DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

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Engineering and Design

USACE PROCESS FOR THE NATIONAL FLOOD INSURANCE PROGRAM (NFIP) LEVEE SYSTEM EVALUATION

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USACE PROCESS FOR THE NATIONAL FLOOD INSURANCE PROGRAM (NFIP) LEVEE SYSTEM EVALUATION

1. Purpose. This document is applicable for all U.S. Army Corps of Engineers (USACE) riverine, lake, and coastal National Flood Insurance Program (NFIP) levee system evaluations. The purpose is to provide a consolidated document that will guide USACE procedures for levee system evaluations in support of the NFIP as administered by the Federal Emergency Management Agency (FEMA). This Engineer Circular (EC) will supplement and clarify existing policy, procedural, and technical guidance and provide an overview of documentation requirements. Technical and procedural guidance in this EC are intended solely for use in USACE process for NFIP levee system evaluation of existing and new levee systems; it is not intended as design guidance.

2. Applicability. This EC applies to all USACE commands having civil works responsibilities and to military installations. It applies to existing and new levee systems.

3. Distribution Statement. Approved for public release; distribution is unlimited.

4. References. References are presented in Appendix A.

5. Background.

a. Mapping for NFIP Purposes. The U.S. Department of Homeland Security's FEMA is the federal agency responsible for administering the NFIP. As part of the NFIP, FEMA develops Flood Insurance Rate Maps (FIRMs) to identify areas that may be subject to flooding, for both determining flood insurance rates and flood plain management activities. Floodplain maps have been published by FEMA since the beginning of the NFIP. Starting in 2003, FEMA embarked on a nationwide program called the Flood Map Modernization (Map Mod) Program. Through Phase 1 of the Map Mod Program, FEMA provided digital flood hazard data and maps known as Digital Flood Insurance Rate Maps (DFIRMs), which are more reliable, easy to use, and readily available than the previous hardcopy FIRMs for 92% of the population (65% of the continental US land area). As part of this process, FEMA worked with federal, state, and local agencies to ensure that the most up-to-date information possible is incorporated into this new digital product. FEMA recognized that many levees may have changed considerably or deteriorated since the previous effective maps were published. As part of the remapping process, FEMA is verifying that all levees recognized on previous FIRMs meet the requirements outlined in Title 44 of the Code of Federal Regulations, Section 65.10 (44 CFR 65.10), *Mapping Areas Protected by Levee*

Systems. Title 44 CFR 65.10 requires that specific structural requirements must be certified by a registered professional engineer or a federal agency with responsibility for levee design, such as USACE, that the levee has been adequately designed and constructed to provide reasonable assurance of excluding the base flood (as defined in 44 CFR 59.1) from the leveed area and thus meet NFIP levee system evaluation requirements. Should USACE be requested to provide an NFIP levee system evaluation, USACE will review all components of the entire levee system as outlined in this EC, not only design and construction issues as noted in the CFR.

b. <u>NFIP Levee System Evaluation</u>. The purpose of an NFIP levee system evaluation is to determine how flood hazard areas behind levees are mapped on FIRMs. A levee is a man-made structure, usually an earthen embankment or concrete floodwall, designed and constructed in accordance with sound engineering practices to contain, control, or divert the flow of water so as to provide reasonable assurance of excluding temporary flooding (as defined in 44 CFR 59.1) from the leveed area. A levee system consists of a levee or levees, and associated structures, such as closure and drainage devices, which are constructed and operated in accordance with sound engineering practices to provide reasonable assurance of excluding flood water from an associated separable floodplain (and further defined in 44 CFR 59.1). The resultant maps are used to determine flood insurance rates; federal, state, and local floodplain management requirements; and other floodplain management decisions. It should be noted here that the definition of 'NFIP levee system evaluation' for the purposes of USACE application under this EC is consistent with definitions in 44 CFR 65 as described in subsequent paragraphs. If a positive finding is made in an NFIP levee system evaluation, FEMA will use this information to determine how the floodplain behind the levee system is mapped.

(1) USACE Use of *NFIP Levee System Evaluation* Rather Than *Certification.* USACE has chosen to use the phrase 'NFIP levee system evaluation' to emphasize that EC 1110-2-6067 is focused on the complete levee system's status with regard to requirements of both 44 CFR 65.10 and USACE guidelines. This choice of terminology better respects the 44 CFR 65 definition of certification in Part 65.2(b) – "For the purpose of this part, a certification by a registered professional engineer or other party does not constitute a warranty or guarantee of performance, expressed or implied. Certification of data is a statement that the data is accurate to the best of the certifier's knowledge. Certification of analyses is a statement that the analyses have been performed correctly and in accordance with sound engineering practices. Certification of structural works is a statement that the works are designed in accordance with sound engineering practices to provide protection from the base flood. Certification of "as built" conditions is a statement that the structure(s) has been built according to the plans being certified, is in place, and is fully functioning." This definition extends to, but may not be limited to, the analysis and data submitted in support of Part 65.6 and in support of the structural requirements for paragraphs (b).(1) through (7) of Part 65.10. This change in terminology does not affect the FEMA process or requirements for the NFIP.

(2) NFIP levee system evaluation determination by USACE is a technical finding that, for the floodplain in question, there is a reasonable assurance (to be defined in paragraph 9.f. (1)) that the levee system will exclude the 1% annual chance exceedance flood (or base flood) from the leveed area based on the condition of the system at the time the determination is made. NFIP levee system evaluation only addresses the levee system with regard to the 1% annual chance exceedance flood. If a levee meets NFIP levee system evaluation requirements, it may be

'accredited' by FEMA and the area behind the levee thus mapped on the FIRM in accordance with 44 CFR 65.10. If a levee is not found to meet the requirements of NFIP levee system evaluation, the area behind the levee that is subject to inundation by the base flood could be mapped as a high-risk area (or Special Flood Hazard Area) on the FIRM. The USACE use of the phrase 'NFIP levee system evaluation' reflects the requirements of the 44 CFR 65.2, and extends the coverage beyond paragraphs (b) (1) through (7) to include paragraphs (c) and (d) of Part 65.10 and the additional items as listed in paragraph 9.a. of this EC, which include analysis of capacity exceedance, and residual risk and public safety. These items are included in the USACE NFIP levee system evaluation as they are integral, and typically devised in concert with the intended design objectives and the proper operation of the system.

c. AR Zone Designation Requests. Under the NFIP, a community can request a Flood Protection Restoration Designation (also termed AR zone), which requires minimum floodplain management requirements; while a levee system is in the process of being restored. Requirements for restoration zones can be found in 44 CFR 65.14, *Remapping of areas for which local flood protection systems no longer provides base flood protection*. This requires verification by a federal agency responsible for design or construction that the existing levee system shown on the effective FIRM was originally built using federal funds, that there is no reasonable assurance of excluding the base flood event from the leveed area, but there is reasonable assurance of excluding at least a 3% annual chance exceedance flood from the leveed area. For requests to USACE to provide NFIP levee system evaluation for an AR zone designation, USACE will follow procedures in this EC using the target event for evaluation as the 3% annual chance exceedance flood.

d. A Note on Risk Terminology. The term 'risk' is used among professionals in various fields to convey a variety of concepts ranging from probability or chance of occurrence or exceedance, to uncertainty, to probability of occurrence or exceedance and consequences, and sometimes all of these. Consistent with USACE agency-wide changes, the definition in the glossary "**Risk –** Measure of the probability and severity of undesirable consequences" is used for this document. In USACE dam safety policies and other USACE technical engineering guidance, 'risk' is used consistent with this definition, sometimes expressed as a probability-consequence diagram. Please note, however, that various USACE official guidance documents and policy letters have 'risk' in their titles, and that these documents often address only probability of occurrence or uncertainty. These documents will be corrected as they are updated and revised. For this document, where 'risk' might have come from a document or common vocabulary usage, 'probability of occurrence,' 'exceedance probability,' or 'uncertainty' is substituted to ensure consistency. Titles of existing official documents that may use 'risk' inconsistent with the definition, however, are not changed.

6. Policy Guidance.

a. Policy History Overview. USACE has issued several policy guidance letters, described below, specifically addressing NFIP levee system evaluation. In the mid-1990s, USACE adopted a risk analysis approach for flood damage reduction project development. That policy, Engineer Regulation (ER) 1105-2-101, *Risk Analysis for Flood Damage Reduction Studies*, which emphasized risk-informed decision making, full disclosure of uncertainties, and residual risk and public safety, was updated in January 2006, continuing and strengthening the principles previously put forth. In April 1997, the policy guidance letter, *Guidance on Levee Certification for the National Flood Insurance Program*, was issued to ensure consistency throughout USACE with the application of the policy to perform NFIP levee system evaluations. This letter was updated and reissued with the policy letter *Guidance on Levee Certification for the National Flood Insurance Program – FEMA Map Modernization Program Issues,* dated June 23, 2006. This updated letter and attachments describe USACE policy in the area of freeboard criteria by providing a performance target that is statistically based, reflecting stream profile variability and uncertainty. A second policy letter, *Geotechnical Activities in Support of Levee Certification for Federal Emergency Management Agency (FEMA) Flood Insurance Purposes*, dated June 20, 1997, established that geotechnical analysis for NFIP levee system evaluation would continue to be based on traditional deterministic analysis. In the future, there will be a transition from a solely deterministic approach to an integrated approach combining widely understood and accepted deterministic assessment procedures with appropriate risk analysis methodology once the risk approach is developed and finalized. A third policy guidance letter, *Authority and Funding Guidance for USACE Levee Certification Activities*, dated August 15, 2006, outlined current authorities, programs, and funding mechanisms, which are applicable to USACE in relation to performing or supporting NFIP levee system evaluations. This EC supersedes all previous policies specific to NFIP levee system evaluation, previously referred to in policy as levee certification.

b. <u>Authority and Funding for NFIP Levee System Evaluations</u>. USACE policy is a NFIP levee system evaluation is the responsibility of the local levee sponsor or community seeking recognition of the levee system on the FIRM. In limited cases as listed below, USACE may perform the NFIP levee system evaluation.

(1) Upon request by the local community, USACE may perform NFIP levee system evaluation for federally authorized levees that USACE operates and maintains (or has major maintenance responsibilities) and may be funded from project appropriated funds if available. Requests for funding to perform the NFIP Levee System Evaluation shall be made through the budget process for O&M activities. Project funds shall only be used upon full agreement of district project review board after defining scope, duration, and product required. Any reprogramming requests for NFIP levee system evaluations not falling under normal budget cycles will only be approved after full review and approval by the district PRB or higher level approval if appropriate.

(2) Upon request by a state, local, or tribal government, USACE may perform a NFIP levee system evaluation on a federally authorized levee in the USACE Levee Safety Program using funding provided by the requester, provided USACE has statutory authority to do so. Consult ER 1140-1-211, *Work for Others – Support for Others: Reimbursable Work*, 22 June 1992 and applicable policies for the Intergovernmental and Cooperation Act, as amended.

(3) Upon request by a state, local, or tribal government, USACE may perform a NFIP levee system evaluation on a non-federal levee in the USACE Levee Safety Program (under the Rehabilitation and Inspection Program in accordance with Public Law (PL) 84-99) using funding provided by the requester, provided USACE has statutory authority to do so. Consult ER 1140-1-211, *Work for Others – Support for Others: Reimbursable Work,* 22 June 1992 and applicable policies for the Intergovernmental and Cooperation Act as amended.

(4) Upon request by another federal agency, USACE may perform a NFIP levee system evaluation using funding provided by the requesting federal agency for federally authorized levees that are operated and maintained by the requesting federal agency. Consult ER 1140-1-211, *Work for Others – Support for Others: Reimbursable Work*, 22 June 1992.

(5) USACE cannot initiate a cost-shared project for the sole purpose of a NFIP levee system evaluation. However, if there is an active USACE authorized cost-shared study or project which includes a levee, then if applicable and requested by the local sponsor, an NFIP levee system evaluation can be performed by USACE on that levee using project appropriated funds. The cost incurred by USACE for the NFIP levee system evaluation work would be included as part of the budget of the current authorized cost-shared project.

(6) USACE does not have funding that is legally available to perform a NFIP levee system evaluation for non-federal levee systems that are not within the USACE Levee Safety Program or part of an ongoing USACE authorized study or project.

(7) For any levee system, USACE may provide technical analysis support for an NFIP levee system evaluation to be performed by others under the Floodplain Management Services (FPMS) Program, reference ER 1105-2-100, *Planning Guidance Notebook*, 22 April 2000. A final NFIP levee system evaluation may not be completed under the FPMS Program. Technical support may also be provided via other applicable authorities. Consult ER 1140-1-211, *Work for Others – Support for Others: Reimbursable Work*, 22 June 1992 and applicable policies for the Intergovernmental and Cooperation Act, as amended.

c. Waivers. No waivers granting exemptions from the application of uncertainty analysis to the hydraulic and hydrologic evaluation for NFIP levee system evaluations will be issued. This refers to the replacement of a minimum freeboard requirement with a quantified assurance of containing the base flood. Any related previous waivers are no longer valid.

d. Partial and Conditional NFIP Levee System Evaluation. Levee systems are a collection of components that must function as a complete, integrated system to be effective. NFIP levee system evaluation will be based on a professional assessment of the complete levee system when subjected to the 1% annual chance exceedance flood and the condition of the system at the time the NFIP levee system evaluation is made. Thus, the concept of "partial NFIP levee system evaluation" is not appropriate. A request for USACE to issue a letter stating "the design and construction" is adequate to exclude the base flood from the leveed area is considered a partial NFIP levee system evaluation. In some instances, USACE may be requested to perform NFIP levee system evaluation for a particular aspect of a system, such as: "Will the height of a levee provide the necessary protection?" In such an instance, USACE must be careful to provide their results in the context of an engineering evaluation of a specific component but that such findings are not to be interpreted as an NFIP levee system evaluation. The NFIP levee system evaluation will be reserved for the levee system as a whole. 'Conditional' NFIP levee system evaluation implies that something must be accomplished for the NFIP levee system evaluation to be valid or that something must not happen in the future – such as additional rise of a closed basin lake. Thus, except as provided for in 44 CFR 61.12, *Rates based on a flood protection system involving Federal funds*, in regard to insurance rates based on adequate progress towards construction completion, conditional NFIP levee system evaluations are not to be issued. In situations involving 44 CFR 61.12, the findings may be transmitted in a letter indicating that when such conditions are satisfied, then a NFIP Levee System Evaluation Report (NLSER), reference paragraph 10.a, may be issued and the 10 year validity period, as described below, will begin.

e. Negative Finding for NFIP Levee System Evaluations. Levees that are accredited by FEMA in accordance with NFIP levee system evaluation requirements may, at any point in time, be found to no longer be in accordance with NFIP levee system evaluation requirements and thus may lose their accreditation for mapping purposes. Examples of levee systems that may be candidates for negative findings are levee systems that failed or exhibited great distress during one or more recent floods; levee systems found during inspections or risk assessments to have major deficiencies in structural features or operation and maintenance (O&M); or levee systems that upon study are no longer in accordance with NFIP levee system evaluation requirements (e.g. changed hydrology or hydraulics or updated structural and geotechnical design criteria). If a levee system previously determined to be in accordance with NFIP levee system evaluation requirements by USACE is found to no longer meet USACE NFIP levee system evaluation requirements, USACE will immediately notify FEMA and the levee sponsor in writing. Decisions to recommend negative findings of NFIP levee system evaluation should be closely coordinated with the FEMA regional office and the levee sponsor, after which results are presented in a joint public meeting to the affected community. FEMA and USACE will work jointly with the levee owner to develop a strategy for returning the levee to a positive finding for NFIP levee system evaluation if that is the desire of the non-federal sponsor. Negative findings for NFIP levee system evaluation are to be documented as described in Section 10.

f. Maximum Period of Validity for NFIP Levee System Evaluations. USACE existing policy letters and other guidance documents do not specifically address the period for which NFIP levee system evaluation is to be considered valid. Through numerous deliberations, for the purposes of this EC, USACE has chosen 10 years as the agency maximum period of validity. NLSERs issued by USACE will include a statement that the NFIP levee system evaluation is valid for a stated period not to exceed 10 years. The LSO shall notify the levee sponsor and FEMA at the 9-year mark that the 10-year NFIP levee system evaluation milestone is approaching. Should a district determine that a levee system is no longer in accordance with NFIP levee system evaluation requirements at any time prior to the end of a 10-year NFIP levee system evaluation period; the district shall inform the appropriate parties that the levee system is no longer in accordance with NFIP levee system evaluations requirements. An example of a condition for such negative NFIP levee system evaluation might be inadequate O&M or changed hydraulics conditions.

(1) For levee systems in which USACE is on record with FEMA as providing a positive NFIP levee system evaluation (or a letter providing a partial/conditional positive finding) that is comprised of only a letter or a letter with incomplete documentation, then a new NFIP levee system evaluation is required and all provisions of this EC apply. The 10-year validity begins with the date on the USACE signs and approves a new NLSER. If none of the provisions in paragraphs 6.b. (1) – (5) can be applied, USACE shall notify the levee sponsor and FEMA that USACE can no longer remain on record as providing a positive NFIP levee system evaluation. In addition, USACE shall notify the levee sponsor and FEMA that the current USACE letter on file with FEMA will be considered invalid 3 years from the date of this EC or when the letter becomes 10-years old, whichever comes first. If there is a known deficiency or information

available to determine that the levee system does not meet USACE criteria for NFIP levee system evaluation, then USACE shall communicate that information to FEMA and the levee sponsor and the 3 year timeframe will not be provided.

(2) For levee systems with USACE NFIP levee system evaluation documentation (including a letter providing a partial/conditional positive finding) on file that is 10-years old or more, then a new NFIP levee system evaluation is required and all provisions of this EC apply. The 10-year validity begins with the date on which the USACE signs and approves a new NLSER. If none of the provisions in paragraphs 6.b. (1) – (5) can be applied, USACE shall immediately notify the levee owner and FEMA that the current documentation previously provided by USACE is no longer valid.

(3) For levee systems with USACE NFIP levee system evaluation documentation on file that is less than 10 years old, and the depth of analysis and documentation were accomplished in accordance to draft ETL 1110-2-570, *Certification of Levee Systems for the National Flood Insurance Program* or meet the intent of this EC, the levee system status and documentation should be carefully reviewed in comparison to the requirements of this EC and if found sound can remain valid. The 10-year validity begins with the date USACE signed the documentation.

g. NLSER Approving Official. The final approving official for NFIP levee system evaluations is the Levee Safety Officer (LSO) for the geographic district where the levee system is located. The LSO is required to be a registered professional engineer as defined in the Levee Safety Program Implementation, CECW-HS Memorandum, USACE, 16 November 2007.

h. Flood Fighting and NFIP Levee System Evaluation. Flood fighting activities are measures taken under emergency conditions that are required to attempt to keep the system from overtopping or failing. Such measures typically include: sand-bagging of boils resulting from seepage under or through an embankment or floodwall that if left unattended, would threaten the integrity of the levee or floodwall; temporary erosion protection from waves or current; and raising the crest elevation by sand-bagging or other flood fighting temporary structure, such as geotextile lined gabions. Flood fighting activities will not be recognized as a measure that can be employed to ensure that a levee system can be found to meet evaluation requirements. While close monitoring and maintenance of features might arise during a high water event and flood fighting might prove prudent and is expected, the principle here is that there must be reasonable assurance that the levee/floodwall system will reduce flood risk against the base flood with "hands off" during the occurrence of an event. Stated another way, if the levee/floodwall system requires flood fighting activities either before or during an event to exclude the base flood from the leveed area, the system cannot meet evaluation requirements. A levee/floodwall system that is on record as being in accordance with levee system evaluation requirements must have deficiencies that were exposed during a flood event repaired and the repair validated in order for the levee system to continue to meet evaluation requirements.

7. USACE Process for NFIP Levee System Evaluation.

a. Introduction. This section summarizes the USACE process of performing a NFIP levee system evaluation or supporting a NFIP levee system evaluation.

b. USACE Role in NFIP Levee System Evaluation. The specific types of levee systems for which USACE has the authority to perform a NFIP levee system evaluation are described in paragraph 6.b. Because the local community or local entity must officially adopt the FIRM, that community or local entity must also decide whether or not it will desire to have an existing levee shown as accredited on their FIRM. USACE will perform NFIP levee system evaluations for systems it operates and maintains (or has major maintenance responsibilities) if requested by a non-federal government entity with interest in achieving accreditation by FEMA, such as a county or local government. For all other levee systems, USACE districts will work closely with their corresponding FEMA regional office and the non-federal sponsor to determine the applicable authority and ability to support the NFIP levee system evaluation request.

(1) Request is made by a non-federal sponsor. These requests may come into various offices within a district, such as Operations, Engineering, or Planning. Each district office should designate a single point of contact (POC) such as the LSO or LSPM for requests and develop coordination standard operating procedures (SOPs). It is important for the district office that receives the request to coordinate with other USACE district offices that will be involved with the work.

(2) District determines type of system and which authority, if any, applies.

(3) District coordinates with FEMA regional office and non-federal sponsor to determine scope of work and schedule. Scope and cost may be based upon availability of data and engineering analyses to be performed.

(4) District determines applicable funding mechanism. Should the work or part of the work be performed on a reimbursable basis, each district will follow its process to initiate the agreements under the applicable authority.

(5) District performs technical analysis. The district will develop an investigation strategy (see Section 8) and provide a detailed scope of technical studies (e.g. Project Management Plan) based on the results of a data/literature search and on-site field inspection. The specific technical analysis scope will be based on a step-wise data collection process including design and construction documentation, O&M inspection procedures and inspection reporting, specific event performance records (see Section 9.c.) and the NFIP levee system evaluation field inspection (see Section 9.d.). The level of detail of technical analysis would then depend on completeness of the technical background available to demonstrate the elevation adequacy and structural soundness of the levee system. The technical analysis will adhere to the guidance presented in this EC; deviations from this guidance must be requested and approved by HQUSACE through the Regional Integration Team (RIT) process prior to the initiation of the proposed analyses. District will coordinate with FEMA regional office and non-federal sponsor throughout the technical analysis process.

(6) District compiles documentation and completes an NLSER, as described in Section 10.

(7) District performs required review of the NLSER.(8) District coordinates findings with FEMA regional office, non-federal sponsor and local community, if different than the non-

federal sponsor. District then provides final NLSER to FEMA regional office and non-federal sponsor.

c. Technical Support. In certain instances, as referenced in paragraph 6.b., USACE may be engaged as a technical resource in support of a NFIP levee system evaluation as follows:

(1) To provide existing information, such as as-built drawings, construction documents (including foundation reports and shop drawings), mapping, geotechnical data, construction documentation, and previous and current inspection reports; or

(2) To perform specific technical analyses, such as hydrologic and hydraulic modeling or slope stability and seepage analyses; or

(3) To review analyses performed by others if requested by a non-federal sponsor.

d. Levee Safety Program. USACE has developed the Flood Risk Management Program (FRMP) as a comprehensive approach for sustainable national flood risk management to improve public safety and reduce flood damages to the nation. The Levee Safety Program is the component of the FRMP framework that will establish USACE policies and procedures for managing, executing, and integrating levee safety activities. The purpose of the Levee Safety Program is to assess the integrity and viability of levees and recommend actions to assure that levee systems do not present unacceptable risks to the public, property, and the environment. One of the objectives of the program is to create a consistent risk-based methodology to evaluate levees nationally and to prioritize actions to maximize flood risk reduction to the public. As an initial step, USACE has created a National Levee Database to serve as a living database of information relative to the status and safety of the nation's levee systems. Over the next few years, USACE will continue to develop and implement policies and procedures to assess, evaluate and communicate risks associated with levees. As the Levee Safety Program moves forward, the overlap with NFIP levee system evaluation procedures will be identified to ensure leveraging of resources and effort where possible.

8. Investigation and Evaluation Strategies.

a. General. This section suggests a strategy for ensuring efficient and effective use of time and funds for performing NFIP levee system evaluations. The scope of the investigations to support NFIP levee system evaluations could vary widely depending on original design intent and availability of design information, construction procedures and records, age of levee system, geomorphic dynamics of system, or completeness of O&M documentation. Developing the level of study and documentation would in turn influence the cost of a NFIP levee system evaluation investigation. System evaluation underpins strategies outlined in this and the following section. A basic tenet of system evaluation is determination of whether the individual components will function individually as intended, and also perform together as a system with reasonable assurance that the base flood will be excluded from the leveed area. Such analysis is expected to be overarching, integrated, and include: hydrology, hydraulics and coastal floods containment, structural and geotechnical performance, mechanical and electrical requirements, O&M plans, encroachments, and recent inspection results that adequately meet NFIP levee system evaluation requirements. Also key to system evaluation is ensuring that interaction among the components will not result in possible system failure, as a result, it is strongly suggested that a Potential Failure Mode Analysis (PFMA) approach be applied. A discussion of PFMA as applied to dam safety investigations is contained in the Federal Energy Regulatory Commission (FERC) document, Dam Safety Performance Monitoring Program (DSPMP), Chapter 14 - Potential Failure Modes Analysis (PFMA), updated in 1 July, 2005 and in Dam Safety Risk Analysis Methodology, U.S. Department of the Interior, Bureau of Reclamation, May 2003. The methods used by FERC and USBR are acceptable to use until formal USACE guidance is published.

b. Existing Data. Reasonably obtainable data will be collected including, but not limited to:

(1) O&M manuals, including levee sponsorship and entity responsible for maintenance,

(2) Parcel boundary, right-of-ways, and easement information,

(3) Performance reports,

(4) Engineering and design documents (e.g., an assessment of the flood hazard, structural components, interior drainage components, geotechnical configuration and placement),

(5) Construction records and as-built drawings,

(6) Records of any and all modifications,

(7) Up-to-date or current surveys of top of levee,

(8) Flood Insurance Study text and maps,

(9) Current hydrologic and hydraulic modeling results,

(10) Base maps showing cross-section locations, construction techniques, and inspection reports,

(11) Flood fighting, maintenance, repair, modification, and rehabilitation records,

(12) Annual and after-event inspection documentation, permits for utility crossings (including encroachments),

(13) Available geology, geomorphic information, and Natural Resources Conservation Service Maps,

(14) National Levee Database (NLD) - https://corpsmap.usace.army.mil/, and Cold Regions Research and Engineering Laboratory (CRREL) Ice Jam Database - https://rsgis.crrel.usace.army.mil/icejam/.

Existing data would include current levee system condition as determined by a NFIP levee system evaluation field inspection (see Section 9.d.) of all features and components in the system.

c. Initial Screening. Initial screening is an incremental approach to categorize levees as a way of identifying the appropriate actions and level of effort required to complete the NFIP levee system evaluation prior to performing a detailed (and possibly costly) effort which may not be needed or justified. Levee systems under study will be tentatively placed in one of three categories based on the data collected. The categories can be defined as: those likely to meet or exceed NFIP levee system evaluation requirements; those likely not to meet NFIP levee system evaluation requirements; and those levees requiring additional or more detailed engineering studies on which to base a determination. The initial screening should include a preliminary probability and uncertainty of levee overtopping analysis as detailed in paragraph 9.e.

(1) Likely to Meet or Exceed NFIP Levee System Evaluation Requirements. Levee systems that have full documentation of system performance including engineering design, construction reports, O&M and inspection documentation, and demonstrating high likelihood of excluding the 1% annual chance exceedance flood from the leveed area can clearly be identified as in accordance with NFIP levee system evaluation requirements. To fit in this category, the preliminary flood frequency and uncertainty analysis should show at least a 90% assurance of containing the 1% annual chance exceedance flood assuming the appropriate uncertainties. The preliminary 90% assurance with appropriate level of uncertainty (probably greater uncertainty at this level of analysis) should provide the confidence that a more refined final analysis would meet the minimal 90% assurance criteria likely including lesser uncertainties. Additionally, preliminary geotechnical and structural assessments and related performance predictions must indicate a high likelihood of adequate performance for critical system components when subjected to the 1% annual chance exceedance flood (e.g. including seepage and piping modes of failure). Such levee systems will typically be those designed for substantially higher design elevations. The process would then be to (1) perform the field inspection to verify the documentation, (2) perform a hydrologic and hydraulic uncertainty analysis using existing data if appropriate to verify adequate height to exclude the 1% annual chance exceedance flood from the leveed area with 90% assurance, (3) consolidate the information in an NLSER (see paragraph 10.a.), (4) perform required reviews, and (5) prepare a NLSER.

