 3. Flow in a highly irregular channel

e. Expansion and Contraction in a 1-D Model. Sager, McNair and Keulegan conducted a physical model study to determine Manning's n-value for a channel formed by nuclear explosives, (Sager, et al, 1961). Their model is shown in figure 3. Note the irregularity in the cross section shape, both the plan and elevation views. This is translated to the variations in cross sectional area shown at the top of the figure. However, the variation in average velocity, also shown at the top of the figure, is much less than that of the cross sectional area. Selected stream lines are shown on the "Normal Flow" view. They vary from rather straight near the center of the section to distorted to even reverse flows as one moves toward the side of the section.

The resulting general equation for predicting an effective Manning's n that could be used for channels with similar expansions and contractions is:

$$n_e = \sqrt{0.732n^2 + 0.000443\ R^{1/3}}$$

where

n is the Manning n-Value for no contractions and expansions

R is the geometric-average-hydraulic-radius of the channel with contractions and expansions

The application of the above technique to the test channel, which had a geometric-mean-hydraulic radius of 189-ft, gives n-values of 0.051 to .061 when the n for prismatic channel constructed in the same material varied from .01 to .04. This illustrates how strong the domination of contraction and expansion can be. It is probably significant that Dr. Keulegan chose not to use expansion and contraction coefficients!!!

7. Uncertainties Embedded in Forecasting the Future.

a. Water Diversions. Some flood protection projects include water diversions. It is difficult to design a diversion in a mobile boundary stream, but it is even more difficult to forecast the long term performance of one. Suffice it to say that in the general case, a diversion is an extremely complex problem in 3-dimensional hydraulics and one should not relegate its design to a 1-dimensional calculation without considerable justification.

b. Hydraulic Roughness Values.

(1) Seasonality. Floods which occur in the growing season show a significant increase in water elevation over those in the dormant season when a significant percentage of water flows over the flood plain.

(2) Tubeworms and barnacles. The corps built a concrete channel in Corte Madera Creek only to find that a marine critter called Tubeworms

were attracted to it. They create a substantial increase in the surface roughness in the zone below sea level. Rather than the usual Ks of .007-ft, WES estimated the zone with the tubeworms had a Ks of .08-ft (Copeland, 1989).

(3) Roughness from Gravel Moving in a Concrete Channel. In recent experiments at WES gravel movement was modeled along a hard bottom flume to determine how much the n-value would increase (Stonestreet, 1991). As long as it moved, the increase was only about 10 percent. That was the case for concentrations up to about 3000 ppm. When the concentration exceeded that, bed deposits started to form. That effect on n-value is very significant and requires a sedimentation investigation. Those tests are still in progress.

(4) Bed Form Roughness in Concrete Channels. After the Corte Madera Creek channel went into operation, sediment deposited over the smooth concrete bed in the downstream portion. A sedimentation study was conducted, after the fact, using HEC-6 (Copeland, 1989). He determined the channel n-value was .028 by using high water marks and the known water discharge. The calculated depth and gradation of bed deposits matched prototype values very nicely. This n-value is not suggested as a design value. It is presented to illustrate surprises that come from fixed bed hydraulics.

(5) Large Woody Debris. Large woody debris refers to downed trees and log jams. This is an condition that exists but its effect on the water surface profile during large floods is not well documented. It should be considered in connection with project maintenance requirements.

(6) Ice. Call Cold Regions Research and Engineering Laboratory.

(7) Wetlands. Measurements by the South Florida Water Management District in connection with the restoration of the Kissimmee River produced n-values of 1.0!! That coincided with flow depths below the top of the marsh vegetation. They chose to use 0.3 for the levee design calculations. This was judgement rather than experiment. Once flow depth exceeds the top of vegetation by 2 or 3 times the height, it seemed reasonable to reduce n-values.

(8) Marsh. Studies by a private consultant (Dr. Bob McArthur) for a flood at Kawanui Marsh, Hawaii, resulted in an n-value of 0.95. The process was like an ice jam. That is, there was a dense vine growing on the water surface. It was attached to the bed from place to place, but when the flood occurred it piled the vine into accordion-type folds. Subsequent measurements, on smaller floods, were used to develop the n-value.

(9) Sedimentation Processes. Sediment deposits in the channel are an obvious problem. These are not considered an uncertainty in water surface profile calculations. They should be addressed in a separate study. There will be uncertainties in the sedimentation studies in addition to those for the water surface calculation. Two distinct cases need to be evaluated: the long term sedimentation and sedimentation during the design flood hydrograph.

[a] Maintenance. The objective of the long term sedimentation study is primarily project maintenance. How does one know when to remove sediments? It is not reasonable to disturb a channel every year to restore it to the design cross section. Perhaps a decision can be based on some allowable rise in the water surface elevation. For example, if the water surface becomes as much as a foot above the design elevation for the levee design discharge, the deposits should be removed back to the constructed grade. Such maintenance is required to preserve the design capacity of a project. Should it be allowed in the freeboard?

[b] Reliability Reliability is another issue. It relates to the project's being able to pass the design water discharge hydrograph without overtopping the levees. There is no time for dredging. Often this means a sediment trap or some allowance for sediment deposition in the project channel design. Perhaps freeboard is a proper means. In this case, a sedimentation study is needed to identify zones of deposition and to quantify the rise in water surface during the design flood hydrograph.

Another consideration in forecasting the future is that no two 100-year floods will have the same sediment concentrations. There is usually a log-cycle of scatter in a water-sediment discharge relation. Perhaps the best estimate of the future sediment inflows is the average of that scatter of data, but the extremes can be permitted and handled by within the levee freeboard. Presently there are no reliable techniques for forecasting future sediment concentrations because it depends so strongly on man's activities and land use.

[c] Examples In the San Lorenzo River at Santa Cruze, CA, a sand deposit forms on the channel bed during normal flows (Copeland, 1986). There was a similar case on the Waimea River, Island of Kauai, Hawaii. During floods the deposit may be completely eroded. In any case they will degrade some because of the M2 backwater curve. Perhaps another reasonable use of freeboard is to allow the levees to contain that rise in water surface as the deposit is being eroded.

In the general case, one should not assume that sediment deposits will scour out of a channel during an extreme event. It depends on whether the incoming sediment discharge is less than transport capacity (Thomas, 1970).

In cases like Mount St Helens, where sedimentation was the key issue, we eventually accumulated a good data set. In the long term analyses we used the average through the scatter of data. In single event analysis, we multiplied that average by 3. That result was toward the upper limit, but still within, the scatter of data.

c. Base Level Change. Base level refers to what ever controls the water surface elevation at the downstream end of the project. It is important to forecast future behavior of that gage. Such changes may be minor over a year or two whereas the trend over 50 years would be significant. This could affect the calculated water surface profiles along the entire project.

Appendix A. REFERENCES

1. Brownlie, William R. 1983 (Jul). "Flow Depth in Sand-Bed Channels," Journal of the Hydraulic Engineering, American Society of Civil Engineers, Vol 109, No 7, pp 959-990.

2. Barnes, Harry H., Jr. 1977. "Roughness Characteristics of Natural Channels," Geological Survey Water-Supply Paper 1849, United States Government Printing Office, Washington, D.C.

3. Chow, Ven Te. 1959. "Open-Channel Hydraulics," McGraw-Hill, New York.

4. Copeland, Ronald R. 1989 (Apr). "Corte Madera Creek Numerical Sedimentation Study." TR HL-89-6, Hydraulics Laboratory, US Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

5. Copeland, Ronald R. 1986. "San Lorenzo River Sedimentation Study." Numerical Model Investigation, Hydraulics Laboratory, US Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

6. Copeland, Ronald R. 1986. "Waimea River Sedimentation Study, Kauai, Hawaii." Numerical Model Investigation, Hydraulics Laboratory, US Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

7. HEC. 1986 (Dec). "Accuracy of Computed Water Surface Profiles," Research Document 26. Prepared by The Hydrologic Engineering Center for The Federal Highway Administration. Davis, CA.

8. HEC-2, Generalized Computer Program. 1990 (SEP). "Water Surface Profiles," Users Manual, the Hydrologic Engineering Center, US Army Corps of Engineers, Davis, Calif.

9. James, Maurice and Bobby J. Brown. 1977 (Jun). "Geometric Parameters that Influence Floodplain Flow." US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

10. King, Horace W. and Ernest F. Brater. 1963. "Handbook of Hydraulics," McGraw-Hill, N Y.

11. Limerinos, J. T. 1970. "Determination of the Manning Coefficient from Measured Bed Roughness in Natural Channels," Geological Survey Water-Supply Paper 1898-B, Prepared in cooperation with the California Department of Water Resources, US Government Printing Office, Washington DC.

12. Schwab, Glenn O., Richard K. Frevert, Talcott W. Edminster, and Kenneth K. Barnes. 1966, (Second Edition). "Soil and Water Conservation Engineering," John Wiley & Sons, Inc., New York, NY.

13. Sager, Richard A., E. Clark McNair, Jr., and Garbis H. Keulegan. 1969 (May), "Hydraulic Characteristics of Nuclear Excavated Channels." Miscellaneous Paper H-69-5, US Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

14. Thomas, William A. 1970 (Apr). "A Digital Model for Simulating Sediment Movement in a Shallow Reservoir." Proceedings of a Seminar on Sediment Transport in Rivers and Reservoirs. The Hydrologic Engineering Center, Davis, CA.

15. Thomas, William A. 1975. Water Surface Profiles, Vol 6 of 12 Vol set of Hydrologic Engineering Methods for Water Resource Development, distributed worldwide by Corps of Engineers as contribution to the International Hydrologic Decade.

16. Thomas, William A., Ronald R. Copeland, Nolan K. Raphelt, and Dinah N. McComas. (1991) DRAFT. "Hydraulic Design Package for Flood Control Channels" (SAM), User Manual. US Army Engineer Waterways Experiment Station, Vicksburg, MS.

17. Stonestreet, Scott E., Ronald R. Copeland and Darla C. McVan. 1991 (Aug). "Bed Load Roughness in Supercritical Flow," Hydraulic Specialty Conference, American Society of Civil Engineers, Nashville TN.

18. U.S. Department of Transportation(USDT). Arcement, George J., Jr and Verne R. Schneider. 1989. "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains," Water-Supply Paper 2339, United States Geological Survey, Department of the Interior, Books and Open-File Reports Section, Federal Center, Box 25425, Denver, CO 80225.

19. US Army Corps of Engineers(USACE). 1970. "Engineering and Design - Hydraulic Design of Flood Control Channels," EM 1110-2-1601, US Army Corps of Engineers, Washington, DC.

INTERNATIONAL SURVEY OF LEVEE FREEBOARD DESIGN PROCEDURES

by

Robert C. MacArthur[1] and Teresa Bowen MacArthur[2]

1. INTRODUCTION

1.1 Study Purpose

The Corps of Engineers is responsible for the design and construction of flood control projects throughout the United States. As part of their responsibility, the Corps develops uniform design standards and guidance for use by Corps offices and other State and Federal agencies. The Corps is currently reexamining the way freeboard is used in their flood control projects. As part of the Corps' reexamination, Water Engineering & Technology, Inc. (WET) was asked to conduct a survey of freeboard design practices used in other countries.

An international survey was conducted to determine the analytical methods and design procedures used by other countries to establish freeboard heights for stream channels and flood control levees. A detailed questionnaire was sent to more than 200 individuals, universities and water control agencies around the world. The questionnaire requested information regarding how the design flood and top elevation of flood control levees are determined, if freeboard is used, what computational methods are used to quantify these parameters, and if risk, uncertainty or environmental issues are considered during levee design. A total of 205 questionnaires were sent to 45 countries. Thirty five meaningful responses were received from 18 different countries. This paper summarizes the results from the international freeboard survey and discusses the different freeboard policies and design procedures according to individual country and specific methods used.

1.2 Limitations

The survey was limited to a relatively small population of water ministries because of time and monetary constraints. Its intent was to determine if other countries are equally concerned about flood control and levee design as we are in the United States. Therefore, the results from this preliminary study may have missed a few key flood control agencies or water ministries and may not present all of the different methods presently being used internationally, however, it does present an example of the diverse nature of freeboard design procedures and underscores the fact that the United States is not the only country concerned about safe levee design and flood control adequacy.

---

[1]Principal Civil Engineer, Water Engineering & Technology, Inc., 621 Fourth Street, Suite No. 1, Davis, CA 95616, USA

[2]Senior Water Resources Engineer, Water Engineering & Technology, Inc., 621 Fourth Street, Suite No. 1, Davis, CA 95616, USA

The following information was requested by the questionnaire: (1) how is the top of levee elevation determined, (2) is freeboard used, (3) how are the design discharge and frequency determined, (4) are rural and urban areas handled differently, (5) are there uniform national standards, (6) are risk and uncertainty included in the determination of the levee elevation, (7) how are the water surface profiles determined, (8) are sediment transport or mobile boundary problems considered in the design elevation, (9) are environmental concerns considered, and (10) what are your country's most serious levee design problems?

2. STUDY APPROACH

A thorough literature search to determine if information on international freeboard practices is readily available in published papers, reports or conference proceedings was conducted as Phase I of the study. An exhaustive literature search and review was performed using the University of California interlibrary computer survey system and the University of Colorado research library system. No significant information was found regarding international freeboard practices in readily available national or international publications. This made the tasks of the second phase of the investigation more difficult because there was little solid information to begin with.

Phase II of the study was directed towards assembling names, addresses and points of contact in each of the individual countries targeted by the survey. These were gathered from a variety of sources, including: university professors; international water ministries, flood control agencies, environmental and engineering organizations, the membership roster for the World Meteorological Organization, colleagues in the profession, and personal contacts of the authors.

Phase III of the investigation involved mailing, FAXing and, in many cases hand delivering the questionnaires to contacts all over the world. The first mailings were made in early May, however, mailings and FAXing of the questionnaires continued through July as more addresses and potential sources of information were obtained. Two hundred and five questionnaires were sent to forty five different countries.

Phase IV included compiling and reporting of the survey results. This was not easy because each response was received in its own unique format and level of detail. Some questionnaires were filled out thoroughly and returned with additional supporting literature. Those responses were the most valuable because they were fairly complete. Other questionnaires were only partially completed with little or no back up information and very brief answers to the questionnaire. Each response, however, contained significant information and every response was regarded as important and was summarized and logged onto a spread sheet. Unopened questionnaires that were returned with incorrect addresses were also noted.

There were many difficulties in interpreting the information that was received. Much of the literature received was written in Japanese, Chinese, German, Swedish or other foreign languages and needed translation. Several of the most promising documents were roughly translated by personal friends of the authors in order to provide critical information to the survey in a timely manner. Many materials still require translation. Complete translation and compilation of the information should be done as soon as possible.

WET began mailing questionnaires in early May even though they did not have a contract

WET began mailing questionnaires in early May even though they did not have a contract until June. It is difficult to get foreign agencies to respond to international requests for information in less than two months time. Therefore, several responses were received late, or were prepared in a hurry with less detail than may have occurred with sufficient time. There was also insufficient time for WET to correspond back to those responding for further information or clarification of the materials sent. Future international surveys should allow 8 to 12 months for the collection of the information and completion of the survey report. The authors feel, however, that this survey was indeed very successful and has provided a great deal of valuable information. It has also opened new channels of communication with 18 different foreign countries who are all very interested in continuing to share technical information and data with the Corps of Engineers. Results from the international survey on freeboard design procedures are presented in the following sections.

3. SUMMARY OF COUNTRIES SURVEYED AND RESPONSES

Table 1 presents a list of the countries and organizations that returned completed questionnaires and/or other published materials pertaining to freeboard design procedures. A total of 205 different sources were queried in 45 different countries. More than 35 responses were received from 18 different counties. Additional responses were still arriving while this paper was being prepared. Tables 2 and 3 summarize the responses that have been received from each responding country according to the questions asked in the survey. Information presented in these tables reflect actual responses from the materials returned with the surveys. The authors have taken care not to elaborate or infer additional information other than actually reported.

Sections 3.1, 3.2 and 3.3 present survey results tabulated as follows: Procedures for Determining Design Discharge and Water Surface Elevations, Determination of Freeboard, and Summary of Freeboard Design Procedures, respectively. Subsequent sections deal with International Problems and Research and Needs, Conclusions, and Recommendations for Further Study.

Table 1. List of Countries Responding to the Survey

	COUNTRY	ORGANIZATION	CITY/PROVINCE
1	AUSTRALIA	Gutteridge Haskins and Davey Pty Ltd	New South Wales
2	AUSTRALIA	Gutteridge Haskins and Davey Pty Ltd	Western Australia
3	AUSTRALIA	Water Authority of Western Australia	Western Australia
4	AUSTRALIA	Gutteridge Haskins and Davey Pty Ltd	Victoria
5	BRAZIL	Enge-Rio, Engenharia e Consultoria S.A.	Rio de Janeiro
6	CANADA	Northwest Hydraulics Consultants	Edmonton
7	CANADA	Hydro-Quebec, Div. Ouvrages hydrauliques, Service Hydraulique	Quebec
8	CANADA	Water Planning and Management Branch, Environment Canada	British Colombia
9	CANADA	Ontario Ministry of Natural Resources, Engineering Branch	Ontario
10	CANADA	Upper Thames River Conservation Authority	Ontario
11	CANADA	Manitoba Water Resources Branch Environment Canada	Manitoba
12	CANADA	Environment Canada, Inland Waters Directorate	Ontario
13	CHINA	Inst. of Mountain Disasters & Environment, Chinese Academy of Sciences	Sichuan
14	CHINA	Ocean & Coastal Engineering Research Institute	Nanjing
15	CHINA	Yellow River Conservancy Commission	Zengzhou
16	CHINA	Nanjing Inst. of Hydrology & Water Res., Ministry of Water Resources	Nanjing
17	ENGLAND	A. T. Pepper Engineering Consultant	Surrey
18	ENGLAND	Institute of Hydrology Maclean Bldg	Oxfordshire
19	FRANCE	CACG Chemin de l'Alette	Tarbes Cedex
20	GERMANY	Institut fur Hydrologie und Wasserwirtschaft Universitat Karlsruhe (TH)	Karlsruhe
21	GERMANY	Oberste Baubehorde im Bayerischen Staatministerium des Innern	Munich
22	HUNGARY	Research Centre for Water Resources Development "VITUKI"	Kvassay
23	INDIA	Central Water & Power Commission Ministry of Irrigation and Power	New Delhi
24	IRELAND	Office of Public Works	Dublin
25	JAPAN	Department of Civil Engineering, Kobe University	Kobe
26	NETHERLANDS	Int. Institute of Hydraulic Engineering (IHE)	Delft
27	NEW ZEALAND	Auckland Regional Council, Regional House	Auckland
28	NORWAY	Norwegian Water Resources & Energy Administration	Oslo
29	NORWAY	Univ. of Trondheim, Norwegian Inst. of Tech., Div. of Hydraulic Engineering	Trondheim
30	POLAND	Institut of Hydroengineering Polish Academy of Sciences	Koscierska
31	SWEDEN	Swedish Meteorological and Hydrological Institute	Norrkoping
32	SWITZERLAND	Swiss Federal Office for Water Management	Bern
33	SWITZERLAND	Lab. of Hyd., Hydrol.&Glaciology of the Swiss Federal Inst of Technology	Zurich
34	TAIWAN	Water Resources Plng Commission, Ministry of Economic Affairs	Taipei
35	TAIWAN	National Chung Hsing Univ, Graduate Inst of Soil & Water Conservation	Taichung

Table 2. Summary of Survey Responses Pertaining to International Freeboard Design Criteria

	COUNTRY	TOP OF LEVEE DETERMINED?	FREEBOARD FIXED VALUE?	COMPUTATIONAL METHOD FOR VALUE OF FREEBOARD	FACTORS INCLUDED IN FREEBOARD
1	AUSTRALIA	DFWS + FB	no	.3m + uncertainties + afflux due to works	Waves, "hidden safety factor," risk is applied to design Q not FB
2	AUSTRALIA	DFWS + FB	no	Varies (.3m-.5m)	Experience, land value, flooding history
3	AUSTRALIA	100-yr + .5m	0.5m	Not computed	Fixed (urban = 0.5m, rural = .15-.3m)
4	AUSTRALIA	100-yr + FB	varies	Experience, land value, flood history	Q, waves, inaccuracy in survey & fld level est., risk is included in FB
5	BRAZIL	max wsl + wave ht + FB	no	0.5-1.5m	Levee characteristics and risks to d/s areas
6	CANADA	varies	no	Greater of 200-yr: (daily max+1' or inst+2')	Varies according to Province and flow
7	CANADA	max ws elev	3-4m	Wind + wave runup	Wind effects only: wind setup and wave runup
8	CANADA	DFWS + FB	2 ft	See text	Fixed @ 2' but includes inaccuracies of DFWS, waves, levee roads
9	CANADA	Design Flood Criteria	0.2-0.5m	Not computed	No other factors, FB is fixed at 0.2 - 0.5m above DFWS
10	CANADA	Regulatory flood of Auth.	no	Not computed	Variable, depends on levee height and needs
11	CANADA	100-yr + wave + wind	min 0.6m	Greater of wave + wind or .6m	Wave runup, wind setup or 0.6m which ever is greatest
12	CANADA	DFWS + FB	yes	Varies	Varies throughout Canada; fixed or includes wind and wave effects
13	CHINA	DFWS + FB	.25-.8m	Varies	Wind waves, high sediment loads, debris flows
14	CHINA	DFWS + FB	no	Wave amplitude + wave setup + FB safety	Wave runup, wind setup, factor of safety = f(potential damage), FB=.3-.7m
15	CHINA	DFWS + FB	no	Wave + fixed value(1.2m 1st grade structures)	Waves and factor of safety, risk is accounted for in Factor of Safety
16	CHINA	DFWS + FB	no	Windwave runup + deposition pot. + safety	Wave runup, sediment deposition/erosion, factor of safety
17	ENGLAND	DFWS + FB	no	Wave + settlement + accuracy + stability	Wave action, pot. settlement, overtopping survival, acc of computed DFWS
18	ENGLAND	DFWS + FB	min 3m	Not computed	Margin of safety added as protection against loss of life
19	FRANCE	DFWS + FB + wave	3.5-4m	Wave + absolute FB	0.5m for waves, 0.5m for safety factor, other factors (total 3.5-4m)
20	GERMANY	DFWS + FB	.1-2m	Varies	Wave,wind,ice,settlement,H&H uncertainties,meas inacc,floating debris,sed.
21	GERMANY	DFWS + final settle.+FB	min .8m	Ht of levee and d/s risk determine FB	Minimum .8m, greater for areas with high damage and loss potential
22	HUNGARY	DFWS + 1m (or 1.5m)	1.0-1.5m	Experience	Wave action, potential erosion of the upper portion of the levee
23	INDIA	DFWS + FB	1.5-1.8m	Q<3000cms FB=1.5m, Q>3000cms FB=1.8m	Waves, wind setup, sediment
24	IRELAND	DFWS + FB	no	Levee ht & material, d/s property, fetch	Waves, subsidence pot., possible damage and value of d/s property
25	JAPAN	DFWS + FB	.6-2.0m	Varies according to Q and top width	Waves, wind, social and psychological perceptions (factor of safety)
26	NETHERLANDS	DFWS+FB+sea level rise	min .5m	Varies with levee purpose, Q	Waves and pot. overtopping flow rate, sophisticated opt. of risk & cost
27	NEW ZEALAND	hydraulic calcs	no	Uncertainties	Uncertainties in hydrology & hydraulics, n values, variable amount
28	NORWAY	Flood Frequency	30-50cm	Not normally computed	FB = 30-50cm, varies with value of property protected
29	NORWAY	DFWS + other	.5m,adj.	Not a computed value	Fixed at 0.5m, adjusted for flow and ice conditions, risk in design Q
30	POLAND	DFWS + FB (1000 yr prop)	no	Varies depending on d/s risk	FB = 0.5-1.2m, depending on danger for loss of life, 4 danger categories
31	SWEDEN	DFWS + FB	no	Varies (translation needed)	No details provided
32	SWITZERLAND	DFWS + FB	min .5m	FB proportional to $v^2/2g$ (avg.5-1.0m)	Risk and floating debris
33	SWITZERLAND	DFWS + FB (100yr or HiQ)	no	Not computed	Depends on river type and danger, possibility of debris flows,ice
34	TAIWAN	DFWS + FB (50-200yr)	1-1.5m	No Response	Fixed: 1m for concrete structures, 1.5m for earthen embankments
35	TAIWAN	DFWS + FB	no	Computed value	Accounts for floating debris and debris flows, function of d/s danger

DFWS = Design flood water surface
FB = Freeboard

Table 3. Summary of Other Considerations Often Included In Freeboard

	COUNTRY	MIN. LEVEE HEIGHT?	DESIGN FLOOD DETERMINED BY	RURAL VS. URBAN STDS?	RISK AND UNCERTAINTY?	WSP COMPUTATIONAL METHODS	SEDIMENT/EROSION CONSIDERED?
1	AUSTRALIA	no	Varies	yes	yes	HEC-2,CELL2D,RORB,RAFTS	where needed
2	AUSTRALIA	2m	Hydrological models and freq analysis	no	in design Q	HEC-2, CELL2D, MIKE11	not generally
3	AUSTRALIA	no	Gaging sta, models	r .15-.3m, u 0.5m	no	SS,1-D, Manning's eq	no
4	AUSTRALIA	no	Natl guidelines or regional freq analysis	yes	yes	SS, HEC-2, MIKE11	no
5	BRAZIL	no	Natl stds according to risk (see Tables)	yes	yes	Std step	yes
6	CANADA	no	freq analysis, ref. included	no	sometimes	HEC-2, unsteady	no
7	CANADA	no	flood freq analysis	no	yes	HEC-2, ONED	no
8	CANADA	no	highest Q on record	no	yes	SS, hist. highwater	where applicable
9	CANADA	no	synthetic unit hydrographs, reg freq analysis	no	yes	SS, HEC-2	not a problem
10	CANADA	no	ann. flood series stat., reg. analysis	no	yes	HEC-2	no
11	CANADA	no	100-yr min required	no	inc in FB	HEC-2	erosion considered
12	CANADA	no	flood frequency	no	yes	SS, HEC-2	yes
13	CHINA	1m	frequency statistics	yes	yes:wave/debris	Std stp, computer models	yes
14	CHINA	NR	statistical Log Pearson Type III Dist.	yes	beginning to	1-D or 2-D	depends
15	CHINA	NR	max flood discharge	no	yes	NR	yes
16	CHINA	no	statistics, historical, regional	yes	yes	SS, HEC-2,1-D, 2-D	yes
17	ENGLAND	no	varies	yes	yes	SS, 1-D, HEC-2	occasionally
18	ENGLAND	NR	flow frequency, historical data	yes	yes	NR	NR
19	FRANCE	NR	"methode du Gradex"	NR	yes	NR	NR
20	GERMANY	no	flow frequency analysis, hist. data	yes	yes	NR	yes
21	GERMANY	no	statistical eval of ppt, 100-yr event	no	yes	SS, 1-D, others	scour
22	HUNGARY	yes	1% prob. of ann max floods at all gaging stas	FB yes, Design Q no	yes	SS, 1-D or 2-D	no
23	INDIA	no	Flood frequency analysis	yes	no	SS, 1-D, HEC-2	yes
24	IRELAND	no	hist. fld levels, unit hydrograph	yes	NR	SS, std stp	yes
25	JAPAN	.6m	Flow freq. and hist data	yes	yes	NR	yes
26	NETHERLANDS	no	Sophisticated models	yes	yes	1-D, 2-D, Steady&Unsteady	yes
27	NEW ZEALAND	no	hydrologic data: rainfall, stage/time	yes	yes	1-D, Unsteady flow	included
28	NORWAY	yes	freq from observations &normal log procedures	yes	yes	SS, 1-D	yes
29	NORWAY	no	flood frequency analyses	yes	yes	Std step, 1-D, SS (HEC-2)	yes
30	POLAND	no	0.1% probability determined by DEBSKI	yes	yes	1-D models	no
31	SWEDEN	no	Probabilistic models, hist data	yes	yes	1-D, 2-D, Steady&Unsteady	yes
32	SWITZERLAND	no	100-year flood or highest observed	yes	not normally	Std step, 1-D, SS	yes
33	SWITZERLAND	no	statistical methods, usually 100-year is DF	yes	yes	SS	yes
34	TAIWAN	no	Log Pearson Type III	yes, natl criteria	no	SS(HEC-2);UNET,FESWMS-2DH	no
35	TAIWAN	no	NR	yes, natl criteria	yes	NR	yes

NR = no response
SS = steady state
std stp = standard step method

3.1 Procedures for Determining Design Discharge and Water Surface Elevation

3.1.1 Freeboard Freeboard refers to the vertical distance between either the top of the channel or the top of the levee and the water surface which prevails when the channel is carrying the design flow at normal depth. The purpose of freeboard is to prevent overtopping of either the channel or levee caused by (1) waves, (2) wind setup, (3) tidal action, (4) hydraulic jumps, (4) superelevation of the water surface as flow rounds curves at high velocity, (5) the occurrence of greater than design depths of flow caused by channel sedimentation or an increased friction coefficient (due to bed forms and/or the growth of vegetation), (6) effects of floating debris or ice, (7) settlement of the levee or channel banks, and (8) hydrologic and hydraulic uncertainties. Figure 1 presents a diagram showing many of the factors often included in freeboard according to the results from the survey. As shown in the diagram, the design flood water surface is determined first and fixed or variable freeboard height is usually added to that elevation.

Components of Freeboard

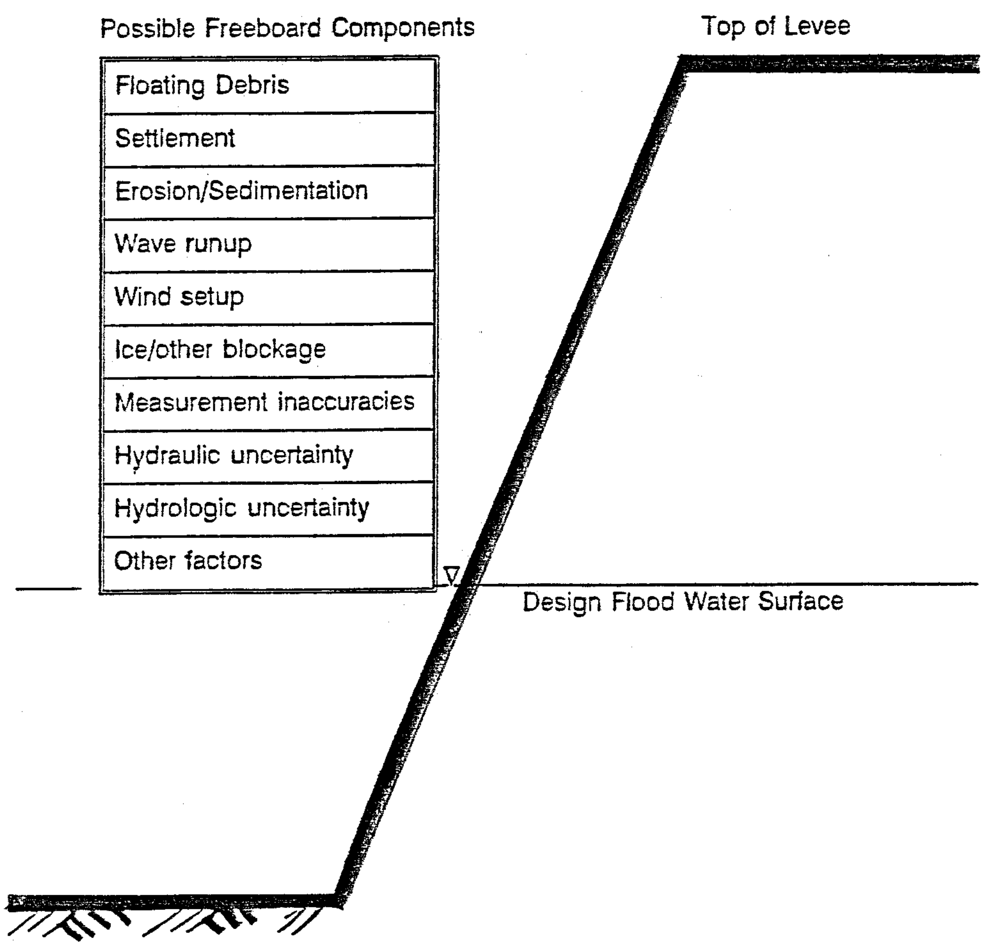


Figure 1. Components of Freeboard

3.1.2 Design Flood Discharge Most countries responding to the survey indicated that they use traditional statistical and empirical methods for determining the design flood discharge. The expected peak flow for the 1 % chance flood is typically calculated from the following procedures: (1) statistical analysis of past flood events using Log-Pearson Type III or other acceptable methods, (2) hydrologic methods of analysis and synthesis of rainfall runoff relationships to estimate the likely flood flow resulting from critical rainfall events, and (3) other accepted empirical methods which may incorporate historical, geomorphologic and other similar information. Examination of Table 3 shows that several countries, such as Germany, China, Switzerland, The Netherlands, Canada and others use very sophisticated hydrologic procedures to develop the design discharge. In Ontario, Canada, there are four Regulatory Floods (Design Floods) for designating flood plains. Three of the four are based on observed data from past events, the fourth is the frequency based 1 % chance (100 yr) flood. Selection of the magnitude of the Regulatory Floods in a particular area of the Province is largely dependent upon the susceptibility of that area to tropical storms, thunder storms, snowmelt, rainfall or a combination of these meteorological events.

Most countries use the 1 % chance (100 yr) event for their basis of design. However, in areas where there is a great threat for loss of life if there is a failure of the flood control facilities, much rarer design events are used. For example, in China along some sections of the Yellow River where millions of people could be lost if the levees were to fail, their design discharges may be as great as the 0.2 % to 0.05 % (500 yr to 2000 yr) event. In the Netherlands where the entire country is primarily at or below sea level, they also use very high standards for protection. In Central Holland, sea dikes are designed for the 0.001 % (10,000 yr) storm surge event; river levees are designed to carry the 0.008 % (1250 yr) flood event. Poland uses the 0.1 % (1000 yr) deign event for their levee systems. Several countries also include other factors such as hydrologic and hydraulic uncertainty, measurement inaccuracies and risk in their design discharge and design water surface elevation computations. A slightly different approach is taken in Sweden, where a special Flood Control Committee recently concluded that "hydrological modeling techniques may be more reliable and consistent than frequency analysis methods for the assessment of design floods." The main reasons for their conclusion are attributed to the great uncertainty when extrapolating probability distribution functions and that the combined effect of critical flood generating factors can now be accurately estimated due to the progress of hydrological modeling.