(2) Likely to Not Meet NFIP Levee System Evaluation Requirements. Levee systems that display significant or critical deficiencies in any particular area of levee performance, predicted system design performance, or structural condition or levees that clearly fail O&M pre-NFIP levee system evaluation inspections and which therefore have obvious inability to exclude the 1% annual chance exceedance flood from the leveed area, should clearly be identified as not in accordance with NFIP levee system evaluation requirements. The process would then be to (1) perform a field inspection to verify the documentation, (2) perform a hydrologic and hydraulic uncertainty analysis with existing data if appropriate to verify inadequate height to contain the 1% annual chance exceedance flood with 90% assurance and/or (3) perform appropriate geotechnical or structural analyses to illustrate critical deficiencies, then (4) consolidate this information regarding those deficiencies in a NLSER, (5) perform required reviews, and (6) finalize and issue the NLSER documenting the negative findings.

(3) Systems Requiring Additional Studies. Levee systems that have partial or unavailable documentation to clearly demonstrate the capability of the levee system to exclude the 1% annual chance exceedance flood from the leveed area with the necessary assurance will require additional studies. The next section will provide guidance on the overall approach to parsing the

levee system into assessable components that need to be addressed. In other words, define the reaches where the hydraulic and hydrologic uncertainty analysis would be applied, how the hydrologic and hydraulic loadings would be integrated with elements subject to structural and geotechnical analysis, and other analyses related to a system's evaluation of performance.

9. Technical Evaluation Guidance.

a. Basis for NFIP Levee System Evaluation. This section provides the overall approach used to define the flooding hazard, including the elements that are relevant to assessing the 1% annual chance exceedance flood. These elements include the definition of the exceedance criteria, design and construction details, as-built and in-situ status for the full array of engineering assessment factors, O&M status and plans, residual risk, and emergency response plans. Each section will detail references and guidance documents, other sources of information, and professional state-of-the-practice documents. This section will define the overall method for combining levee system assessment elements to establish whether or not that system is in accordance with NFIP levee system evaluation requirements.

b. General. FEMA guidelines for assessing the eligibility of a levee system to be shown as providing reasonable assurance of excluding the base flood from the leveed area on FIRM's and DFIRM's are based on operation plans and criteria, maintenance plans and criteria, and certified data, analysis, design, and construction requirements. Technical evaluations performed by USACE for the purposes of NFIP levee system evaluation are aligned with these criteria. Note that, while the CFR components continue to be relevant, design approaches and criteria have evolved from those that were applicable for the CFR publication date of 1986. Furthermore, USACE has more recently adopted policies and perspectives re-emphasizing public safety that are appropriate to apply in the USACE process for NFIP Levee System Evaluation. These up-to-date policies, engineering concepts and perspectives, and criteria will be applied for NFIP levee system evaluation and are described in subsequent paragraphs. Major factors to be considered in technical evaluations include:

O&M Plan,
NFIP levee system evaluation field inspection,
Flood hazard characterization,
Capacity exceedance/failure criteria,
Levee height assurance determination,
Closure structures and devices,
Embankment protection from current or wave action,
Seepage/underseepage analysis,
Embankment and foundation stability,
Settlement,
Seismic analysis (where appropriate),
Construction records and quality control testing,
Performance records,
Major maintenance and rehabilitation,
Interior drainage,
Residual risk and public safety,
Encroachments,

Ice issues, and
Other applicable unique design criteria.

c. Operation and Maintenance. Both an initial and final component of the NFIP levee system evaluation is determining if the system O&M is adequate in order to ensure the overall integrity and functionality of the levee system during the base flood event. An initial evaluation will help facilitate the identification of observable deficiencies and/or areas which may need further analysis. The system under evaluation shall have an officially adopted O&M manual detailing specific actions and procedures. The manual shall include information such as frequency of O&M activities, provisions for routine inspections (with no more than one year between inspections), and assignments of responsibility for the activities. Typically, these responsibilities are that of the levee sponsor or in the case of federally cost shared projects the responsibility of the local sponsor. All O&M activities shall be under the jurisdiction of a Federal or state agency, an agency created by Federal or state law, or an agency of a community participating in the NFIP to ensure all O&M requirements are met. Information sources may include O&M documentation, rehabilitation measures, and inspection reports. Resources to evaluate O&M adequacy include USACE inspection checklist in conjunction with the USACE publication, *Levee Owner's Manual for Non-Federal Flood Control Works*, March 2006, and subsequent publications of this manual. Additionally, ETL 1110-2-571, *Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, and Appurtenant Structures*, outlines minimum standards for vegetation should be used to ensure the vegetation management practices of the levee system being evaluated are adequate. The NFIP levee system evaluation team should review all deficiencies to ensure the deficiencies will not prevent the system from reasonable assurance of excluding the base flood from the leveed area. For example, a levee system designed for the 0.2% annual chance exceedance (500-year) level could have deficiencies and still has reasonable assurance of excluding the base flood from the leveed area. This levee system could still be in accordance with NFIP levee system evaluation requirements. As a final evaluation it may be appropriate to review the O&M after other technical analyses in order to verify that the system components are integrated in order to ensure operability during the base flood event.

d. NFIP Levee System Evaluation Field Inspection. An NFIP levee system evaluation field inspection will be an on-the-ground field visit (windshield inspections are not acceptable) to be conducted by an appropriately staffed and equipped team. The inspection team should be comprised of disciplines similar to the Periodic Inspection team under the Levee Safety Program (see Policy Guidance Letter – Periodic Inspection Procedures for the Levee Safety Program, dated 17 December 2008). Previous Routine Inspection reports may not be substituted for a NFIP levee system evaluation field inspection but certainly could guide the focus or areas of concern. Periodic Inspections reports may be used to fulfill the inspection requirement if planned in advanced or if the Periodic Inspection has been conducted no more than twelve months prior to the start of the NFIP levee system evaluation. If a previous Periodic Inspection report is to be used, then a site-visit shall be conducted and documented to ensure no changes in field conditions have occurred since the inspection was conducted. Part of the field inspection could confirm deficiencies or reveal repairs that have been completed on required maintenance items. However, the main purpose of the inspection is to collect pertinent information to support the NFIP levee system evaluation or identify the areas that need further analysis. The field inspection should consider all aspects of the levee system's capability for providing reasonable assurance of excluding the 1% annual chance exceedance flood from the leveed area, including upstream watershed changes that may impact the levee system.

e. Context of Uncertainty Analysis for NFIP Levee System Evaluation. USACE policy is to apply a probability and uncertainty analysis framework to NFIP levee system evaluations for all engineering elements. As of the publication date of this EC, probability and uncertainty-based methodologies for the hydrology and hydraulics in riverine situations is more advanced; elements of the uncertainty for NFIP levee system evaluation exist and will be applied as outlined below. Probability of exceedance and uncertainty assessment methods for coastal, estuarine, and lake settings are less mature, but being developed currently, and should be used as noted in paragraph 9.f., but with caution and appropriately applied until they become more codified into engineering guidance. Probability of exceedance and uncertainty-based methodologies are under development and emerging for structural and geotechnical engineering elements but are not yet sufficiently mature for direct application in NFIP levee system evaluations. Thus, initially, the risk and uncertainty analysis framework is applicable for only the uncertainty in flow and stage-chance exceedance (still-water-level-frequency) aspect of probability of exceedance and uncertainty assessment, and to a lesser degree other components. As methodologies for these and the remaining engineering assessment elements mature, they will be incorporated into future versions of this EC. To provide a vision of the way forward, each of the following technical sections describes the base NFIP levee system evaluation methodology (probability and uncertainty-based for flow and stage-frequency, probability of exceedance and uncertainty-based for some other elements, and deterministic for a number of other elements), summarizes the status of developing probability of exceedance and uncertainty-based methods, and to varying degrees, outlines the expected future state when the methods mature.

(1) Risk-analysis methodologies for analyzing the full suite of engineering and operational elements of a levee system or dam are under development for application to USACE dam and levee safety assessments. These methodologies are expected to be adapted to NFIP levee system evaluations and made part of this EC as they mature.

(2) Existing guidance for characterizing the storm surge and waves in lake, estuarine, and coastal settings is neither complete nor well organized. Many aspects are covered in sections throughout Engineer Manual (EM) 1110-2-1100, *The Coastal Engineering Manual*, and other guidance documents and information products; however, guidance is not organized well around the subject of flood hazard assessment. Therefore, clear and concise, step-by-step, guidance is not provided for characterizing the hazard and frequency of flooding associated with storm surge and waves. Also, based on work done by the Interagency Performance Evaluation Task Force (IPET) and in the Louisiana Coastal Protection and Restoration (LaCPR) study, certain aspects of the guidance were found to be deficient. Until guidance is updated and made more complete and more effective, it is important to create awareness of certain aspects that are critical to NFIP levee system evaluations [9.f.(5) and its sub-paragraphs, 9.f.(6), and 9.f.(7)], especially where current guidance is deficient.

(3) A probability of exceedance and uncertainty assessment approach for levee systems in coastal/estuarine settings is being refined, advanced, and applied as part of the LaCPR, Mississippi Coastal Improvement Project (MsCIP), and other work. The approach utilizes technologies applied by the IPET to investigate performance of the Southeast Louisiana

Hurricane Protection System in response to Hurricane Katrina (see Volume IV, *The Storm*). Flood hazard characterization is being based, in large part, on work done in the LaCPR and MsCIP projects to carefully examine the characteristics of hurricanes that have occurred in the Gulf of Mexico since the 1940s, as well as tendencies of the most intense hurricanes. Documentation and a number of calculation tools are being developed in the LaCPR study to aid in computing wave overtopping and wave forces; water level, wave, and overtopping frequencies; and uncertainties. Appendix D briefly describes the approach being taken by the New Orleans District for NFIP levee system evaluation. It represents a good first step toward flood probability and uncertainty assessment in a complex setting where the flood hazard is dictated by storm surge and waves and is indicative of the future state of guidance on this topic. When these levee system investigations are completed, the methods will be adapted more fully into this EC, and into subsequent guidance and other technology-transfer efforts. The approaches being developed are also applicable to those river and lake settings where wind-driven water level changes and/or wind-generated waves are important considerations in levee system design and NFIP levee system evaluation.

f. Hydrology and Hydraulics in Riverine and Coastal Environments. Probability of exceedance and uncertainty analysis of levee containment is required for NFIP levee system evaluation of all new and existing riverine, coastal/estuarine, and lake levees. The method includes a probabilistic assessment of water levels and waves (to the greatest extent possible), as well as their uncertainty, for the present conditions. The analysis must include a proper treatment of the types of events (for example, elevated river discharge due to far-field precipitation or snowmelt, local precipitation, strong wind events, elevated ambient lake levels) that alone, or in concert with another type of event, produce a level of flooding that must be considered in assessing the 1% annual chance exceedance flood performance for a levee system. The analysis must also include proper consideration of whether or not different types of events can be treated as statistically independent event populations, or if there is a statistical dependence of one type of event on another. Typically these evaluations employ deterministic modeling results combined with various methods for determining the uncertainties (and distribution of those uncertainties) about the best estimates.

(1) The probability of exceedance and uncertainty analysis procedure for riverine levees or floodwalls) and incised channels is described in Chapters 4 and 5 of EM 1110-2-1619, *Risk-Based Analysis for Flood Damage Reduction Studies*. For riverine levees (or floodwalls) and incised channels the analysis will usually include the uncertainty in the discharge-probability function and in the stage-discharge function. To obtain the chance of non-exceedance of the levee elevation, the uncertainties in these two functions are combined to get the uncertainty in the stage-probability function. The Monte Carlo analysis in the Hydrologic Engineering Center's *Flood Damage Analysis* (HEC-FDA) program can be used to compute this combined uncertainty as well as the assurance (conditional non-exceedance probability (CNP)) of the levee (or floodwall) or incised channel excluding the 1% chance exceedance flood from the leveed area. To meet NFIP levee system evaluation requirements, a levee or incised channel must have at least a 90% assurance of excluding the 1% annual chance exceedance flood for all reaches of the system. **For levees (and floodwalls)**, if top of levee elevation is less than the FEMA required freeboard (FEMA freeboard requirements are shown in Section 65.10 (b) (i) of the National Flood Insurance Program regulations (Title 44, Part 65, of the Code of Federal Regulations), generally 3 feet with additional freeboard needed at structures, constrictions, and at the upstream

end of levees) above the 1% annual chance exceedance flood stage, then the levee can only be in accordance with NFIP levee system evaluation requirements if the assurance (CNP) is 95% or greater. Top of levee elevation shall not be less than two feet above the 1% annual chance exceedance flood elevation, even if assurance is 95% or greater. Once overtopping assurance (CNP) is determined the levee will fall into one of three categories: 1) assurance is less than 90% - levee cannot be found in accordance with NFIP levee system evaluation requirements, 2) assurance between 90 and 95% - levee can be found in accordance with NFIP levee system evaluation requirements if it is at least the FEMA required freeboard above the 1% annual chance exceedance flood, and 3) assurance greater than 95% - levee can be in accordance with NFIP levee system evaluation requirements if it is at least 2 feet above the 1% annual chance exceedance flood. As uncertainty analysis methodologies improve and more data are gathered, the two feet minimum requirement will be reviewed. **For incised channels**, the top of channel elevation should be checked for containment of the 90% assurance flood level; containment of the 1% annual chance exceedance flood; and in accordance with the "freeboard" guidance provided in EM 1110-2-1601, *Hydraulic Design of Flood Control Channels*. It is important to note that this assurance is only for containment by levees or incised channels; it does not include the probability of failure by any other mode or the combined probability of all failure modes (See Appendix C for examples of these methods).

(2) NFIP levee system evaluation for existing levees will sometimes not have current hydrologic and hydraulic data with defined uncertainty. The engineer doing the NFIP levee system evaluation shall determine if the existing data are adequate for assessing the performance of the levee for current conditions (e.g. the available flows and stages are believed to be substantially higher than a current estimate of flow and stage for the 1% annual chance exceedance flood. If not, it must be updated for the NFIP levee system evaluation. For example, data that characterize the water levels reached during recent flooding events in the region should be examined (river discharges, hurricanes, extra tropical storms, etc.). If major events have occurred since the hazard was last evaluated and characterized, the analysis should be updated. Data and analysis that is more than 10 years old should undergo at least a cursory update to ensure the data and analysis is reasonably representative of current conditions. Data and analysis less than 10 years old can generally be accepted unless there is information clearly indicating an update is needed.

(3) A field inspection can indicate if major changes have occurred in the watershed, surrounding topography/bathymetry, or in the channels that would make the existing hydrologic, hydraulic, and coastal data or hazard assessment out-of-date. Changes such as a large amount of urbanization of the watershed, new upstream dams, diversions, new or changed bridges, encroachments, new or altered channels, dam construction or removals could mean the current/most recent flood hazard assessment is outdated and should be modified to reflect current conditions. In a riverine situation, for example, major channel erosion, sandbars, or vegetation could make the hydraulic data suspect. GIS and LIDAR data may also be employed to assess watershed changes and in some cases may be more timely and cost efficient. If there is a long-term gage in the levee system area, the discharge-probability curve can often be checked by a relatively quick reanalysis of the discharge-frequency curve at the gage. It should be noted that EM 1110-2-1619, *Risk-Based Analysis for Flood Damage Reduction Studies,* addresses watersheds with relatively large gaged data sets or watersheds with little or no gaged data. Compute the discharge-frequency curve based on the gage annual peaks for the period-of-record

used for the past study and then compute with the annual peaks extended through the latest available data. If the change in the 1% annual chance exceedance flow is small, the old discharge-frequency curve can be used for the NFIP levee system evaluation. The definition of a small change depends on the particular river, but might be (1) less than a 5-10% change in design flow, (2) less than 0.5 foot change in the 1% annual chance exceedance flood level, or (3) if the old 1% annual chance exceedance discharge fits within the 75 and 25 percent confidence limits (50% confidence interval) of the new curve, which is part of the FEMA criteria for when old hydrology should be used for flood insurance studies, as noted in Guidelines and Specifications for Flood Hazard Mapping Partners, FEMA, April 2003. This method of comparing results using only gage data can be used to determine if the old hydrology is adequate, even if the old hydrology was based on a regional analysis. In this last case, the analysis of the gage data for the period of record previously used will not match the actual curve used in the past study, but the comparison of the gage-only data will indicate if a change in the frequency curve is needed. The adequacy of the hydraulic data can also be assessed at the gage by comparing recent discharge measurements to older ones. This assessment will show if the stage-discharge relationship has been changing for the range of flows measured. If the relationship has changed, it might be possible to recalibrate the existing hydraulic model to match current measurements. However, in some cases new channel surveys will be needed.

(4) Sensitivity analysis can be used to test the need for updating the existing data. For example, the performance of the levee system can be checked using existing hydrologic and hydraulic data and also using conservative assumptions for how they might have changed. If the levee meets the NFIP levee system evaluation requirements with the conservative assumptions, it can be found to be in accordance with NFIP levee system evaluation requirements. Conservative assumptions for the hydrology could include increasing the curve statistics (mean of logs, standard deviation, and skew) by more than the analysis of the gage indicated or by using the old discharge frequency if there have not been any major floods since it was completed. Conservative hydraulic assumptions could assume the flood profiles have increased from encroachments to the maximum allowable in the Flood Insurance Study (FIS) with floodway profiles, or could use the old profiles if the river has been enlarging and river stages have been decreasing over a reasonable period of time.

(5) For lake and coastal/estuarine levee systems or for riverine levee situations where fetches are sufficiently long, a proper analysis of wave conditions that can accompany extreme water levels must be done. The potential exists for wave overtopping to occur while the still-water level is below the levee crest. Wave overtopping is defined here as the condition in which individual waves break on the levee slope, broken wave bores advance up the slope, onto and across the levee crest, and down the landward side. Adequacy of the analysis procedures used in the original levee system design should be examined in light of guidance provided below; if it was not addressed in accordance with the guidance below, this should be done as part of the NFIP levee system evaluation. The following paragraphs reflect current and emerging guidance for addressing the role of wave overtopping in NFIP levee system evaluation. Due consideration should be given to each method presented. A number of revisions stem from lessons learned by the IPET in its forensic examination of hurricane protection system performance during Hurricane Katrina (see Appendix E), from subsequent work on levee design and NFIP levee system evaluation for the region, and experience in the international community. Note that, because this topic is not addressed in a concise and up-to-date manner in existing USACE

documents, the following paragraphs 9.f.(5)(a) through 9.f.(5)(o) contain an extensive and lengthy discussion intended to educate, caution, and provide guidance regarding evaluation of waves and wave overtopping of levees.

(a) The general hydraulic requirement for NFIP levee system evaluation is a 90% assurance that the leveed area will not be overtopped by the 1% annual chance exceedance flood. When wind waves are not present, the flooding event is simply an elevated still-water level or stage. When waves are present, the flooding event is the possibility of intermittent wave overtopping, which is a more complicated situation. In any wind-generated sea-state, variability is great among the heights and periods of individual waves. For example, over a time interval of 30 to 60 minutes, it is not unusual to experience an occasional wave having a height (crest to trough distance) that has nearly twice the value of the significant wave height (defined here to be the average of the highest one-third of the waves). Most waves will have heights that are much less than the significant wave height. A single broken wave bore or even a few bores that occasionally reach the top of the levee or even overtop it during any 30 to 60 minute period of time will generally not be problematic from the perspectives of either flooding or levee integrity, so the complete absence of wave overtopping is not nearly as critical for NFIP levee system evaluation as it would be for an overtopping situation when the still-water level exceeds the levee crest (steady overtopping is not tolerable). The volume of water, in an average sense over a time span of tens of minutes, which would overtop the levee for the two situations described above (very infrequent/intermittent versus steady) differs by about four orders of magnitude or more. In light of the random nature of waves, the intent for NFIP levee system evaluation is to have little to no wave overtopping of the levee such that the magnitude of overtopping creates insignificant interior flooding and the integrity of the levee is not threatened, all with a high level of assurance. Any small overtopping volume allowed must be adequately handled by the existing drainage system.

(b) For the present time, the approach for examining the issue of wave overtopping in the context of NFIP levee system evaluation determination is to use a conservative deterministic approach for treating the wave contribution and a probabilistic approach for treating the still water level contribution. Complete probabilistic methods are currently being developed and applied. Examples of those emerging methods are presented in Appendix D, but these methods have not yet been codified into guidance, and calculation tools are not yet generally available to fully implement them. The deterministic approach for treating wave overtopping in river and small lake situations involves the following general steps for each levee reach:

– Develop the stage-frequency curve for still-water level that considers all the important processes that alone or in concert with one another can generate extreme still-water levels;

– For the types of events that produce still-water levels (stages) which range from the 1% annual chance exceedance value up to the value associated with the 95% confidence interval, define the fetch conditions that can occur for water levels of these magnitudes, and identify the maximum wind conditions that can occur for several hours along different fetches in front of the levee. Determination of the maximum wind conditions might require examination of winds that occurred during discrete storm events considered in the stage-frequency analysis or of wind conditions that occur during seasons in which extreme water levels occur;

– Use the appropriate maximum wind speed along each different fetch to compute the wave conditions, significant wave height, peak wave period, and mean wave direction, that can be created at the toe of the levee under these conditions (note that the methods for computing winds and waves for large lakes, estuarine, and coastal situations are different, more complicated, and additional discussion is provided in later paragraphs);

– Using this set of wave conditions, as input to a wave overtopping discharge calculation method along with an appropriate overtopping threshold-for-damage value which is discussed later, calculate the maximum freeboard that is required to reduce the level of overtopping to the requisite threshold value for these wave conditions (note that wave overtopping is reduced for oblique wave incidence compared to normal incidence);

– Add the required freeboard to the 1% annual chance exceedance flood still-water elevation; and

– Adopt the larger of either this value or the 1% annual chance exceedance flood still-water level (with 90% assurance) to establish the required levee height.

If the existing levee crest elevation equals or exceeds the required height, the levee can be found in accordance with NFIP levee system evaluation requirements. Because a rigorous probabilistic treatment of waves and wave overtopping is not being done, in order to provide the higher level of assurance that is sought, a very conservative approach to defining the wind, fetch, and wave conditions is adopted (i.e., selection of the maximum wave overtopping condition possible at the 1% annual chance exceedance flood still-water level, assuming it occurs concurrently with the 1% annual chance exceedance still-water level). It is also believed that the wave overtopping rate thresholds cited below are conservative. These several sources of conservatism provide the higher level of assurance in this deterministic approach. Additional information about two methods for computing wave overtopping potential (the recommended wave overtopping discharge approach and a second approach, the 2% wave run-up method), as well as more information regarding methods for conducting wave overtopping assessments in river and small lake settings [paragraph 9.f.(5)(k)] and large lake/estuarine/coastal settings [paragraph 9.f.(5)(l)] are provided in subsequent paragraphs. Due consideration should be given to each method.

(c) The discussion below examines the two methods for defining the freeboard required to reduce the wave overtopping volume to a very low, acceptable level. Both are covered in EM-1110-2-1100 (Part VI, Chapter 5) and both should be considered when selecting the NFIP levee system evaluation requirements. The recommended approach involves computation of wave overtopping discharge rate and comparison of that rate with a maximum allowable overtopping rate value, or threshold value. Methods for computing wave overtopping rates for two possible levee situations (with and without an embedded wall) are covered in Part VI-5-2 of EM-1110-2-1100. A second method uses the 2% wave run-up elevation, which is the elevation above the still water level that is exceeded by only 2% of the waves. The 2% run-up elevation is a traditional coastal engineering elevation parameter. At low wave energy levels and for short wave periods, and for simple sloping levees, the 2% run-up elevation is a reasonable surrogate for a very low level of wave overtopping rate. However, for high wave energy and longer period wave conditions, which are typical for most large lake, estuarine, and coastal settings, the 2%

run-up parameter becomes an increasingly inconsistent surrogate for a certain level of wave overtopping rate. Also, some levees have a wall located on and embedded in the crest. The 2% run-up method should not be used for this more complicated levee situation; the concept of run-up is appropriate for a sloping levee surface but does not apply to a vertical wall face. The wave overtopping rate method is applicable to both levee situations. Wave overtopping rate is the best parameter to use in NFIP levee system evaluation because interior flooding and levee erodability are both intuitively and directly relatable (from an engineering perspective) to the amount of water coming over the levee; an elevation parameter is not.

(d) It is informative to examine and compare the two methods for a simple sloped levee case with no embedded wall. Table 1 shows values of required freeboard elevation above the still-water level, computed using both methods for several wave conditions. In Table 1, the parameter H_s is significant wave height (a statistical measure that represents the average of the highest one-third of the waves) at the toe of the levee; the parameter T_p indicates the peak spectral wave period, also defined at the toe. If a numerical model is being used to generate the wave parameters to be used with these methods, it is likely that the model is generating an energy-based significant wave height parameter (computed from the full wave energy spectrum), not the statistical measure. In some situations the two measures of significant wave height can be assumed to be equal; in some situations they cannot, and a conversion from one to the other must be done. Part II-1-3 of EM 1110-2-1100 addresses this topic. The first two wave-condition

Table 1. Freeboard Requirement for Significant Wave Heights.

Calculation Method	Incident Wave Conditions					
	H_s = 1.5 ft T_p = 2.5 sec	**H_s = 2 ft T_p = 3.5 sec**	**H_s = 3 ft T_p = 5 sec**	**H_s = 5 ft T_p = 7 sec**	**H_s = 8 ft T_p = 7 sec**	**H_s = 8 ft T_p = 15 sec**
$R_{u2\%}$ (CEM)[1]	2.8 ft	4.5 ft	7.8 ft	14.2 ft	17.9 ft	31.2 ft
Q = 0.001 cfs/ft	2.4 ft	4.3 ft	8.2 ft	16.2 ft	21.5 ft	31.2 ft
Q = 0.01 cfs/ft	1.7ft	3.0 ft	6.0 ft	12.2 ft	16.5 ft	24.2 ft
Q = 0.1 cfs/ft	0.9 ft	1.8 ft	3.8 ft	8.4 ft	11.6 ft	17.1 ft
$R_{u2\%}$ (TAW)[2]	2.8 ft	4.5 ft	7.7 ft	14.2 ft	17.9 ft	26.6 ft
Q = 0.001 cfs/ft	2.7 ft	4.7 ft	8.9 ft	18.0 ft	23.8 ft	35.3 ft
Q = 0.01 cfs/ft	1.9 ft	3.3 ft	6.5 ft	13.6 ft	18.3 ft	27.3 ft
Q = 0.1 cfs/ft	1.0 ft	2.0 ft	4.2 ft	9.3 ft	12.9 ft	19.3 ft

Note: H_s = significant wave height of incident waves at the toe of the structure; T_p = wave period corresponding to the peak of the wave spectrum; $R_{u2\%}$ = 2% run-up elevation; cfs/ft = cubic feet per second per foot.
[1] CEM = Coastal Engineering Manual (EM 1110-2-1100, Part VI, Chapter 5).
[2] TAW = Technische Adviescommissie voor de Waterkeringen; Technical Advisory Committee on Flood Defence in The Netherlands (TAW 2002).

columns, perhaps the third as well, are representative of conditions that might be experienced in a river or small lake under high winds. Wave conditions reflected in columns three and four are representative of waves in larger lakes and in estuaries under high winds. Wave conditions in column five are representative of conditions in large lakes, in estuaries, and along the coast under hurricane-force wind conditions (storm waves generated in the far field within the ocean would have higher wave periods). Wave conditions in columns four and five are quite similar to the conditions that were experienced along the south shore of Lake Pontchartrain during Hurricane Katrina, albeit well seaward of the levee toe. The east-facing levees of St. Bernard and Plaquemines Parishes were exposed to significant wave heights of 5 to 10 ft and peak periods of up to 15 sec during Katrina well seaward of the toe (conditions in column 6). A typical ocean wave peak period is 12 to 15 sec for storm conditions, up to 20 sec on the west coast.