The storms of 1987 in the Swiss Alps killed eight people and caused significant damages. The consequences of these flood events lead to the development of the following suggested criteria for design events in Switzerland:

* The level of protection depends on the importance of the object or area being protected.
* Flooding characteristics have to be distinguished according to their potential threat to life and damage potential. The damage will be different if only submersion occurs; submersion with high flow velocities is more dangerous and causes more damage; and extreme flooding conditions that cause extensive bank erosion and massive debris flows require special consideration.
* The possibility (the risk) of higher flow rates than the standard design flood needs to be considered, especially when structural flood control measures are planned.
* Passive protection measures (mainly land use planning alternatives) are preferable to active (structural) protection measures against these higher discharges.
* Flood protection measures should not allow the transfer of the flood risk into other zones outside the project area.

Based on the Swiss experiences during the 1987 floods, Table 4 presents their new design guidelines. This is a good example of regional design methods being applied in Europe.

Table 4. Proposed Design Discharge Criteria for Switzerland

Type of Land Use Design Q:	Q_{10} Q_{50} Q_{100} Q_{1000} PMF
Forest, pasture	/ / = = =/==========================/
Agricultural land	/ /= = = = = /===================/
Isolated housing, local infrastructure	/ /= = = = /===================/
Trade and industry centers	/ /= = = /===================/
Major settlements, large industrial complexes	/ /= = =/===========/
National infrastructure	/ /= = = = = = = =/===========/
Objects with high danger and risk	individual design discharge

/ / no damage should occur
/= = = / small damages possible, low flooding likely
/======/ severe erosion and deposition expected, greater threat of damages

<u>3.1.3 Design Water Surface Elevation</u> Table 3 summarizes the procedures used by various countries to compute the design water surface elevation. Steady state, 1-dimensional, fixed bed, back water methods (such as standard step and HEC-2) are the standard procedures used most often to compute the design water surface elevation from the design discharge. Germany, England, Canada, Switzerland, The Netherlands, Taiwan and China may also apply advanced 1-D and 2-D unsteady flow models when necessary to account for complex hydraulic or geometric effects. Some countries (China, Switzerland, Taiwan, India, Canada) account for mobile boundary effects (sediment transport and the potential for hyperconcentrated flows and debris flows) under special circumstances.

3.2 Determination of Freeboard

Tables 2 and 3 summarize the various methods used in different countries to determine freeboard and some of the unique factors that are occasionally included in the freeboard height.

<u>Australia</u>. In Western Australia, freeboard is used on all levees. In urban areas, a fixed value of 0.5m above the 100-year flood level is generally used. In rural areas, freeboard is generally 0.15 to 0.30m above the design flood. In Victoria, freeboard is also used on all levees and varies with flow rate, wave action, inaccuracies of survey data and flood level estimates. The freeboard value is the sum of:

(1) minimum 300mm
(2) allowance for uncertainties (0-400mm)
(3) allowance for "afflux due to works" (50-300mm)

In New South Wales, freeboard is selected as an arbitrary value, taking into account wave action and fetch. Freeboard may not be theoretically determined and may incorporate a "hidden" safety factor. Generally a freeboard allowance is made for levees in excess of 2m.

Brazil. In Brazil, the top of levee elevation is determined by the normal maximum water level plus the wave height plus freeboard. The wave height is determined by the Beach Erosion Board Method (Saville, 1962). The freeboard above the height of the wave varies from 0.5m to 1.5m depending on the characteristics of the levee and the safety risks to downstream areas. Freeboard is used on levees of all heights.

Canada. In some Canadian provinces, freeboard is a fixed value, in others, computed factors such as wave runup and wind setup are taken into account. There is no minimum levee height for using freeboard. In Quebec, for preliminary studies, freeboard is set as a fixed value of 3-4m. For major or more advanced studies, freeboard is calculated. It is equal to the wind setup plus the wave runup. It takes into account the 1/1000 year wind, the effective fetch length and the reservoir depth. In Ontario, freeboard is used as warranted, generally it is a fixed value between 0.2 and 0.5m above the design flood level. In British Columbia, freeboard is a minimum value of 0.6m. Freeboard is determined by taking into account the inaccuracy of design flood water level, wave action and trafficable surface on top of levee at design flood condition. In Manitoba, the minimum freeboard is either 0.6m or the wave approach plus wind set up, whichever is greater. Most of the Canadian design criteria are presented in "Hydrology of Floods in Canada: A Guide to Planning Design," by the National Research Council of Canada, 1989.

China. Freeboard is used on all levees in China. It is based on flow, land use, potential hazards (downstream safety) and practical experiences as well as engineering judgement. To compute freeboard two methods were presented:

Method 1:

$$\text{Freeboard} = h_b + \Delta h$$

where h_b represents the height of water surface on the sloping surface of levee due to wave impingement and Δh represents a factor of safety:

$$h_b = 3.2K * 2h * tg\,\Theta$$

where K= roughness factor of the sloping surface (for earth material K=1, for stony masonry K = 0.75 to 0.80)
$tg\Theta$ = tangent of the angle of inclination of the sloping surface facing the water
2h = wave height in meters which may be determined by:

$$2h = 0.0208\ V^{5/4}\ L^{1/3}$$

where V = wind speed in m/s
L = water surface length in direction of wind in km

The safety factor, Δh, is determined according to Chinese design standards for earth dams (for first grade structures, Δh = 1.2m).

Method 2:

$$\text{Freeboard} = H1 + H2 + H_c$$

where $H1$ = wave amplitude = 0.017*(V**1.25)*(D**0.33)
V = wind velocity (m/s)
D = fetch length (km)
$H2$ = wave setup = 3.1416*H1*H1/L1
$L1$ = 10.4*(H1**0.8)
H_c = factor of safety determined by Table 5

Table 5. Freeboard Factor of Safety Used in China

CLASS OF DAM/FLOOD STRUCTURE	1	2	3
DESIGN FLOOD	0.7 m	0.5 m	0.4 m
VERIFICATION FLOOD	0.5 m	0.4 m	0.3 m

England. Freeboard is the summation of a number of empirical values, which relate to: potential for wave action, settlement, ability to withstand overtopping, and the accuracy of computed flood levels. There is no minimum levee height for freeboard; however no freeboard is added where existing ground levels coincide with the flood level. Hence, when levees run into natural ground, the natural ground is lower and will overtop first, reducing the risk of levee failure by overtopping. In the Yorkshire Region, the minimum freeboard standard is 300mm.

France. A procedure has been developed to optimize design floods. This optimization results in a freeboard ranging from between 3.5 and 4m high.

Germany. Freeboard is always applied in Germany. The lowest allowable value is 0.8m and can go up to 1.5m to protect populated areas. Germany uses a variety of very sophisticated methods for computing the design discharge, water surface elevations and freeboard.

Hungary. A fixed value of 1.0m or 1.5m is added to the design flood water surface elevation. This value is not computed, but determined by experience, knowing the possible wave heights and the "usual degradation of the upper 0.5m layer of the levee. "Freeboard is not used for levees of minor importance (classified as Second Order)".

India. "Structural Analysis of Flood Management" by N.V.V. Char, Director FCCDte states: "In earthen embankments along rivers carrying a design discharge up to $3000m^3/s$, a freeboard of 1.5m above the design HFL shall be provided. In earthen embankments along rivers carrying more than $3000m^3/s$ design discharges, a freeboard of 1.8m over the design HFL shall be provided. This shall also be checked for ensuring a minimum of about 1m freeboard over the design HFL, corresponding to the 100 year return period."

A similar regulation is given for levee top width: "Earthen embankments along rivers carrying design discharge up to 3000m^3/sec shall have top widths of 5m. In the case of protective embankments along major channels carrying design discharges above 3000m^3/sec, the top width shall generally be 5.5m. Turning platforms, 15m to 30m long and 3m wide with side slopes of 1:1.5 shall be provided along the country side slopes of the embankment at every km."

Ireland. Freeboard is used on all channels, even those not embanked. It is basically to allow for errors in the estimation of flood discharge and water surface profile values. The value of freeboard is dependent on embankment height and material (likelihood of subsidence), value and vulnerability of property protected, and the degree of exposure to waves (fetch) on the water side of the embankment. Depending on these factors, freeboard varies from about 1 foot to 3 feet.

Japan. In Japan, National Regulations govern levee design. The height of levees is determined by adding freeboard to the design water surface elevation. Freeboard is added as a factor of safety to take into account the relative structural integrity of levees (how and what they are made of), wind and wave action, settling, and for social/aesthetic reasons for maintaining uniform levee heights and widths in highly urbanized areas. The minimum freeboard to be applied is 0.6m. The estimation of freeboard is based on design discharge according to Table 6. The exception to this rule is if the existing ground level is higher than the design water surface elevation, then the minimum freeboard of 0.6m may be added.

Table 6. Discharge vs. Freeboard (Japan)

Discharge (m^3/s)	Freeboard (m)
$Q < 200$	0.6
$200 < Q < 500$	0.8
$500 < Q < 2000$	1.0
$2000 < Q < 5000$	1.2
$5000 < Q < 10000$	1.5
$10000 < Q$	2.0

In addition, there are rules governing the levee height at the transition between tributaries and the main stem. The same levee height and width must be maintained where there are no controls. Where a control structure prevents reverse flow, the levee can be lowered on the tributary. In the safest case, where the existing ground elevation is higher than the design water surface elevation, 0.6m may be used.

Similarly, levee top width is regulated based on the design Q according to Table 7. If the height of the levee is less than 0.6m, then this rule need not be followed. If the ground level is greater than the design water level, then the minimum top width is 2m. For psychological reasons (social confidence in the level of protection), uniformity in both levee height and top width is required in levee designs. Where a tributary enters the main stem, transitions in levee dimensions must be gradual.

Table 7. Top Width of Levee vs. Discharge (Japan)

Discharge (m^3/s)	Levee Top Width (m)
$Q < 500$	3
$500 < Q < 2000$	4
$2000 < Q < 5000$	5
$5000 < Q < 10000$	6
$10000 < Q$	7

Netherlands. The Netherlands uses very detailed and sophisticated methods to compute the heights and geometry of levees. Scientific analyses of levee design has been ongoing since 1953. In general, freeboard is always used for river and coastal levees and is a minimum of 0.5m.

In Central Holland, sea dikes are designed for the 0.001 % (10,000 yr) storm surge event; river levees are designed to carry the 0.008 % (1250 yr) flood event.

For river dikes, an overwash (wave overtopping) criterium is used. Freeboard should be such that the overwash during design conditions is less than a design constant, "A":

A = 0.1 liter/second for normal dikes
A = 1 liter/second for dikes with an inner slope of 1:3 or less and a good grass on clay cover
A = 10 liter/second for specially designed inner slopes and good facilities to get rid of the water.

Besides freeboard, a forecasted value for sea level rise is added to the design height (20cm/century) to cover the design period (50 years). Settlement of the subsoil is also added to the design height. This settlement is calculated for each dike section depending on local soil physics. Each year, the equivalent of approximately US $4 million is spent in the Netherlands on research on levee-related research, not including project-related research.

New Zealand. Freeboard is a factor normally added as a precautionary measure against uncertainties (factor of safety). Its value may depend on the sensitivity of flow and levee height, and on uncertainties in the design flow or Manning's n.

Norway. A value of 0.3m to 0.5m is normally used. It is adjusted for flow and ice conditions. For flood embankments based on 10-20 year frequencies, there is normally no freeboard because overflow for these projects is a calculated risk. Overflow is planned to occur at a location where the slope is small and where protection is provided to prevent erosion and levee failure.

Poland. Freeboard varies according to class of structure and location. Four classes are defined based on the degree of danger for loss of life. The range is from 0.5m to 1.2m. Poland typically uses the 0.1% chance (1/1000 year) storm event for design.

Sweden. In the recently published "Swedish Guidelines For Spillway Design Floods", (Bergstrom, 1990), freeboard considerations have deliberately been left out as it was felt that there were "too many local aspects that had to be accounted for". The committee only recommended wind speeds for estimation of water and wave heights. Providing adequate technical freeboard is then a responsibility of the dam owner and local flood control district.

Switzerland. Freeboard is added to the design water surface elevation to set the top of levee elevation. There is no minimum height for the addition of freeboard. Freeboard depends on the river type, flow velocities, damage potential, risk of floating debris and debris flows. Freeboard is generally computed as a function of velocity head ($V^2/2g$) in steep channels. The minimum is normally not less than 50 cm and the maximum is generally 100cm.

Taiwan. Two brief responses were obtained from Taiwan. One said freeboard is a fixed value of 1m for concrete structures, and 1.5m for embankments. Another said freeboard is a computed value adjusted to meet the debris flow requirements. Design criteria may differ between different locations.

3.3 Summary of Freeboard Results

Results from the survey conclude that the value of freeboard usually varies with (1) height of the levee, (2) type of construction material used to build the levee, (3) levee top width, (4) design flow, (5) velocity head and degree of curvature in the channel, (6) value of the land protected, and (7) potential for loss of life if the project were to fail. Many of the countries agreed that "the amount of freeboard should be increased to protect areas with high value and high damage and loss potential." The following factors were mentioned in the survey results as important factors to include in freeboard if they are not already included in the design water surface elevation: (1) wave runup, (2) wind setup, (3) settlement potential, (4) ice effects, (5) potential for floating debris, (6) measurement inaccuracies, (7) sediment scour or deposition, (8) possibility of debris flows, (9) uncertainties in hydrology and hydraulics, (10) uncertainties in n values, (11) factor of safety, and (12) social and psychological perceptions of the level of protection required.

3.4 Risk and Uncertainty

Most countries reported that risk and uncertainty principles are included in their design procedures. We did find, however, that there is wide range of interpretations and understandings of what "risk and uncertainty" are and how they may be applied to determine the top of levee elevation. Examples from the survey include: (1) risk is applied directly to the design discharge and not to freeboard, (2) risk is included in freeboard and not in the design discharge, (3) an "expected probability adjustment" is used in lieu of freeboard, (4) risk is included in flood frequency analyses, (5) freeboard is increased in cases of higher risk (damage potential), (6) freeboard is a "factor of safety," and (7) risk is used in cost/benefit methods to determine project viability. Table 3 summarizes the responses made to the questions pertaining to risk and uncertainty.

4. OTHER INTERNATIONAL PROBLEMS AND RESEARCH NEEDS

The survey identified the following kinds of design problems, data deficiencies and research needs:

Category of Problem or Need	Discussion
Data:	Lack of data for calibration of computer models (New South Wales, Australia) Need long term continuous data to establish accurate design discharges, need measured high water marks to estimate n values
Debris Flows:	Need to understand more about the physics of debris flow events (Switzerland, China, Japan)
Environmental:	Lack of information on environmental effects of flood control works (Victoria, Australia), constructing levees to provide as natural a condition as possible (Switzerland)
Erosion:	(Norway), erosion protection (Ontario and British Columbia, Canada), riverbed erosion and aggradation (Switzerland)
Funding:	governments are reluctant to commit funding until after a major flood event has occurred (Canada), financial difficulties (Hungary), methods of financing projects (Norway), general funding problems (Ontario, Canada)
Ice:	Canada, Norway, Sweden, Germany, Poland
Institutional/ Political:	Lack of good administrator (Poland), not sufficient consideration given to flood risk by municipal authorities (Norway)
Land Use Changes:	Increase in runoff due to clearing of native vegetation for agriculture (Western Australia), urban development in flood prone areas (Canada), dikes are multi-functional, i.e. houses are on the dikes, roads, natural reserves on both sides (Netherlands), limited space available (Netherlands)
Maintenance:	Lack of adequate maintenance due to budget and political problems (Brazil), due to ice conditions (Canada), maintenance hindered by historical encroachment (Canada), lack of maintenance costs (Ontario, Canada)
Rodents:	Destruction of levees (Poland)
Sediment:	Taiwan, Japan, sediment problems and debris flows are common (Switzerland)
Social:	Objections due to altering natural landscape by levees (aesthetics)
Technical:	Estimation of very low frequency floods: (Ontario, Canada), upgrading to a full probabilistic design of a full dike-circle (Netherlands), selection of design floods (Switzerland)
Telecommunication:	Lack of reliable telecommunication systems (Poland, Hungary)
Vegetation:	Norway
Weak foundations:	Embankments on weak foundations or with unsuitable materials (peat), bank protection (Ireland)

5. SUMMARY AND CONCLUSIONS

A thorough search of literature within the U.S. shows that very little information is readily available on international freeboard practices from traditional library sources in the United States. This led to an international survey to determine the analytical and design methods used by other countries to establish freeboard heights for stream channels and flood control levees. Over 200 questionnaires were distributed to 45 countries around the world. Approximately 50 written responses were received; 35 contained completed questionnaires and/or materials dealing with the topic of freeboard design practices. The survey was not intended to be an exhaustive investigation and may have missed some important information sources. The responses discussed herein represent a good sample of freeboard practices from 18 different countries.

The following conclusions are made based on the results from the international survey:

1. There is a wealth of information from foreign countries regarding freeboard practices, however, it is not readily available through normal library resources in the United States. In general, foreign water ministries are interested in sharing design procedures and information with the United States.
2. All of the 18 countries responding to the survey use some form of freeboard.
3. Freeboard is most often determined by various factors that are estimated and is not often a computed value.
4. The most common design discharge is for the 1% chance (100-year) flood event, however, in some countries where levee failure could lead to tremendous loss of life, design standards are often based on the 0.1 to 0.01% chance (1000 year to 10,000 year) event.
5. The range of freeboard reported is from 0.1 meters to 4 meters.
6. Factors included in freeboard vary significantly from country to country and often regionally within a country.
7. Risk and uncertainty methods are being applied by some but there is a great deal of confusion on how to approach this type of consideration.
8. **Freeboard is not a uniform or universally agreed upon term.**

6. RECOMMENDATIONS FOR FURTHER STUDY

The responses and results from this preliminary investigation of international freeboard practices are excellent. The authors suggest that the present leads and information be expanded and that new contacts be pursued. Many of the materials received were in different languages. Little time was available for adequate translation. It would be very valuable to translate the remaining materials and include them with the present results. The authors also recommend that copies of the final workshop proceedings be sent to the responding countries with a request that they review and comment on the materials contained in this paper and clarify any statements the authors may have misinterpreted. Additional foreign contacts and addresses can be requested at that time as well. Finally, we suggest that the Corps consider the possibility of an international workshop to address freeboard and related flood control topics.

7. REFERENCES

The following references contain all the information that was received. Omissions of dates or locations are not introduced by the authors.

1. Bergstrom, Sten, August 1990, "New Swedish Guidelines for Spillway Design Floods", Swedish Meteorological and Hydrological Institute.

2. Central Water and Power Commission, Ministry of Irrigation and Power, 1960, "Embankment Manual," New Delhi, India.

3. Char, N.V.V., Director, FCCDte, "Structural Measures of Flood Management,", India.

4. Goetz, Antreas, Head of Division of Swiss Federal Office for Water Management, 1990, "Protection of Habitat Against Floods and Debris Flows in Switzerland."

5. Japan Ministry of Construction, Department of Rivers, Japan Society for Rivers, "Regulations for Design of Flood Control Levees."

6. Ministry of Natural Resources, Ontario, Canada, March 1977, "Guidelines and Criteria for Approvals Under the Lakes and Rivers Improvement Act."

7. National Research Council Canada Associate Committee on Hydrology, 1989, "Hydrology of Floods in Canada: A Guide to Planning and Design."

8. Natural Environment Research Council, London, England, 1975, "Flood Studies Report Procedures."

9. New South Wales Government, December 1986, "Floodplain Development Manual."

10. Saville Jr., T.; McClendon, E.W., and Cochran, A. L., May 1962, "Freeboard Allowances of Waves in Inland Reservoirs", Journal of the Waterways and Harbours Division, Proceedings of the American Society of Civil Engineers.

11. Swiss Federal Office for Water Management, 1982, "Guidelines for Protection Measures Against Floods."

12. Water Resources Council, Victoria, Australia, September 1978, "Flood Plain Management in Victoria."

8. ACKNOWLEDGEMENTS

Water Engineering & Technology, Inc. wishes to thank the Corps of Engineers for supporting the research project. The authors also express their deep appreciation to all of those individuals and agencies around the world who took the time to respond to the survey. Without their help this project would not have been successful. The opinions and results presented in this paper are those of the authors and not necessarily those of the Corps of Engineers or of the countries that were surveyed.

SUMMARY OF SESSION 4: PROPOSED STRATEGIES FOR SIZING LEVEES CONSIDERING RISK AND UNCERTAINTY

Overview

This session included two papers describing aspects of a risk analysis framework approach to sizing a levee without reference to freeboard. The session also included an open discussion period led by HQUSACE to explore steps that needed to be taken to implement new levee design procedures incorporating the risk analysis framework approach described in this session.

Paper 12. Darryl Davis, Director, Hydrologic Engineering Center, presented a paper entitled, "A Risk and Uncertainty Based Concept for Sizing Levee Projects." Levee projects are sized to provide economical protection to flood prone areas. Current policy is that levee height is determined by analyzing the flood damage potential of the flood prone area, damage prevention performance and cost for a range of levee heights, and the plan selected that maximizes net economic benefits. Consideration is given to risk issues but deviation from the NED plan must be justified and approved. During preliminary planning, an increment of levee height over the flood level "freeboard" is included in the analysis and is usually assigned a fixed value such as three feet. Freeboard is meant to ensure performance of the project during occurrence of the design flood by including an allowance for the many intractable factors involved in determining design flood levels. During later project development phases, levee freeboard is designed by more explicitly considering hydraulic uncertainties and the need to configure the levee height and alignment to control the location and manner of failure from possible overtopping.

Much debate has occurred regarding the economic value of freeboard. Adding height to a levee is expensive. The economic investment view is that we should receive a return for the increment of investment. The typical design view is that freeboard must be provided to ensure the project provides the benefits claimed for the design flood.

The risk analysis framework described uses statistical distributions of the error in the estimates of the several important variables including discharge-frequency, stage-discharge, stage-damage relationships, and perhaps cost, which are used to determine levee height. The statistics quantify the uncertainties in the relationships. Exhaustive trials are made using Monte Carlo simulations to account for all the possible combinations of the errors in the relationships. The result of the risk analysis framework approach is a matrix of levee height, probability distributions of the several variables, expected cost and benefits and exceedances for selected design heights. The National Economic Development (NED) plan is identified in accordance with current policy. The matrix is further used as appropriate, to evaluate project sized larger or smaller than the NED plan. The performance of the selected plan would be expressed in reliability terms such as, "the levee has a 90% chance of protecting against the .005 chance exceedance (200-year) flood event, should it occur and a

75% chance of protecting against a .002 chance exceedance (500-year) flood event, should it occur", in lieu of stating a specific level-of-protection, as is currently done.

Paper 13. David A. Moser, Technical Analysis and Research Division, Institute for Water Resources, presented the final paper entitled, "Risk and Uncertainty and the Economics of Levee Level of Protection and Freeboard." David emphasized that design and evaluation of engineering structures in water resources inherently requires extensive consideration of risk and uncertainty. This is especially true for flood control structures. The Corps of Engineers has long used frequency based methods in flood control benefit evaluation. The incorporation of risk, however, has generally been limited to representing peak annual discharge or flow by a frequency distribution such as log-Pearson Type-III. There are numerous other relationships in the evaluation of a project that also exhibit risk and uncertainty. Indeed, the discharge-frequency function itself retains significant uncertainties due in part to the limited number of historical data points used in its derivation. Similarly, there is uncertainty. Along with the engineering related sources, there are also economics related sources of risk and uncertainty that Yshould be assessed in the evaluation of any flood control project.

His paper describes freeboard as an important planning issue for other, non-engineering reasons. It supports the notion that freeboard should not be given special status except for specific considerations such as assuring a "safe fail." A levee should be evaluated and the choice of scale recommended using a combined risk model with risk analysis methods.

A RISK AND UNCERTAINTY BASED CONCEPT FOR SIZING LEVEE PROJECTS

by

Darryl W. Davis[1]

INTRODUCTION

Levee projects are sized to provide economical protection to flood prone areas. Current policy is that levee height is determined by analyzing flood plain damage potential, and damage prevention performance and cost for a range of levee heights. The plan selected is based on maximizing net economic benefits consistent with acceptable risk and functional performance. An increment of levee height over design flood level, "freeboard", is included. The freeboard is provided to ensure performance of the project during occurrence of the design flood.

Debate is ongoing regarding the economic value of freeboard. Adding height to a levee is expensive. An economic viewpoint is that we should receive a return for the increment of investment. A design viewpoint is that freeboard must be provided to ensure the project provides the benefits claimed for all floods up to and including the design flood. A simple formula is now used to claim partial credit for benefits in the freeboard range.

This paper explores the issues involved from the hydrologic engineering, flood damage, risk, and uncertainty perspectives and proposes a general concept for sizing levees without explicit identification or addition of freeboard. The concept is based on practical incorporation of risk and uncertainty in determining the levee height. An appendix presents a example application of the concept.

PLANNING/DESIGN GOAL

The goal of planning and design of levee projects is to provide economical and safe protection to flood prone areas. This is a common goal for all flood damage reduction projects. Levee projects are attractive for protecting developed riverine areas remote from locations offering potential for effective flood control storage projects. The performance of levees is predictable for floods up to the design flood. When the design flood is exceeded, the project may overtop and in some instances, fail structurally. Flooding may then suddenly occur to flood prone areas up to depths that would have existed without the project. Other types of flood damage reduction projects do not exhibit this characteristic to the same degree. Reservoirs simply cease to regulate flow but do not suddenly fail when the design flood is exceeded. For channel projects, flood stage reduction continues to occur when the design flood is exceeded since flooding is limited to the depth that

[1]Director, Hydrologic Engineering Center, Davis, Calif.

exceeds the design. The unique characteristic of levees, potential for sudden loss of protection, places a burden on project formulation and design so that economical protection is provided while assuring that a catastrophe will not occur when the design flood is exceeded.

A levee project is defined by the height, grade, configuration and location of the embankment. In a technical sense, determining these items are the objectives of planning and design. Other issues of community acceptance, cost sharing, and environmental impacts are certainly important and critical but are not discussed herein. Nor is the issue of geotechnical aspects of the levee embankment discussed.

The technical task is to balance risk of design exceedance with flood damage prevented, uncertainty of flood levels with design accommodations, and provide for safe, predictable performance. The task is made difficult because economics dictate that less than complete protection be accepted, risk of overtopping is real and must be planned for because it most likely will occur within the life of the project, and uncertainty in flood levels exists because of imperfect knowledge.

WHAT WE DO NOW

The basic approach taken at present is to make best estimates for the several items discussed above and consider uncertainty by application of professional judgement, designing freeboard, and sensitivity analysis.

Risk. Risk is considered in that several projects are formulated and evaluated considering a range of target protection levels. Several target levels of protection are nominated, typically above and below the 1% chance exceedance for urban areas and at lesser protection levels for agricultural areas. The consequence of design exceedance is evaluated and is considered in the selection of the recommended project. Policy directs that the selected plan will be the project that maximizes net economic benefits. Deviations from this policy are possible given acceptable justification. The underlying assumption is that only projects that are appropriate are considered in the selection process. In other words, projects that may result in unacceptable risk of catastrophe or are inherently unsafe are not considered.

Uncertainty - Discharge. Uncertainty in discharge for nominated design risk level of protection is considered to the extent that using an expected probability frequency curve yields the best single estimate of the design flow. The design discharge corresponding to the nominated protection level is read from a discharge-frequency curve. The discharge-frequency curve is developed applying recommended methods contained in Corps regulations generally following federal guidelines (U.S. Department of Interior, 1982). In some instances, confidence limits shown on the frequency curves (the range of possible error in discharge values) are used in sensitivity analysis.

Uncertainty - Stage. Uncertainty in stage is considered to the extent that extensive studies are undertaken to adopt and verify "design" energy loss coefficients, debris blockages, geometry and bed profiles, and other special flow conditions for computation of flood profiles. The adopted design profile tends to be an envelope representing the "best" estimate of the upper bound of water surface elevations. Accommodation of uncertainty tends to be in the direction of increasing profile elevations to ensure performance against the design flood. Freeboard is a concept that serves the basic purpose of accommodating uncertainty in stage for design flow resulting from intractable hydraulic (conveyance) factors. An important design goal is subsumed into the concept of freeboard that insures initial levee overtopping from design exceedance occurs at a planned location, typically the downstream end. Another design goal is to extend the interval between major maintenance, such as tree removal, for cost saving reasons as well as practical scheduling and avoidance of disruption of the embankment.

Uncertainty - Damage. Uncertainty in damage is generally not considered other than as might occur in sensitivity studies associated with project economic justification.

The present approach has served the Corps and the public well. The criticism recently surfaced is that freeboard is expensive and our studies do not quantify the return expected for the investment. One approach would be to intensify efforts to enable placing an economic value on freeboard as now used. Another would be to consider an approach that incorporates elements of risk and uncertainty more directly in project formulation, evaluation, and design and abandons the concept of freeboard. Both approaches have merit toward ultimately resolving the issue. For discussion purposes, the presentation that follows will deal with the latter approach.

THE RISK AND UNCERTAINTY PROBLEM

Figure 1 is a conceptual schematic of the problem from a risk and uncertainty perspective. Risk here is used as the classic case of ..."possibility of loss or injury," (Webster, 1983). We reflect risk by terms such as "1 percent chance design exceedance flood." Our basic task is to find the level of risk that will constitute project design. The risk relationship used for flood problems is depicted in the upper left corner of Figure 1. This relationship, the discharge frequency curve, displays exceedance probability versus peak discharge.

Uncertainty in corresponding design flood is illustrated in the upper right corner of Figure 1. A probability density function representing possible statistical sampling error in the basic discharge-frequency function is superposed. This depiction reflects that there is "uncertainty" in the discharge that corresponds to the stated design risk. This is because gaged record lengths are short (in a statistical sense), or do not exist where they are needed. We thus have imperfect knowledge of the statistical parameters needed for the discharge estimate. Uncertainty here conforms to the classic case of, ..."the quality or state of being uncertain ... lack of sureness about something" (Webster, 1983). This uncertainty can be represented by confidence bands shown on the flow frequency curve, or a probability distribution of error about the design flood.

Flood stage uncertainty corresponding to a stated design discharge is represented in the lower left corner of Figure 1. This depicts a rating curve with a probability density function superposed, representing possible statistical error in elevation estimate for stated design discharge. This reflects that there is "uncertainty" in the stage that corresponds to the stated design discharge because we have imperfect knowledge about the channel

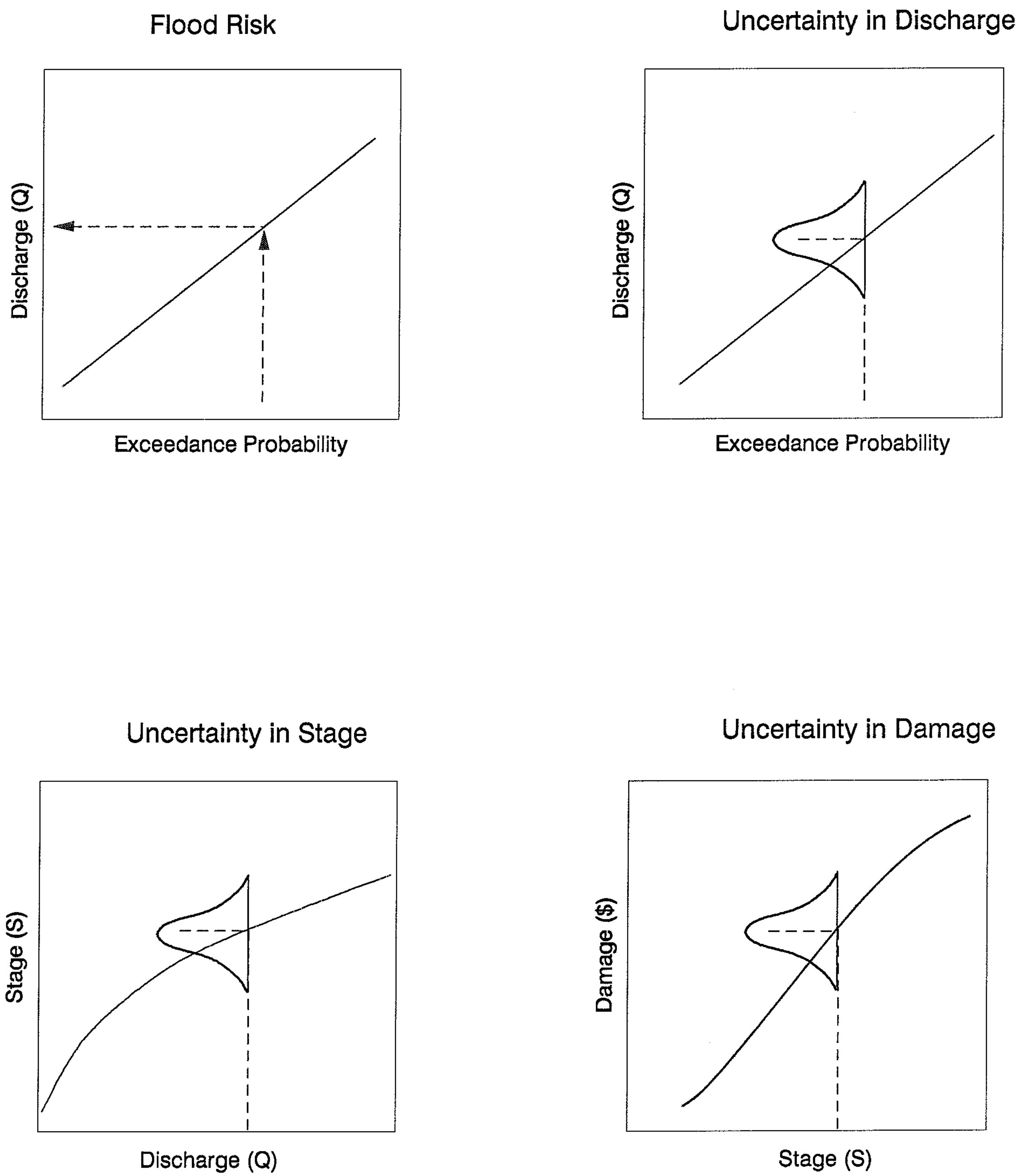


Figure 1. Uncertainity in Discharge, Stage, and Damage

geometry and roughness, flow regime, flow debris content, and our models of the phenomena are not exact. This uncertainty could be represented by confidence bands, similar to those used for frequency curves, about flow-elevation rating curves but is more typically noted by such terms as, ... "the profile is accurate to the nearest foot," or a tabulation of the results of sensitivity analysis.

Flood damage (or damage prevented) uncertainty corresponding to a stated design stage is reflected as shown in the lower right corner of Figure 1. A probability density function representing possible statistical error in damage estimate for design stage is superposed on the stage damage function. This reflects that there is "uncertainty" in the flood damage that would result should the design stage occur in the flood plain because we have imperfect knowledge about the nature and mix of improvements, elevation of improvements, and physical structure and content damage potential. This could be represented by confidence bands, similar to those used for frequency curves, about stage-damage curves but is more typically noted, ... "accurate to within the nearest foot," or a tabulation of the result of sensitivity analysis.