(e) In all calculations shown in the table, a typical levee slope of 1:4 is assumed. Wave run-up and overtopping are dependent upon levee slope, in addition to incident wave height and period. Other assumptions made in the calculations are: normal wave incidence, no berm present in front of the levee, a smooth grass levee slope (i.e., no frictional reduction), and Rayleigh-distributed waves. Part VI, Chapter 5 of EM-1110-2-1100 provides guidance for considering these other factors. Oblique wave incidence, the presence of a berm in front of the levee, a rougher levee slope, and the presence of shallow water in front of the levee all act to reduce these values of required freeboard. Values in row 1 of Table 1 reflect the 2% run-up elevation computed using the method outlined in EM-1110-2-1100, denoted as "CEM" for *Coastal Engineering Manual*, (Equation VI-5-3 with coefficients from Table VI-5-2). The next three rows show the freeboard elevations required to reduce the average wave overtopping discharge rate to each of three threshold values that are cited and discussed later: 0.1, 0.01 and 0.001 cfs/ft, using the method of van der Meer and Janssen that is cited in EM-1110-2-1100 (Equation VI-5-24). Recall that the average wave overtopping rate reflects an average rate over a time span of tens of minutes, not the rate that is associated with any individual wave. The last four rows of the table show values of freeboard that are calculated using methods adopted in recent guidance produced by the Technical Advisory Committee on Flood Defence in The Netherlands, TAW (2002); specifically, equations 3a and 3b in TAW (2002) for the 2% run-up values, and Equations 22 and 23 in TAW (2002) for average wave overtopping discharge rate values. The TAW methods are noted by its authors to be slightly conservative, which they recommend for deterministic design. The TAW guidance recommends using less conservative methods in probabilistic assessments of run-up and overtopping. Note that the TAW guidance uses a different wave period parameter than the peak spectral wave period. Therefore, in the computations made using the TAW methods, the peak period was divided by 1.1, which is the factor the authors cite to relate the two wave period measures. The TAW guidelines use the energy-based significant wave height, which was assumed to be equal to the statistically based significant wave height for these computations. The results for 2% run-up elevation values computed using USACE guidance and those computed using TAW (2002) guidance are nearly the same. Freeboard values computed using TAW (2002) guidance for calculating wave overtopping rates are quite similar to, but slightly more conservative than, those values computed using USACE guidance.

(f) Table VI-5-6 of EM-1110-2-1100 suggests that the start of damage to an earthen grass-covered sea dike subjected to wave action begins when the average wave discharge overtopping rate is between 0.01 and 0.1 cfs/ft. For a wave overtopping discharge of 0.001 cfs/ft the table suggests there is no damage. This overtopping rate reflects an extremely small volume of water; expressed another way, 0.001 cfs/ft is equivalent to 0.6 cups of water over a levee, every 5 sec (a typical short wave period), per foot of levee length. For wave conditions in the first three columns, which are typical for rivers and small lakes, the table shows quite a bit of consistency between freeboard values computed using the 2% run-up method and values computed using the wave overtopping discharge value of 0.001 cfs/ft. Both are considered very conservative criterion for NFIP levee system evaluation, but might be appropriate if absolutely no water on or over the levee crest can be tolerated. However, for more energetic, longer period waves, freeboards computed using the 2% run-up elevation method are similar to those computed using the 0.01 cfs/ft threshold. This inconsistency is one reason why the wave overtopping discharge method is recommended over the 2% run-up method; others were cited previously. Historically, for NFIP levee system evaluation in riverine situations, a freeboard of 3 ft was used. For the lowest wave condition in the table, 3 ft of freeboard suggests the levee system will exclude, for all practical purposes, any water over the levee crest. The freeboard values for the second column (conditions that might be expected in some riverine situations) suggest that a freeboard value of 3 ft is consistent with use of the 0.01 annual chance exceedance wave overtopping discharge value, which would also be consistent with Table VI-5-6. However, for higher wave energy conditions, significant wave height of 3 ft or higher, a freeboard of 3 ft might not be adequate. Also note the strongly non-linear variation of required freeboard with increasing wave energy. Use of a constant freeboard across the range of wave conditions is not appropriate.

(g) Whether or not wave overtopping degrades a levee depends upon the overtopping magnitude and subsequent velocities at the crest and on the protected side, soil properties, vegetation cover, duration of overtopping, presence and nature of perturbations on the protected slope, and the levee’s quality of construction and maintenance. Levee erosion due to wave overtopping is most likely to occur on the protected side of the levee crest for higher overtopping rates where water velocities can approach critical speeds, on the protected slope where speeds can be supercritical, and at the toe where flow transitions from supercritical to subcritical. Erosion on the protected side of the levee begins as head-cutting, which then often advances toward the flood side. Guidelines for establishing the overtopping rate threshold (i.e., the threshold associated with the onset of levee erosion and damage) for earthen sea dikes found in EM 1110-2-1100 (Part VI), Table VI-5-6, are consistent with threshold values adopted by the TAW (1989 and 2002) for three different levee conditions (each described qualitatively). The TAW guidance cites metric equivalents that are approximately equal to the following values: 0.0011, 0.011, and 0.11 cfs/ft; so the threshold values adopted here are slightly more conservative than values adopted in the TAW guidance.

(h) For an unarmored earthen levee, the maximum acceptable value of average wave overtopping is 0.1 cfs/ft, unless a higher value can be well-supported by site-specific results from large-scale testing involving wave overtopping. Use of this wave overtopping threshold value must be reserved for well-maintained levee systems that were highly engineered and constructed with good field control, constructed of highly compacted clay or similar erosion-resistant material, with a protective layer thickness of 3 feet or more, and with steps taken during construction to control gullying, which can compromise the thickness of the protective layer and introduce undesirable perturbations. An important feature that promotes resistance to erosion during overtopping is a high-quality protective vegetation cover. These levees should have high quality grass as slope protection, with high-quality protective grass that extends well beyond the levee toe on the protected side. The highest-quality levees are constructed with a thin layer of top soil that is placed to promote smooth levee surfaces, vegetated to promote a dense root system and penetration of the root system into the clay layer. The levee should be actively maintained to retain a high-quality grass cover that is free of significant gullying or other perturbations on the slopes that might induce local head-cutting in the event of overtopping. The levee should be free of pockets of more erodable soil that would also tend to promote head-cutting in the event of overtopping.

(i) Evidence shows the maximum threshold value of 0.1 cu ft/sec/ft for well-maintained high quality clay levees with high quality grass cover is conservative (i.e., a higher value of wave overtopping can be withstood before any damage begins). Emerging data suggest that this is the case for high quality clay levees with very good turf conditions; that acceptable values might be as high as 0.3 to 0.5 cfs/ft, perhaps higher. Also, based on Hurricane Katrina experience, some levees in St. Bernard Parish were undamaged despite apparently being subjected to significant overtopping that exceeded 0.1 cfs/ft. Levees along the south shore of Lake Pontchartrain experienced some overtopping in places but no significant damage to speak of. Peak significant wave heights of about 8 ft or more and peak periods of 7 sec were measured just offshore. The highest high-water marks found on the levees along the south shore (based on debris deposited on levee slopes) were about 6 ft above peak still-water levels. This suggests that freeboard estimates of 8 to 12 ft (from Table 1), even for a threshold value of 0.1 cfs/ft, might be overly conservative. However, until more data are acquired, the maximum threshold value of the overtopping discharge rate should be retained as 0.1 cfs/ft in a second pass through the NFIP levee system evaluation, and this value should be reserved for the highest quality, most erosion-resistant levee conditions.

(j) The wave overtopping rate threshold for lesser quality levees and lesser quality grass cover are lower. The overtopping rate threshold for a clayey soil with relatively good grass cover is 0.01 cfs/ft. This value is expected to be more typical for the wave overtopping threshold. As was mentioned previously, levee sections constructed of silts, silty or fine sands, and all hydraulically placed materials will have even lower overtopping thresholds; and they must be considered highly susceptible to erosion and breaching in the event of any significant wave overtopping. The wave overtopping rate threshold for levees constructed with these types of materials, or via hydraulic placement, or levees with poor turf, is much lower (0.001 cfs/ft). If the soil composition of a levee is unknown or highly uncertain, this low threshold value of overtopping (0.001 cfs/ft) should be used. Selection of a threshold overtopping rate should be well-supported and documented as part of the NFIP levee system evaluation. Until more is learned about how the thresholds relate to all the factors that dictate levee erosion and degradation, these are the adopted values. Past experiences by both the Dutch and Japanese have been considered in developing these values, and their experiences have shown these values to be reasonable. Simple methods for calculating average wave overtopping rates are adequate if the actual levee cross-section is similar to those used to develop the method (e.g., simple uniform slopes or similar berm dimensions). For more complex cross-sections that are unlike those used in the development of the simple methods, such as cross-sections having elevated roadways, benches, or steps, or offshore submerged breakwater features, application of a more rigorous and accurate tool is warranted; either a Boussinesq-type numerical model such as BOUSS2D (see EM-1110-2-1100, Part II, and http://chl.erdc.usace.army.mil/chl.aspx?p=s&a=Software;23) or physical scale model tests involving actual cross sections and wave conditions. For complex open coast wave situations (e.g., narrow-banded sea states that occur on the west coast, or double-peaked wave energy spectra, where the wave field is comprised of both long-period energy generated in the far field and wind seas generated in the near field) the simple calculation methods are less accurate. Again, for these conditions, use of a more rigorous numerical model might be warranted.

(k) Wave Overtopping in Rivers and Small Lakes. At present, waves can be treated deterministically in riverine and small-lake situations because there is no standard accepted methodology for quantifying the uncertainty distribution for riverine waves, or for combining wave uncertainty with the discharge-probability and stage-discharge uncertainties discussed above. The method for defining the annual chance of exceedance for still-water levels was described above in paragraph 9.f.(1). For lakes, increased still-water levels due to wind forcing and/or changes to ambient lake levels, and to waves, are the most likely and dominant contributors to the 1% annual chance exceedance lake level. In light of the dependence of wind forcing on water depth, lake levels and wind-driven events might not be statistically independent events, such as in shallow lakes. Wave conditions are a function of the following: wind speed and direction in relation to the available fetch, duration, spatial and temporal wind variability, water depth throughout the fetch, depth gradients that control wave refraction and shoaling, and local depth that controls wave breaking. Paragraph 9.f.(5)(o) discusses several important considerations in estimating surface over-water winds for use in computations of wind-generated waves and water levels. Part II (Chapters 2 and 3) of EM 1110-2-1100 provides guidance for estimating wave conditions. The restricted-fetch (or narrow-fetch) assumption and simplified one-dimensional wind wave prediction methods are generally adequate for use in most flooded river and small lake situations, unless the fetches become quite large and two-dimensional (variable in both horizontal dimensions). Where currents in rivers are strong, they can influence wave characteristics. Part II (Chapter 3) of EM 1110-2-1100 discusses wave-current interaction and predictive methods that consider this process. Note that the wave overtopping discharge rate method should be used as the threshold for NFIP levee system evaluation in all riverine, lake, estuarine and coastal situations. However, if a run-up elevation method has been used previously in the levee design, and if use of that run-up elevation provides a more conservative (higher) value for the required levee elevation than that determined using the analysis method outlined above (based on wave overtopping rate threshold), then the levee is in accordance with NFIP levee system evaluation requirements from a wave overtopping perspective.

(l) Wave Overtopping in Estuarine/Coastal and Large Lake Situations. For the coastal and estuarine situations, extreme water levels due to a combination of astronomical tide and storm surge (which is primarily, but not exclusively, wind-driven) associated with extra-tropical events and/or hurricanes, combined with waves generated during these events, are primary contributors to the flooding hazard for the 1% annual chance exceedance flood. In most cases it is a reasonable assumption to consider astronomical tide and storm surge to be statistically independent phenomena. Astronomical tide-frequency and storm surge-frequency distributions can be convolved, using the assumption of statistical independence, to yield still-water level-frequency curves; or astronomical tide (which is periodic and quite predictable) can be considered as an aspect of uncertainty in generation of still water level-frequency curves. Part 2, Chapter 5, of EM 1110-2-1100 provides information on computing water levels associated with tides and wind-driven events. Multi-dimensional analysis of winds and waves is needed for estuaries, coastal regions, and very large lakes, where the surface wind fields themselves exhibit two-dimensional (in space) variability, where the water bodies (available fetches) are large relative to the size of the wind system, and where moving wind systems produce complex and highly energetic wave fields. Wave conditions can vary considerably along the shoreline in these situations. An approach for conducting a probabilistic analysis of water levels, waves, and wave overtopping in coastal/estuarine settings is being developed and applied to examine NFIP levee system evaluations which comprise the hurricane protection system in southeast Louisiana. The approach utilizes technologies that were applied by the IPET (see Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System: Draft Final Report of the Interagency Performance Evaluation Task Force, Volume IV, *The Storm*, 2006). Definition of the storm threat is being based on work done in the LaCPR and MsCIP projects to carefully examine the characteristics of hurricanes that have occurred in the Gulf of Mexico since the 1940s, as well as tendencies of the most intense hurricanes. Documentation and a number of calculation tools are being developed in the ongoing studies to aid in computing wave overtopping and wave forces; water level, wave, and overtopping frequencies; and uncertainties. Appendix D briefly describes the approach being taken by the New Orleans District to evaluate the issue of NFIP levee system evaluation; it represents a first step toward a probabilistic treatment of wave overtopping in a complex coastal/estuarine setting where the flood hazard is dictated by storm surge and waves. The approach is the recommended one at this time for coastal/estuarine and large lake situations; it is indicative of the future state of guidance on this topic. When these investigations are completed, the methods will be adapted more fully into this EC, and into subsequent guidance and other technology-transfer efforts.

(m) Defining the Hurricane Hazard. Hurricanes are the primary flooding event of concern for determining the 1% chance exceedance along the Gulf of Mexico coast and most of the Atlantic coast. Thus, they are extremely important in NFIP levee system evaluation along the coast and in estuaries. Storm surge can propagate tens of miles up rivers and deep navigation channels. Hurricane surge can inundate wetlands and barrier islands, and expose estuaries to high storm surge and long-period, high-energy ocean wave conditions. Hurricanes might also be the primary events for some interior river and lake systems because of the diminished (but still high) wind-generation potential. Current guidance in Part 2 of EM 1110-2-1100 for assessing the hurricane hazard is deficient. Special care must be taken to characterize the hurricane-induced flood hazard. Where hurricanes are the primary cause for severe flooding, land-fall hurricanes are the events that are most relevant to assessments of the 1% annual chance exceedance flood. The occurrence of major land-fall hurricanes at any one location is rare. Central pressure, radius-to-maximum winds (a measure of the storm’s size), and the coastline shape and continental shelf width and configuration are the most important factors in determining hurricane storm surge. Also, the hurricane surge experienced at a location is greatly dependent upon the hurricane track. Sole reliance on the local historical hurricane experience (the limited range of central pressure, radius-to-maximum-winds, and track characteristics that are reflected in the small number of hurricanes that have had a major impact, locally), even as part of an Empirical Simulation Technique (EST) application, will not yield a sufficiently accurate assessment of the hurricane flood hazard. An accurate assessment must properly consider other hurricane characteristics that are possible in a local area. Only data acquired since 1940 should be used to characterize hurricane probabilities and their characteristics. Only since the 1940s have aerial reconnaissance, radar, and other sensing technologies enabled hurricanes to be characterized accurately. An exception would be a major hurricane that impacted the region prior to the 1940s, with characteristics that can be reliably incorporated into the analysis. To assess the 1% annual chance exceedance, a Joint Probability Method (JPM) approach should be used to properly define the hurricane hazard (in terms of still-water level-frequency relationships). The recommended approach for applying the JPM to define the hurricane hazard in an optimal manner has been developed and applied in the LaCPR and MsCIP projects (see the as yet unpublished whitepaper by Resio et al. 2007). The JPM approach is the best approach for properly considering other hurricane characteristics and tracks that are possible for a region but have not occurred historically, as opposed to consideration of historical occurrences only, or use of the EST applied to only historical hurricanes. In the implementation of a JPM approach, in order to accurately characterize what is possible in the way of intense hurricanes, close examination should be made of the characteristics and tracks associated on the most intense hurricanes, and the decay in intensity as hurricanes approach landfall. The JPM tends to be a computationally intensive method to use, and that has been why the EST method has been adopted in the past. However, the LaCPR project has developed an optimal sampling approach that minimizes the number of storms that need to be considered in applying the JPM. If a JPM approach has not been used to define the hurricane hazard for a levee system where it is a primary design event for 1% annual chance exceedance, then NFIP levee system evaluation will require that it be done. If a JPM analysis was done previously, but it includes consideration of storms prior to the 1940s and/or does not account for the major hurricanes that have occurred since 1995, then the analysis should be updated. A pattern of increased hurricane activity and intensity began around 1995.

(n) Defining the Non-Hurricane Hazard. Reliance on historical water level data and fitting of an extreme probability distribution to those data in small-lake situations, model simulation of extreme water levels associated with historical storms in a large lake or coastal/estuarine situation, or application of the EST approach are sound approaches for characterizing the still-water level-frequency relationship associated with extratropical storm or non-hurricane wind-driven events, if sufficient historical data are available. A data record length of 40 to 50 years is considered sufficient for accurate assessment of the 1% annual chance exceedance. Data must accurately reflect conditions in the area of interest. Gages where measured data are available are often located in areas with levees that may or may not reflect conditions at the locations of interest. Sufficient data are often not available for this length of time, and other prediction methods or models must be used to characterize the water level (and wave) conditions associated with extratropical storms or other extreme wind events that have occurred over the past 40 to 50 years. In large water bodies, model simulation is usually required because of the two-dimensional nature of storm wind fields. If the still-water level-frequency analysis for non-hurricane events has been based on the historical record, but the data record is older and/or of insufficient length, the flood hazard characterization should be reassessed as part of the NFIP levee system evaluation process.

(o) Defining Wind Conditions. The accuracy inherent in estimates of water levels and wave conditions associated with wind events is only as good as the accuracy of the wind input. Since wind stress is non-linearly related to wind speed, errors in wind speed are amplified in water level and wave estimates. Therefore, when defining winds, it is important to maximize the accuracy of wind estimates; it is important to use a wind drag law that changes with wind speed. The Garratt (1977) wind drag is recommended for all wind-driven water level computations. It is extremely important that consistent measures of wind speed be used for water level and wave prediction, and they can be different for each type of calculation. Wind estimates should reflect estimates of over-water winds, not over-land winds, and they should consider air/water temperature differences. Chapter 3 of EM 1110-2-1100 describes procedures for properly estimating over-water winds and how to account for air-water temperature differences. It is extremely important that the measure of wind speed that is used as input to water level and wave calculations is exactly the same as the wind speed measure that is inherent in the method(s) being used to do the calculations. For example, the Garratt drag law assumes the wind speed reflects a

10-min average wind speed at 10-m elevation above the water surface. Therefore, water level computations using the Garratt drag law should use over-water wind input that reflects a 10-min average value at a 10-m elevation above the water surface. In wave calculations, a 30-min average over-water wind speed at 10-m elevation is often used. Consistency is the key. Chapter 3 of EM 1110-2-1100 describes procedures for making corrections to reference winds to different wind averaging intervals and to different elevations. Use of an inappropriate measure of wind speed, such as use of a 1-min gust speed with the Garratt drag law, can lead to significant over-predictions (errors) in computations of water level and wave conditions. Another example of inappropriate use is using flight level wind measures (which the National Oceanic and Atmospheric Administration often publishes) with the Garratt drag law, which requires a surface (10-m) elevation wind measure. This error can also lead to significant over-predictions of water levels and wave conditions. It is advisable to have a trained coastal meteorologist review the method that is being used to create wind input to water level and wave computations because of the crucial nature of wind input.

(6) Levee Erosion on the Flood Side. As part of the levee NFIP levee system evaluation, the potential for erosion of the levee during an episodic storm/flood event should be examined. The current condition of the levee also should be examined to determine if currently eroded, as should the potential for erosion during the period of time between NFIP levee system evaluations. Erosion could be a result of the land in front of a levee experiencing severe erosion during coastal storms, or a result of gradual river channel evolution such as meandering, or due to bed degradation, which has the potential to encroach upon the levee and compromise the levee cross-section, or from general bed degradation. Loadings due to stream velocity and/or wind-wave action produce hydraulic shear stresses that can act on the levee slope or soil mass in front of the levee. The resultant erosion will depend upon the loadings, duration of loadings, topography and bathymetry characteristics, the soil characteristics, vegetation, and any armoring and nature of that armor. Levee fragility can be assessed based on scour depth or erosion rate and an acceptable limit state such as the acceptable width of the berm or soil mass in front of the levee, or any loss of the levee cross-section. In a general gross sense, the erosion rate multiplied by the duration of an event will define a critical condition when that product is greater that the acceptable erosion limit state. Levee erosion on the flood side, due to waves and/or currents, can influence wave heights and wave loadings by changing the local depth and bottom slope; this can have implications for wave overtopping and required levee elevation, and loadings on structures that are embedded in the levee. Erosion potential should be checked when generating floodway (i.e. encroached condition) mapping and hydraulic conditions.

(a) Current-Induced Erosion. The levee can be attacked by a current producing erosion of its surface dependent on the hydraulic shear stress that is calculated from velocity. The response of the surface will depend on the magnitude and duration characteristics of the current, regional geometry (location along a channel, e.g., on the outer edge of a curve in the channel), soil characteristics, vegetation and armoring if present, and characteristics of the armor. This response can be in terms of a scour depth or an erosion rate. The velocity may be a function of water elevation (flood stage or storm surge). The evaluation of levee fragility is then determined by comparing the erosion rate multiplied by time of an event to an existing volume that must be eroded before the levee is compromised. Erosion rate is often treated as the product of an erodability coefficient and the difference between the effective hydraulic stress on the soil boundary and the critical shear stress. Hanson and Temple (2002) and Hanson and Cook (2004) provide some guidance for estimating erosion rates in this manner. Methods for applying this type of approach are available for Corps use in a toolbox application developed by URS Corporation for the purposes of levee assessment. Also the U.S. Department of Agriculture has a spreadsheet application "Bank Stability and Toe Erosion Model" (http://ars.usda.gov/Research/docs.htm?docid=5044) for multi-layer stream-banks that that can be applied. Critical shear stress values and erodability coefficients are highly dependent on the local soil properties. The best method to estimate erosion rate as a function of shear stress, and critical shear stress, is to perform an analysis using a mobile erosion flume and site-specific sediment cores that are as undisturbed as possible. The sediment samples are subjected to various flow conditions to evaluate erosion rate as a function of imposed shear stress, see http://chl.erdc.usace.army.mil/CHL.aspx?p=s&a=ARTICLES;630. This erosion flume can examine sediment erosion due to currents alone, waves alone, or waves superimposed on steady currents. EM 1110-2-1601, *Hydraulic Design of Flood Control Channels*, Change 1 ENG 4794-R provides some guidance on the subject of channel erosion, along with guidance for armoring to prevent erosion. Hughes et al. (2007) provides considerable guidance covering alternatives for erosion prevention.

(b) Wave-Induced Erosion. Erosion of the flood-side of a levee by wave action can also be an issue of concern at elevated water levels. Breaking waves on the flood-side slope, if present for a sufficiently long duration, can erode the levee, diminish its cross-section and potentially lower the levee crest elevation. Erosion potential is a function of incident wave characteristics, duration of wave action, levee soil properties, and presence of protective vegetation cover and armoring, if any. Levees comprised of non-cohesive soils (silts and sands) are especially prone to erosion by this process, even if exposed to wave action for only several hours. NFIP levee system evaluation should consider the possibility of erosive wave action, the possible duration of this wave action, and the likelihood that erosion that would degrade the levee at the annual chance exceedance water level, considering the wave conditions that can occur at this water level. At present there are no reliable methods for predicting wave-induced erosion on flood-side levee slopes, for levees comprised of cohesive and mixed sediments. Methods are available for predicting erosion of sand embankments by increased water levels and wave action (see EM-1110-2-1100, Part V, Chapter 4). Part III of EM 1110-2-1100 provides some guidance for examining cohesive and non-cohesive sediment transport processes in a coastal environment.

(7) Dynamic Wave Loadings. At locations with potential for dynamic wave loading, an important part of NFIP levee system evaluation for structures is determination of loads and related stresses, deformations, and stability conditions of structural members. If waves are present, dynamic wave loadings can be an important consideration in design and NFIP levee system evaluation for a levee system that contains flood walls or other structures, especially those with vertical faces. Dynamic wave loadings on vertical faces can greatly exceed hydrostatic forces, cause damage through repetitive high-impact loadings, and lead to failures of the levee system. Part VI, Chapter 5, *Fundamentals of Design*, of EM 1110-2-1100 contains information relevant to structures that are part of levee systems exposed to wave action.

(a) In the case of rubble-mound structures exposed to waves, specific procedures cannot be followed as theory does not cover wave loading on single stones or blocks. Instead, the structure must be considered as a whole; an integrated approach is thus used to establish relationships between certain wave characteristics and structural response, sometimes in terms of armor movements. These types of loading situations are covered in Part VI-5-3 of EM 1110-2-1100.

(b) For vertical-front monolithic structures such as seawalls and floodwalls, wave loadings can be estimated from theory or experiments. This type of analysis is covered in Part VI-5-4 of EM 1110-2-1100. Different types of wave forces on vertical walls are identified. Non-breaking wave loads can be treated as a pulsating load. It can be treated as a static load in stability calculations. Breaking waves that break in a plunging mode with an almost vertical front just before contact with the wall can generate very high pressures with short duration, resulting in a large single-peaked force with each wave impact. A breaking wave with a large air pocket produces a double-peaked force, in which the first and largest load occurs when a wave crest hits, producing a hammer shock. The second peak is induced by compression of the air pocket, or compression shock. Frequent wave breaking at a vertical structure will not occur for oblique waves with angle of incidence greater than ±70 deg relative to normal incidence to the structure, so hydrostatic analysis can be performed for these conditions. Otherwise, for waves that approach from angles less than ±70 deg relative to normal incidence to the wall, wave impact pressures must be considered.

(c) The ability to predict the impact pressures of breaking waves is difficult due to their extremely stochastic nature. The loads can be very large, and chance of failure increases with the number of loads. To the greatest extent possible, design of vertical structures should avoid waves breaking directly on the vertical face. This can be aided by placing armor units in front of the vertical wall to help dissipate wave energy, maintaining a mild slope of 1:50 or less over a distance of several wavelengths in front of the structure, or using a sloping-front face from still-water level to the crest (however, a sloping-front structure allows more overtopping than a vertical wall of equivalent crest height). For walls without gentle slopes or armored slopes, or for walls imbedded into steep-sloped earthen levees, dynamic wave pressures on the walls need to be assessed as part of the NFIP levee system evaluation for situations where waves are present at elevated water levels.

(8) Assess System Weaknesses and Vulnerabilities. Analyses should carefully examine the potential for locations of critical weakness links in the flood risk reduction system as a precursor to identification of potential failure locations and modes. Critically weak locations can compromise an otherwise robust levee system. Water naturally seeks out low spots in a levee or weaknesses in the soil. Low spots in levee crest elevation can occur over lengths much less than the reach length used in design or NFIP levee system evaluation. The same is true for local conditions of poor soil or vegetation properties, a poorly maintained section, areas exhibiting tendencies for settlement, areas where waves and water levels might be locally higher than in adjacent areas, such as a particularly long stretch of river aligned with the predominant wind direction during severe storm conditions, or features that would tend to concentrate wave energy or flow. Levee-structure transitions (e.g., between earthen section and sheet pile or flood wall section) must be examined as vulnerabilities. It is extremely important that the entire levee system be thoroughly reviewed to characterize spatial variability in levee/system crest elevation and identify critically weak locations at the highest spatial resolution possible, particularly in systems that are not well compartmentalized on the protected side. Data acquired using traditional survey methods, or high-resolution LIDAR (Light Detection and Ranging) or other survey techniques, with correct and consistent datum's, imagery and other visual data sources, should be maximized to search for and identify critically weak locations in the system. The top of levee survey shall be performed in accordance with National Levee Database specifications to identify potential low spots in the crest of the levee system. An assessment of critically weak

locations, or potential critically weak locations, should be included as part of the NFIP Levee System Evaluation Report (NLSER). Information developed in this section will provide valuable input for identifying potential failure modes as noted in paragraph 9.h.(5).

(9) Interior Drainage. Drainage of storm water from floodplains behind levee systems is impeded by the presence of the levee. Flooding that may occur from the drainage-impeded water must be analyzed and the resulting impacted floodplain reflected on flood insurance maps. Ensuring that interior runoff is delivered to the levee is a local responsibility. The NFIP levee system evaluation does not include computation or display of interior flooding that is not impacted by the levee. However, depending on the interior drainage system, the levee can impact interior stages some distance from the levee and the total impacts of the levee on the interior drainage system and flooding need to be analyzed. The analysis of interior flooding is based on a coincident analysis of exterior and interior stages that includes the capacity of gravity and blocked gravity drainage features. Coincident analysis for interior areas is explained in Chapter 4 of EM 1110-2-1413, *Hydrologic Analysis of Interior Areas*. For riverine levee systems, the interior analysis considers interior rainfall events during both low river stages (gravity conditions) and high river stages when the gravity outlets are closed (blocked conditions) and the performance of pumping stations as might exist. As described in the referenced EM, there are several ways to compute the combined probability of interior stages from these events. For NFIP levee system evaluation of existing levees, the need for a reanalysis of interior stages should be based on similar criteria to the need for reanalysis of the river hydrology and hydraulics. Requirements for a restudy of the interior flooding include: interior flooding since the levee system was constructed, loss of interior ponding areas to development, significant land-use changes in the interior of the levee system, changes in interior flood risk reduction facilities (outlets, intercepting sewers, pump stations) or their operating plans, etc. Data and analysis that is more than 10 years old should undergo at least a cursory update to ensure the data and analysis is reasonably representative of current conditions. Data and analysis less than 10 years old can generally be accepted unless there is information clearly indicating an update is needed. For lake and coastal levee systems, in addition to impeded flow from interior runoff, the volume of water that may enter the floodplain from wave overtopping of levees from wind-driven waves, as discussed in paragraph 9.f.(5) and its sub-paragraphs, must be considered. This can be particularly important if the floodplain under study is small and confined to near the coastal protection levee. Preparation of the final maps and coordination of these with residents is the responsibility of the community, the state, and FEMA.