A CONCEPTUAL PROPOSAL FOR LEVEE SIZING

Represent the basic information as accurately as possible, quantify the uncertainty, and proceed with the analysis essentially as presently performed. Specifically:

a. Develop basic data, e.g., best estimate discharge-frequency, elevation-flow rating, and stage-damage relationships,

b. Develop statistical description of uncertainty for each of the above relationships,

c. Develop cost versus levee design elevation relationship,

d. Nominate levee design elevations, compute costs and damage prevented, array and choose according to economic and other, as appropriate, criteria, and

e. Make design refinements to ensure planned management of overtopping and subsequent flooding.

This proposal while similar to present practice, is markedly different in several respects. The basic data is the same except that uncertainty must be explicitly quantified. The levee sizing parameter is design elevation - not the protection level or risk. There is no "design flood" as such, and levee freeboard to account for uncertainty is not added. The issues associated with these differences are significant and are discussed in the order presented above.

<u>Basic Data</u>. The basic data identified in step a. is prepared now in typical levee sizing studies. No change is required.

<u>Statistical Description of Uncertainty</u>. There is a long history and documented technical methods for quantifying uncertainty in discharge for stated risk exceedances. Corps policy now requires that confidence limits be shown for frequency curves and further, that the curve developed represent

"expected" probability. The technical analysis needed to develop uncertainty (in design discharge) information in the form that will be needed in subsequent analysis thus exists. The frequency curve to be used in this analysis should not have the expected probability adjustment. The uncertainty is automatically incorporated in the sampling strategy that will be employed.

No significant history nor extensively documented methods exist for stage uncertainty related to design discharge. The accuracy of computed steady flow profiles has been investigated (Hydrologic Engineering Center, 1986). This reference should be considered to assist in quantifying uncertainty in flood profile elevations. In a typical Corps study, uncertainty in stage for design flow is addressed by using upper bound values of parameters when computing the design flood profile, or performing sensitivity analysis. It is believed that by use of published uncertainty data, carefully performed sensitivity analysis, and professional judgement that uncertainty in stage related to discharge can be accurately quantified in statistical terms as needed for the analysis proposed herein.

No history nor documented methods exist for estimates of damage uncertainty related to design stage. Typically such uncertainty is dealt with, if at all, by modest (in scope) sensitivity analysis. It is believed that use of emerging uncertainty data emanating from risk studies, carefully performed sensitivity analysis, and professional judgement will allow accurate quantification of uncertainty in damage related to stage in statistical terms, as needed for the analysis proposed herein.

Cost Relationships. The cost data identified in step c. is prepared now in typical levee sizing studies. Although not proposed herein, cost could be considered uncertain, similar to the other factors. An increment in realism/refinement would be to perform preliminary design studies (to address step e.) for a range of levee heights and incorporate the results into the cost relationships. This would ensure that the full cost of levee construction would be integral to selecting the levee height.

Sizing the Levee. Step d. is where the elements are brought together to determine the selected design height. In present studies, the best estimate for each relationships would be used to compute the average annual value of flood damage, damage reduced, and cost. To correctly incorporate uncertainty in the several elements, they must be allowed to interact with one another. For example, the possibility of error for larger flows (or lower flow) must be allowed to couple with the full range of possible stage and damage errors. Because of the nature and complexity of the error distributions, the interaction cannot be accomplished analytically. Instead a Monte Carlo analysis is necessary. In this approach, the basic relationships and error distributions are sampled by exhaustive trial to allow the interactions to take place. See, for example, (Benjamin, 1970) and (Palisade Corporation, 1988) for explanations of the Monte Carlo method for sampling of interaction among uncertain relationships. A simplified example of Monte Carlo simulation applied to levee sizing is included in an appendix.

The results of the analyses are probability distributions of the various parameters (design flow, stage, and residual damage) as a function of levee height. The expected cost and benefit can then be computed and the levee height selected according the appropriate criteria. One could (and likely would) prepare reliability tabulations of the likelihood of exceedance by various flood events to enable characterizing performance by assignment of a level-of-protection, with stated reliability.

Final Refinements. The remaining task of levee height determination is that of making design refinements to ensure planned management of overtopping and subsequent flooding, and any other closely associated operational issues. Adding or subtracting to the levee height at selected locations has most often been included within the freeboard determination step. This is not appropriate; it is a legitimate and necessary task that should be dealt with directly in project formulation as proposed herein.

SUMMARY

Imperfect knowledge of the "true" nature of the hydrology and hydraulics in an area creates uncertainty in project designs and in our estimate of their reliability. Additionally, uncertainties in expected damage with and without the project influence the selection of an alternative plan for design. A risk and uncertainty based method is proposed that incorporates these elements directly in project formulations and abandons the concept of freeboard. A simple example is presented that demonstrates that the concept is possible to apply in Corps studies. Probabilistic output distributions of various variables add a significant increment of information over traditional methods. Project risk is implied in our traditional evaluation of several target protection levels. The reliability of these target design levels could also be presented in a probabilistic manner. The methods described in this paper are recommended to further develop the concepts of risk and uncertainty in flood control project design and evaluation. Additional effort is needed to establish the reliability of these methods to accurately analyze the desired relationships. Similarly, clearer representation of uncertainty as an error density function would be helpful. Ideally, simplified, widely used procedures to better analyze and present design reliability can only improve the quality and acceptability of our projects.

CONCLUDING THOUGHTS

On the technical side, the significant issue in explicitly incorporating uncertainty into the sizing problem is development of defensible error density functions. Much effort would be required at present for a typical study. With a concerted research effort and accumulation of experience, it is believed that this issue will resolve itself. In addition, the concepts associated with more involved risk/uncertainty must be accepted by technical staff and further, they will need to develop a degree of proficiency in performing the analysis for the studies to succeed. This could be helped by a concentrated research effort to develop technical information, relationships, and guidelines that might enable the essence and value of analysis as proposed herein to be achieved without explicitly requiring Monte Carlo simulation in each instance.

From the institutional side, there are issues to be addressed. The freeboard concept is deeply imbedded within the established way of sizing and designing levee projects in the Corps. In fact the Corps invented it. A significant effort would be needed to successfully achieve the change. An associated issue is that of representing the performance of existing levee projects. The approach suggested explicitly acknowledges that there is not a specific, unequivocal, performance level. Levels-of-protection would have to be couched in a reliability context such as "this levee project has a 95% chance of protecting against the 100-year exceedance interval flood, should it occur". Acceptable reliability criteria would have to be adopted and education of others (Congress, other federal agencies, local sponsors, etc.) in appropriate interpretation would need to be accomplished.

The critical question is .. Would the proposed approach result in better formulated, more acceptable and defensible levee projects than the present approach? I believe it would.

ACKNOWLEDGEMENT

Karen Kuhn assisted the author in preparation of the example presented in the appendix. Karen developed the Lotus 123 spreadsheet, assembled and debugged the data, performed the simulation analysis using the @RISK add-in to Lotus, and prepared the appendix materials. The work was performed while Karen was on a developmental assignment to HEC. At the time, she was a Civil Engineer in the Planning Division, Sacramento District.

REFERENCES

1. U.S. Department of Interior, Geological Survey, "Guidelines for Determining Flood Flow Frequency," Bulletin #17B, March 1982.

2. A Merriam-Webster, Webster's Ninth New Collegiate Dictionary, 1985.

3. Hydrologic Engineering Center, "Accuracy of Computed Water Surface Profiles," 1986.

4. Benjamin, Jack R. and Cornell, Allin C., Probability, Statistics and Decision for Civil Engineers, McGraw-Hill Inc., New York, New York, 1970.

5. Palisade Corporation, @RISK, Risk Analysis and Simulation Add-in for Lotus 123 User's Guide, March 1988.

6. U.S. Army Corps of Engineers, Philadelphia District, "Status Report on Feasibility of Interior Flood Central Studies, Metropolitan Chester Creek Basin, Pennsylvania (not a final report)," undated.

7. Lotus Development Corporation, Lotus 123 Release 2.2 User's Manual, 1989.

8. Strait, Peggy Tang, Probability and Statistics with Applications, Harcourt Brace Jovanovich, Inc., New York, 1983.

9. Hydrologic Engineering Center, "Expected Annual Flood Damage Computation User's Manual," March 1989.

APPENDIX

EXAMPLE: LEVEE SIZING BASED ON RISK, UNCERTAINTY CONCEPTS

This paper proposes a general concept that incorporates risk and uncertainty in levee sizing without the explicit identification or addition of freeboard. This appendix presents a simplified example application of this concept. A Monte Carlo type analysis is used to sample the interaction among uncertain relationships associated with flood discharge, stage, and damage estimation. The example application explores the selection of levee size, the sensitivity in the quantification of uncertainties, and the representation of risk for a selected design.

DATA AND UNCERTAINTY

Setting. Data from Chester Creek, Pennsylvania were selected for the example. The 68 square mile Chester Creek Basin, located in the southeastern portion of Pennsylvania, ranges from rolling rural hills to flat tidal urban areas. Localized heavy rainfall combines with this relatively long narrow, steep basin to produce quick rises in flood stages. Periods of maximum runoff generally occur in summer and fall. In the period from 1945-1971, approximately 13 events resulted in overbank flooding and subsequent damage along Chester Creek.

Thirty years ago, the Corps constructed a floodwall to protect a portion of the Chester Creek floodplain. Flooding that occurred in September of 1971 (flood of record) overtopped the floodwall causing extensive damage and the loss of lives. Subsequent studies investigated alternatives for flood-damage reduction along approximately 10 miles of the main stem of Chester Creek upstream from the Delaware River, and approximately 3200 feet of the West Branch of Chester Creek (U.S. Army Corps of Engineers, undated). Data compiled for these studies are used in this example.

Example Data. To compute the damage-probability function used in flood damage analyses, three relationships are commonly used: the discharge-probability function, the stage-discharge function, and the stage-damage function. For the original Chester Creek analyses, the river was subdivided into numerous subreaches with different stage-discharge and stage-damage functions. The following uncertainty experiments utilize an index stage-discharge function and aggregate elevation-damage function. When these functions are combined with the expected probability-discharge relationship, the resulting damage-probability function can be integrated to compute the expected annual damage. Project studies are performed by evaluating the impact of changes to these functions resulting from alternative project proposals.

Discharge. A 65 year equivalent period-of-record was used to derive the discharge-probability function for the Dutton Mill gage. The annual peak streamflow data was fit to a Log-Pearson Type III distribution with a logarithmic mean equal to 3.507 and a logarithmic standard deviation of 0.295.

A regional skew coefficient of 0.4 was used. This distribution is utilized directly in the experiments and is defined with 23 carefully chosen coordinates. The expected probability adjustments are not made for the Monte Carlo simulations. However, expected probability is utilized in the "traditional" analysis shown for comparison.

The simulations quantify uncertainty in discharge associated with sampling errors in the mean and standard deviation for a stated risk exceedance. This method is often used to develop confidence limits for the distribution function using the noncentral t-distribution, as defined by approximation equations (U.S. Department of Interior, 1982). With given values for parameters of the frequency curve (i.e. mean, standard deviation and skew), the sample size (i.e. years of record), and the exceedance frequency associated with a particular discharge, a distribution of errors about the given discharge can be developed.

Stage. The original Chester Creek analysis defined the stage-discharge relationship utilizing the HEC-2 computer program with the information provided by the September 1971 event for calibration. An index stage-discharge relationship as described above was adopted and used in the simulations.

Water surface elevations associated with specific discharges are generally based on steady-flow water surface profile computations. A study performed by HEC for the Federal Highway Administration analyzed errors in such computations due to uncertainties in stream geometry and roughness coefficients (Hydrologic Engineering Center, 1986). Information from this report is useful for estimating the magnitude of error for a profile.

In the following analysis, errors are taken to be normally distributed about the calculated profile. A one foot standard deviation of error associated with the estimated rating curve was used for the first and the third examples. The standard deviation of error was raised to 1.5 feet and 2.0 feet in the second example to explore the sensitivity of uncertainty estimation.

Damage. The basic categories analyzed in the damage assessment for the Chester Creek Basin were Residential, Commercial, Industrial, Railroads, Utilities, Highways, Public Facilities, Agricultural and Flood Emergency costs. As mentioned previously, an aggregate stage-damage function was utilized in the following experiments for convenience. Combining the aggregate stage-damage relationship with the index stage-discharge relationship and the expected probability-discharge relationship yields a damage-probability function. Integration of this function yields an expected value of $79,000 per year.

Stage-damage uncertainty reflects uncertainty in the flood damage given that the design stage occurs in the floodplain. This is due to imperfect knowledge of the elevations, characteristics, and distribution of improvements, and their "actual" damage potential. There is little documented data for estimating damage uncertainty related to design stage. For the purpose of this analysis, damage uncertainty is taken to be related to accuracy in structure inventory mapping and is therefore represented by units of elevation. A standard deviation of error of one foot is used for the first and third experiments and is incorporated within the stage uncertainty in the following manner. The standard deviation of error for the stage-discharge function (SSD) is combined with the standard deviation of error for the stage-damage function (DSD) to form a composite standard deviation of error (CSD) using the following relationship: $(CSD) = (SSD^2 + DSD^2)^{1/2}$ (Strait, 1983). The second experiment varies the damage uncertainty in the same way as the stage uncertainty is varied from 1.0 foot to 1.5 feet and 2.0 feet.

MONTE CARLO SIMULATION

Overview. Uncertainty is present in the estimation of discharge, stage and damage due to imperfect knowledge of statistical parameters, flow regime and structure inventory information. Derivation of the damage-probability distribution, a necessary relationship in the planning and design process, requires the interaction of the uncertainties associated with the discharge-probability, stage-discharge, and stage-damage functions. Theoretically, the probabilistic nature of the distribution for the function of one or more random variables may be derived, but it is the completion of the resulting integrations that proves to be very difficult (Benjamin, 1970).

The Monte Carlo method uses the probabilistic nature of the derived distributions to obtain the results of many repeated experiments. Monte Carlo sampling uses random numbers to sample from a probability distribution. With enough iterations, Monte Carlo sampling will mimic the original distributions. Some of the input distributions used for the following experiments have low probability outcomes that strongly impact the results. In an effort to avoid "missing" important low probability values through standard random Monte Carlo sampling, a refined sampling method, Latin Hypercube, was employed (Palisade Corporation, 1988). This sampling method stratifies the input distribution and samples without replacement, thereby ensuring inclusion of important outlying events that could be missed. Additional technical references for simulation, Monte Carlo techniques, and Latin Hypercube sampling techniques are suggested in the @RISK Users Guide (Palisade Corporation, 1988).

Application. Chart 1 is the basic Lotus 123 (Lotus Development Corporation, 1989) spreadsheet developed for the sampling experiments used in conjunction with @RISK add-in program (Palisade Corporation, 1988). The nominal target level of protection (FD), in probability units, is requested of the user and design discharge (QD) and design stage (SD) are interpolated from existing data. The amount of freeboard (FR) included here for comparison purpose to be added is also input by the user to determine the appropriate annual cost of the plan (CD) and to be used during the simulation as a design stage (SD+FR) for comparison with the sampled stage. The annual stage-cost function was created for the example simulations for comparison purposes based on typical levee cross-sections. While these design parameters in the upper portion of the spreadsheet retain the same value throughout the simulation, the variables in the lower portion are recalculated with each iteration. The following paragraphs illustrate a single iteration for a simple simulation.

A discharge (QS) value is sampled from a cumulative distribution defined with selected data points from the sample distribution function (Log Pearson III distribution). For this discharge, a cumulative distribution of discharge with uncertainty is derived using techniques described in Bulletin #17B (U.S. Department of Interior, 1982). From this distribution a new value for discharge (discharge with error, QE) is sampled. Stage (SS) is then

Example Lotus Spreadsheet
Monte Carlo Simulation Experiment

LEVEE SIZING SIMULATION
(Experiment #1, 23 August 1991)

VARIABLE	VALUE	EXPLANATION
Target Level (FD)	0.0100	Target probability entered by user.
Design Discharge (QD)	18991	Interpolated from Discharge-Probability data.
Design Stage (SD)	21.8	Interpolated from Stage-Discharge data.
Freeboard (FR)	0	Freeboard entered by user.
Annual Cost (CD)	20	Interpolated from Stage-Discharge data for (SD+FR).

MONTE CARLO SIMULATION FOR UNCERTAINTIES

Discharge (QS)	18991	Sampled from Discharge-Prob. function defined by 23 coord. points.
Q With Error (QE)	23106	Noncentral t-distrib. used for Q w/error distrib., then QE sampled.
Stage (SS)	24.6	Interpolated from Stage-Discharge data.
Stage With Error (SE)	25.9	Normal error function sampled and added to SS.
Damage With Error (DE)	4464	Interpolated from Stage-Damage data.
Residual Damage (RD)	4464	Equals DE if SE>=SD+FR; otherwise equals 0.
Net Benefit (NB)	-20	DE-RD-CD
No. Exceedances/Simulation	2121	Exceedances for simulation (Iterations=4000 for this example).

Chart 1

interpolated from the rating curve data. The error associated with the stage-discharge function and the stage-damage function are both taken to be normally distributed about the chosen stage with mean error of zero and standard deviation in units of elevation. The combined representation of error is sampled and added to the sampled stage (SS) to define the new stage with error (SE) for the iteration. Damage with error (DE) is then interpolated from the stage-damage data.