(10) Ice Conditions. For many levees ice can impact flood stages and needs to be considered in the analysis. Flood stages can be impacted by ice jams and by ice cover. Quantifying the impact of ice on flood stage-frequency can be difficult and may require assistance from the USACE ERDC Cold Regions Research and Engineering Laboratory (CRREL). When a levee system is known to be affected by ice jams, the combined ice-jam and open-water stage-frequency should be used in place of the open-water stage-frequency. CRREL has developed and maintains an Ice Jam Database (https://rsgis.crrel.usace.army.mil/icejam/) which has information on many known ice jam locations in the US. Because the database is not a complete record, USGS records for gages in or near the levee system area should be searched as well (see http://nwis.waterdata.usgs.gov/nwis/peak). If there is a USGS gage in or near the levee system area, the annual peak stages should be reviewed for evidence of ice impact, which is often reported by the discharge code 9 (Discharge due to snowmelt, hurricane, ice-jam or debris dam

breakup) or the stage code 1 (Gage height affected by backwater). Note that the tab-separated file format contains more information than the watstore format. Another source of data is the annual water date report series (http://wdr.water.usgs.gov/). A third source of historic information is the USGS data contained in Streamflow Measurements for the Nation (http://nwis.waterdata.usgs.gov/nwis/measurements), where the codes AICE, SICE, and CICE (stream control is impacted by anchor ice, ice, or shore ice, respectively) indicate the occurrence of ice. When historic data is available it can be used to help develop discharge-elevation relationships for open water and ice-impacted conditions. When historical data is not available it may be possible to generate the ice-affected stage-frequency curve indirectly. One method is to develop a discharge-frequency curve for the ice impacted portion of the year and a synthetic discharge-elevation curve for ice conditions from HEC-RAS. The HEC-RAS program includes ice cover and wide river ice jam routines that can be used to determine river stages for ice impacted events. The ice season discharge-frequency curve is then combined with the ice impacted discharge-elevation curve to get the ice-affected stage-frequency curve. The synthetic method will usually be much more complicated than this, and assistance from CRREL is recommended. A technique for combining the open water and ice impacted stages into an overall stage-frequency curve is described in "Ice-Affected Flood Frequency" Ice Engineering Information Exchange Bulletin Number 24, January 2000, by the U.S. Army Cold Regions Research and Engineering Laboratory, available on the CRREL web site. CRREL has a Fortran program called Combiner that can be used to combine the open-water and ice-affected stage-frequency curves, which can be a tedious and error-prone process otherwise. Engineering Manual 1110-2-1612, *Ice Engineering*, 15 Oct 1982 provides background information on ice processes. Appendix F, "Guidelines for Ice-Jam Flooding and Analyses and Mapping", of FEMA "Guidelines and Specifications for Flood Hazard Mapping Partners", also gives additional guidance on ice impacts on flood elevation frequencies.

(11) Assumptions Regarding External Flood Risk Management Features. For the NFIP evaluation of levee systems that are situated upstream or downstream of measures that make up flood risk management projects for external systems, certain assumptions need to be made regarding the performance of the those external systems. The performance of the external systems could dictate the load delivered to the system to be evaluated and the assumptions of that performance are key to the findings of the system being evaluated. For example, the NFIP evaluation of a downstream system, it will be assumed that an upstream external system(s) will operate and function as intended. If the upstream measures include levees, it will be assumed that they will not fail prior to loads reaching top of levee and after overtopping if capacity is exceeded. If the 1% annual chance exceedance flood is imposed on the upstream external system and that external system includes levees that can not contain the 1% annual chance exceedance flood, those levees will overtop and assuming no failure, the residual 1% annual chance exceedance flow would pass on to the downstream system. This residual flow (with uncertainty) would then be used to determine the 0.01annual chance exceedance stage (with uncertainty) to analyze the assurance (CNP) of the downstream levee system for NFIP evaluation purposes. An approach to evaluating this situation is to use the Inflow-Outflow Transform capability in HEC-FDA where the inflow represents the flow approaching the upstream external system and the outflow represents the residual flow that is imposed on the downstream system under evaluation. Uncertainties can be evaluated about the transform function by performing sensitivity on overtopping locations, overtopping lengths, and weir coefficients. Additionally, the NFIP

evaluation of an upstream system, it will be assumed that a downstream external system(s) will operate and function as intended. If the downstream measures include levees, it will be assumed that they too will not fail prior to loads reaching top of levee and after overtopping if capacity is exceeded.

g. Structural Technical Evaluation Guidance. Currently no explicit USACE guidance exists for the structural evaluation of existing levee structures. Most structural evaluations have a number of basic steps in common. The first and most important step is to study past performance and define the existing condition of all the levee system's structural components. This step should be done by reviewing all available information, including inspection results for the NFIP levee system evaluation (or recent Periodic Inspection report); previous Routine and Periodic Inspection reports; specific flood event "After Action Reports"; "As built" drawings; levee system permits; original construction photographs; and any instrumentation data that have been collected.

(1) Review of Conduit Inspection Records/Reports. Conduits, culverts and pipes of all diameters, dimensions and materials must be assessed. The structural engineer must review the latest conduit condition assessment reports for the levee system to see if any conditions or performance issues are noted that would reduce the engineer's confidence in the levee system. All conduits are to be evaluated every five years, using video technology. If the conduit in the levee system under study has not been evaluated in the past five years with video technology, then such an evaluation is required as a component of the study. If any of the conduits are rated as Unacceptable (as outlined in the latest version of the USACE levee inspection checklist), then a positive finding for NFIP levee system evaluation should not be recommended. Pump station discharge pipes that are buried within the levee cross-section must have their conditions assessed, but there is no requirement to have discharge pipes evaluated within the previous five years. Video or visual inspection of the entirety of all such conduits is mandatory in accordance with the requirements in the inspection checklist.

(2) Condition Assessments. Condition assessments for video inspected pipes shall follow the Pipeline Assessment Certification Program (PACP) guidelines published by NASSCO. In general most local sponsors would be expected to hire specialty contractors to perform the video inspections. The reports from these contractors (or by the sponsors themselves) shall be prepared using PACP guidelines, and each conduit shall be rated in accordance with these guidelines. Pipes rated a "4" or a "5" using the PACP coding system may be subject to sudden failure or collapse; therefore it is suggested that such a conduit should be replaced or slip-lined in order for the levee system to meet NFIP levee system evaluation requirements. The technical evaluation must include a justification for why each conduit is considered to be either in acceptable or unacceptable condition. If any of the conduits are rated by the engineer to be unacceptable, then a positive finding for NFIP levee system evaluation should not be recommended. Additionally in many cases portions of conduits are partially submerged at the exit end; such situations are common on the Ohio River where dams were raised in the 1960's - years after the original gravity flow outlets were constructed. Current video inspection technology is useless for submerged portions of conduits. In such cases sonar inspections of the submerged portions of the conduits should be completed, and the resulting condition assessments reviewed. There are currently no standards for rating conduits using sonar methods; engineering judgment must be used for such cases.

(3) Site Visit. A detailed and well documented site visit to visually assess the condition of the structural elements is required. Movement of monoliths, including tilting and lateral translation, should be noted and documented. Use of the USACE inspection checklist is encouraged to provide an orderly basis for assessing the levee system's condition.

(4) Review Design Analysis. The next step is to procure a copy of the design analysis, including calculations, prepared by the original designers. This is used to determine how the structural elements were designed, including what loading was selected. The results of the original analyses should be compared to current USACE guidance to verify that the structures meet current design requirements. If the original design analysis is not available, then a detailed analysis may be performed in accordance with current USACE guidance. This step is outlined in some detail within this guidance document (see paragraph 9.a). The actual condition of the structures, gleaned from either a site visit or a review of the most recent inspection reports, may cause the section properties in the analyses to be revised from the original analyses.

(5) Fitness-for-Purpose. Alternately an evaluation can be performed to determine if the structural elements are "Fit for Purpose," similar to the methodology proposed in ERDC TR-07-15, *Fitness-for-Purpose Evaluation of Hydraulic Steel Structures*, published in November 2007. The basis for a Fitness for Purpose analysis is that hydraulic structures may have fabrication defects that, while not allowed based on stringent interpretation of the project specifications, may not in fact be harmful to the structure. Similarly while a structural floodwall element that was not designed and constructed strictly in accordance with current standards may not be expected to meet all of those standards, shortcomings in regard to those standards may not actually be harmful to the performance of the structure during a flood event. The analysis considers a particular structural component to be adequate as long as the conditions which would lead to failure are not reached. The method of this analysis is ultimately left to the judgment of the engineer. The technical evaluation must include a justification for performing a Fitness for Purpose analysis, and also a description of the method of analysis. The determination of whether a levee system meets NFIP levee system evaluation requirements should be based on the results of the prescribed evaluations as well as accompanying engineering judgment whether there is a high likelihood of adequate performance of the structure or component when subjected to the 1% annual chance exceedance flood. Finally the lessons learned from Hurricane Katrina, as outlined in the IPET report, must be reviewed to see if any of the levee system's structural elements, particularly I-walls, need to be further evaluated in light of the catastrophic failures that occurred in New Orleans. This includes the possibility of barge impacts to floodwalls or other structural features (see Paragraph 9.g.(2)(c)).

(6) Deterministic Criteria. At the present time, deterministic criteria should be used to perform the NFIP levee system evaluation. The deterministic criteria will be based on the current stability and strength criteria developed from and based on current design standards. Analysis should show the existing structure (floodwall monolith – T-wall, I-wall, or L-wall; closure monoliths; closure gates; pump stations; and gatewells) meets the criteria below. It is recommended that at least one typical monolith of each size range on the levee system should be evaluated. For example, if there are 30 different 10-ft high T-wall monoliths on the levee system, 60 different 15-ft high T-wall monoliths on the levee system, and 120 different 20-ft high T-wall monoliths on the levee system, an analysis of one such monolith of each height should be

completed. However, if the soil conditions are noted to be different for some of the monoliths, an analysis of one such monolith of each height for each discrete foundation condition should be completed. An engineer specializing in geotechnical engineering with specific experience and knowledge of flood damage reduction systems should be consulted when determining how many different foundation conditions must be evaluated for the levee system. Selection of the monoliths to be analyzed is ultimately left to the judgment of the engineer. The technical evaluation must include a description justification of the section selection and also a description of the method of analysis.

(7) Stability Requirements for Concrete Structures.

(a) Structural stability criteria shall be based upon EM 1110-2-2100, *Stability Analysis of Concrete Structures*, USACE, 1 December 2005. Required factors of safety for sliding, flotation, and resultant location (overturning) are provided in Tables 3-2, 3-4, and 3-5, respectively, in EM 1110-2-2100. Using the above criteria, the engineer should establish the stability factors of safety for all structures on the levee system. This includes pump stations that are integral parts of the levee system. All analyses shall be prepared and submitted as part of the NLSER. A determination must be made whether the structure(s) in question are defined as “Critical” or “Normal”; Appendix H of EM 1110-2-2100 provides guidelines for making this determination. In general, however, a flood damage reduction system for which FEMA NFIP levee system evaluation is sought would have people within the leveed area and would be considered a critical structure. Once this determination has been completed, the Load Condition Category must be determined. For purposes of NFIP levee system evaluation, the 1% annual chance exceedance flood elevation shall be used which, by definition, is considered an “Unusual” event. The next step is to determine whether the site information available is “Well Defined” or “Ordinary.” EM 1110-2-2100 does not allow a structural evaluation to be completed for a Critical structure for which only “Limited” site information is available; for such a case, NFIP levee system evaluation cannot be completed unless additional site information is obtained. The definition of “Well Defined,” “Ordinary,” and “Limited” are presented within EM 1110-2-2100.

(b) Seismic Stability. Structural floodwalls and pump stations within levee systems that are subjected to water loading for extended periods of time every year shall be evaluated for seismic stability. The median annual water level shall be combined with the 1% annual chance exceedance earthquake (100yr return period) load for the evaluation. The peak ground acceleration (PGA) should be determined for the levee system using the current United States Geological Survey (USGS) ground-motion database ground motion parameter calculator found at http://earthquake.usgs.gov/research/hazmaps/design/. If the PGA for the 100 year earthquake is less than 0.10g, no evaluation is required. For PGA greater than 0.10 g, structural stability should be checked using the seismic approach discussed in Appendix G of EM 1110-2-2100, *Stability Analysis of Concrete Structures*, USACE, 1 December 2005. Structures which meet all stability factors of safety requirements when evaluated by this procedure are considered to be safe and no additional seismic stability analyses are required. Structures with an indication of an inadequate factor of safety cannot be found to be in accordance with NFIP levee system evaluation requirements unless a more rigorous seismic deformation study demonstrates the wall system will have minor deformation and will continue to provide reasonable assurance of excluding the base flood from the leveed area. The failure to meet these requirements should only suggest the need for other seismic coefficient and dynamic analyses to fairly assess the demands placed on the structure and foundation during a major earthquake. From these advanced analyses engineers can determine if the displacements and stresses experienced by the structure and foundation will place the structure at risk of a stability failure. In many instances, it is acceptable for sliding and rocking to occur at the base of the structure during extreme earthquake load conditions. Stability in such cases is evaluated using dynamic analysis methods, and performance is ensured by limiting permanent displacements to acceptable levels. Concerns include relative sliding between adjacent monoliths; if relative movement is great enough, waterstops could fail and excessive leakage could occur if repairs are not made in time. Where sliding of floodwall monoliths, pump stations or relative movement between floodwall monoliths and pump stations are indicated, the capability to repair all earthquake-damaged reaches prior to the next flood event shall be taken into account in the NFIP levee system evaluation process.

(c) Barge Impacts. During a flood event the risk exists that a loose barge could impact a floodwall located directly along a navigable waterway. Such an impact has the potential to impart significant loads. It is not necessary to check floodwalls for barge impact if the walls are located behind lines of trees or buildings that would intercept a loose barge during a flood. Floodwalls located away from the navigation channel do not need to be evaluated for stability in the event of a barge impact. For floodwalls located directly adjacent to navigation channels, the methodology outlined in Appendix B of ETL 1110-2-563), *Engineering and Design: Barge Impact and Analysis for Rigid Walls*, 30 July 2004, may be used to develop the impact load. It is recommended that a single empty inland barge should be used as the basis for the barge mass. Since levee systems are generally oriented roughly parallel to the orientation of the waterway, a loose barge will travel parallel to the direction of river flow. Therefore use the lower range of the "Usual" impact angle of 5 degrees; use either the "Unusual" or "Extreme" parallel flow velocity of between 3 to 6 feet/second (as appropriate for the levee system's 1% annual chance exceedance event); and use the lower end of the "Usual" lateral flow velocity range of about 0.1 feet/second. This impact force should be applied to the floodwall at a series of elevations ranging from the water surface up to seven feet above the design water surface. Selection of the monoliths to be analyzed for barge impact, and the magnitude of the impact forces applied is ultimately left to the judgment of the engineer. The technical evaluation must include a description/justification of the selection and also a description of the method of analysis.

(d) Review of Inspection Reports. The NFIP levee system inspection report and latest Routine/Periodic Inspection reports for the levee system shall be reviewed to see if any conditions or performance issues are noted that would reduce confidence in the factors of safety already determined by analysis. For example, an “Unacceptable” rating for a floodwall monolith because a monolith has tilted between two recent inspections is likely sufficient reason to suspect that the floodwall may not have the capacity to safely withstand the 1% annual chance exceedance flood. In such a case, additional analysis is needed. It may be that the movement of the monolith occurred as a result of a truck collision with the wall, and that the wall still will provide a reasonable assurance of excluding the 1% annual chance exceedance flood from the leveed area. Any such analysis performed in response to such issues shall be prepared and submitted as part of the NLSER.

(8) Concrete Structures Strength Requirements.

(a) American Concrete Institute (ACI) 318, *Building Code Requirements for Structural Concrete*, Chapter 20, and ACI 437, *Strength Evaluation of Existing Concrete Buildings* both provide a basis for evaluating the strength of existing flood damage reduction structures. Paragraph 4.2.1 of ACI 437 states that evaluation solely by analysis can be performed if sufficient information is available about the physical characteristics of the structure, and if load testing is impractical or unsafe. It further states that analytical evaluation is appropriate if an accepted methodology exists for analyzing the type of structural system under consideration, and if characteristics of the structural elements can be determined and modeled within acceptable limits of error. Accepted methodologies exist for analyzing flood damage reduction structures. These methodologies are discussed in paragraph 9.g.(3)(c) below.

(b) ACI 437 paragraph 4.2.1 also implies that, if the conditions outlined above do not hold true for the structure or system under consideration, evaluation by analysis together with in-place load testing is appropriate. This is the basis for relying heavily and placing a high value on past performance of levees systems during floods (discussed in detail in paragraph 9.h.(3) of this document). It may be very difficult and expensive to perform full-scale load testing of flood damage reduction system components because of the effort required to provide the flood loading. In general a cofferdam must be built on the flood side of the structure being tested; inside this cofferdam the test flood load must be impounded. Additionally instrumentation should be installed to measure deflections, stresses, and piezometric head. If load testing is to be used, the duration of the hydrostatic loading should match or exceed the estimated duration of the river system's 1% annual chance exceedance with 90% confidence flood. Consideration should be given to overloading the wall segment(s) to an elevation higher than the 1% annual chance exceedance with 90% confidence flood elevation because a cofferdam's relatively small scale can result in the reduction of the global forces applied to the soil-structure system. A decision to perform a load test is ultimately left to the judgment of the engineer. The technical evaluation must include a description justification for the selection of the monolith(s) to be load tested, the duration of the load, and the height of the load. Additionally a detailed description of the load test and any instrumentation should be provided in the technical evaluation. Full-scale flood load tests were performed for I-walls in Paducah, Kentucky in 1939; at Tell City, Indiana in 1941 and on a series of specially built T-walls in Cincinnati, Ohio in the early 1950s. At the time of this document's publication, existing I-walls were being tested along one of the New Orleans Canals near Lake Pontchartrain. None of the test programs listed here included tests to failure.

(c) Analysis of existing flood damage reduction structures should be performed using the design methods outlined in EM 1110-2-2104, *Strength Design for Reinforced-Concrete Hydraulic Structures.* This Engineering Manual is consistent with the design methodology of ACI 350, *Code Requirements for Environmental Concrete Structures*. These documents both utilize the Strength Method for design, which uses partial load and resistance factors that are calibrated to achieve a structural design with uniform reliability for all members across a range of loading sources such as wind, gravity, and applied live load for all limit states. For example, live loads (such as flood events) are more random and have higher uncertainty than dead loads; hence, code calibration results in a higher load factor for live loads than dead loads. On the strength side of the equation, the strength reduction factor is calibrated to account for the accuracy of strength prediction models, repeatability/inspectability of construction methods, and the type of failure mode (brittle or ductile). The strength prediction model for flexure is more accurate than shear; hence, the strength reduction factor is higher for flexure than shear. Multipliers less than 1.0 are generally applied to load combinations of wind, live load, dead load, and earthquake to reflect the lower joint probability of all events occurring at their maximum magnitude at the same time; the joint probability is low. EM 1110-2-2104 requires that the Single Load Factor Method be used, which requires that both the dead and live loads be multiplied by the same load factor of 1.7. Because flood damage reduction structures are all considered hydraulic structures, all analyses shall include multiplying the basic load factor by the additional hydraulic load factor, H_f. EM 1110-2-2104 does not require that earthquake loads be combined with flood loads in analysis. For the NFIP levee system evaluation, therefore, earthquake loads do not have to be included in any load combinations evaluated.

(d) Appropriate concrete and reinforcing steel strengths should be used in the analysis. Table 3.1 in ACI 437 provides yield strength properties for reinforcement going back to 1911. The evaluating engineer should review as-built drawings and specifications, if available, to determine the steel type and grade used. If as-built information is not available, the best estimate should still be made using the referenced Table 3.1 together with the closest known age of the component being analyzed. For concrete, the best way to determine compressive strength is to test core samples in accordance with paragraphs 3.1.1.1 and 3.1.3.1 of ACI 437, as there are no in-place tests that provide direct measurement of the compressive strength of concrete.

(e) Using the above criteria, the structural engineer should establish that the required strength is at least equal to the effects of factored loads for all structures on the levee system. It is not intended that each individual floodwall monolith be analyzed, but that representative structures for the levee system be analyzed. This includes pump stations that are integral parts of the levee system. For example, if any structures on the levee system are in deteriorated condition, those structures should be analyzed utilizing reduced section thicknesses and reduced reinforcement cross sectional areas, as would be appropriate for the structure. Such damaged or compromised structures are likely weak points in the levee system and, therefore, represent elements with a higher likelihood of failure. Additionally, all representative floodwall types (I-walls, T-walls, and L-walls) should be analyzed, and a range of heights should be represented, including the maximum wall height for each representative structure. It is acknowledged that older existing pump stations may not meet this test. Older design criteria did not require minimum reinforcement and will not meet current criteria. Therefore an alternate evaluation can be performed to determine if the structural elements meet the "Fitness for Purpose" criteria. The analysis considers a particular structural component to be adequate as long as the conditions which would lead to failure are not reached. The method of this analysis is ultimately left to the judgment of the engineer. The technical evaluation must include a justification for performing a Fitness for Purpose analysis, and also a description of the method of analysis. All analyses shall be prepared and submitted as part of the NLSER.

(9) Closure Evaluations. Closure structures, which are temporary structures placed in openings in the levee system such as rail and other transportation crossings, shall be evaluated based on three primary factors as discussed in 9.g.(4)(a) through (c) below:

(a) Ensure that the structure has been designed consistent with the specific characteristics of the flood threat. Considerations include: rate of water level rise; duration of need for closure; and velocity and other relevant hydrologic and hydraulic parameters. Care must be taken to ensure that time is available to accomplish the closure by also designing and implementing an associated flood threat recognition and warning system. This implies that, for a levee system designed for flash-flooding, where the time from flood threat recognition and warning until floodwaters arrive may be only minutes, it is unlikely that an assembled truss/ stoplog type closure or a sandbag closure is appropriate because construction/assembly of these types of closures may not be possible within the time allowed. A swing gate or rolling gate closure would be more appropriate for a situation of short closure times. On a large river system where there may be several days to more than a week of forecast time to rise, closures that require a team of laborers several days to construct may be appropriate. Many levee systems that fall between these extremes will require careful consideration of the reliability of flood forecasts and warning times, hydrograph shape and duration, and analysis of how long a particular closure assembly takes to complete to determine whether or not a certain type of closure is appropriate.

(b) Structural evaluation of steel closure structures (assembled truss closures; slide gates; roller gates; or swing gates) is required. This shall be based upon a determination that the structures satisfy both current design criteria and are in an acceptable service condition and are operating. Structural design review shall be based upon EM 1110-2-2105, *Engineering and Design - Design of Hydraulic Steel Structures*, 31 May 1994 and EM 1110-2-2705, *Structural Design of Closure Structures for Flood Control Project*, 31 March 1994. The condition of the structure(s) shall be determined in accordance with EM 1110-2-6054, *Inspection, Evaluation, and Repair of Hydraulic Steel Structures*, 1 December 2001. Based on the above criteria, verify that all steel closure structures meet all required design criteria, and that the conditions of these structures are acceptable for service. Alternatively, an evaluation can be performed to determine if the closure elements meet the "Fitness for Purpose" criteria. The analysis considers a particular structural component to be adequate as long as the conditions which would lead to failure are not reached. The method of this analysis is ultimately left to the judgment of the engineer. The technical evaluation must include a justification for performing a Fitness for Purpose analysis, and also a description of the method of analysis. All analyses shall be prepared and submitted as part of the NLSER.

(c) An operational analysis of the closure structure must demonstrate confidence that the closures can all be placed into service, with all materials being available, and the proper manpower and equipment available for the performance of the required tasks. All closures must be operated in accordance with an officially adopted operation manual. Routine or Periodic Inspection results should be reviewed to verify that all of the levee system's closures have been assembled within the recent past (within Routine Inspection period), that all parts are available, and that non-federal sponsors have work crews available with a working knowledge of how to install the closures.

(d) Special case of sandbag closures. Many of the larger and older levee systems, such as those located along the Ohio and Mississippi Rivers, have sandbag closures that are generally a maximum of about 3 feet in height. These sandbag closures are generally used only at locations well above the 1% annual chance exceedance flood elevation with 90% assurance, and thus do not come into play in NFIP levee system evaluation. For levee systems where sandbag closures are required to provide reasonable assurance of excluding the 1% annual chance exceedance flood from the leveed area, the closures must be designed and implemented as an integral and structurally sound component of the levee system, designed to include proper tie-in to the levee system such that when closed it prevents leakage and resists failure, and demonstrated that the sandbag closures will reliably perform their task as well as other alternative, more permanent closure devices. Such sandbag closures are not considered an emergency flood-fighting measure, but rather shall be a part of the adopted Operation and Maintenance Plan. Additionally, these closures must meet the following criteria: (1) the sandbag closure design must be commensurate with the flood hazard as noted in (a) above, e.g., generally not subject to flash flooding, nor used for extended durations (not to exceed one flood event or one week, whichever is longer), and no more than 3 feet in height; and (2) the inspection report used for the NFIP levee system evaluation verifies that the non-federal sponsor has the required number of sandbags available, has a source of sand readily available, and has a means for filling, transporting and placing the sandbags in a timely fashion prior to a flood event.

(e) Earthen Closures. Earthen closures are acceptable provided the closures meet the requirements outlined in (a) above. Earthen closures with the base below the 1% annual chance exceedance elevation are acceptable; however, the closures must be fully evaluated in respect to the requirements outlined in (a) above. Specific items which must be evaluated for earthen closures include: the fill volume required for the closure including compaction shrinkage, the estimated time to install the closure; a geotechnical analysis to determine the sliding factor of safety for the closure; and an operating plan for the closure that includes a trigger to mobilize and a trigger to install the closure based on the rate of rise, forecasts and/or a flood warning system. The operating plan should include general compaction requirements such as the maximum lift and the number of complete passes with a bulldozer. The plan should also include the source of borrow material including whether is it stockpiled on site or will be obtained from a nearby pit when an event is forecast. A list of acceptable sources should be included in the operating plan if the borrow material will be obtained from a nearby pit. The local sponsor must provide written documentation that they have the personnel, equipment and/or contracting capability to install the closure as outlined in the operating plan. All analyses and the local sponsor documentation shall be prepared and submitted as part of the NLSER. Example 3 in Appendix C is an example closure analysis. The annual inspection report of record should verify that the borrow material is available, the closure geometry has not changed, and the local sponsor states they still have the capability to install the closure.

h. Geotechnical Evaluation Guidance.

(1) Available Information. In performing NFIP levee system evaluation for existing levees, the geotechnical engineer must collect and review all available pertinent information. The list of available resources includes, but is not limited to:

(a) Regional geology reports, site-specific geology reports, aerial imagery, boring logs, soil testing data, foundation material characteristics, and inferred stratigraphy,

(b) Design documents or design memoranda and all design computations,

(c) As-built drawings showing levee geometry, material zoning, construction methods,

(d) All other construction records (such as daily reports, quality assurance/quality control (QA/QC) reports, excavations, dewatering requirements, settlements, utility relocations, and construction failures),

(e) Records of utility crossings constructed by permit,

(f) All post-construction investigations,

(g) Annual and periodic inspection reports, along with the most recent surveys of the levee system and any other available geospatial information,

(h) Groundwater studies, relief well and piezometer installation reports (to include piezometer data during floods) relief well and piezometer maintenance reports, and testing performed on the relief wells and piezometers,

(i) Instrumentation installation reports, instrumentation data, and data interpretation,

(j) Performance history, especially reports and records of performance during floods (see paragraph 9.h.(3) below),

(k) Data on any repairs or upgrades made to the levee system, plus records of permits for any alterations made to the levee since its construction. Alterations include any changes (additions or subtractions) to the as-designed and constructed levee cross section or adjacent foundation soils. Alterations also include construction or abandonment of utilities (conduits, force mains, water lines, oil or natural gas pipelines, electrical or telecommunication cables), roadways, railroads, crossings either over or under the levee,

(l) The levee system's O&M manual,

(m) Geomorphology and Natural Resources Conservation Service maps,

(n) Geomorphologic Analyses.

(2) Site Visit. In conjunction with the field inspection (paragraph 9.d.), the engineer should review all available information including inspection reports and the system's O&M manual. During the site visit/inspection, the engineer shall take care to observe, assess, and document the condition of the levee system, including the adequacy of the maintenance afforded the levee system. During the site visit the engineer should:

(a) Note any physical damage to seepage control measures (relief wells, seepage berms, cutoffs, riverside blankets, evidence of sand boils). Interview maintenance personnel about the proper functioning of relief wells, seepage berms, cutoffs, riverside blankets during recent flood events. Examination of recent pumping tests for relief wells, and any other measured seepage, is suggested. Verify that the levee's turf provides adequate cover, is regularly mowed, and that all undesired vegetation, especially woody growth, is controlled. Note where large trees have been felled but the roots have not been grubbed out. Where roots have been grubbed out, note if repair fill was properly placed. The engineer should note any locations where turf does not provide adequate cover, such as areas of livestock grazing or bike paths.

(b) Note any locations of erosion or scour due to runoff, current velocity, or wave run-up, and where riprap or other slope protection systems are failing. Check erosion adjacent to concrete structures.

(c) Note any locations and treatment (if any) of burrowing rodent activity, including tunnels, dens, or nests.

(d) Note conditions at all active utility crossings, especially conduits through or under the levee system. Special attention shall be paid to known locations of abandoned utility crossings. Note if pipes are removed or grouted “after” abandoned utility crossings. Identification of deteriorated conduits through the levee must be considered dangerous and may be cause for a negative finding of NFIP levee system evaluation. Note any obstructions, undermining, leaking, and clogged trash racks.

(e) Note locations where the existent embankment cross-section or template is substandard or irregular with respect to as-built drawings. Note locations of settlement, cracks, or signs of slope instability (slides, slumps, and tension cracks).

(f) The engineer should note any encroachments or alterations in the levee system not identified by the post-construction permitting process. The engineer should also note any activities immediately adjacent to the levee, even those that do not appear to affect the structural integrity of levee.

(g) Note access road conditions for ruts and ponded water.

(3) Previous Flood Fighting Information. A very important piece of information for NFIP levee system evaluation available to the geotechnical engineer is knowledge of how the levee system performed during past floods. If the system in question has successfully withstood a 1% annual exceedance flood event, records of its performance during that event will provide essential information to the NFIP levee system evaluation. Levee performance can be determined by reviewing flood fighting records and other written accounts of flood performance and from interviews with witnesses having first-hand experience during the flood event. These witnesses can be levee commissioners, flood fighting personnel, and property owners. This review should identify weak spots and other problem areas in the levee system. Any observed and documented deficiencies should be considered in the NFIP levee system evaluation process. Performance records of seepage related issues from previous flood events are generally good indicators of future performance. Observed seepage problems at low flood stages may indicate potentially larger problems at higher flood stages. Measured piezometric levels should be plotted against the flood elevation and trends extrapolated to the 1% annual chance exceedance stage. Measured piezometric levels and measured relief well flows should generally match values predicted in original designs. The engineer should determine the locations of seepage-induced soft spots, pin boils, or sand boils (if any), and at what river stage these features appeared. Levee structures that have withstood a 1% annual chance exceedance flood stage are likely to do well again at the

same flood stage. A key item to note is that sand boils noted during one flood event that have become active at lower flood stages in subsequent flood events, especially if a substantial amount of soil has piped out of the foundation in the prior event, likely indicates a problem. This clearly implies that previous piping problems may worsen with each subsequent flood event of equal or even lesser severity such that seepage may be approaching the danger point.

(4) Additional Information. If a definitive determination cannot be made concerning capability of the levee system to withstand the 1% annual chance exceedance flood based on the information review, site visit, and subsequent analyses, the geotechnical engineer may determine that additional data must be obtained and additional analyses completed. The request for additional data may require new field exploration, soil testing, or surveying to help characterize existing conditions. Additional analyses may also need to be performed to resolve problem areas and to quantify missing information. The geotechnical engineer should understand and identify the likely potential failure modes when determining the need for additional investigations and analyses. These modes are identified in paragraph 9.h(5) below.

(5) Identification of the Potential Failure Modes. Identifying and understanding potential failure modes are the basis for scoping and prioritizing additional investigations and analyses. The typical and significant failure mode categories are:

(a) Piping, uncontrolled seepage through the levee, through the foundation, or through the levee into the foundation can all cause failure of the levee.

(b) Levee stability, sliding or deformation may be the result of insufficient strength of levee fill and/or foundation materials to resist gravity and seepage forces, may be induced by seepage-induced excessive pore pressures, or could be induced by an external loading.

(c) Failure from erosion on both the water side (riverine and wave-induced erosion) and landward side (from overtopping) will be considered.

The likely potential failure modes should be identified. For existing levee structures, review the NFIP levee system evaluation inspection results and the latest Routine or Periodic Inspection reports should be performed to see if there are conditions or performance issues relevant to the potential failure modes described above.

(6) Seismic Stability. Levee systems that are subjected to water loading for extended periods of time every year shall be evaluated for seismic stability. The median annual water level shall be combined with the 1% annual chance exceedance earthquake (100yr return period) load for the evaluation. The peak ground acceleration (PGA) should be determined for the levee system the using the current United States Geological Survey (USGS) ground-motion database ground motion parameter calculator found at http://earthquake.usgs.gov/research/hazmaps/design/. If the PGA for the 100 year earthquake is less than 0.10g, no evaluation is required. For PGAs greater than 0.10g, the levee and its foundation should be checked for liquefaction using a simplified approach (Seed, et al. 2003). Where liquefaction is indicated, the geotechnical engineer should perform a post-earthquake limit equilibrium stability analysis using an estimate of undrained residual strength for the liquefied soils. These estimates should be based on published empirical correlations (Idriss and Boulanger 2007). Appropriate drained or undrained soil strength

parameters should be used elsewhere. If post earthquake factors of safety are greater than 1.2, then no further evaluation is necessary. For factors of safety less than 1.2, a more detailed seismic deformation analysis will be required to determine how the levee will perform in the seismic event. Levee systems with an indication of widespread liquefaction and/or an inadequate post-earthquake factor of safety cannot be found compliant unless a more rigorous seismic deformation study demonstrates the levee system will have minor deformation and will continue to provide the required design level. Detailed seismic evaluations should assess the impacts on relief wells, toe drains, or other seepage control measures linked to the stability of the levee. Damage to these measures, the loss of (or reduced effectiveness of) these features could result in levee failure. Seepage control measures are typically located along the protected side levee toe, and are especially susceptible to damage from even relatively minor shear deformations and differential settlements.

(7) Analytical Methods. The analyses and criteria described in EM 1110-2-1913, *Design and Construction of Levees*, and ETL 1110-2-569, *Design Guidance for Levee Underseepage*, should be used as a guide to evaluate levee structures for NFIP levee system evaluation. ETL 1110-2-569 made changes to EM 1110-2-1913, these changes are current USACE policy and should be used instead of the requirements in specified paragraphs in the EM. Failure modes and features subject to failures which must be considered include, but are not limited to: erosion, erosion protection, and erosion rates; foundation and levee stability; through-seepage, underseepage, performance of relief wells, and seepage-induced piping; structural performance and stability of floodwalls (including L-, T-, and I-walls); effectiveness of gravity drain closure structures; corrosion of drain pipes; bearing capacity; settlement; and overtopping performance. All of these failure modes shall be evaluated to ensure the levee system meets criteria established in all applicable guidance.

(8) Method of Analysis. For the near future and until appropriate uncertainty-based protocols are developed and fielded by future guidance, NFIP levee system evaluation in the area of geotechnical engineering shall be based on widely accepted deterministic analyses using appropriate factors of safety against unacceptable performance, experience, and engineering judgment. The minimum stage for the geotechnical deterministic analysis shall correspond to the 90% non-exceedance (assurance) of the 1% chance exceedance flood. Uncertainty analyses may be performed to enhance the geotechnical engineer’s understanding of the fragility of various system components. Such analyses will also help the engineer understand the impact of potentially random variables (such as widely varying material properties and in-situ conditions) on levee integrity and the probability of failure. The results of uncertainty analyses shall be used as an aid to experience and engineering judgment as to whether levees are stable with respect to specific failure modes. However, at this time, the existing uncertainty assessment tools available to the USACE Geotechnical Community of Practice only determine probabilities within relative orders of magnitude. Mature probability and uncertainty assessment procedures are not presently available that allow estimates of the probability of failure with the accuracy needed to incorporate these probabilities into the overall NFIP levee system evaluation process.

i. <u>Electrical and Mechanical Components</u>.

(1) General. Stormwater pump stations generally operate as a component of the interior drainage system. The interior drainage system is the system of culverts, canals, ditches, storm

sewers, and drainage structures with associated gates and valves that convey interior water from rainfall or seepage by gravity to outside the leveed area, to interior ponding areas, or to a pump station to be pumped outside the leveed area if gravity drainage is not possible. Failure of any of these components could prevent water from collecting in ponding areas or from being pumped from the leveed area causing flooding or damages to structures or infrastructure. This section will address how the electrical and mechanical components of the interior drainage system, namely pump stations and gates and valves, will be incorporated into the NFIP levee system evaluation process. Evaluation of the structural components of stormwater pump stations and other interior drainage features is discussed in Section 9.g. A significant cause of failure of pump stations has been uplift. A discussion of evaluating pump stations for stability under various loading conditions is included in Section 9.g.

(2) Probability-based Analysis. Characterizing the performance of pumping stations probabilistically is possible, but with varying degrees of uncertainty, depending on the level of analysis performed and the accuracy of information on component failure rates. The general approach would be to determine these failure rates and apply them in properly constructed event or fault trees. For such analysis, two more components are required to calculate the risk of pump station failure besides the probability of failure: the probability of loading and the consequences of partial or complete pump station failure. The probability of loading would be the coincidental probability of both the base flood on the river and on the interior drainage system, producing a given inflow to the pump station. Consequences would typically be economic damages resulting from partial or complete failure of a pump station. The hydraulic loading information will likely be available from the interior drainage analysis [paragraph 9.f.(9)], but the aged pump station performance data will most likely be problematic. As a consequence, an alternate assessment method is presented based on the concept of ‘Condition Assessment.’ In the future, as performance data for aging pumping stations of reasonable fidelity become more readily available, and probability and uncertainty analysis methods and tools mature, the preferred analysis method is expected to evolve consistent with a comprehensive risk-based event and fault tree approach.

(3) Condition Assessment. Condition assessment is the analysis method that will be used in the NFIP levee system evaluation to determine failure modes of pump stations and drainage structures. Performance of certain components of the pump station may be affected by the static head encountered such as the pumps and drivers and gates and valves. These components should be evaluated for satisfactory performance up to the 1% event. In the case of pumps and drivers, this evaluation may only be necessary in order to determine the suitability of replacement or retrofitted equipment to meet the original design criteria. The USACE inspection checklist for levee systems provides the process to evaluate the condition of those components contributing to the failure modes. An “Unacceptable” rating for any components that directly contribute to one or more of the failure modes could be cause for the levee not receiving a positive finding for NFIP levee system evaluation. An “Unacceptable” rating is defined as the deficiency being so serious that the item will not adequately function in the next flood event, compromising the levee’s ability to exclude the base flood from the leveed area. Judgment should be exercised in determining the significance of component(s) with unacceptable ratings. For example, a single inoperable 12-in. flap valve may not be cause for a negative finding of NFIP levee system evaluation, because the volume of water that could enter the leveed area would not cause damages in the amount of time the water level would cover the opening. A similar judgment

should be made in the case of pump stations. If, over time, the purpose of a pump station has changed to the point where it is no longer considered a critical component of the interior drainage system, then only those components of that pump station contributing to active failure modes should be considered. For example, those components could include flap valves or sluice gates preventing exterior water from entering the interior. Changes to pump stations and other components should be evaluated against mapping of interior flooding, particularly if evidence points to these changes having a relationship to increased interior flooding.

(a) The failure modes for pump stations that will be assessed include failure of the pumps, drivers, controls, or the backflow of water through the pump station to the leveed area. The following list of items contributes to these pump station failure modes and should be considered in a NFIP levee system evaluation:

Plant building,
Pumps,
Motors, engines, fans, gear reducers, and back stop devices, etc.,
Power supply,
Megger testing on pump motors and critical power cables,
Motor control center,
Enclosures, panel, conduit, and ducts,
Circuit breakers,
Instruments,
Sumps/wet well,
Intake and discharge pipes,
Flap gates/flap valves/pinch valves (when component of pump station),
Mechanical operating trash rakes,
Non-mechanical trash racks,
Structural failures (uplift, stability, etc., see Section 9.g)
Sluice/slide gates (when component of pump station), and
Fuel system for pump engines.

In addition to the criteria defined for “Power supply,” the reliability of the commercial power system and presence or lack of backup power should be considered. If commercial power is judged to be unreliable, then backup power must be provided in the form of an alternate commercial power source, a properly sized generator set, or provisions for connection of a properly sized generator set that is readily available in an emergency. Commercial power reliability in conditions possible up to the 1% event should be investigated with cooperation of the local utility. Frequency and duration of outages and the size and importance of the pump station should be considered when determining the need for alternate power provisions. These considerations are described in Chapter 13 of EM 1110-2-3105, *Mechanical and Electrical Design of Pumping Stations*.

(b) Pump stations that are part of hurricane protection systems merit additional scrutiny of certain features that may contribute to station failure only as a result of exposure to a hurricane. These features include the ability of the pump station structure and equipment to withstand high winds, the provision of safe housing for operators, and the ability of the pump station to be operated while not exposing the operators to dangerous conditions. For example, trash raking equipment should not require an operator outside during raking operations. During Hurricane Katrina, a significant proportion of pump stations were abandoned. This resulted in not only the loss of pumping capacity but, combined with the loss of commercial power (which is more likely in a hurricane), significant backflow in several cases. The mechanical engineer shall review all of the assembled data, reports, levee system photographs, plans and specifications with emphasis given to the IPET recommendations and lessons learned in their report: *Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System* Draft Final Report. The text of this report is available at https://ipet.wes.army.mil/. In particular, Volume VI, The Performance – Interior Drainage and Pumping, and the appendices under Volume VI shall be reviewed and the levee system under consideration for NFIP levee system evaluation shall be considered in this context. A result of the lessons from Katrina has been the retrofitting of pump stations with hurricane safe houses to lessen the risk of abandonment by operating personnel under threat of potentially unsafe conditions. Hurricane safe house should meet the requirement of ASCE 7, Minimum Design Loads for Buildings and Other Structures, for critical structures. Pump stations in hurricane prone areas should be evaluated against these criteria.

(c) The mechanical failure mode of concern for drainage structures, pipes, and other conduits that convey water out of the leveed area is the drainage structure becoming unable to exclude water from the interior during flooding. This can happen due to failure of a flap or pinch valve or due to a sluice gate in the open position. An “Unacceptable” rating of any one of these components could be cause for a negative NFIP levee system evaluation.

(d) Mechanical or electrical components necessary for the operation of closure structures should be considered in NFIP levee system evaluation. These components would include winches, rollers, etc. An unsatisfactory rating of any of these components could be cause for a negative finding of NFIP levee system evaluation. If the structure can be closed in a reasonable time without the use of a component that has an unacceptable rating, then that consideration may be used in making a determination for NFIP levee system evaluation. For example, a gate that relies on a winch for closure may also be closed with help of mobile machinery if the need arises.

(e) If it is likely that a pump station or other drainage structure is in an earthquake prone area, a seismic investigation of the structure should be conducted in accordance with EM-1110-2-3104 *Structural and Architectural Design of Pumping Stations*. If the investigation finds that damage from earthquakes is possible, the pump station should be evaluated to determine if mechanical and electrical components are anchored or supported in accordance with UFC 3-310-04.

j. System Evaluation. The objective of the NFIP levee system evaluation is to verify that the levee system performs as an integrated set of features and components functioning individually and collectively to provide reasonable assurance that the 1% annual chance exceedance flood will be excluded from the leveed area. The focus is thus upon the levee system that is associated with a given separable floodplain (or separable consequence area in an extensive floodplain). The term ‘system’ as used herein thus is inclusive of all components, and associated interfaces and interactions among components, that are necessary to ensure flood risk reduction of the associated floodplain – levee sections, closure structures, pumping stations, culverts, interior drainage works, any other works or components not necessarily designed/built as components of the flood system, and system O&M. Such analysis will generally require coordinated multi-disciplinary analysis wherein the team in consensus, should seek and understanding of the risks and how best to manage them. In some instances, for example, the system may include components such as high ground areas, road and railroad embankments, bridge abutments, navigation walls and channels, etc., and the system analysis must ensure that these components also meet design guidelines. Note that reaches of levee systems can be determined to be found in accordance with NFIP levee system evaluation requirements if the associated floodplains (or consequence areas in an extensive floodplain) are sufficiently separable as to be unaffected by performance of other reaches of the levee system. In such circumstances, this could then describe two or more separate systems.

(1) As the analysis method matures and becomes adapted to levee systems, Potential Failure Mode Analysis (PFMA) may provide a useful framework for system evaluation wherein possible failure modes, including interfaces and interactions among components, are postulated and analyzed. Examples of potential failure modes occurring at component interfaces might include: (1) erosion on the water side undermining embankment or floodwall stability; (2) permissible overtopping criteria for levee overflow that might result in erosion on the landward side that would degrade the embankment or floodwall stability; (3) deteriorating drainage pipe through levee detected in inspection that might provide critical through-seepage path; (4) intersection of structural features such as levees abutting floodwalls; or (5) ponding at interior drainage works that might saturate and weaken the land-side of levee system.

(2) Identifying Critical Locations and Considering Complex Systems. Levee systems must be parsed into their individual components and manageable length increments and then each systematically evaluated based on their specific location and features.

(a) The levee system associated with a specific floodplain could comprise several lengths and combinations of features. This might include a relatively short reach of levee or long reach of levee. The levee system could also be on a single stream or section of lake shore or coastline. The levee system could also comprise several reaches of levees on a mainstream or coastline and one or more tributaries or even completely separate streams, coastline, or lake shore. Levee system reaches will likely have other appurtenant features, such as interior drainage culverts, closure structures, outlet gates, pumping stations, and perhaps other works. The complexity of the system will determine the degree to which a ‘systems components’ perspective is needed for the NFIP levee system evaluation. Note that, as probability and uncertainty analysis technology evolves, it is expected that eventually it will be possible to describe the performance characteristics of each component of simple or complex systems and perform a system uncertainty analysis to make the NFIP levee system evaluation. In the interim, the following basic concepts are presented for application.

(b) For a levee system comprising a short reach with the usual embedded features of culverts and closure structures, it is appropriate to conduct a preliminary investigation to identify the critical location or feature (levee height with respect to flood profile, embankment/structural strength, etc.) and then perform the analysis for that feature/location. If the critical location(s) or component(s) pass the criteria of excluding 1% annual chance exceedance flood from the leveed area, then the levee system would be determined to be found in accordance with NFIP levee system evaluation requirements, provided that the system evaluation that considers interaction among components would validate the critical section analysis findings. Evaluation of backup systems and other alternatives for ensuring performance should be examined.

(c) For a levee system comprising a long reach with the usual embedded features, there may be several critical features at several locations along the leveed area that will need to be analyzed for performance. Typically, the most critical of the several critical locations analyzed will govern the NFIP levee system evaluation. This would be the circumstance if the levee reaches surrounding the floodplain are subjected to essentially the same event/loading, thus loading at the several sections would be considered to be perfectly correlated.

(d) For levee systems comprising of two or more systems providing flood water exclusion from main stream or coastal or lake flooding, tributary, or independent streams, the appropriate systems approach will depend on each site-specific circumstance. **It is important to note that, when flooding may occur from more than one source, the likelihood of the floodplain experiencing flooding is higher than the likelihood from either source independently.** The following flooding circumstances and approach for system performance determination is presented as general guidance. Note that the required analysis is focused on the floodplain and is based on the flood water exclusion performance of each levee reach expressed as Annual Exceedance Probability (AEP) of each reach.

Main stem or coastal or lake flooding, and tributary or two separate streams: For perfectly correlated flood loading (e.g., 1% annual chance exceedance flood loading would occur on both simultaneously), then the condition of paragraph 9.j.(2)(c) above applies.

Main stem or coastal or lake flooding and tributary or two separate flooding sources: For completely independent flood loading (e.g., loading may occur for one or the other or both but are uncorrelated), the system flood water exclusion performance is determined from the Total Probability Methods – see *Probability, Random Variables, and Stochastic Processes,* 2nd ed. New York: McGraw-Hill – as noted in the following formula:

$$P(S) = P(A) + P(B) - P(A)*P(B)$$

where:

P(S) = AEP of flooding the floodplain from the levee system
P(A) = AEP of flooding from capacity exceedance or failure from stream/coast/lake A
P(B) = AEP of flooding from capacity exceedance or failure from stream/coast/lake B.

For example, if the two independent flooding sources are both provide for excluding the 1% annual chance exceedance flood, then the chance of the floodplain experiencing flooding would be: .01+.01-(.01)*(.01) or about 2%. In this circumstance, the system comprising these two reaches could not be determined to be in accordance with NFIP levee system evaluation requirement as having reasonable assurance of excluding the1% annual chance exceedance flood from the leveed area.

Main stem/coastal/lake source and tributary or two separate streams: For partially correlated flood loading (e.g., loading is correlated but does not always occur simultaneously on both streams), the system protection is determined from the two conditions as follows:

$$P(S) = P(A) + P(B) – P(A)\cap P(B)$$

where P(A) and P(B) are as before, and P(A)∩P(B) is the probability of both A and B occurring simultaneously.

For floodplain with flood water excluded from three (or more) streams/coasts/lakes: For systems comprising several perfectly correlated flood loading (1% annual chance exceedance flood loading will occur on all simultaneously), the condition of paragraph 9.j.(2)(c) above continues to apply.

For systems comprised of three completely independent/uncorrelated sources, system performance is determined from the following formula:

$$P(S) = P(A) + P(B) + P(C) – P(A)*(P(B) – P(A)*P(C) – P(B)*P(C) + P(A)*P(B)*P(C)$$

where:

P(S) = AEP of flooding the floodplain from the levee system
P(A) = AEP of flooding from capacity exceedance or failure from stream/coast/lake A
P(B) = AEP of flooding from capacity exceedance or failure from stream/coast/lake B
P(C) = AEP of flooding from capacity exceedance or failure from stream/coast/lake C.

For complex systems with more than three flooding sources, particularly for systems with partially correlated flooding sources, (e.g., loading from several sources is correlated but will not always have the same exceedance probability), the analysis can be quite complex and likely will require customized case-specific analysis. In this instance it is recommended that experts be engaged that specialize in complex system uncertainty analysis, such as staff from USACE Engineer Research and Development Center (ERDC) and USACE Institute for Water Resources, Hydrologic Engineering Center (HEC).

k. Residual Risk and Public Safety. NFIP levee system evaluation is only concerned with the levee system performance associated with the 1% annual chance exceedance flood. While only one of the elements of residual risk and public safety (Emergency Response Plan, paragraph 9.k.(4)) is a required technical factor in the NFIP levee system evaluation, USACE will examine and report in the NLSER, several other elements of residual risk and public safety as noted in paragraphs 9.k.(1) through 9.k.(4). The issues addressed here are focused on residual risk and public safety that are significant during the occurrence of flood events exceeding the capacity of the levee system, which includes floods that may result in overtopping or floods that may result in system failure prior to overtopping. The elements to be addressed here are: probability of capacity exceedance; consequence of capacity exceedance; levee system features that address capacity exceedance; and emergency plans to ensure public safety in the event of a flood that exceeds the levee system capacity or results in failure prior to capacity exceedance.

(1) Probability of Capacity Exceedance. The threat to floodplain residents and businesses is best described by the probability of flooding of the floodplain. The probability is to be estimated based on the information compiled on flood flow and stage and associated uncertainties, levee embankment and associated structures potential failure probabilities and closure devices, interior drainage facilities and other component operations analysis. For levee systems that have high

probability of excluding flood water to the top of the levee, the probability of capacity exceedance is the overtopping probability. For levee systems that may fail before overtopping, the probability of capacity exceedance is the failure probability. Probability of capacity exceedance is to be expressed as AEP as defined herein and in USACE ER 1105-2-101.

(2) Consequence of Capacity Exceedance. The scope and nature of impact on floodplain residents, businesses, transportation systems, other critical infrastructure systems, and the environment will be assessed in general terms and reported. The intent is not a detailed impact analysis, but an assessment that reflects the threat to floodplain occupants that indicates the approximate numbers and demographics of residents, such as Population at Risk (PAR) as may be available in the National Levee Database, business and transportation disruption that would likely occur from one to several reasonably possible capacity exceedances (overtopping or system failures), depending on the extent and complexity of the system. This information would clearly distinguish between floodplains that may experience slow, shallow flooding, for example less than five feet in 24 hours, and those that may experience rapid, deep flooding, for example up to 30 feet in 12 hours or less; floodplains that have adequate routes and capacity for occupant evacuations and those that do not; and floodplains that, if flooded from a levee system failure, could result in the crippling of a regional economy or have significant national economic impact and those that would not. A further consideration is the issue of time that would be needed to repair/reconstruct failed system components and consequent vulnerability of the floodplain during that time period. As concepts and methodologies mature, potential lives lost from capacity exceedance as a measure of life risk is expected to become a key public safety metric.

(3) Levee System Features for Capacity Exceedance. Levee systems may to have features and other means of accommodating the inevitable capacity exceedance. The concept is that, where feasible and practical, the system may have been designed and constructed such that capacity exceedance is accounted for so that, should capacity exceedance likely result in breaching and failure of the levee system, the failure will occur gradually in a predictable manner, allowing orderly floodplain evacuation and minimizing the reconstruction requirements (time and cost) after an exceedance or failure. A description of such measures (if they exist) and a qualitative assessment of their likely performance will be documented in the NLSER. Levee superiority is one physical means for such accommodation, and another is to harden sections to withstand overtopping. See ETL 1110-2-299, *Overtopping of Flood Control Levees and Floodwalls*, for further discussion of this topic.

(4) Emergency Response Plan. The system under evaluation should have an emergency response plan supported by a flood warning system. Such plans are often a component of floodplain management plans developed by USACE and levee system non-federal sponsors as a component of agreements signed when the levee system is transferred to the non-federal sponsor at completion (Reference ER 1105-2-100, Chapter 3, paragraph 3-3). Levee systems developed by others may have such plans developed in support of communities' obligations under the NFIP and associated mitigation grant programs. The emergency response plan should be under the jurisdiction of federal, state, or community officials. The flood warning system must provide warning time sufficient to ensure that the system will be operated as planned during the occurrence of a flood event. The emergency response plans must also seek to maximize public safety from the occurrence of flood events that exceed levee system capacity, demonstrating that the possibility of exceedance and failure has been planned for, and that the plan has a high

likelihood of being successful should such a flood event occur. Emergency response plans will address the key issues of flood threat recognition, warning dissemination, evacuation, and search and rescue. It shall also be demonstrated that such plans are current and tested (updated and tested at an interval of 5 years or less).

10. Documentation and Review.

a. NFIP Levee System Evaluation Report. An NLSER shall be prepared to document and describe the basis for the NFIP levee system evaluation. The NLSER shall contain full documentation of data, information, assumptions, and explanation sufficiently clear so that an individual not familiar with the levee system could review the NLSER and understand how the NFIP levee system evaluation (positive or negative finding) was made. It shall be sufficient to support execution of the review process as described in paragraph 10.c. The following is a general example outline for a NLSER. The report format could be structured by system feature and then by discipline as appropriate. The NFIP levee system evaluation team should add any additional system specific information, considerations, or analysis applicable to the final NFIP levee system evaluation.