Residual damage (RD) for an iteration is equal to the sampled damage with error (DE) if the sampled stage with error (SE) is greater than the design stage (SD) plus freeboard (FR), SD+FR, which were defined at the beginning of the simulation. If the sampled stage with error (SE) is less than SD+FR, a zero is returned. Net benefit (NB) for the iteration is calculated by subtracting the sampled residual damage (RD) and the annual design cost (CD) from the damage with error (DE). The values displayed in Chart 1 for QS, QE, SS, SE, DE, RD and NB are actual values for one iteration. Finally, the number of exceedances for this simulation is used as a counter for comparison with the design probability.

Sampling the distribution functions (i.e.: QS, QE, and the combined stage error) and recalculating SS, SE, DE, RD, and NB for many iterations results in several probability distributions of the various parameters. The resultant QS distribution mimics the Log Pearson III distribution of the actual data. The discharge with error (QE) distribution is a composite of the discharge distribution with random error included. The SS distribution defines a stage-probability relationship for the simulation given the sample discharge distribution and associated uncertainty. The SE distribution is similar to the SS distribution but includes the estimate of rating curve and damage relationship uncertainty. The damage with error function (DE) is comparable to the damage-frequency function that would be defined using expected probability methods. The mean or expected value (sum of all the values in the set divided by the total number of values in the set) for a variable in a simulation is called the expected result, and is used in the following examples for purposes of comparison.

RESULTS

Levee Sizing. Table 1 tabulates the computed residual damage, cost, and benefits using traditional "expected probability" methods. Table 2 tabulates the computed residual damage, cost, exceedance frequencies and stages for the proposed risk, uncertainty method. Traditional methods incorporate expected probability in the discharge-frequency function to account for uncertainty associated with discharge. Using expected probability, residual damage is determined for the various design events utilizing the EAD computer program (Hydrologic Engineering Center, 1989). The residual damage at the project design level of 1.0 is the existing average annual damage. As is common design practice, freeboard (a nominal 3 feet) is included to ensure performance of the project during the occurrence of the design flood. The addition of freeboard influences the annual cost calculation and the estimation of net benefits. Current planning guidance suggests one estimate of freeboard benefits to be half the area under the damage-frequency curve between the design event and the maximum event that can be carried within the freeboard. This estimate was used in the calculation of net benefits for the

Table 1
Levee Sizing Example, Conventional (Present) Methods
(in $1000)

Expected Probability (Present Methods)					
Project Design Exceedance Probability	Design Stage[1]	Top of Levee Stage	Residual Annual Damage[3]	Annual Cost[2]	Net Benefits[3]
1.000			79		
0.040	17.4	20.4	76	17	-10
0.020	20.3	23.3	71	23	-4
0.010	23.6	26.6	47	35	10
0.004	26.8	29.8	22	57	8
0.002	28.8	31.8	12	66	7

1/Design Stage is interpolated from rating function for discharge corresponding to project design exceedance probability (expected probability).
2/Cost includes 3 feet of freeboard.
3/Residual annual damage assumes failure at design stage. Net benefits include an approximation of half the area under damage/frequency curve between design and top of levee; Computations are performed with EAD programs (Hydrologic Engineering Center, 1989).

Table 2
Levee Sizing Example, Proposed Risk, Uncertainty Method
(in $1000)

Monte Carlo Simulation[1]					
Nominal Exceedance Probability[2]	Top of Levee Stage[2]	Residual Annual Damage	Annual Cost	Net Benefits	Simulation (True) Exceedances[3]
1.000		78			
0.040	16.5	75	9	-6	.046
0.020	19.1	73	14	-9	.026
0.010	21.8	57	20	1	.013
0.004	25.7	30	31	17	.0058
0.002	27.0	19	37	22	.0035
.0014	28.7	11	48	19	.002
.0008	30.0	6	58	14	.001

1/Expected result for Residual Annual Damage and Net Benefits determined with 4,000 iterations of Latin Hypercube sampling using @RISK (Palisade Corporation, 1988).
2/Discharge - probability curve used does not have expected probability adjustment. Top of Levee Stage is interpolated from the rating function for the discharge corresponding to the nominal exceedance probability.
3/Simulation (True) Exceedances are the ratio of exceedances during simulation to the total.

conventional method. Maximization of net benefits occurs at approximately the 100-year exceedance interval level of project design.

Table 2 tabulates the computed residual annual damage, cost, net benefits and simulation exceedances for the proposed risk, uncertainty method. This simulation allows the full interaction of uncertainty for discharge, stage and damage. The top of levee corresponds to the design stage associated with the discharge corresponding to the "nominal" exceedance probability. The term "nominal" is used to distinguish these probabilities that derive from common flood frequency analysis from probabilities associated with frequency curves that are derived with the expected probability adjustment performed. The residual damage at the nominal exceedance of 1.0 is shown to be $78,000; very close to the $79,000 determined by conventional analysis. This suggests the sampling/simulation method can accurately reflect the uncertainties involved. Notice that the simulation exceedances are more frequent than the nominal frequencies; an expected result. The simulation exceedances reflect uncertainty in both discharge and stage and therefore are the most accurate estimate of design exceedances. While not directly comparable because stages are different, the residual annual damage for the Monte Carlo simulations approximates that of the traditional method.

Maximum net benefits occur at approximately the .0035 exceedance probability (simulation) design level, a bit higher top of levee than that indicated for the expected probability method. Additional simulation studies could further refine the maximum net benefit design. Note that the top of levee for the two methods are close even though the proposed method did not add freeboard while the traditional method did. This was not expected nor would it likely prove to be true in other projects since the magnitudes and character of the various uncertainties and stage and damage consequences thereof would be different.

<u>Sensitivity of Uncertainty Estimation</u>. This example, shown in Table 3, allows the same interaction of uncertainties as used in the first example (Table 2) for the nominal 100-year exceedance interval project design level. Existing and residual annual damage are then compared for varying degrees of uncertainty associated with the stage-discharge and stage-damage relationships. Intuitively, one would think that the expected result of the existing annual damage function would not change as uncertainties in stage are varied because they are represented as normally distributed. Instead, existing and residual annual damage increases. This phenomena deserves further exploration but is likely the result of the range of damage uncertainty being bounded by the lower end but unbounded at the upper end.

<u>Reliability of Design Stage</u>. The purpose of the final experiment is to present the reliability of various design levels in a probabilistic manner. Discharge (QS) was held constant at either the nominal .01, .02, or .04 percent chance exceedance flow for each simulation. The error distribution associated with each discharge was derived from the noncentral t-distribution as described earlier, and a new discharge with error (QE) could be sampled for each iteration within a simulation. Error in stage was sampled from a normal distribution with a standard deviation of error for the stage-discharge relationship of one foot. This was added to the interpolated stage (SS) to define the stage with error distribution (SE). The stage (SE) distribution for the 100-year exceedance interval event simulation as output from @RISK is shown in Chart 2 (Palisade Corporation, 1988). The dashed vertical line at

Table 3
Residual Damage for Nominal 100-Year Level of Protection with Varying Uncertainty in Stage

Damage Type	Damage (in $1000)[1]		
	Standard Deviation Error = 1.41[2]	Standard Deviation Error = 2.12[3]	Standard Deviation Error = 2.83[4]
Existing Annual Damage	78	84	93
Residual Annual Damage for 100-Year Design	57	61	70

[1] Expected result for 4,000 iterations of Latin Hypercube Sampling using @RISK (Palisade Corporation, 1988).

[2] Uncertainty in discharge/frequency according to Bulletin #17B (Water Resources Council, 1982); rating curve standard deviation of error = 1 foot; stage/damage standard deviation of error = 1 foot.

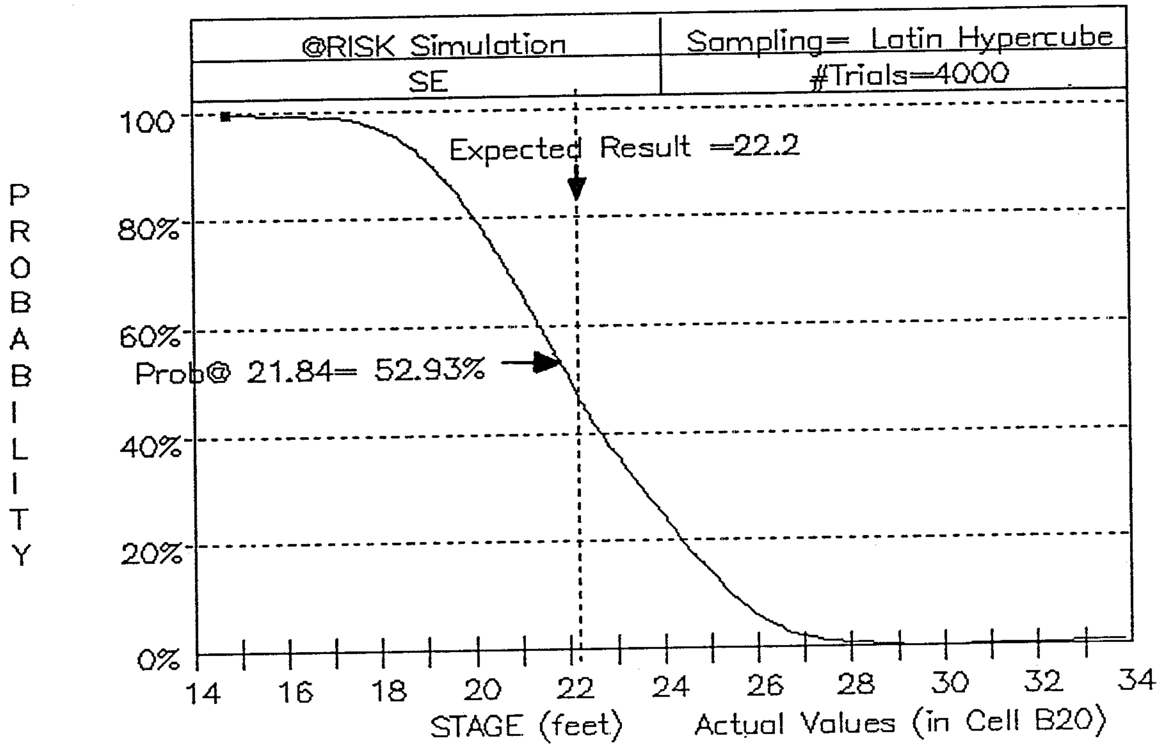
[3] Uncertainty in discharge/frequency according to Bulletin #17B (Water Resources Council, 1982); rating curve standard deviation of error = 1.5 feet; stage/damage standard deviation of error = 1.5 feet.

[4] Uncertainty in discharge/frequency according to Bulletin #17B (Water Resources Council, 1982); rating curve standard deviation of error = 2 feet; stage/damage standard deviation of error = 2 feet.

stage 22.2 feet indicates the expected result (mean value) for the simulation. The probability of exceedance for the design stage, which is about 21.84 feet (no expected probability) for the 100-year exceedance interval event, is about 53 percent. This would be consistent with a stage error that is normally distributed around the stage-discharge function while errors associated with discharge tend to be skewed to the higher, less frequent discharges. A statistical summary of the distribution is also shown in Chart 2.

A summary of the stage (SE) distributions for all three simulations is shown in Table 4. From this table one could look up a specific design stage and describe its reliability during an occurrence of the 100-year, 50-year or 25-year exceedance interval events. For example, a 100-year design stage of 21.8 feet has a 53% chance of being exceeded during a 100-year event, a 12% chance of exceedance during a 50-year event, and less than 1% chance of being exceeded in a 25-year event. Looking at the probability of non-exceedance (1-Probability of Exceedance), a 50-year design stage of 19.1 feet has a 10%, 48%, and 92% chance of keeping the 100-, 50-, and 25-year floods, respectively, within the levee. It can also be seen that by adding three feet of freeboard above the 100-year design stage (approx. 25 feet), exceedance frequency is reduced to approximately 14%, 1% and 0.4% for the 100-, 50-, and 25-year events respectively. It is interesting to note that the 100-year design stage using the conventional (expected probability) method with 3 feet of freeboard added (design stage 26.6 feet) has about a 95% chance that the 100-year event will stay within the design levee height. As a comparison, the proposed method design levee height of 27.0 feet (slightly higher) has about a 98% chance of passing the 100-year event, should it occur.

Example Probability Distribution For Stage Assuming Occurrence Of 100-Year Event



SE (in Cell B20)
@RISK Risk Analysis 23-Aug-1991
==============================
Expected/Mean Result = 22.18134
Maximum Result = 33.83104
Minimum Result = 14.78829
Range of Possible Results = 19.04275
Probability of Positive Result = 100%
Probability of Negative Result = 0%
Standard Deviation = 2.497021
Skewness = .2812773
Kurtosis = 3.094899
Variance = 6.235115
ERRs Calculated = 0
Values Filtered = 0
Simulations Executed = 1
Iterations = 4000

Percentile Probabilities:
(Chance of Result <= Shown Value)
(Actual Values)
==============================
<= 14.7883= 0%
<= 18.3486= 5%
<= 19.044 = 10%
<= 19.5685= 15%
<= 20.0162= 20%
<= 20.3701= 25%
<= 20.7263= 30%
<= 21.0626= 35%
<= 21.3725= 40%
<= 21.7114= 45%
<= 22.0201= 50%
<= 22.3254= 55%
<= 22.6847= 60%
<= 23.1041= 65%
<= 23.5301= 70%
<= 23.9796= 75%
<= 24.3959= 80%
<= 24.9224= 85%
<= 25.447 = 90%
<= 26.2346= 95%
<= 33.831 = 100%

Probabilities for Selected Values:
==============================
Probability of Result > 0 = 100%
> 13 = 100%
> 16 = 99.73%
> 19 = 90.6%
> 22 = 50.48%
> 25 = 14.1%
> 28 = 1.15%
> 31 = .2%
> 34 = 0%
Probability of Result <= 0 = 0%

Chart 2

Table 4
Reliability of Design Stage for Various Events[a]

Stage (ft)	Percent Chance Exceedance for Nominal 100-Year Event (Design Stage = 21.84)[b]	Percent Chance Exceedance for Nominal 50-Year Event (Design Stage = 19.10)[c]	Percent Chance Exceedance for 25-Year Event (Design Stage = 16.50)[d]
11			100
12			99.8
13		100	99
14		99.7	94
15	99.9	99	83
16	99.7	96	64
16.5	99.5	92	52
17	99	87	40
18	96	73	21
19	91	54	9
19.1	90	52	8
20	80	35	4
21	66	20	2
21.8	53	12	0.9
22	50	11	0.8
23	36	6	0.5
24	25	3	0.4
25	14	1	0.35
26	6	0.8	0.28
27	2	0.4	0.21
28	1	0.3	
29	0.7	0.1	
30	0.5		
31	0.4		
32	0.38		
33	0.33		
34	0.29		

[a] Probability distributions based on 4,000 iterations of Latin Hypercube Sampling using @RISK (Palisade Corporation, 1988).

[b] Given that the nominal 100-year event occurs, the probability that the stages in the first column will be exceeded due to uncertainties in frequency versus discharge, and discharge versus stage.

[c] Given that the nominal 50-year event occurs, the probability that the stages in the first column will be exceeded due to uncertainties in frequency versus discharge, and discharge versus stage.

[d] Given that the nominal 25-year event occurs, the probability that the stages in the first column will be exceeded due to uncertainties in frequency versus discharge, and discharge versus stage.

Risk and Uncertainty and the Economics of Levee Level of Protection and Freeboard

David A. Moser[1]

Introduction

The design and evaluation of engineering structures in water resources inherently requires extensive consideration of risk and uncertainty. This is especially true for flood control structures. The Corps of Engineers has long used frequency based methods in flood control benefit evaluation. The incorporation of risk, however, has generally been limited to representing peak annual discharge or flow by a frequency distribution such as log-Pearson Type-III.

There are numerous other relationships in the evaluation of a project that also exhibit risk and uncertainty. Indeed, the discharge-frequency function itself retains significant uncertainties due in part to the limited number of historical data points used in its derivation. Similarly, there is uncertainty in the discharge-stage relationship stemming from both model and parameter uncertainty. Along with the engineering related sources, there are also economics related sources of risk and uncertainty that should be assessed in the evaluation of any flood control project.

Traditionally, freeboard has had the status of a design standard and has been added to levees as part of the design process. The addition of freeboard to levees is an explicit recognition of the great uncertainties confronted in designing a flood control project. Typically, freeboard is thought of as the exclusive province of the design engineer.

This paper will discuss freeboard as an important planning issue for other, non-engineering reasons. The succeeding sections will also support the notion that freeboard should **not** be given special status except for specific considerations such as assuring a "safe fail." Rather a levee should be evaluated and the choice of scale recommended using a combined risk model with risk analysis methods.

Before proceeding, the following section provides an overview of the historic role of freeboard and its evaluation.

The Economic Evaluation of Freeboard

For the purposes of this paper the design height of a levee is the height that will contain the design flood. The estimated stage of this flood is used in the economic evaluation of the benefits from the project. Freeboard is the addition to the design height to insure against overtopping from the design flood. Many of the hydraulic and hydrologic sources of risk and uncertainty that comprise the estimate of flood stages have been used as arguments for the addition of freeboard to local protection projects. The table in the Appendix summarizes some of the reasons for freeboard that have been provided in Corps guidance

[1]Economist, Technical Analysis and Research Division, Institute for Water Resources, Fort Belvoir, VA

over the last 35 years. An examination of the Appendix indicates that the situations justifying the routine addition of freeboard have been significantly reduced as more sophisticated models have been developed.