I. Table of Contents
II. Summary of the NFIP levee system evaluation findings (signed and approved by the Levee Safety Officer (LSO)). Summary should include key findings, assumptions, and expiration date of the validity period for positive NFIP levee system evaluations
III. System Description (location, levee system authorization (type), main features, and local sponsor, etc). Determine entity responsible for operation and maintenance of the levee, and include pertinent information that illustrates all easements, R-O-W, and ownership boundaries
IV. References (including design documents, reports, as-builts, models, etc. used for the analysis)
V. NFIP Levee System Evaluation Team Members
VI. Previous NFIP levee system evaluation/FIRM or DFIRM
VII. Letter of Intent, Memorandum of Agreement, Scope of Work (whichever applicable) between district and requester
VIII. Overall Performance History/O&M (from inspection reports, past flood events and associated flood-fighting activities, rehabilitation measures, etc.)
IX. Engineering Studies, Investigations, and Analyses
    a. Site Visit Summary (participants, scope, itinerary, and summary of findings)
    b. Hydrology and Hydraulics Evaluation
        1. Summary of Available Information
        2. Characterization of the Flood Hazard
        3. Capacity Exceedance/Criteria and System Performance
    c. Structural Evaluation
        1. Summary of Available Information
        2. Closure Devices
        3. Stability and Strength Requirements
        4. Corrugated Metal Pipe (CMP) Condition Assessment
    d. Geotechnical Evaluation
        1. Summary of Available Information

2. Embankment Erosion Protection
3. Seepage and Piping
4. Embankment and Foundation Stability
5. Settlement
6. Seismic Issues

e. Electrical and Mechanical
f. Interior Drainage
g. Other Analysis/Pertinent Data

X. System Evaluation
a. Emergency Response Plan and Status
b. Probability of Failure and Consequences
c. System Capacity Exceedance Provisions

XI. Residual Risk and Public Safety

XII. Appendices
a. Site Visit Report
b. Applicable Meeting Minutes and Decision Milestones
c. Review Documentation
d. Additional Appendices (as needed)

b. Transmittal Letters. A transmittal letter shall be bound with the NLSER or with data provided for NFIP levee system evaluation purposes. Example transmittal letters are found in Appendix B.

c. Review Requirements. Review requirements for each NLSER shall be determined in accordance with EC 1165-2-209, *Civil Works Review Policy*, 31 January 2010. Typically, an Agency Technical Review (ATR) will be required. All review documentation shall be included in the final NLSER.

d. Requirements for Final Signature. The NLSERs shall be reviewed, concurred with, and signed by the district LSO.

e. Notification. A copy of the final NLSER shall be provided to the non-federal sponsor requesting the NFIP levee system evaluation, the corresponding FEMA regional office, state NFIP coordinator, and the county NFIP coordinator. District offices shall upload final NLSERs to the National Levee Database.

FOR THE COMMANDER:

6 Appendices
(See Table of Contents)

JAMES C. DALTON, P.E.
Chief, Engineering and Construction
Directorate of Civil Works

APPENDIX A

References

This appendix is a list of USACE Engineer documents (Circulars, Regulations, Manuals, and Technical Letters) and other USACE and non-USACE appropriate references. The intent is to provide a comprehensive listing of appropriate guidance referenced in the text. Often the text includes annotations as might be necessary to identify outdated or contradicting components thereof and point to contemporary technical material not yet in USACE Manuals.

A.1. Required Publications

Public Law 106-554
Data Quality Act, USC Section 515 of Consolidated Appropriation Act 2001.

31 USC 1535
Subtitle II, Chapter 15, Subchapter III
Economy Act

33 USC 408
Navigation and Navigable Harbors
Chapter 9, Subchapter I, Section 408
Taking Possession of, Use of, or Injury to Harbor or River Improvements

33 CFR 208.10
Local Flood Protection Works; Maintenance, and Operations of Structures and Facilities.

44 CFR 61.12
Rates Based on a Flood Protection System Involving Federal Funds, 10-1-02 Edition

44 CFR 65.2(b)
Definitions, 10-1-02 Edition

44 CFR 65.10
Mapping of Areas Protected by Levee Systems, 10-1-02 Edition

ERDC TR-07-15
Fitness-for-Purpose Evaluation of Hydraulic Steel Structures

ER 11-2-220
Civil Works Activities, General Investigation

ER 500-1-1
Emergency Employment of Army and Other Resources - Civil Emergency Management Program

ER 1105-2-100
Planning Guidance Notebook

ER 1105-2-101
Risk Analysis for Flood Damage Reduction Studies

ER 1110-2-1150
Engineering and Design for Civil Works Program

ER 1110-2-1806
Earthquake Design and Evaluation of Civil Works Projects

ER 1140-1-211
Support for Others: Reimbursable Work

ER 1165-2-119
Modifications to Completed Projects

EP 500-1-1
Emergency Employment of Army and Other Resources Civil Emergency Management Program – Procedures

EM 1110-1-1904
Settlement Analysis

EM 1110-1-1905
Bearing Capacity of Soils

EM 1110-2-301
Guidelines for Landscape Planting at Floodwalls, Levees & Embankment Dams

EM 1110-2-1100
Coastal Engineering Manual

EM 1110-2-1413
Hydraulic Analysis of Interior Areas

EM 1110-2-1416
River Hydraulics

EM 1110-2-1601
Hydraulic Design of Flood Control Channels, Change 1 ENG 4794-R

EM 1110-2-1612
Ice Engineering

EM 1110-2-1619
Risk-Based Analysis for Flood Damage Reduction Studies

EM 1110-2-1901
Seepage Analysis and Control for Dams (Change 1).
EM 1110-2-1902
Slope Stability

EM 1110-2-1908
Instrumentation of Embankment Dams and Levees

EM 1110-2-1913
Design and Construction of Levees

EM 1110-2-1914
Design, Construction and Maintenance of Relief Wells

EM 1110-2-2100
Stability Analysis of Concrete Structures

EM 1110-2-2104
Strength Design for Reinforced Concrete Hydraulic Structures

EM 1110-2-2105
Design of Hydraulic Steel Structures Change 1

EM 1110-2-2502
Retaining Walls and Floodwalls

EM 1110-2-2504
Sheet Pile Walls

EM 1110-2-2705
Structural Design of Closure Structures for Flood Control Projects

EM 1110-2-2906
Design of Pile Foundations

EM 1110-2-3104
Structural and Architectural Design of Pumping Stations

EM 1110-2-3105
Mechanical and Electrical Design of Pumping Stations

EM 1110-2-4000
Sedimentation Investigations of Rivers and Reservoirs, ENG 1787

EM 1110-2-6054
Inspection, Evaluation and Repair of Hydraulic Steel Structures

EC 1110-2-6001 (Draft)
Seismic Analysis of Dams and Levees (targeted for publication in 2008)

EC 1165-2-209
Civil Works Review Policy

ETL 110-2-569
Design Guidance on Levee Underseepage

ETL 1110-2-299
Overtopping of Flood Control Levees and Floodwalls

ETL 1110-2-563
Engineering and Design: Design Guidance for Levee Underseepage

ETL 1110-2-571
Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, and Appurtenant Structures

ACI 318
Building Code Requirements for Structural Concrete

ACI 350
Code Requirements for Environmental Concrete Structures

ACI 437
Strength Evaluation of Existing Concrete Buildings

Bulletin 17B 1982
Guidelines for Determining Flood Flow Frequency. Bulletin 17B of the Hydrology Subcommittee of the Advisory Committee on Water Data.

Federal Emergency Management Agency (FEMA) 2003
Guidelines and Specifications for Mapping Partners, Federal Emergency Management Agency, National Flood Insurance Program, April 2003.

Federal Emergency Management Agency (FEMA) 2005
FEMA Memorandum No. 34, Interim Guidance for Studies Including Levees. Federal Emergency Management Agency, National Flood Insurance Program, 22 August 2005

Federal Energy Regulatory Commission (FERC) 2005
Dam Safety Performance Monitoring Program, Chapter 14, Potential Failure Modes Analysis

Hanson and Cook 2004
Hanson, G. J., and Cook, K R. 2004. Apparatus, Test Procedures, and Analytical Methods to Measure Soil Erodability in Situ. *Applied Engineering in Agriculture*, ASAE, Vol. 20, No 4, pp. 455-462.

Hanson and Temple 2001
Hanson, G. H., and Temple, D. M. 2001. Performance of Bare-Earth and Vegetated Steep Channels Under Long-Duration Flows. Presented at the 2001 American Society of Agricultural Engineers (ASAE) Annual Meeting, ASAE Paper No. 012157.

Hughes et al. 2008
Hughes, S. A., Ward, D. L., Wibowo, J. L., and Lee, L. T. (Draft 2008). Protection Alternatives for Levees and Floodwalls in Southeast Louisiana: Phase 1 Evaluation. U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Hydrologic Engineering Center – Flood Damage Reduction Analysis (HEC-FDA). 1998
Flood Damage Reduction Analysis, User's Manual. CPD-72. USACE Hydrologic Engineering Center, Davis, CA, March 1998.

Idriss, I. M. and Boulanger R. W. 2007
Residual Shear Strength of Liquefied Soils. *Proc. 27th Annual USSD Conference. Modernization and Optimization of Existing Dams and Reservoirs, Philadelphia, Pennsylvania.*

Papoulis 1984
Papoulis, A. 1984. *Probability, Random Variables, and Stochastic Processes*, 2nd ed. New York: McGraw-Hill.

Seed, R. B. et al. 2003
Recent advances in soil liquefaction engineering: A unified and consistent framework." *Keynote Presentation, 26th Annual ASCE Los Angeles Geotechnical Spring Seminar*, Long Beach, CA.

TAW 1989
Guidelines for Design of River Dikes, Part 2 – Lower River Area, Technical Advisory Committee on Flood Defence (Technische Adviescommissie voor de Waterkeringen), The Netherlands, September 1989.

TAW 2002
Wave Run-up and Wave Overtopping at Dikes, Technical Advisory Committee on Flood Defence (Technische Adviescommissie voor de Waterkeringen), Den Haag, The Netherlands, May 2002.

UFC 3-310-04
Seismic Design for Buildings

U.S. Army Corps of Engineers 1997
Guidance on Levee Certification for the National Flood Insurance Program, CE CECW-EG Memorandum, USACE, 10 April 1997.

U.S. Army Corps of Engineers 1997
Geotechnical Activities in Support of Levee Certification for Federal Emergency Management Agency (FEMA) Flood Insurance Purposes, CECW-EG Memorandum, USACE, 30 June 1997.

U.S. Army Corps of Engineers 2006
Levee Owner's Manual for Non-Federal Flood Control Works, U.S. Army Corps of Engineers, March 2006.

U.S. Army Corps of Engineers 2006
Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System; Volume IV – "The Storm**,"** Interagency Performance Evaluation Task Force, Washington, DC.

U.S. Army Corps of Engineers 2006
Guidance on Levee Certification for the National Flood Insurance Program – FEMA Map Modernization Program Issues, CECW-P/CECW-E Memorandum, USACE, 23 June 2006.

U.S. Army Corps of Engineers 2006
Authority and Funding Guidance for USACE Levee Certification Activities, CECW-P/CECW-E Memorandum, USACE, 15 August, 2006.

U.S. Army Corps of Engineers 2006
Actions for Change - 12 Actions for Change, a set of actions that the Corps will focus on to transform its priorities, processes and planning.

U.S. Army Corps of Engineers 2007
Levee Safety Program Implementation, CECW-HS Memorandum, USACE, 16 November 2007

U.S. Army Corps of Engineers 2008
Policy Guidance Letter – Periodic Inspection Procedures for the Levee Safety Program.

U.S. Department of Agriculture 2006
Bank Stability and Toe Erosion Model. U.S. Department of Agriculture, 18 April 2006.

U.S. Department of the Interior 2003
Dam Safety Risk Analysis Methodology, U.S. Department of the Interior, Bureau of Reclamation, May 2003.

APPENDIX B

Templates for NFIP Levee System Evaluation Report Transmittal Letters

The following are examples of transmittal letters to be bound in NLSERs or data reports documenting a NFIP levee system evaluation effort:

1) Letter for a positive finding for NFIP levee system evaluation.
2) Letter for a negative finding for NFIP levee system evaluation.
3) Letter for providing technical information in support of NFIP levee system evaluation effort performed by others.

Positive NFIP Levee System Evaluation Report Transmittal Letter

District Letterhead

(*Date here*)

(*Name and address of requester of determination here*)
[Mr./Ms.] (Full Name of Requester)
(Title of Requester**)**
(Requester Address**)**
(City, State Abbreviation**, and Zip Code)**

Dear [Mr./Ms.] (Last Name of Requester),

The (*district name here*) District of the U.S. Army Corps of Engineers (USACE) submits this final NFIP Levee System Evaluation Report (NLSER) for (*levee identification and location here*). This report concludes that (*levee identification here*) has met all of the requirements established by USACE for determining that the levee system can be reasonably expected to exclude the 1% annual chance exceedance flood, also referred to as the base flood, from the leveed area. This NLSER documents the NFIP levee system evaluation requirements, assumptions made, and analyses conducted to make this levee NFIP levee system evaluation finding and that they are consistent with requirements outlined in Title 44 of the Code of Federal Regulations, Section 65.10 (44 CFR 65.10), *Mapping Areas Protected by Levee Systems*.

Under the National Flood Insurance Program, a NFIP levee system evaluation is a prerequisite for receiving levee accreditation from the Department of Homeland Security, Federal Emergency Management Agency (FEMA). If the levee is in accordance with NFIP levee system evaluation requirements and thus accredited, FEMA will not show the area located behind the levee as a Special Flood Hazard Area, an area that would subject to flooding by the base flood. The area instead will be designated as a shaded Zone X or moderate risk zone. The purchase of flood insurance and elevation of structures is not federally mandated in a moderate risk zone; however, it is encouraged.

This NFIP levee system evaluation expires on (*date equal to 10 years from the date of letter*). After this time, USACE will no longer consider the (*levee identification here*) to be in accordance with NFIP levee system evaluation requirements and you and FEMA will be notified. At any time prior to this date, it is at the (*district name here*) District's discretion to revoke the positive finding for NFIP levee system evaluation should the District decide that (*levee identification here*) is no longer in accordance with NFIP levee system evaluation requirements, which may include reasons such as inadequate operation and maintenance, excessive settlement/subsidence, or change in hydraulic conditions. USACE will notify you and FEMA

Region (*number of appropriate FEMA region here*) should this situation occur. It will be the responsibility of the local community or other entity that desires to retain accreditation of this levee system to pursue a reevaluation. At that time, we recommend that we are contacted to discuss potential next steps.

This NFIP levee system evaluation does not assure that (*levee identification here*) will exclude flood water from all future flood events. Even with a levee in place that meets NFIP levee system evaluation requirements, a possibility of flooding that overtops or otherwise fails the levee exists. Flood risk management measures to reduce the consequences of this possibility are strongly advised, such as elevating structures, maintaining a current flood warning system and evacuation plan, and wisely managing floodplain development.

A copy of this NLSER has been transmitted to FEMA Region (*number of appropriate FEMA region here*). They have also been sent to the National Flood Insurance Program (NFIP) Coordinator for the County of (*appropriate county here*), and State of (*appropriate state here*).

For any questions regarding this report, please contact (*name and title of district point of contact here*) at (*contact information here*). For questions about accreditation or the NFIP, please contact (*name and title of FEMA region contact*) at (*contact information here*).

Sincerely,
(Name of LSO here)

(*district name here*)
U.S. Army Corps of Engineers

Enclosure

Copies furnished:
Point of contact for FEMA Region
County NFIP Coordinator
State NFIP Coordinator

Negative NFIP Levee System Evaluation Report Transmittal Letter

District Letterhead

(*Date here*)

(*Name and address of requester of determination here*)
[Mr./Ms.] (Full Name of Requester)
(Title of Requester)
(Requester Address)
(City, State Abbreviation, and Zip Code)

Dear [Mr./Ms.] (Last Name of Requester),

The __(*district name here*)__ District of the U.S. Army Corps of Engineers (USACE) submits this final NFIP Levee System Evaluation Report (NLSER) for (*levee identification and location here*). This report concludes that USACE finds __(*levee identification and location here*)__ is not in accordance with all of the NFIP levee system evaluation requirements established by USACE for determining that the levee system can be reasonably expected to exclude the 1% annual chance exceedance flood from the leveed area. Specifically, the levee system failed to meet the following necessary requirements for NFIP levee system evaluation:

> __(*provide a brief description of the specific NFIP levee system evaluation requirements that were not met and levee conditions that resulted in a negative finding for NFIP levee system evaluation*)__.

This NLSER documents the NFIP levee system evaluation requirements used, assumptions made, and analyses conducted to make this finding.

Under the National Flood Insurance Program (NFIP), a NFIP levee system evaluation is a prerequisite for receiving levee accreditation from the Department of Homeland Security, Federal Emergency Management Agency (FEMA). If levee accreditation is denied by FEMA, the purchase of flood insurance and the elevation of structures may be required. Flood risk management measures to minimize the consequences associated with the possibility of the levee overtopping or failing, such as elevating structures, maintaining a current flood warning system and evacuation plan, and wisely managing floodplain development are strongly advised.

We understand that you may be interested in discussing your options for improving the levee system or implementing other flood risk reduction measures and we will work collaboratively with you and other stakeholders to determine the next steps.

A copy of this NLSER has been transmitted to FEMA Region (*number of appropriate FEMA region here*). It has also been sent to the National Flood Insurance Program Coordinator for the County of (*appropriate county here*), and State of (*appropriate state here*). For any questions regarding this report, please contact (*name and title of district point of contact here*) at (*contact information here*). For questions about accreditation or the NFIP, please contact (*name and title of FEMA region contact*) at (*contact information here*).

Sincerely,
(Name of LSO here)

(*district name here*)
U.S. Army Corps of Engineers

Enclosure

Copies furnished:
Point of contact for FEMA Region
County NFIP Coordinator
State NFIP Coordinator

Transmittal Letter for Technical Support

District Letterhead

(*Date here*)

(*Name and address of requester of determination here*)
[Mr./Ms.] (Full Name of Requester)
(Title of Requester)
(Requester Address)
(City, State Abbreviation, and Zip Code)

Dear [Mr./Ms.] (Last Name of Requester),

The __(*district name here*)__ District of the U.S. Army Corps of Engineers (USACE) submits the following (*information, technical data, or analysis result described here*) for (*levee identification and location here*). This submittal is information to be used in support of a NFIP levee system evaluation to be performed by another entity and does not constitute a positive NFIP levee system evaluation or partial NFIP levee system evaluation finding by USACE.

Under the National Flood Insurance Program, a NFIP levee system evaluation is a prerequisite for receiving levee accreditation from the Department of Homeland Security, Federal Emergency Management Agency (FEMA). If the levee is found to be in accordance with NFIP levee system evaluation requirements and thus accredited, FEMA will not show the area located behind the levee as a Special Flood Hazard Area, an area that would subject to flooding by the base flood. The area instead will be designated as a shaded Zone X or moderate risk zone. The purchase of flood insurance and elevation of structures is not federally mandated in a moderate risk zone; however, it is encouraged.

For any questions regarding this letter, please contact (*name and title of district point of contact here*) at (*contact information here*). For questions about accreditation or the NFIP, please contact (*name and title of FEMA region contact*) at (*contact information here*).

Sincerely,
(Name of LSO here)

____(*district name here*)____
U.S. Army Corps of Engineers

Enclosure

APPENDIX C

Example 1

Example Procedures for Systems that Clearly Meet NFIP Levee system Evaluation Requirements

Example of a Generalized Hydrology and Hydraulics Component

Note: This is an example of one method for determining if an existing riverine levee system meets the H&H criteria for a system that clearly meets NFIP levee system evaluation requirements. This method assumes the design analysis was based on discharge-probability and stage-discharge relationships, that there is a current USGS gage at the levee system site, and that the HEC-FDA program is used to determine the overtopping assurance. There are other methods that can be used.

<u>DATA NEEDED</u>

1. Latest flow frequency curve and statistics for the gage
2. USGS discharge measurements for the gage
3. Levee elevation at the gage
4. Design Memorandum (DM) or similar for the levee system.
5. Peak flows since the frequency curve was done
6. Official FIS 100-yr flow at the gage
7. Official FIS 100-yr stage at the gage

<u>DISCHARGE-PROBABILITY RELATIONSHIP</u>

Determine if the most current relationship is still adequate. The usual assumption is the relationship is adequate for the clearly meets NFIP levee system evaluation requirements analysis unless there are clear reasons it is not correct. Synthetic statistics for the discharge-frequency results can be computed from Appendix 5 of Bulletin 17B if not available from the original analysis. Possible reasons for not using the current relationship:

1. Have there been large floods approaching the flood of record have there been since the relationship was developed?
2. Are there significant changes to the basin or river system, such as major land use changes, new reservoirs or diversions, etc.?

<u>STAGE-DISCHARGE RELATIONSHIP</u>

Determine if the most current stage-discharge relationship is still adequate. The usual assumption is the relationship is adequate for the clearly meets NFIP levee system evaluation requirements analysis unless there are clear reasons it is not correct. It will usually be necessary to extend the relationship and to estimate the uncertainty:

1. To determine if the existing relationship is adequate read the hydraulics portion of the DM to see if the river at the levee system was stable prior to construction. Check river stability since the levee system was completed. Talk to inspectors, sponsors/operators, check aerial photos, do field inspection, etc. For a clearly certifiable analysis a field trip required. Check the USGS discharge measurements over time for changes in stage for a given flow.
2. The USGS discharge measurements can be used to modify the discharge-elevation curve and to determine the uncertainty in the curve. One method is to plot the measured flows vs. stages in Excel. (The USGS on-line text data can be input directly into Excel. Copy the "Surface Water Field Measurements" tab-separated data then save as a *.txt file, and open with Excel.) If the DM and the measurements show that the rating at the gage has been changing, use only the recent measurements. Usually the non-flood measurements do not have to be included.
3. Extend the discharge-elevation curve. The curve should extend higher than the top of the levee. Provided below are two possible ways to extend the curve.
    a. Method 1 - Plot the log of the measured flows vs. the log of the stages and fit a curve through them. This could be a straight line or a polynomial. You need to try different types until you have a good eye-ball and R squared fit. Make sure you extend the fit curve to above the top of the levee. Sometimes a good fit to the measured data looks terrible when extended – it could bend over and go down. Check the curve extension against the physical characteristics of the floodplain. A wide, flat floodplain will have a much flatter curve extension than a restricted floodplain with multiple bridges with high embankments.
    b. Method 2 - Use computed design conditions flood profiles from the DM to extend the curve. This method is especially good if the extension in step 3.a. looks bad and if the design profiles were computed with a backwater model. You need to check to see if these points fit the rating curve based on measurements.
4. Find uncertainty in the discharge-elevation relationship. EM 1110-2-1619 gives methods for determining the uncertainty. One method is to use the equation for the fit curve to find the computed stage for every measurement used in the analysis. Then find the differences between the actual and rating curve predicted stages. The standard deviation of these differences is the standard deviation of the rating curve for the portion where there are measurements. Generally you would expect the uncertainty in the discharge-elevation relationship to be greater for stages higher than the largest discharge measurement. You could assume the standard deviation for the upper unmeasured part of the curve to be twice that of the measured portion.

<u>RUN HEC-FDA</u>

1. Hydrology. For LP III curves: mean, standard deviation, skew and record length are used. For graphical curves there is a procedure to use in FDA that includes the record length.
2. Hydraulics. Input the extended rating curve points and uncertainty.
3. Enter the top of levee at the gage.
4. Economics. For the current version of FDA you will need to enter fictitious economic values to get it to run.

OTHER CONSIDERATIONS

Superiority. Newer levee systems include superiority (extra levee height to ensure overtopping at a predefined location) with varying freeboard (old design) or assurance (new design with risk and uncertainty). For levee systems with uneven freeboard or overtopping assurance the gage may not be the critical location for assessing performance. In these cases you should also do an analysis at the location with minimum freeboard or overtopping assurance – usually the downstream end of the levee system. Without gage data at the critical point it is more complicated to determine the discharge-elevation relationship and its uncertainty. The data from the gage can be used to help guide the analysis.

Changes in levee profile. Known locations with levee settlement, modifications made to the levee, etc. Generally the Corps levee database should have current top of levee profiles but in some cases there may be too much uncertainty in the levee elevations to do a clearly certifiable analysis.

Major basin, floodplain, or channel changes. Changes that would obviously impact the previous hydrology or hydraulics and that could not be quickly analyzed will rule out the possibility of a quick likely meets requirements analysis. These could be changes in regulation plans for upstream reservoirs, major land-use changes, flow diversions, new bridges and other development in the floodplain, etc.

Example 2

Example of a Hydrology and Hydraulics Analysis
To
Check for a System that Clearly Meets NFIP Levee System Evaluation Requirements.

MINNEOTA NFIP Levee System Evaluation

The Minneota, MN, levee is on the South Branch of the Yellow Medicine River. The levee system design is shown in the February 1961 Detailed Project Report (DPR). The levee system was designed for the then estimated 110-yr flood, 6900 cfs, with 3 feet of freeboard. USACE found that NFIP levee system evaluation requirements were met for the levee system in 1994 using freeboard criteria. A new positive finding for NFIP levee system evaluation requires the use of uncertainty and assurance of overtopping for the hydrologic and hydraulic analysis.

The NFIP levee system evaluation procedures proposed in the previous part of this appendix could not be applied directly. The USGS gage at the site was discontinued in 1958 so it could not be used to for the elevation-discharge analysis.

<u>Hydraulics:</u>

Aerial photos and discussion with the levee system inspector indicates the channel and floodplain are very stable at this location. HEC-2 hydraulic models were used in 1994 for NFIP levee system evaluation. One model is for downstream of the city and the other goes through the city and upstream. With the stable geomorphology the geometry in the existing models are considered representative of today's conditions. The comment cards in the HEC=2 models include good documentation. Converting the models to Hydrologic Engineering Center's River Analysis System (HEC-RAS) program was attempted but the special bridge routine for the bridge at the d/s end of the levee system, State Highway No. 3, did not transfer well. Therefore, the HEC-2 models were used for the analysis.

From Table C-1, the upstream end of the upstream levee has the minimum freeboard. This is the critical location for overtopping since the uncertainty in the hydraulics is assumed to be relatively constant based on the characteristics of the system. To estimate the uncertainty in the rating curve, the n values were increased 10 and 20%. The results are shown in Table C-2. In the levee system reach, the original channel and overbank n values were 0.045 and 0.120. The 20% increased n values for the channel and overbank were 0.054 and 0.144. The n values resulting from the 20% increase are quite high for this type of stream and floodplain. The probability of n values this high is probably less than 16% (one standard deviation), but to be conservative it is assumed the increased stages from an n value increase of 20% represent one standard deviation in the stage uncertainty. As seen below there was about a 0.6 foot increase in stage for the 20% increase in n value, and 0.6 foot was set as the standard deviation for the discharge-elevation rating curve in the Monte Carlo analysis for the levee assurance analysis.

Table C-1. HEC-RAS Results				
Location	Top of Levee Elevation, ft	HEC-2 Cross Section	1% Stage (Q = 5870 cfs)	DIFF - Top of Levee – 1% Stage
D/S end of D/S levee	1166.1	32.3	1160.75	5.35 ft
U/S end of D/S levee	1167.0	34.2	1162.93	4.07 ft
D/S end of U/S levee	1168.8	34.4	1163.53	5.27 ft
U/S end of U/S levee	1169.4	1.4	1167.57	1.83 ft
Note: All elevations 1929 datum. Levee elevations from the 1961 DPR.				

Table C-2. HEC-RAS Results at Section 14						
Cross Section	Discharge	Original n Stage	n plus 10% Stage	n plus 20% Stage	DIFF 10% Stage	DIFF 20% Stage
1.4	0	1155.00	1,155.00	1,155.00	0.00	0.00
	2240	1164.90	1,165.18	1,165.43	0.28	0.53
	4600	1166.89	1,167.25	1,167.54	0.36	0.65
	5550	1167.57	1,167.91	1,168.20	0.34	0.63
	5870	1167.78	1,168.12	1,168.39	0.34	0.61
	6900	1168.41	1,168.71	1,168.96	0.30	0.55
	7180	1168.56	1,168.85	1,169.11	0.29	0.55
	9470	1169.90	1,169.91	1,170.20	0.01	0.30

<u>Hydrology:</u>

The hydrology for Minneota from the 1994 NFIP levee system evaluation was used. The details of the analysis were not found. The only results available were the 10-, 50-, 100-, and 500-yr flood discharges. The discharge-probability curve in the 1961 DPR was developed by a transfer from the gage on the Yellow Medicine River near Granite Falls, MN. In the 1961 analysis, the period of record for the gage was 1931-1958. The Granite Falls discharges were reduced by the ratio of drainage areas to the 0.6 power, resulting in a factor of 0.353. The 1961 analysis was graphical but using the LP III statistics for the gage record and a weighted skew of -0.37 gave a good match to the 1961 Granite Falls curve. The Granite Falls station skew for that period was -0.62 and the regional skew is -0.20, thus a weighted skew of -0.37 is reasonable. To determine the statistics for the 1994 curve it was assumed that the 1994 curve was also developed from the Granite Falls gage data. Using the Granite Falls 1931-1993 records and a weighted skew of -0.32 gave a very good match to the 1994 Minneota curve (using the same 0.353 reduction factor). The results are shown in Table C-3.

Table C-3. Flow Frequency Data					
Flood	1961 DPR	LP III curve developed to match 1961	1994 NFIP levee system evaluation	LP III curve developed to match 1994	LP III curve for 1931-2005
10-yr	1900	2082	2240	2051	1973
50-yr	4700	5177	4600	4510	4176
100-yr	6600	7008	5870	5880	5375
200-yr	9200	9126	N/A	7418	6706
500-yr	N/A	12393	9470	9735	8686
LP III = Log Pearson Type III					

The last column in the above table uses the Granite Falls period of record up to 2005 and the skew of -0.32 used to match the 1994 analysis. It shows that the frequency curve would probably be a little lower if updated. Thus the 1994 curve is conservative.

Appendix 5 of Bulletin 17B gives a way to determine the LP III statistics if you know the various frequency flood discharges. This method was not used since it would not allow a comparison of different periods of record and a check of the impact of using the period through 2005.

Table C-4 shows the hydrologic parameters from the 1994 analysis used in the Monte Carlo analysis for the levee assurance analysis.

Table C-4. LP III Statistics	
MATCH 1994 ANALYSIS	
AVE =	2.64150
STD DEV =	0.53992
SKEW =	-0.32
YEARS =	62

Monte Carlo Results:

The Monte Carlo analysis in the FDA program was used to determine levee overtopping assurance. The Monte Carlo results of this approximate analysis showed that the levee at the upstream end has a 92.15% probability of not being overtopped by a 100-yr flood. The minimum assurance required by the EC for a positive finding for NFIP levee system evaluation is 90% when a detailed analysis is performed. When performing an approximate analysis like this one, the EC recommends requiring an assurance of at least 95% to determine clearly certifiable systems. In this case the hydrology is conservative since an updated frequency curve would likely be lower, and assuming a 20% increase in n value only represents one standard deviation is also conservative. Therefore, it could be argued that the 92% assurance of the levee should meet the required risk and uncertainty assurance. However, the EC also requires that if the assurance is less than 95%, the levee has to have at least 3 feet of freeboard and also requires at least 2.0 feet of freeboard even if the assurance is more than 95%. As shown in Table C-1 there is only 1.83 feet of freeboard at the upstream end of the levee system. Therefore, the requirement for 3 feet of freeboard with an assurance less than 95% is not met and the minimum of 2 feet of freeboard requirement is also not met, therefore **the levee cannot be found to continue to meet NFIP levee system evaluation requirements based on this approximate analysis. To find the NFIP levee system evaluation requirements continue to be met**, a detailed analysis would either have to result in at least 3 feet of freeboard with 90% assurance of non-overtopping or give more than a 95% assurance for the 100-yr flood, and could not have less than 2 feet of freeboard no matter what the overtopping assurance.

In a detailed analysis the upper end of the hydraulic model should be checked. The new model gives stages much higher than the original DPR at the upstream end of the levee. This could be the reason for the reduced freeboard at this location. The levee system area has fairly recent topography and the model should be compared to the topography to ensure it correctly reflects the prototype geometry. Since the hydrology is fairly old, 1994, and back up analysis is missing, it should be updated. Updating the hydrologic model could decrease the probability of overtopping. The bridge modeling also needs to be looked at closely since there have been bridge replacements since the original modeling and the bridge routines had problems when converted to HEC-RAS.

Since the levee was not shown by the hydrology and hydraulic analysis to be clearly certifiable the field inspection required by the EC was not performed. The inspection and analysis by other disciplines would be required before a final determination that the levee system clearly meets NFIP levee system evaluation requirements.

Example 3 – Closure Analysis

Montevideo, MN, Section 205 Flood Control Project, Closure Analysis

Closures

1. There are five closures in the levee system as shown on Plate 1, four earthen road closures and one stoplog railroad closure. The information in paragraphs 2 through 14 was evaluated before it was decided that the closures were acceptable.

2. The base of the Canton Avenue closure is at the 1-percent water surface elevation while the other three earthen closures are above the 1-percent water surface elevations; therefore, the closures are acceptable from this perspective.

3. Paragraphs 4 through 8 contain a short description of each closure. The closure lengths cited in these paragraphs are based on the required top-of-levee elevations and do not include overbuild required for settlement in some locations. Fill amounts computed for the earthen closures include an additional 30-percent for compaction shrinkage. Compaction requirements in the operation and maintenance manual will be three (3) complete passes with a bulldozer in lifts of no more than 9 inches. Typical production rates for emergency levee construction during flood events were considered in the estimated time to install the earthen closures.

4. The low point of the Highway 212 earthen closure is about 3.3 feet above the 1-percent water surface profile and is about 109 feet long with a maximum height of 2 feet. The closure requires about 101 cubic yards (CY) of fill and the estimated time to install the closure is 3 hours.

5. The low point of the Highway 59/7 earthen closure is about 1.6 feet above the 1-percent water surface profile and is about 63 feet long with a maximum height of 3 feet. The closure requires about 104 CY of fill and the estimated time to install the closure is 3 hours.

6. The rail recesses for the stoplog railroad closure are 3.1 feet below the 1-percent water surface profile and the closure is 20 feet long. The maximum height of the closure is about 7.7 feet including the rail recesses and about 6.9 feet excluding the rail recesses. The closure requires about 10 stoplogs and the estimated time to install the closure is 6 hours.

7. The low point of the Canton Avenue earthen closure is at the 1-percent water surface profile and is about 85 feet long with a maximum height of 4.6 feet. A small road raise was required so the base of the closure would be would be at the 1-percent water surface profile. The closure requires about 342 cubic yards (CY) of fill and the estimated time to install the closure is 6 hours. Borrow material for this closure is stockpiled adjacent to the closure.

8. The low point of the Parkway Drive earthen closure is 0.5 feet above the 1-percent water surface profile and is about 58 feet long with a maximum height of 4.0 feet. The closure requires about 166 cubic yards (CY) of fill and the estimated time to install the closure is 4 hours. A road

raise was required so the base of the closure would be would be above the 1-percent water surface profile.

Closure Operating Plans

9. Numerous factors were considered in developing operating plans for the closures. These factors included the amount of fill or stoplogs required for the closures, the time required to install the closures, the time available to mobilize and install the closures, the spring and summer rates of rise for the Minnesota River, geotechnical sliding stability considerations and the City's capability to install the closures in the available time.

Rate of Rise for Minnesota River

10. The rate of rise for the Minnesota River was used to assess the time available to install the closures. To develop the rate of rise information, discharge hydrographs for the Minnesota River at Montevideo were downloaded from the USGS website and converted to elevation based on the HEC-RAS model rating curve at the USGS gage. Spring flood events that were considered include 1951, 1952, 1965, 1969, 1979, 1985, 1986, 1994, 1997, and 2001. Summer flood events that were considered include 1919, 1953 and 1993. The 1919 summer flood event hydrograph was adjusted because it occurred before the Marsh Lake and Lac Qui Parle reservoirs were constructed. The maximum spring rate of rise is 3 feet per day and the maximum summer rate of rise is 5 feet per day. However, the maximum elevation for the historical summer events would not require that the closures be installed or even that the city mobilizes to install the closures. Therefore, the maximum spring rate of rise of 3 feet per day was used to assess the time available to install the closures. The mobilize to install the closure trigger elevations were based on when the river is six (6) feet below the base of the closure which would provide 48 hours before the closures may be required. The install closure trigger elevations were based on when the river is three (3) feet below the base of the closure which would provide 24 hours before the closures may be required.

11. There are other factors that will provide the City with additional information and warning time regarding the closures. The National Weather Service starts numerical 5-day forecasts at a stage of 12 feet which is an elevation of 921.1. This is about five (5) feet below the lowest mobilize to install closure trigger elevation which is for the stoplog railroad closure. In addition, the City will know weeks in advance that the closures may be required based on snowfall amounts and snow water equivalents since major floods on the Minnesota River are spring events.

12. A table was developed for each closure based on the trigger elevation requirements in the paragraph 10. If necessary, the elevations at each closure were translated to the USGS gage using the HEC-RAS water surface profiles and rating curves. Then the regulated discharge frequency curve was used to determine how frequently mobilize-to-install and install-closure trigger elevations would be reached. Finally, the annual peaks for the period of record from 1910 to 2007 were reviewed to determine whether the frequency of the closure actions based on the annual peaks were similar to those determined based on the regulated discharge frequency curve.

The operating plans and closure actions for the period of record are summarized in Tables 1 through 5. Based on this information, there is adequate warning time to install the closures.

Geotechnical Requirements

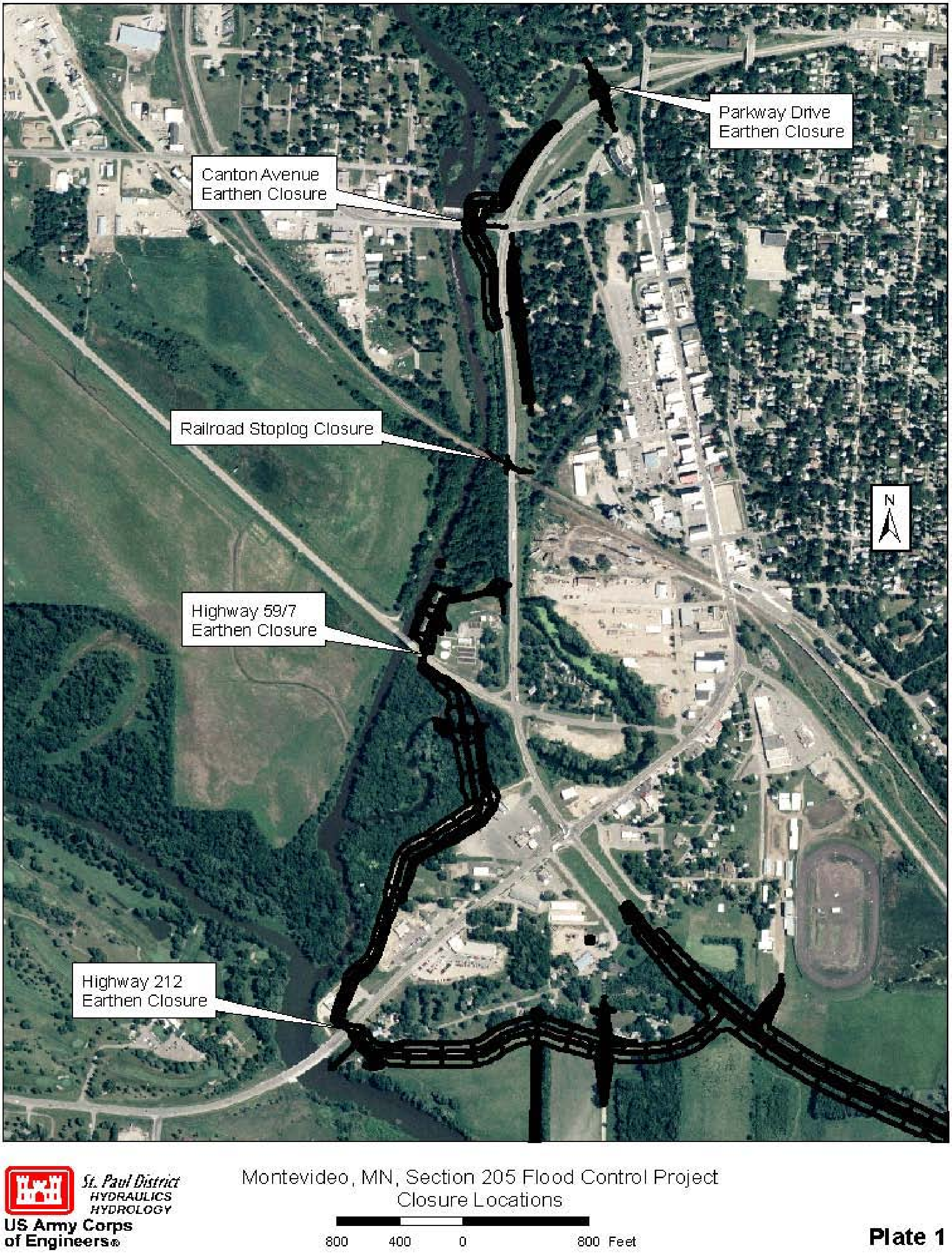
13. A geotechnical (Fast Lagrangian Analysis of Continua (FLAC)) analysis was performed to determine the sliding factor of safety for the earthen closures. The analysis was based on the closures being constructed on a paved surface with a 10 foot top width and 1V on 3H side slopes which is the same as the levee section. Closure heights of 4.65, 3 and 2 feet were analyzed and the resulting sliding factors of safety were 1.49, 1.68 and 1.89, respectively. The lowest computed factor of safety of 1.49 is for the Canton Avenue earthen closure which is 4.65 feet high. The minimum acceptable factor of safety is 1.4 based on Table 6-1b in EM 1110-2-1913. Since the factor of safety for all of the earthen closures exceeds 1.4, the closures are acceptable from a geotechnical perspective.

City's Capability to Install Closures

14. The final factor considered regarding the closures was whether the City of Montevideo had the capability to install the closures during a flood event. Table 6 is a summary of the closure and the closure operation data that was supplied to the City. The City was asked to evaluate this information and provide a response whether they had the ability to install the five closures according to the operation plan. The City provided a detailed response stating they have the ability to install the closures. The response listed personnel, equipment and contracting capabilities. Therefore, the closures are acceptable from this perspective.

Closure Summary

15. The proposed closures at Montevideo are acceptable based on all the criteria that were evaluated and based on the City's ability to install the closures. The summary information in Table 6 is based on the most recent plans and will be incorporated into the Operation and Maintenance manual for the levee system.



Montevideo, MN, Section 205 Flood Control Project
Closure Locations

Plate 1

Table 1
Minnesota River at Montevideo, MN
Highway 212 Closure - Operating Plan
All Elevations in NGVD 1929
Closure Low Point - Elevation 937.2

Action	USGS Gage (XS 111.0) WSEL	EGL
Mobilize to Install Closure - 6' below Low Point		
* - Mn River Q = 31,000 cfs, 2.8%, Exceedance Frequency, 36-Year	930.8	931.2
* - 24 hours prior to Installation trigger based on 3 feet/day rate of rise		
Install Closure - 3' below Low Point		
* - Mn River Q = 46,500 cfs, 1.2% Exceedance Frequency, 83-Year	933.4	934.2
* - 24 hours prior to WSEL being at base of closure based on 3 feet/day rate of rise		
Base of Closure		
* - Mn River Q = 66,000 cfs, 0.55 % Exceedance Frequency, 180-Year	936.0	937.2
Top-of-Levee Elevation - based on 100-Year EGL = 4.5'	**939.2**	
Maximum Closure Height	**2.0**	
100-Year Event, Q = 50,000 cfs	933.88	934.70
Top-of-Levee Event, Q = 81,000 cfs	938.04	939.22

Closure Actions Required based on Period of Record

Period of Record for Annual Peaks from 1910 through 2006
(Period from 1910 through 1937 is before construction of Lac Qui Parle Dam)

Action	No of times / Probability	Date	Annual Peak Elevation
Mobilize to Install Closure	~3 times in 97 years	4/6/1997	933.02
	3.1%	4/14/2001	931.27
		4/12/1969	930.80
Install Closure	~1 time in 97 years 1.0%	4/6/1997	933.02

Table 2
Minnesota River at Montevideo, MN
Highway 59/7 Closure - Operating Plan
All Elevations in NGVD 1929
Closure Low Point - Elevation 936.8

Action	Highway 59/7 Closure (XS 32.0)	USGS Gage (XS 111.0)	
	WSEL	WSEL	EGL
Mobilize to Install Closure			
* - Mn River Q = 27,000 cfs, 3.4% Exceedance Frequency, 29-Year	930.8	929.9	930.3
* - 24 hours prior to Installation trigger based on 3 feet/day rate of rise			
Install Closure			
* - Mn River Q = 41,000 cfs, 1.6% Exceedance Frequency, 63-Year	933.8	932.6	933.3
* - 24 hours prior to WSEL being at base of closure based on 3 feet/day rate of rise			
Base of Closure			
* - Mn River Q = 60,000 cfs, 0.68% Exceedance Frequency, 150-Year	936.8	935.3	936.4
Top-of-Levee Elevation - based on 100-Year EGL = 4.5'	**939.75**	**939.2**	
Maximum Closure Height	**3.0**		
100-Year Event, Q = 50,000 cfs	WSEL - 935.22 EGL - 935.25	933.88	934.70
Top-of-Levee Event, Q = 81,000 cfs	WSEL - 939.63 EGL - 939.64	938.04	939.22

Closure Actions Required based on Period of Record
Period of Record for Annual Peaks from 1910 through 2006
(Period from 1910 through 1937 is before construction of Lac Qui Parle Dam)

Action	No of times / Probability	Date	Annual Peak Elevation
Mobilize to Install Closure - 930.8		4/6/1997	933.02
	~3 times in 97 years	4/14/2001	931.27
	3.1%	4/12/1969	930.80
Install Closure - 933.8	~1 time in 97 years	4/6/1997	933.02
	1.0%		

Table 3
Minnesota River at Montevideo, MN
Railroad Closure with Rail Recesses - Operating Plan
All Elevations in NGVD 1929
Closure Low Point - Elevation 933

Action	Railroad Closure (XS 35)	USGS Gage (XS 111.0)	
	WSEL	WSEL	EGL
Mobilize to Install Closure			
* - Mn River Q = 12,500 cfs, 8% Exceedance Frequency, 13-Year	927.0	926.5	926.6
* - 24 hours prior to Installation trigger based on 3 feet/day rate of rise			
Install Closure			
* - Mn River Q = 22,000 cfs, 4.2% Exceedance Frequency, 24-Year * - 24 hours prior to WSEL being at base of closure based on 3 feet/day rate of rise	930.0	928.9	929.2
Base of Closure			
* - Mn River Q = 35,000 cfs, 2.2% Exceedance Frequency, 45-Year	933.0	931.6	932.1
Top-of-Levee Elevation - based on 100-Year EGL = 4.5'	**939.92**	**939.2**	
Maximum Closure Height	**6.92**		
100-Year, Q = 50,000 cfs	WSEL - 935.37 EGL - 935.42	933.88	934.70
Top-of-Levee Event, Q = 81,000 cfs	WSEL - 935.69 EGL - 935.70	938.04	939.22

Closure Actions Required based on Period of Record
Period of Record for Annual Peaks from 1910 through 2006
(Period from 1910 through 1937 is before construction of Lac Qui Parle Dam)

Action	No of times / Probability	Date	Annual Peak Elevation
Mobilize to Install Closure		4/6/1997	933.02
	~7 times in 97 years	4/14/2001	931.27
	7.2%	4/12/1969	930.80
		4/10/1952	929.14
		6/25/1919	926.57
		4/8/1986	926.38
		8/4/1993	926.30
Install Closure		4/6/1997	933.02
	~4 times in 97 years	4/14/2001	931.27
	4.1%	4/12/1969	930.80
		4/10/1952	929.14

Table 4
Minnesota River at Montevideo, MN
Canton Avenue Closure - Operating Plan
All Elevations in NGVD 1929
Closure Low Point - Elevation 935.7

Action	Canton Avenue Closure (XS 39)	USGS Gage (XS 111.0)	
	WSEL	WSEL	EGL
Mobilize to Install Closure			
* - Mn River Q = 18,000 cfs, 4.6% Exceedence Frequency, 22-Year	929.7	927.9	928.3
* - 24 hours prior to Installation trigger based on 3 feet/day rate of rise			
Install Closure			
* - Mn River Q = 31,500 cfs, 2.7% Exceedance Frequency, 37-Year	932.7	930.9	931.3
* - 24 hours prior to WSEL being at base of closure based on 3 feet/day rate of rise			
Base of Closure			
* - Mn River Q = 50,000 cfs, 1% Exceedance Frequency, 100-Year	935.7	933.9	934.7
Top-of-Levee Elevation - based on 100-Year EGL = 4.5'	**940.35**	**939.2**	
Maximum Closure Height	**4.65**		
100-Year, Q = 50,000 cfs	WSEL - 935.71 EGL - 935.85	933.88	934.70
Top-of-Levee Event, Q = 81,000 cfs	WSEL - 939.75 EGL - 939.76	938.04	939.22

Closure Actions Required based on Period of Record

Period of Record for Annual Peaks from 1910 through 2006

(Period from 1910 through 1937 is before construction of Lac Qui Parle Dam)

Action	No of times / Probability	Date	Annual Peak Elevation
Mobilize to Install Closure - 929.7		4/6/1997	933.02
	~4 times in 97 years	4/14/2001	931.27
	4.1%	4/12/1969	930.80
		4/12/1969	929.14
Install Closure - 932.7	~3 times in 97 years	4/6/1997	933.02
	3.1%	4/14/2001	931.27
		4/12/1969	930.80

Table 5
Minnesota River at Montevideo, MN
Parkway Drive Earthfill Closure - Operating Plan
with Road Raise to meet
14' - 9" Minimum Urban Clearance without Signs
All Elevations in NGVD 1929

Action	Parkway Drive Closure (XS 43)	USGS Gage (XS 111.0)	
	WSEL	WSEL	EGL
Mobilize to Install Closure			
* - Mn River Q = 16,000 cfs, 4.9% Exceedance Frequency, ~20-Year	930.75	927.6	927.8
* - 24 hours prior to Installation trigger based on 3 feet/day rate of rise			
Install Closure			
* - Mn River Q = 32,000 cfs, 2.6% Exceedance Frequency, ~38-Year	933.75	931.0	931.4
* - 24 hours prior to WSEL being at base of closure based on 3 feet/day rate of rise			
Base of Closure			
* - Mn River Q = 53,000 cfs, 0.9% Exceedance Frequency, ~110-Year	936.75	934.3	935.2
Top-of-Levee Elevation - based on 100-Year EGL = 4.5'	**940.73**	**939.2**	
Maximum Closure Height	**3.98**		
100-Year, Q = 50,000 cfs	WSEL - 936.22 EGL - 936.23	933.88	934.70
Top-of-Levee Event, Q = 81,000 cfs	WSEL - 940.07 EGL - 940.08	938.04	939.22

Closure Actions Required based on Period of Record
Period of Record for Annual Peaks from 1910 through 2006
(Period from 1910 through 1937 is before construction of Lac Qui Parle Dam)

Action	No of times / Probability	Date	Annual Peak Elevation
Mobilize to Install Closure		4/6/1997	933.02
	~4 times in 97 years	4/14/2001	931.27
	4.1%	4/12/1969	930.80
		4/10/1952	929.14
Install Closure		4/6/1997	933.02
	~2 times in 97 years	4/14/2001	931.27
	2.1%		

Table 6
Montevideo, MN Flood Control Project
Summary of Closure & Closure Operation Data
All Elevations in NGVD 1929

Closure	Type	Quantity of Earthfill or Stoplogs	Low Point Elevation	100-Year Elevation	Low Point in Respect to 100-Year WSEL	Maximum Height	Sliding Factor of Safety	Estimated Time Required to Install	at Minnesota River USGS Gage 05311000					
									Mobilize to Install Closure Trigger (1)			**Install Closure Trigger (2)**		
									Elevation	Stage	Recurrence Interval	Elevation	Stage	Recurrence Interval
Highway 212	Earthfill	101 CY	937.2	933.9	3.3	2.0	1.89	3 Hours	930.8	21.7	36-Year	933.4	24.3	83-Year
Highway 59/7	Earthfill	104 CY	936.8	935.2	1.6	3.0	1.68	3 Hours	929.9	20.8	29-Year	932.6	23.5	63-Year
Railroad with Rail Recesses	Stoplog	10 Stoplogs	933	935.4	**-2.4**	6.9	N/A	6 hours	926.5	17.4	13-Year	928.9	19.8	24-Year
Canton Avenue	Earthfill	342 CY	935.7	935.7	0.0	4.65	1.49	6 hours	927.9	18.8	22-Year	930.9	21.8	37-Year
Parkway Drive with Road Raise (to meet 14'-9" Minimum Urban Clearance without Signs)	Earthfill	166 CY	936.75	936.2	0.5	4.0	1.56	4 hours	931.0	21.9	38-Year	927.6	18.5	20-Year

(1) - Mobilize to Install Closure Trigger based on when river is 6 feet below base of closure which provides 48 hours before closure may be required based on the 3 foot/day historical rate of rise.

(2) - Install Closure Trigger based on when river is 3 feet below base of closure which provides 24 hours before closure may be required based on the 3 foot/day historical rate of rise.

APPENDIX D

Toward a Probability and Uncertainty-Based Approach for Characterizing the Flood Hazard Associated with Storm Surge, Wave, and Overtopping of Levees

1. High winds during storms (hurricanes and extratropical events) are the primary force for the abnormally high water levels and wave conditions that are experienced in lakes, estuaries, and along the coast. The abnormally high water levels and energetic wave conditions can create a flooding hazard. In efforts to characterize this hazard, the quality of storm-induced water level and wave prediction is only as good as the accuracy and resolution of the wind conditions used to make the assessment. Lakes are generally much smaller in spatial extent than the weather systems that generate extreme winds, so wind conditions over the lake may be assumed to be uniform for smaller lakes. For large lakes, the Great Lakes for example, and for coastal and estuarine situations, the full two-dimensional variation of wind fields must be considered for estimating water level and wave conditions. Time variation of the winds is important in storm situations.

2. Historically most attention has been paid to estimation of water level changes associated with high winds (often referred to as wind setup) or storm surge which arises from forces in addition to the wind, because water level is the first-order parameter of importance in flooding. However, in some cases, insufficient attention was given to waves generated by the same winds, to the dependencies and interaction of waves and surge, and to the role of waves and wave overtopping in levee system design and performance. Generation of significant wave heights of 2 to 4 feet or more is possible even in restricted-fetch situations subjected to high winds, where fetches are only a few miles in length (much higher wave heights are possible for hurricane-force winds). Very large lakes and estuaries can experience wave heights of up to 10 ft or more, and major storm-generated significant wave heights can reach 30 feet or more in open coastal areas. For restricted fetches of several miles and very strong wind events, wave periods are generally 3 to 5 sec; for larger lakes, wave periods are generally 5 to 8 sec; and, for ocean conditions, storm wave periods can range from 12 to 20 sec.

3. As waves are generated across an open-water fetch propagate into shallow water, they begin to break and their energy is dissipated. In very shallow water (shallow in terms of water depth-to-wave length ratio), wave height tends to be limited by breaking and it becomes proportional to the local water depth. To aid in a quick first assessment of the importance of waves to levee system overtopping, the depth-limited significant wave height at the toe of a levee fronted by extensive shallow water areas with very small slopes (such as natural marsh areas) is roughly 40% of the local still-water depth at the toe. The depth-limited significant wave height is about 60% of the local still-water depth at the toe for situations where the water is deeper seaward of the levee toe. Depending on the other factors that govern wave generation and propagation (fetch and wind speed along the fetch are important), significant wave heights may not reach this depth-limited value, but they probably will not exceed this value. More rigorous analysis of waves is required to support levee design and NFIP levee system evaluation, and these depth-limited conditions may not always be realized depending upon the actual incident wave conditions, but

these rough rule-of-thumb estimates provide a starting point to assess whether or not waves are an issue to be considered further.

4. The IPET investigation of Hurricane Katrina and response of the Hurricane Protection System in Southeast Louisiana to that extreme event showed that levee system and floodwall failure was dominated by overtopping, either (1) overtopping of levees and subsequent erosion of the levee from the protected side toward the flooded side, or (2) overtopping of walls, scour on the protected side of the wall, and subsequent failure of the wall. To a lesser degree, failure was caused by scour at transitions between levees and hard structures. Levee response and wave and water level conditions throughout the system were far from uniform. It was not possible within the time frame of the IPET investigation to fully evaluate the relative roles of the following factors in the overtopping and levee erosion process: spatially varying waves and still-water levels, overtopping duration, levee soil variability, vegetation cover, construction method, and protected side inundation.

5. Current coastal engineering design practice is to use an overtopping discharge rate threshold to define the onset of levee damage (see Table VI-5-6 of EM 1110-2-1100; TAW 1989, 2002). These overtopping rate thresholds are rather crude and uncertain. They somewhat account for certain factors that can influence levee resistance to erosion, such as quality of levee soils (clayey versus sandy sediments, for example), condition of vegetation cover, and presence of armoring. However, the thresholds do not incorporate, nor are they so sensitive to, several other key site-specific conditions that influence levee erosion in response to overtopping: thickness of protective soil cover, degree of root system development or lack thereof, degree of cracks and gullies in the levee soil, spatial variations in soil properties or presence of perturbations or structural features on the levee that would promote local scour if overtopped, duration of overtopping, or degree of inundation on the back side that might serve as a buffer to degradation of the levee toe. The overtopping rate threshold criteria identified in the references cited above do not account for differences between the periodic unsteady nature of wave overtopping and steady flow over the levee. The thresholds have been developed primarily based on situations where the still-water level is above the levee crest and overtopping is rather steady. During wave overtopping alone, or wave overtopping in addition to still-water overtopping, the velocities associated with each wave crest passage can be significantly higher than velocities associated with steady flow alone. Computations made by IPET investigators showed that velocities on the back side of levees due to wave overtopping were three or more times greater than velocities associated with steady flow overtopping (which translates into an erosion potential of about 30 times greater, albeit not a steady forcing). The overtopping rate thresholds do not distinguish between steady velocities at lower speeds, and intermittent and shorter duration velocities at much higher speeds. This factor needs to be examined further.