The preceding description differentiates the design flow water surface elevation from the top of the levee in project evaluation by the amount of freeboard. The design flow is used for economic evaluation of flood damages and project benefits and is subject to selection by economic efficiency criteria. The amount of freeboard and the top of the levee, however, seems to be based on risk management on the part of the technical analysts irrespective of its contribution to benefits or costs. The recognition that "freeboard is not free" and the asymmetrical treatment of freeboard as a separate reliability issue is at least acknowledged under current guidance.[2] ER 1105-2-100 states that:

> "In view of the complexity of assessing the ability of a project to pass floods greater that the design flood, a simple method which is acceptable in the absence of better data is to claim one-half the area under the frequency-damage curve between the design level of protection and the largest flood which might be carried with the freeboard."

Typically this guidance is implemented by calculating 1) the expected annual damages from any flow in excess of the design flow and 2) the expected annual damages with no damages from any flow equal to or less than the maximum flow that could be safely passed. The difference between these two values is the expected annual damage in the freeboard range. One-half of these damages are added to the project benefits for the levee design height.

Figure 1 shows this assessment of benefits. Distance A represents the amount of freeboard at the indicated design height. Distance B represents the expected benefits in the freeboard range. The solid curve shows the expected benefit including one-half of the benefits in the freeboard range as a function of design height. This procedure basically assumes that there is a 100 percent chance that the levee will contain the design flow and a zero percent chance that a flow resulting in a water surface elevation equal to or greater than the freeboard will be contained. It must be noted, however, that ER 1105-2-100 adds: "other methods which use project specific data and probability relationships are encouraged and may be employed if documented."

Note that the current approach treats the frequency curve, the rating curve, and the stage damage curve as deterministic functions. Thus, expected values are calculated based only on the frequency distribution of floods.

The effect of incorporating the benefits in the freeboard range changes the NED project selected on the basis of maximizing expected net benefits. Figure 2 shows that the addition of benefits in the freeboard range reduces expected marginal benefits at each levee height. Since freeboard costs are already included in project costs, marginal NED costs are unchanged. The economically efficient scale of project is reduced since marginal costs must be lowered to equal the reduced expected marginal benefits. The inclusion of benefits in the freeboard range effectively transfers some benefits to the lower levee that were previously attributed to a higher levee.

---

[2]Freeboard is one of the most expensive components of a levee since, in terms of construction, it is added to the bottom of the levee.

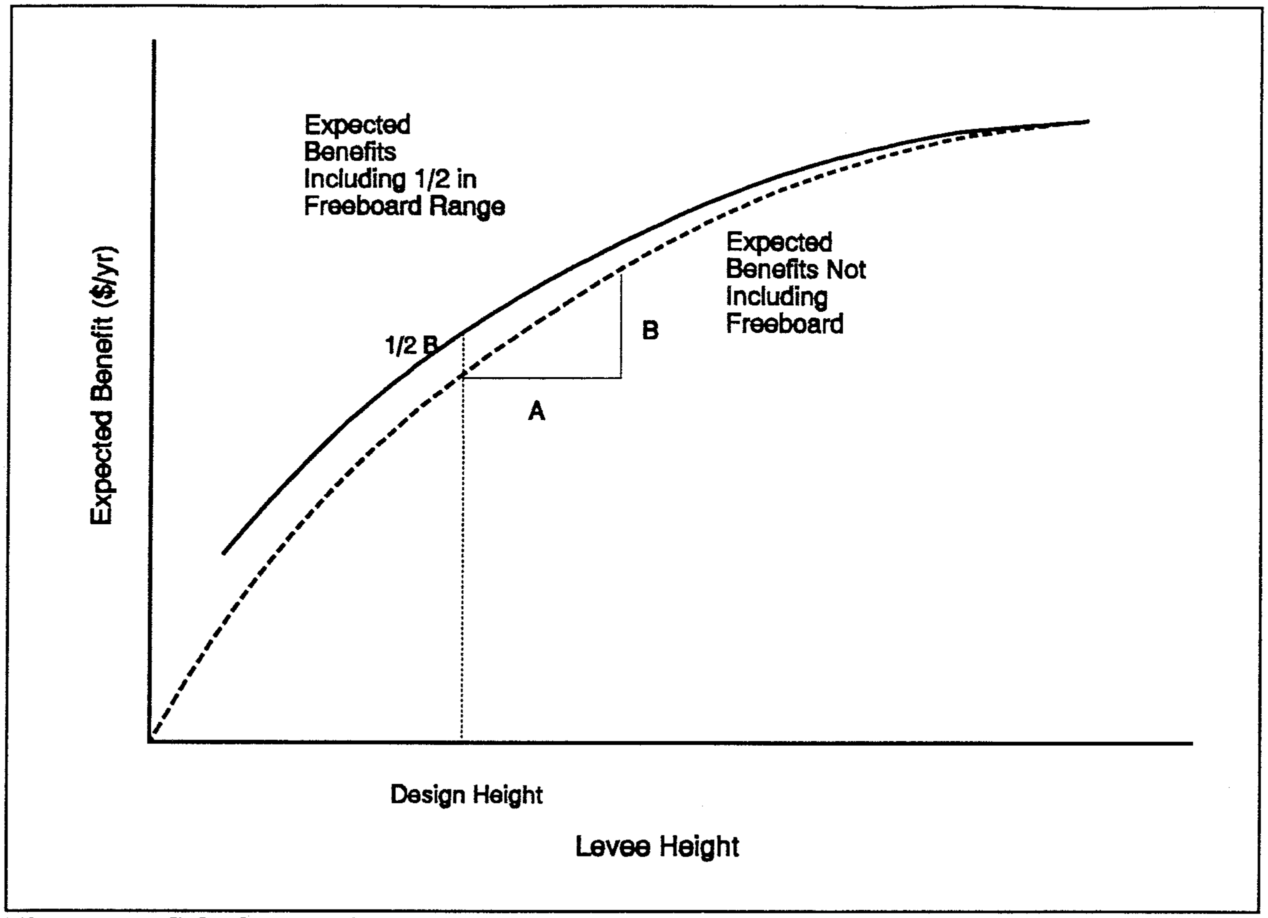


Figure 1: Calculation of Benefits in Freeboard Range

Economics in a Combined Risk Model for Levee Evaluation

As described above there are a variety of sources of risk and uncertainty that can influence the evaluation of a levee project. Many of these arise in the determination of the discharge-frequency and the discharge-stage functions. In addition, relationships that enter into the estimate of flood damages and benefits also contain elements of risk and uncertainty. Table 1 displays some of these sources of risk and uncertainty and potential methods for estimating the risk. Many of the sources could rely on statistical relationships estimated from existing databases or sample data. In all instances expert judgement could be used to provide bounds on possible outcomes and approximate likelihoods.[3]

A complete model incorporating risk and uncertainty in all the sources of risk and uncertainty could be developed and used in the calculation of expected damage reduction and benefits. Until recently the extent of risk analysis applications in the economics of flood control evaluation has been the use of limited sensitivity analysis. Even then risk based approaches have had limited use. Advances in microcomputer computational power and speed now mean that sophisticated modeling and simulation capabilities are easily accessible

[3]The Institute for Water Resources is currently investigating both local sample data and FIA National Flood Insurance claims data to develop statistics based estimates of risk in economic model parameters.

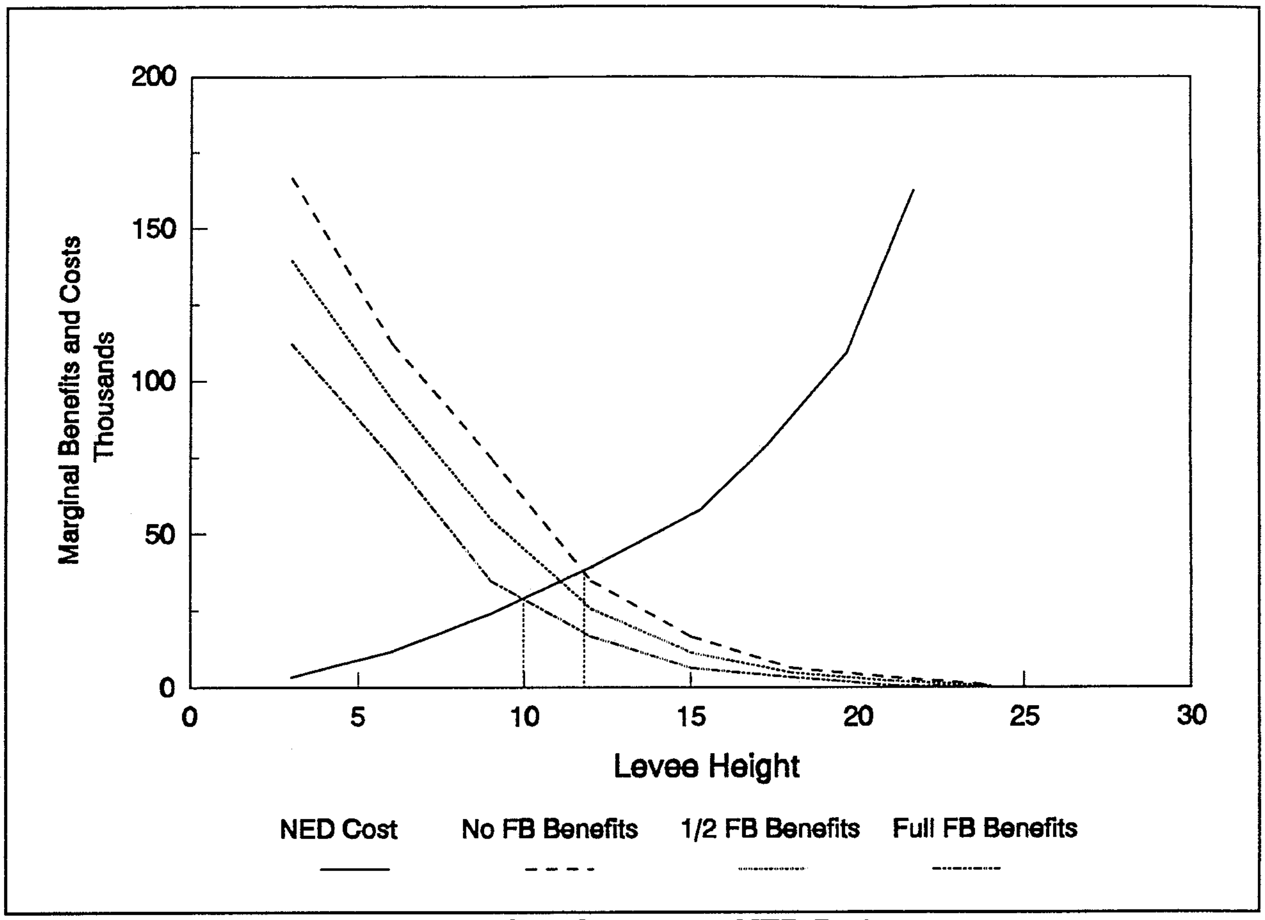


Figure 2: Effect of Benefits in Freeboard Range on NED Project

to planners.

Figure 3 presents a graphical illustration of an economics module in the combined risk model for evaluating levee height. An efficient approach to implementation is to start with a model that calculates expected damages and project benefits. Instead of the usual deterministic assumptions about various model parameters, underlying probability distributions could be entered and a Monte Carlo simulation approach used. For each iteration during the simulation, each of the distributions is randomly sampled and expected damages calculated. Benefits would be calculated by estimating expected damages with a levee of a given height for each simulation. A particular simulation would typically be composed of several thousand iterations with the number depending on the desired accuracy in the output distribution and on the number of independent risk sources. An interesting aspect of this approach is that the variance in the distribution of benefits may not be as great as first presumed. The likelihood that an iteration will returns values simultaneously from all sources of risk that result in extremes is small.

The combined risk model does not assess the risk and uncertainty from the engineering aspects separately from the economic aspects. Instead the approach would necessitate a close cooperation between the technical analysts in these areas.

Table 1: Sources and Methods for Estimating Economic Risk and Uncertainty

Risk and Uncertainty Element	Method for Estimating Risk and Uncertainty
Structure Value	Sample Statistics from Project Area Expert Judgment
Content Value	Statistics for Content to Structure Ratio Estimate Expert Judgment
Depth-Percent Damage for Structure	Statistics for Depth-Damage Function Expert Judgment
Depth-Percent Damage for Content	Statistics for Depth-Damage Function Expert Judgment
First Floor Elevation	Expert Judgement
Building Type	Sample Statistics for Project Area Survey Expert Judgment
Warning Time	Expert Judgment
Evacuation Effectiveness	Expert Judgment

Figure 4 shows what the results of such a approach might yield in terms of an expected benefit function. Notice that there is a distribution of benefits at each levee height based on the risk and uncertainty information in the risk model. Notice also that the variance in benefits is shown to decline with increases in levee height. The presumption is that although the accuracy of the frequency curve, rating curve, and the damage function may be much lower for these events, the overall contribution to expected value calculations may be small.[4]

In the combined risk model method to levee evaluation summarized by Figure 4, freeboard would not be added. Even so, the levee height that maximizes expected net benefits using this approach might not be the appropriate design given residual risk to people and property. Throughout the analysis it is assumed that individuals are risk neutral. This implies that the largest **certain** payment people are willing to pay for flood protection is the expected value of their losses. Therefore, at the economically efficient levee height expected marginal benefits equal expected marginal costs.

The NED analysis ignores 1) the possibility that people are risk averse and 2) human health and safety effects. Allowing aversion to risk implies that people are willing to pay

[4]This presumption has not been verified.

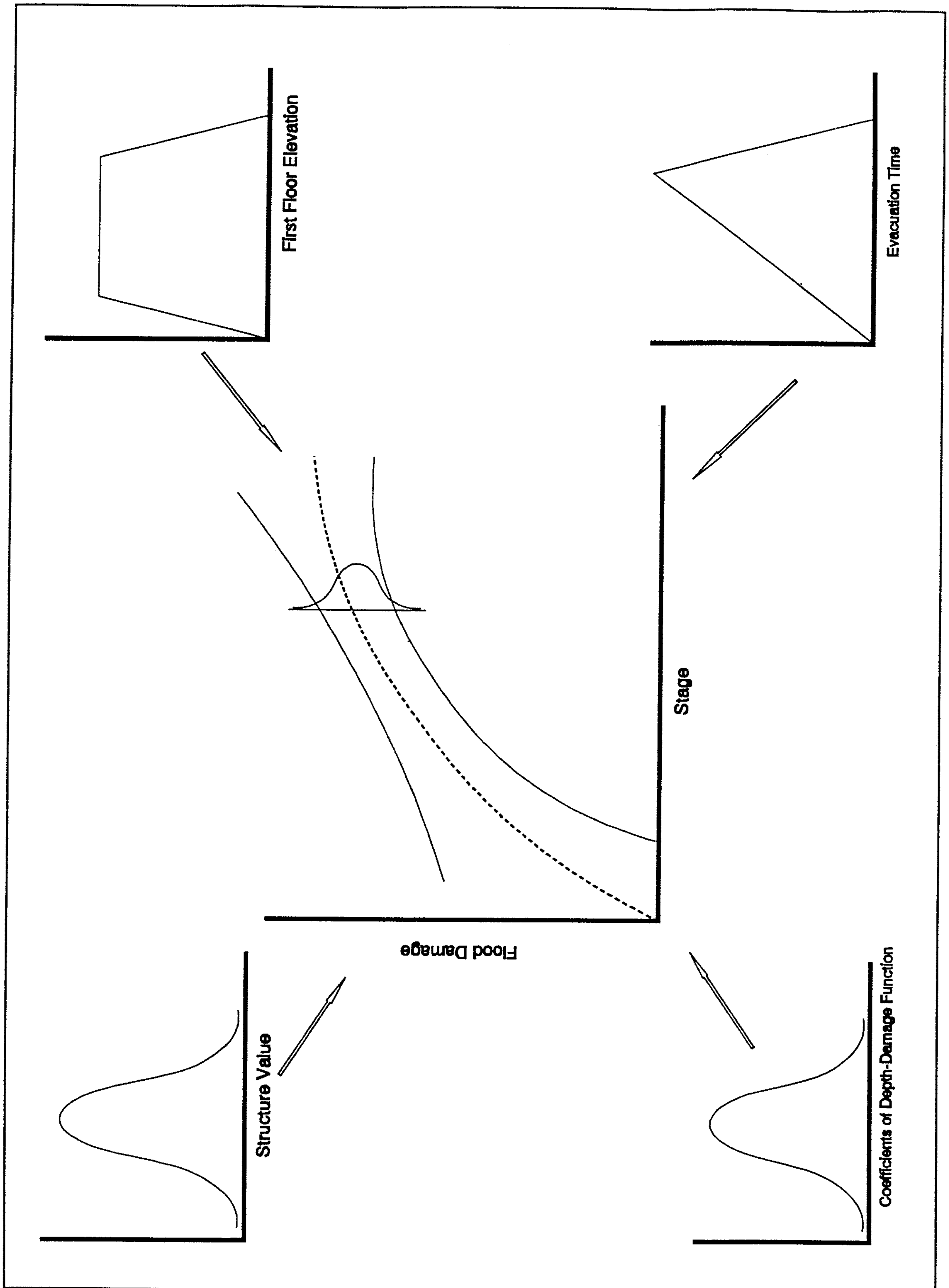


Figure 3: Representation of Components of Economics Module for Combined Risk

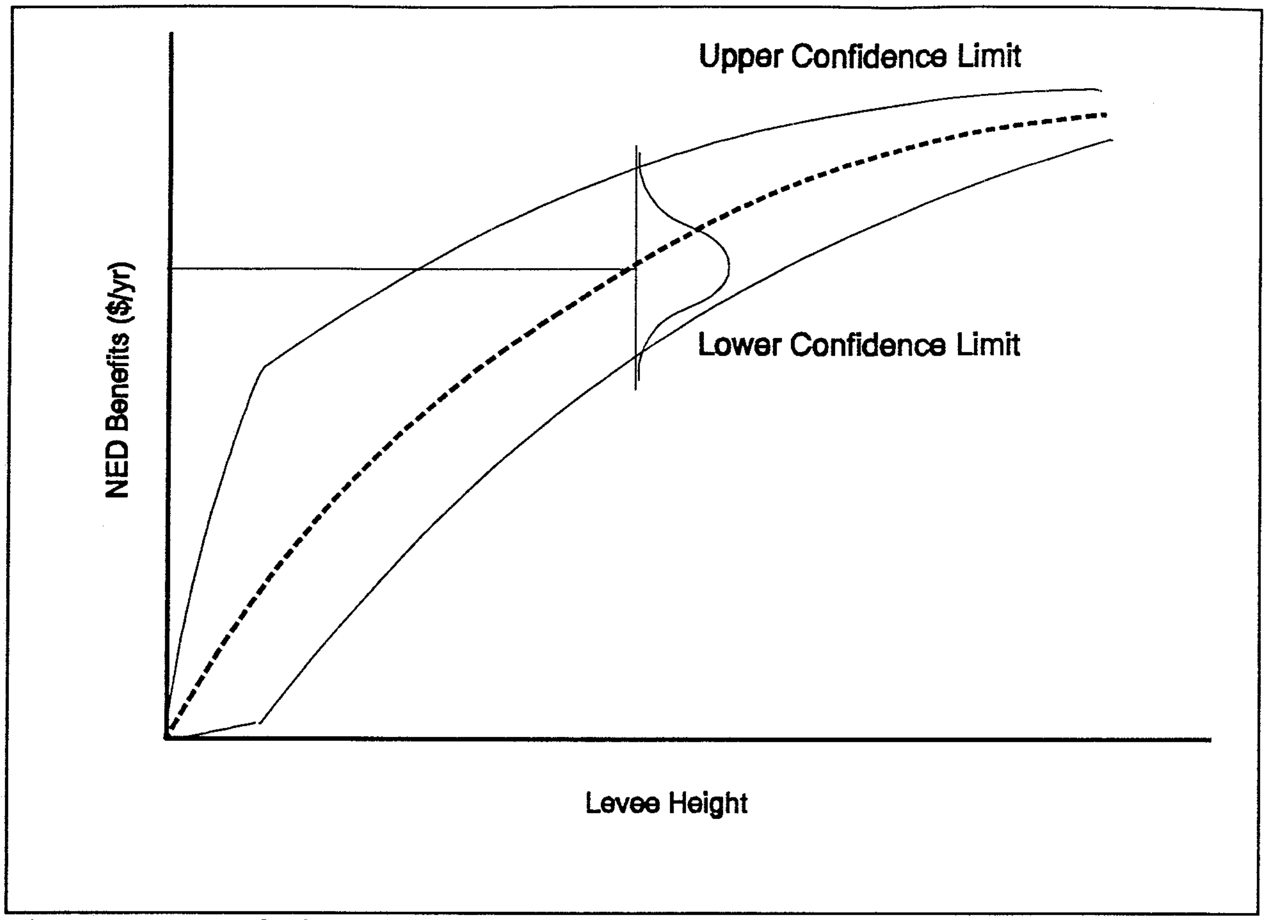


Figure 4: Example Output From Combined Risk Model

more than their expected losses to avoid them. Correspondingly, people may be willing to accept increased project costs in order to obtain increased reliability, and to avoid low probability/high consequence risk especially to the resident population. Risk-based decision making in this context means that the scale of project consistent with maximizing expected net benefits provides the baseline for assessing alternative risk management strategies. These strategies are designed to improve the reliability of project performance and to manage the residual risk to people and property. The basic philosophy in this approach is that the decision about risk management is not made solely by the technical analyst but is shared with those who bear the risk, both physically and financially. In this approach, however, there are no rules for balancing costs against risk. Therefore, risk and uncertainty information on the engineering and financial performance of the scales of projects considered needs to be preserved in the study reports and carried forward to allow an **informed-consent decision** and to become part of the formal record of that decision.[5]

[5]See Leonard Shabman, "Risk-based Decision Making and the Corps of Engineers Planner," in Risk Analysis for Water Resources Planning, Prospect Course Book, IWR, 1990, pp.172-185.

Appendix: Freeboard in Corps Guidance

Reference:	Freeboard Consideration
Levee Freeboard Design: Conference in MRD, 16 March 1953	Objective of Freeboard: a. To insure integrity of protection against design discharges b. To design the freeboard to minimize the possibilities of "chain" failure of levees protecting adjacent units c. Minimize damage to units subjected to levee overtopping or failure by preventing entrance of water at upstream end.
CW Engineer Bulletin 54-14, 23 April 1954	In general, freeboard allowances should be considered as providing only for those factors that cannot be rationally accounted for in design flood profile computations. Such freeboard allowances may be considered as consisting of the following components, insofar as these have not been specifically accounted for in the hydraulic computations involved in the design flood profile estimate: a. in the event of overtopping, initial overflow will occur over the downstream portion of the mainstem levee b. error in profile computations attributable to inadequacies of procedures, basic data, etc. c. dynamic effects and short-period fluctuations not specifically accounted for in computing the design flood profile d. temporary changes in stage-discharge relationship resulting from sedimentation, erosion, etc. e. flow retardance by debris and ice flows

Reference:	Freeboard Consideration
EP 1165-2-1, <u>Digest of Water Resources Policies</u>, 12 Feb 1989: Chapter 13	13-5. <u>Freeboard Allowance</u>. Freeboard is the marginal height provided, above design flow lines, on levees and in certain channels, to insure, as fully as practicable, against overtopping due to uncertainties in the state of project maintenance or flood flow characteristics. In appropriate circumstances, special increments of levee freeboard may be provided to achieve design objectives (e.g. to control, in such an extremity, the location where initial overtopping of a levee would take place; to reduce wave overtopping; to extend the interval between major maintenance efforts for removal of tree growth, sediment deposition, etc., from the channel the levee bounds.) Added height to earth levees is sometimes provided to allow for settlement. In project evaluation, one-half of the inundation reduction benefits creditable to the levee freeboard zone may be included.

Reference:	Freeboard Consideration
EM 1110-2-2502 <u>Retaining and Flood Walls,</u> 29 Sep 1989: Chapter 7	7-2.a. (2) Factors that influence the water surface profile and level of protection, and that can reasonably be quantified, are included in the design water level; not the freeboard. Some examples of these factors are: (a) Changed conveyance, due to changing bed form, sedimentation or scour, and vegetation growth or removal. (b) Dynamic surges and super elevation. (c) Ice, debris, and local anomalies. (d) Transverse slope due to water flowing out of or into the channel or differences in velocity head between the channel and overbank locations. (e) Profile instabilities associated with braids, meanders, etc. (f) Energy losses due to changing flow area, e.g. constrictions, abrupt expansions, and bends. (g) Future changes in flood flows due to changes in the watershed. 7-2.b (1) Examples of (freeboard) design objectives: (a) Assurance of initial overtopping at the most desirable (least hazardous) location. (b) Reduced volume of wave overtopping. (c) Extension of interval between major maintenance such as removal of sediment deposition. (2) Freeboard allowances for water surface uncertainty are allowances that are not otherwise specifically accounted for because they are considered too small to require specific determination or because they are too intractable to be quantified.

PARTICIPANTS

Hydrology and Hydraulics Workshop
on
Riverine Levee Freeboard

Workshop Participants
27 - 29 August 1991

Mike Burnham (CEWRC-HEC-P)
Hydrologic Engineering Center
609 Second Street
Davis, CA 95616
(916) 756-1104

Les Cunningham (CENPW-PL-H)
U.S. Army Engr. Dist., Walla Walla
Bldg. 602, City-County Airport
Walla Walla, WA 99362-9265
(509) 522-6615

Bob Daniel (CECW-PD)
HQ, U.S. Army Corps of Engineers
20 Massachusetts Avenue NW
Washington, DC 20314-1000
(202) 272-8568

John D'Antonio, Jr (CESWA-ED-PM)
U.S. Army Engr. Dist., Albuquerque
P.O. Box 1580
Albuquerque, MN 87103-1580
(505) 766-2635

Darryl Davis (CEWRC-HEC-E)
Hydrologic Engineering Center
609 Second Street
Davis, CA 95616
(916) 756-1104

Darryl Dolanski (CENCE-ED-L)
U.S. Army Engr. Dist., Detroit
P.O. Box 1027
Detroit, MI 48231-1027
(313) 226-4761

Harry Dotson (CEWRC-HEC-P)
Hydrologic Engineering Center
609 Second Street
Davis, CA 95616
(916) 756-1104

Gary Dyhouse (CELMS-ED-HE)
U.S. Army Engr. Dist., St. Louis
210 Tucker Blvd. N.
St. Louis, MO 63101-1986
(314) 331-8362

Earl Eiker (CECW-EH)
HQ, U.S. Army Corps of Engineers
20 Massachusetts Avenue NW
Washington, DC 20314-1000
(202) 272-8500

Robert Engelstad (CENCS-ED-GH)
U.S. Army Engr. Dist., St. Paul
1421 USPO & Custom House
St. Paul, MN 55101-1479
(612) 220-0610

Robert Fitzgerald (CELMK-ED-H)
U.S. Army Engr. Dist., Vicksburg
P.O. Box 60
Vicksburg, MS 38180-0060
(601) 631-5655

Pat Foley (CENCS-ED-GH)
U.S. Army Engr. Dist., St. Paul
1421 USPO & Custom House
St. Paul, MN 55101-1479
(612) 220-0630

Ken Halstead (CEORH-ED-HO)
U.S. Army Engr. Dist., Huntington
502 Eighth Street
Huntington, WV 25701-2070
(304) 529-5606

Ron Hilton (CESAJ-EN-H)
U.S. Army Engr. Dist., Jacksonville
P.O. Box 4970
Jacksonville, FL 32232-0019
(904) 791-2105

Joel James (CESAS-EN-HA)
U.S. Army Engr. Dist., Savannah
P.O. Box 889
Savannah, GA 31402-0889
(912) 944-5513

Jim Lencioni (CENPS-EN-HH-HF)
U.S. Army Engr. Dist., Seattle
P.O. Box C-3755
Seattle, WA 98124-2255
(206) 764-3595

Robert MacArthur
Water Engineering and Technology, Inc.
621 4th Street, Suite 1
P.O. Box 5000
Davis, CA 95616
(916) 758-3611

John Matticks
Assistance Administrator
Office of Risk Assessment, Federal Insurance Administration
Federal Emergency Management Agency
Washington, DC 20472
(202) 646-2768

Jim Mazanec (CENCD-ED-W)
U.S. Army Engr. Div., North Central
111 N Canal Street, 14th Floor
Chicago, IL 60606-7206
(312) 353-7132

Jeffrey McClenathan (CEMRO-ED-HD)
U.S. Army Engr. Dist., Omaha
215 North 17th Street
Omaha, NE 68102-4978
(402) 221-4578

Dave Moser (CEWRC-IWR)
Institute for Water Resources
Casey Building #2594
Ft. Belvoir, VA 22060-5586
(703) 355-2217

Tom Munsey (CECW-EH-D)
HQ, U.S. Army Corps of Engineers
20 Massachusetts Avenue NW
Washington, DC 20314-1000
(202) 272-8504

Dennis Seibel (CENAB-EN-RH)
U.S. Army Engr. Dist., Baltimore
P.O. Box 1715
Baltimore, MD 21203-1715
(301) 962-4840

Lew Smith (CECW-EH-Y)
HQ, U.S. Army Corps of Engineers
20 Massachusetts Avenue NW
Washington, DC 20314-1000
(202) 272-8506

Tony Thomas (CEWES-HR)
Waterways Experiment Station
P.O. Box 631
Vicksburg, MS 39181-0631
(601) 634-2511

Brian Tracy (CESPC-ED-HH)
U.S. Army Engr. Dist., Los Angeles
P.O. Box 2711
Los Angeles, CA 90053-2325
(213) 894-5524

Ming Tseng (CECW-EH-W)
HQ, U.S. Army Corps of Engineers
20 Massachusetts Avenue NW
Washington, DC 20314-1000
(202) 272-8509

Ken Zwickl (CECW-PF)
HQ, U.S. Army Corps of Engineers
20 Massachusetts Avenue NW
Washington, DC 20314-1000
(202) 272-0169

AGENDA

HYDROLOGY AND HYDRAULICS WORKSHOP ON RIVERINE LEVEE FREEBOARD

27 - 29 August 1991

Tuesday, 27 August 1991

Time	Description
8:00 - 8:05 am	**Introductions** (Robert Engelstad, Chief, Hydrology Section, Geotech/H&H Branch, St. Paul District)
8:05 - 8:15 am	**Welcome** (Helmer (Bud) Johnson, Chief, Geotech/H&H Branch, St. Paul District)
8:15 - 8:20 am	**Comments/Introductions** (Earl Eiker, Chief, H&H Branch, HQUSACE)
8:20 - 8:30 am	**Introduction of Technical Program** (Michael Burnham, Chief, Planning Analysis Division, HEC)

Session 1: Levee Freeboard Policy

8:30 - 9:15 am	**Paper 1**	Freeboard Design for Urban Levees and Floodwalls (Earl Eiker, HQUSACE)
9:15 - 10:00 am	**Paper 2**	Levee Freeboard - Deja Vu All Over Again (Bob Daniel, HQUSACE)
10:00 - 10:15 am	Break	
10:15 - 11:00 am	**Paper 3**	FEMA Levee Accreditation Procedures (John Matticks, Asst. Administrator, Office of Risk Assessment, Federal Insurance Administration, FEMA)
11:00 - 12:00 pm	**Panel 1**	Policy Issues
	A.	Freeboard Requirements for Low Levees (Jim Mazanec, North Central Division)
	B.	Levee Freeboard Policy Issues Impacting on Corps Support to the National Flood Insurance Program (Ken Zwickl, HQUSACE)

C. Freeboard, Overtopping and Safety for Levees with Low Levels of Protection (Lew Smith, HQUSACE)

D. Flood Overtopping of Levees - Intent and Implications of Current Design Guidance and Possible Modifications (Tom Munsey, HQUSACE)

12:00 - 1:15 pm Lunch

Session 2: Part 1 - Levee Project Evaluation and Performance

1:15 - 2:00 pm **Paper 4** Lower Santa Anna River Levee and Freeboard Design (Brian Tracy, Los Angeles)

2:00 - 2:45 pm **Paper 5** Levee Freeboard Design for Fort Wayne, Indiana (Darryl Dolanski, Detroit District)

2:45 - 3:00 pm Break

3:00 - 3:45 pm **Paper 6** High Velocity Leveed Channels - Puerto Rico (Ron Hilton, Jacksonville District)

3:45 - 4:45 pm **Panel 2** Levee Project Evaluation and Performance

A. Floodplain Encroachment and Effect on Levee Overtopping Design (Joel James, Savannah District)

B. Levee Freeboard Issues in Omaha District (Jeffrey McClenathan, Omaha District)

C. Unique Freeboard Situations in Seattle District (Jim Lencioni, Seattle District)

D. Levee Freeboard Issues Based on St. Louis District Experience (Gary Dyhouse, St. Louis District)

Wednesday, 28 August 1991

Session 3: Part 2 - Levee Project Evaluation and Performance

Time	Description	
8:00 - 8:45 am	**Paper 7**	Wyoming Valley Levee Freeboard Design (Dennis Seibel, Baltimore District)
8:45 - 9:30 am	**Paper 8**	Levee Freeboard Design in Walla Walla District (Les Cunningham, Walla Walla District)
9:30 - 10:00 am	Break	
10:00 - 10:45 am	**Paper 9**	St. Paul District Experience with Credit to Existing Levees (Pat Foley, St. Paul District)
10:45 - 11:45 am	**Panel 3**	Levee Project Evaluation and Performance
	A.	Levee Freeboard Design for West Columbus, Ohio (Ken Halstead, Huntington District)
	B.	Levee Freeboard Issues in the Vicksburg District (Bob Fitzgerald, Vicksburg District)
	C.	Levee Freeboard Issues - Rio Grande at Alamosa, Colorado (John D'Antonio, Albuquerque District)
	D.	Effect of Channel Erosion on Freeboard at Houston, Minnesota (Robert Engelstad, St. Paul District)
11:45 - 1:00 pm	Lunch	
1:00 - 5:00 pm	Informal Group Discussions	

Evening Session:

7:00 - 7:45 pm	**Paper 10**	Hydraulic Uncertainties in Water Surface Calculations (Tony Thomas, WES)
7:45 - 8:30 pm	**Paper 11**	International Survey of Levee Freeboard Design Procedures (Robert MacArthur, Water Engineering and Technology, Inc.)

Thursday, 29 August 1991

Session 4: Proposed Strategies for Sizing Levees Considering Risk and Uncertainty

Time	Description
8:00 - 8:30 am	**Introductory Remarks** (Earl Eiker, HQUSACE)
8:30 - 9:15 am	**Paper 12** A Risk and Uncertainty Based Concept for Sizing Levee Projects (Darryl Davis, HEC)
9:15 - 10:00 am	**Paper 13** Risk, Uncertainty and the Economics of Levee Level of Protection and Freeboard (Dave Moser, IWR)
10:00 - 10:30 am	Break
10:30 - 11:30 pm	**Open Discussion** (Earl Eiker, Bob Daniel, HQUSACE)
11:30 - 12:00 pm	**Closure** (Earl Eiker, HQUSACE, Michael Burnham, HEC)
12:00	**Adjourn**