6. Despite the crudeness and limitations that are inherent in these threshold values, those listed below and described in more detail in the main text are believed to be conservative. They appear to have worked reasonably well in engineering practice. Site-specific, full-scale tests are the preferred method for defining the threshold values for a particular levee condition/ configuration, but data from these tests is sparse and not nearly inclusive of all the important factors that can effect levee erosion. The threshold values cited here are intended to define the onset of erosion, or levee damage. However, a small amount of erosion on a large earthen levee

might not erode much of the levee cross-section at all, and might not lower the levee crest elevation and therefore not compromise levee performance at all. Levee erosion and damage is a function of duration, which is not considered in these thresholds. This is another reason why the thresholds might be conservative. At present, for unarmored earthen levee system NFIP levee system evaluation, a high value of the average wave overtopping rate threshold, 0.1 cfs/ft, should be reserved for the highest quality levees, unless a higher value is well-supported by large-scale testing. The overtopping rate threshold for a clayey soil with relatively good grass is 0.01 cfs/ft. This value is more typical. The wave overtopping rate threshold for sandy soils and poor turf is lower, 0.001 cfs/ft, which is effectively zero overtopping. Simple methods for calculating average wave overtopping discharge, as a function of still-water level relative to levee system crest, levee cross-sectional shape, and incident wave conditions, are provided in Part VI (Chapter 5) of EM 1110-2-1100. Large-scale field tests, or smaller-scale laboratory tests, or application of numerical models that solve the Boussinesq equations, are more rigorous and accurate alternatives for levee configurations that differ significantly from the laboratory test conditions that were used to produce the data upon which the simple calculation methods in EM 1110-2-1100 are based. Work is underway in the Louisiana Coastal Restoration and Protection Study (LaCPR) to further examine the issue of wave overtopping threshold for the onset of damage to earthen levees; additional work is needed on this topic to refine the thresholds for various types of levee conditions. Also, work is being pursued in the LaCPR project to establish a link between wave overtopping flows and what is currently known about steady flow overtopping.

7. The average overtopping discharge rate due to wave action should be used as a threshold parameter for the evaluation of potential levee failure/damage, instead of the 2% wave run-up elevation, or some other run-up elevation, or some other parameter based directly on wave crest height or wave height as is done in the current HEC-FDA treatment of wave overtopping. The discharge parameter is a more appropriate surrogate for the processes by which levees degrade (velocity, shear stress, and turbulence) than is a run-up elevation or wave crest elevation; and currently, thresholds for levee resistance to erosion (damage) in wave regimes are couched in terms of the average overtopping rate parameter. Also, run-up elevation becomes a nebulous parameter when the computed wave run-up value exceeds the levee crest elevation. Therefore, the average discharge parameter should be used as the threshold for NFIP levee system evaluation in all riverine, lake, estuarine, and coastal situations, until thresholds that more accurately treat both the levee erosion process and levee sediment/vegetation/condition properties can be developed.

8. In addition to changes to the mean water surface due to wind or other forces, wave setup is another process that needs to be considered. Wave setup arises as a result of momentum transfer from the short wave field into the water column as wave height changes, due primarily to breaking. The changing wave height, and changing momentum, exerts a thrust on the water column that can create a local change in the mean, or still, water level. For mild slopes, wave setup can be 15% of the incident significant wave height; 30% of the incident significant height for steep slopes The simple methods for calculating discharge in Part VI (Chapter 5) of EM 1110-2-1100, the Boussinesq-type numerical models and physical-scale modeling, implicitly include the effects of wave setup occurring right at the levee. They do not, however, account for wave setup generated away from the levee. This process is best considered through interactive wave and storm surge modeling, as was done in the IPET investigation, and in the LaCPR and

MsCIP studies. In some coastal situations, interactions among different frequency components in the incident wave field are very important, such as the incidence of energetic narrow-banded wave spectra and the infra-gravity wave motions that can result (for example that which is often experienced on the west coast) or double-peaked wave spectra that are comprised of both short- and long-period components (as was the case for the portions of the southeast Louisiana levee system that were exposed to both long-period Gulf waves and locally generated short-period wave energy). Infra-gravity wave motions can be thought of as longer-period (up to several minutes) time-varying wave setup. In these situations, wave-wave interactions can be extremely important in determining the magnitude of the wave run-up and overtopping, and the simple calculation methods of EM 1110-2-1100 (and the laboratory test cases that produced the data that lead to these calculation methods) might not adequately capture these non-linear effects. Boussinesq-type numerical modeling will produce information that more accurately captures wave overtopping discharge rates for these types of situations.

9. Current engineering calculation tools and methods should be used to assess the storm surge and wave flooding hazard. Two-dimensional models are needed to accurately represent hurricane surface wind and atmospheric pressure fields. Planetary Boundary Layer models are to be used for simulating hurricane wind fields. The ADCIRC model (see http://chl.erdc.usace.army.mil/adcirc) is the recommended tool for computing storm surge. It has been well tested and validated in the IPET, LaCPR, and MsCIP work. The WAM and STWAVE models (see http://chl.erdc.usace.army.mil/chl.aspx?p=s&a=Software;9) are the recommended tools for computing hurricane wave fields. Both have been well tested and validated in these projects. The WAVEWATCH III and SWAN wave models are other acceptable models for making offshore and nearshore wave computations, respectively; both models were evaluated during the IPET examination. Comparisons between results from the ADCIRC, WAM, STWAVE, WAVEWATCH III, and SWAN models and measurements made during Hurricane Katrina, and inter-wave-model comparisons, are presented in the IPET report (https://ipet.wes.army.mil/). The interactions between storm surge and waves (surge changes water depth which influences wave transformation and breaking, and generation of wave setup is a contributor to surge levels in a regional sense) must both be considered. The interactions are important for levee system design and NFIP levee system evaluation coastal and estuarine settings.

10. In general terms, the most technically sound and rigorous approach to assess the probability of levee overtopping associated with hurricanes is to perform the following steps:

a. Define the hurricane hazard by computing the probabilities associated with each hypothetical hurricane considered in a Joint Probability Method Optimal Sampling (JPM-OS) application. Careful consideration needs to be given to decay in intensity of hurricanes as they approach land in the area of interest and the characteristics and tendencies of hurricanes in the region, particularly those of the largest and most intense hurricanes.

b. Simulate the wind/atmospheric pressure fields for each hurricane. Ensure that the specific time-averaging interval (for example a 10-min average) and the reference elevation (typically a 10-m elevation) for the computed winds matches those inherent in the wind shear stress formulations of the storm surge and wave models to be used. If they are not, some scaling of wind speed will need to be done. This is an extremely important issue that is often not properly treated.

c. Simulate the water level and wave fields for each hurricane in an interactive manner to capture regional short wave effects on storm surge and vice versa. For the entire segment of levee being considered, compute the storm water levels and wave conditions just seaward of the levee toe, and identify and use the maximum conditions that occur within a segment of a levee system.

d. Use the water level and wave information to compute wave overtopping discharges for each storm event at all segments where levee overtopping assessments are needed by applying a simple average wave discharge computation method, a Boussinesq-type model (e.g., BOUSS2D; see http://chl.erdc.usace.army.mil/chl.aspx?p=s&a=Software;23), or physical scale model. The Boussinesq model or physical scale model can be applied to compute overtopping for a matrix of wave conditions, water levels, and levee configurations, and a look-up table approach can be adopted to select the appropriate overtopping rate results for any particular levee segment, water level, and set of incident wave conditions.

e. Develop probability-response surfaces for locations of interest that relate the flood response parameter of interest (such as storm surge still-water level, combined still-water level (astronomical tide and surge), wave conditions, or average wave overtopping discharge rate) to the storm probabilities. Finely and consistently discretize those probability-response surfaces, accounting for uncertainty in the surfaces. Then integrate the surface(s) to develop the statistical flooding exceedance probabilities (1% annual chance exceedance flood values with 90% assurance, for example) that are desired, factoring in uncertainty, for each levee segment.

f. Compare values of the 1% chance exceedance wave overtopping rate (having 90% assurance) to the overtopping threshold for each segment or important component of the levee system. If the 1% annual chance exceedance flood wave overtopping value (at 90% assurance) is less than the overtopping threshold, then that segment of levee can be determined to be found in accordance with NFIP levee system evaluation requirements. Levee systems are only as robust as the weakest link, so each segment of the levee system should pass this overtopping assessment before the entire system is in accordance with NFIP levee system evaluation requirements.

11. This is the general direction in which probability of exceedance assessment is heading for the coastal/estuarine/
lake environment. This same approach has applicability to non-hurricane events. Other methods for computing still-water levels and waves associated with extratropical storm events could be used in place of steps (a) and (b) above in the JPM-OS approach.

12. The following step-by-step approach is being used by the New Orleans District for design/NFIP levee system evaluation in the New Orleans vicinity. It represents a reasonable first step toward the more general approach outlined above. It is a somewhat conservative approach in that 1% annual chance exceedance significant wave heights and 1% annual chance exceedance wave periods are computed independently and used with 1% annual chance exceedance flood still water levels, to compute wave overtopping rates; as opposed to computing wave

overtopping rates using the actual wave conditions that were associated with the still-water level conditions for each of the individual storms. The approach outlined below is most valid if the wave conditions are highly correlated with the water levels, which might not be the case.

Step 1: Define Water Level

1.1 Examine the 1% annual chance exceedance surge elevation from the surge-frequency plots at all output points along the levee segments under consideration. The annual chance exceedance surge elevations are the results based on the 152 storm combinations and using the probabilistic tool (JPM-OS method).

1.2 Determine the maximum 1% annual chance exceedance surge elevation for each levee segment and use this number for the entire segment. The maximum is chosen to to be in accordance with NFIP levee system evaluation/design criterion at the most critical point within the segment.

Step 2: Define Wave Characteristics

2.1 Examine the 1% annual chance exceedance significant wave height and 1% annual chance exceedance peak period from the separate wave- and period-frequency plots at all output points along each levee segment. The 1% annual chance exceedance wave heights and 1% annual chance exceedance peak periods are the results based on the 152 storm combinations and using the probabilistic tool based on the JPM-OS method.

2.2 Determine the maximum 1% annual chance exceedance significant wave height and 1% annual chance exceedance peak period for the segment and use these numbers for the entire segment. The maximum significant wave height and wave period are chosen to meet the NFIP levee system evaluation requirements/design criterion at the most critical point in the segment.

2.3 Determine if the foreshore in front of the levee is shallow. The foreshore is shallow if the ratio between the significant wave height (Hs) and the water depth (h) is small ($Hs/h > 1/3$) and if the foreshore length (L) is longer than one deep water wave length L_0 (thus: $L > L_o$ with $L_o = gT_p^2/(2\pi)$). If so, the wave height at the toe of the structure should be reduced according to $H_{smax} = 0.4$ h (the broken wave height limit). This reduction should be applied only if an empirical method is applied for determining the overtopping rate. The breaking wave effect is automatically included in the Boussinesq model results.

Step 3: Define Overtopping Rate

3.1 Determine if Boussinesq results are available for the specific levee segment. If so, use the Boussinesq results from the lookup table. If not, use the van der Meer formulations (see EM 1110-2-1100 or TAW 2002).

3.2 Determine the overtopping rate based on the 1% annual chance exceedance surge level, the significant wave height and the peak period. Use the reduced wave height in case of a shallow foreshore in the empirical approach only.

3.3 Check if the wave overtopping rate is less than the adopted threshold rate, 0.1 cfs/ft. If this criterion is exceeded, the levee geometry should be adapted in such a way that the overtopping rate is lower than 0.1 cfs/ft. Note, the mean overtopping rate should be (much) less than 0.1 cfs/ft in order to meet the criterion of 90% non-exceedance in Step 4, because average values are applied for the 1% annual chance exceedance surge level and 1% annual chance exceedance wave characteristics.

Step 4: Dealing with Uncertainties

4.1 Apply a Monte Carlo simulation to compute the chance of exceedance of the overtopping rate given the levee crest elevation and levee slope from Step 3. This method takes into account the uncertainties in the 1% annual chance exceedance water level, the 1% annual chance exceedance wave height and the 1% annual chance exceedance wave period.

4.2 Check if the overtopping rate will not exceed the overtopping criterion of 0.1 cfs/ft with a 90% assurance. If yes, the design or NFIP levee system evaluation process is finished from a hydraulic point of view and the NFIP levee system evaluation finding is positive. If not, the NFIP levee system evaluation finding is negative (adapt the levee or floodwall height or slope in such a way that this threshold criterion is not exceeded).

4.3 The hydraulic and geometrical parameters in the design/NFIP levee system evaluation approach are uncertain. Hence, the uncertainty in these parameters should be taken into account in a probabilistic treatment. The following sections propose a method that accounts for uncertainties in water levels and waves, and computes the overtopping rate with state-of-the-art formulations. The objectives of this method are to include the uncertainties and check if the overtopping criterion of 0.1 cfs/ft is still met with a certain percentage of assurance, 90%. The parameters that are included in the uncertainty analysis are the 1% annual chance exceedance water level, 1% annual chance exceedance wave height and 1% annual chance exceedance wave period. Uncertainties in the levee geometrical parameters are neglected. Uncertainties in the method used to predict wave overtopping are included.

4.4 The criterion used in this design approach is the overtopping rate, as mentioned above. For this purpose, the probabilistic overtopping formulation was applied but also the Boussinesq results could be incorporated in the method. Besides the geometrical parameters (levee height and slope), hydraulic input parameters for determination of the overtopping rate are the water level, the significant wave height, and the peak period. In the design/NFIP levee system evaluation process, the 1% annual chance exceedance values for these parameters are from the JPM-OS method. Obviously these numbers are uncertain. An additional analysis provided the standard deviation in the 1% annual chance exceedance still-water level (which accounted for a number of sources of uncertainty). Standard deviation values of 10% of the average significant wave height and 20% of the peak period were used, which were based on expert judgment. All uncertainties are assumed to be normally distributed.

4.5 The Monte Carlo analysis that was applied is executed as follows:

a. Draw a random number between 0 and 1 to set the exceedance probability p.

b. Compute the water level from a normal distribution using the 1% annual chance exceedance surge level and standard deviation as parameters and with an exceedance probability p.

c. Draw a random number between 0 and 1 to set the exceedance probability p.

d. Compute the wave height and wave period from a normal distribution using the expected value 1% annual chance exceedance wave height and 1% annual chance exceedance wave period and the associated standard deviations, and with an exceedance probability p.

e. Repeat steps 3 and 4 for the three overtopping coefficients in the overtopping formula, independently, using estimates of variability (standard deviation) in each coefficient.

f. Compute the overtopping rate for these hydraulic parameters and overtopping coefficients.

g. Repeat steps 1 through 5 a large number of times (N).

h. Compute the 50, 90, and 95% value of the overtopping rate (i.e., q_{50}, q_{90} and q_{95}).

13. The procedure was implemented in MATLAB. Several test runs showed that N should be approximately 10,000 to reach statistically stationary results for q_{50}, q_{90}, and q_{95}. The computation time to perform this analysis was on the order of tens of seconds on a current state of the art personal computer.

APPENDIX E

Lessons Learned from the Interagency Performance Evaluation Team's (IPET) Report: *Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System*

OVERVIEW

The performance of the I-walls in New Orleans in the 2005 Hurricane Katrina disaster provides numerous lessons learned, all of which should be understood and considered when performing a NFIP levee system evaluation. This appendix provides a very brief summary of the structural aspects of the disaster, including specific discussions of some of the well-analyzed failures. The entire IPET report is a very large document that is well beyond the scope of this appendix to capture all of the information pertinent to the safety of a levee system. The text of this report is available at https://ipet.wes.army.mil/. In particular, Volume V. The Performance - Levees and Floodwalls, and the structural appendices shall be reviewed and the levee system under consideration for NFIP levee system evaluation shall be considered in this context. Figures shown were part of the IPET report.

Over 220 miles of the New Orleans area's flood risk reduction structures were damaged by storm surges and waves generated by Hurricane Katrina, as well as 34 of 71 pumping stations. Approximately 41 miles of structures were judged to be severely damaged. Initially a total of 50 major breaches were identified, areas where the structures failed, causing a dramatic reduction in design elevation and loss of the ability to prevent the inflow of external water. Of the 50 major breaches, four were caused by foundation-induced failures and the remainder by a combination of overtopping and scour. Three of the four foundation breaches occurred in the outfall canals and one in the Inner Harbor Navigation Canal (IHNC). I-wall structures were particularly vulnerable as were levee sections created from hydraulic fill, and transitions where either elevation or strength differences occurred from changes in structure type or capability. Transitions between types of flood risk reduction structures were also vulnerable, especially where the transition included a significant change in elevation between the structures.

The overtopping waves created very high water velocities down the back sides of the levees, reaching 10 to 15 ft/sec. These velocities were two to three times those experienced on the water side of the levees (4 to 6 ft/sec). The potential for erosion is related to the cube of velocity; thus the back sides of the levees, especially where they were comprised of erodible materials, were scoured away leading to, in many cases, complete breaching.

DESCRIPTION OF FAILURES

Early in the morning of 29 August, around 0500 hours, CDT, a section of I-wall along the Lower Ninth Ward breached. Underlain by the same marsh deposits and clay as the 17th Street Canal, the rising water and waves caused the wall to deflect enough to open a crack that created a direct avenue for high water pressures to reach the foundation. The weak clays underneath, now only reacting with the mass of soil on the protected side of the levee, could not withstand the force and displaced backward, a process that would repeat itself on the 17th Street Canal. The water levels in the IHNC were approximately 9.5 to 10.5 ft when the foundation failure occurred. At the 17th Street Canal, failure began with apparent displacement of a wall panel at about 0630 hours and ended with a full breach by 0900 hours. At about 0630 hours, the water level was estimated to have been 7 to 8 ft, and possibly 1 to 2 ft higher at the time of the catastrophic breach created by displacement of a levee section. No overtopping had occurred and the design water elevations had not been reached at either location at the time of levee displacement.

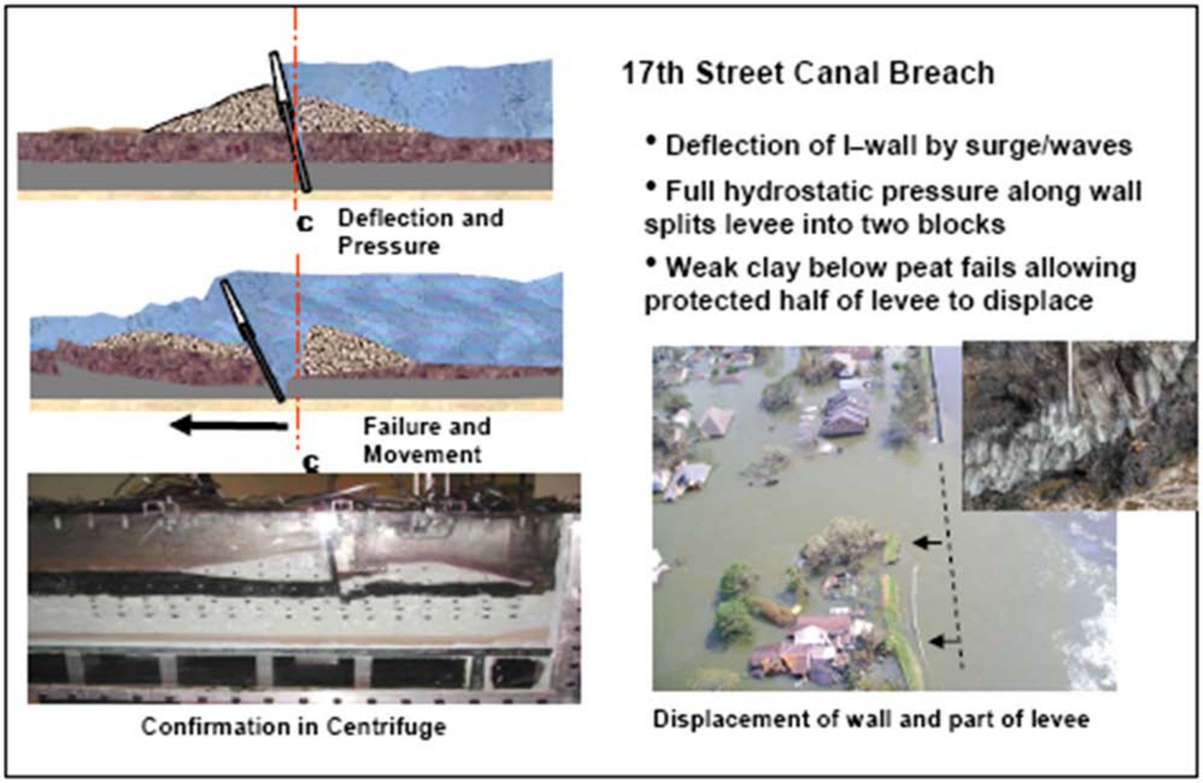


Figure 20. Depiction of failure mechanism for 17th Street and IHNC foundation failures. A crack forming along the front of the I-wall introduced high forces down the face of the sheet pile, resulting in lateral movement of the floodwall along a shear plane in the weak clay foundation.

At around 0700-0800 hours, not long after the 17th Street Canal breach started, the south breach on the London Avenue Canal was initiating. As in the case of the 17th Street Canal and IHNC failures, water elevations below the design levels caused a crack to form on the water side of the floodwall and allowed high pressures to be introduced directly into the foundation materials of the levee, this time relic beach sand. The porous sand quickly conveyed the pressure under the levee and caused significant uplift on the protected side. It also is likely that significant subsurface erosion occurred under the levee and caused a blowout on the protected side through which much sand and water flowed, decreasing the support for the levee and floodwall and causing a narrow failure. The north breach on the London Avenue Canal suffered a similar fate around the same time, 0700-0730 hours. This breach was much wider and involved less erosion, failure being caused by a loss of stability from the uplift. Water levels in the London Avenue Canal reached about 9 ft, below the design levels and well below the height of the I-walls. Figure 21 depicts the breaching mechanisms for the London Avenue breach sites. This finding was arrived at through detailed field investigations, independent seepage and stability analyses by Virginia Tech and Engineer Research and Development Center (ERDC) teams and separate centrifuge modeling by Rensselaer Polytechnic Institute (RPI) and ERDC. Ironically, the Orleans Canal, geographically located between the 17th Street and London Avenue Canals, and having similar geological conditions, did not suffer failure, even though water levels there reached over 10 ft. The geology at the north end of the Orleans Canal is similar to 17th Street conditions (peat underlain by clay) and the south end is similar to London Avenue subsurface conditions (peat underlain by relic beach sands). The design of the levees and I-walls for the Orleans Canal was more conservative than for the other canals, with a broader and stronger levee section and less I-wall height above the levee crest. Soil strength assumptions were also more conservative than for the structures along the 17th Street Canal. This knowledge was an important component of the development of criteria for evaluating the integrity of the sections of the Hurricane Protective System (HPS) that were not severely damaged. Combined with the knowledge of the failure modes for the I-walls and levees, a series of remedial actions were developed that could be used to at least temporarily strengthen sections of the HPS that were deemed least able to withstand a large storm. This led to a large scale examination of the HPS by the New Orleans District to identify areas needing remedial action prior to the 2006 hurricane season.

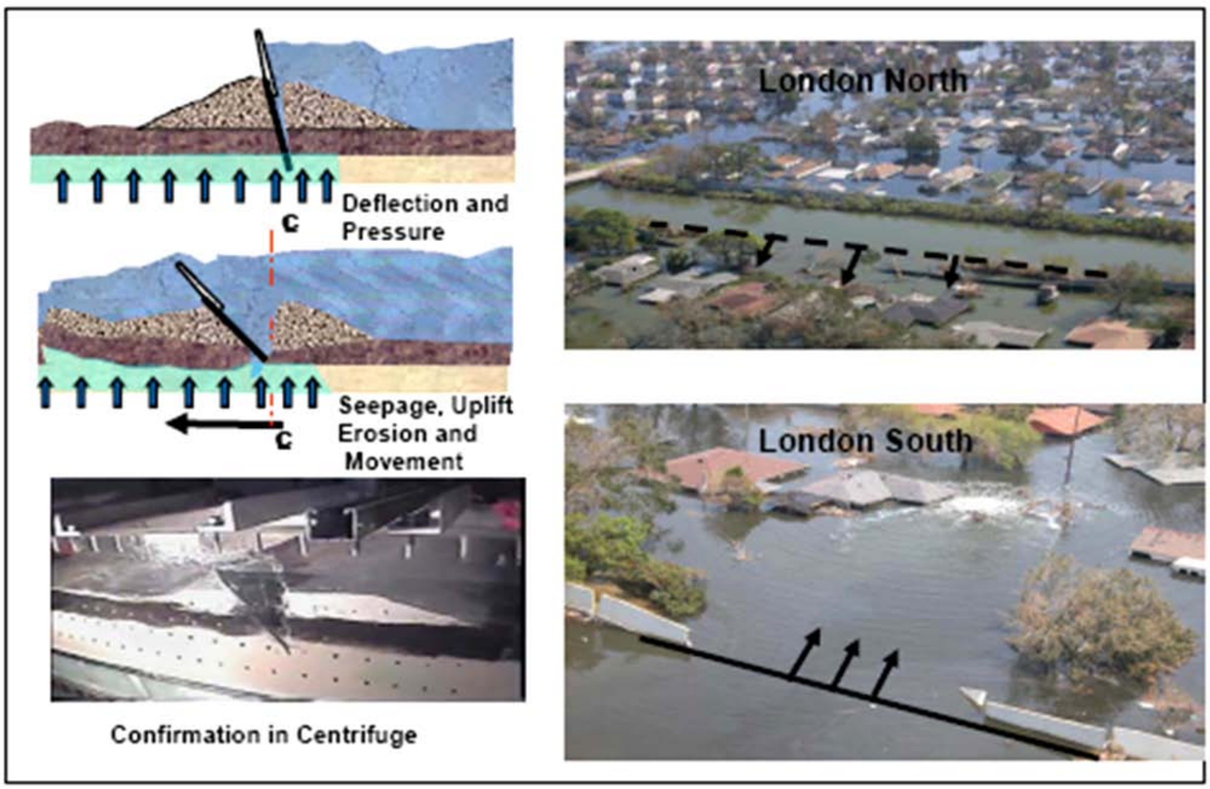


Figure 21. Depiction of London Avenue Canal north and south breaches caused by deflection of the I-wall and high uplift pressures introduced through the underlying porous sands, confirmed in centrifuge testing.

During mid-morning, the I-walls along the IHNC were overtopped and erosion behind the wall reduced their stability, causing three separate sections to fail. The top photograph in Figure 21 shows a section of I-wall along the IHNC collapsed after overtopping created a scour trench behind it and reduced its stability. The bottom photograph shows an adjacent section of I-wall where the scour trench formed but the wall did not fail. Water levels reached over 14 ft in the IHNC. There was also a levee failure along the west side of the IHNC that caused additional flooding into the Upper Ninth Ward. No T-wall failures occurred with the exception of a small section in southern Plaquemines Parish.

SUMMARY OF LESSONS LEARNED:

Of the 50 major breaches experienced by the HPS during Katrina, all but 4 were due to overtopping and erosion. For floodwalls, the overtopping caused erosion behind the walls that eventually caused instability and wall failure. For levees, the scour eroded the back sides and tops of the levees due to high velocities of the overtopping waves in areas of erosion-susceptible soils creating breaching. Areas with high quality levee materials performed well in the face of water conditions that exceeded their design criteria. Structures at authorized design elevations would have reduced the amount of overtopping.

There was no evidence of systemic breaching caused by erosion on the face or water sides of the levees exposed to surge and wave action. The water velocities on the face side were only one-third of those experienced at the crest and back or protected side of the levees. The levees largely performed as designed, withstanding the surge and waves until overtopping, at which time they became highly vulnerable to erosion and breaching, especially those constructed by hydraulic fill.

Four breaches, all in the outfall canals and IHNC and all involving I-walls, occurred before water levels reached the top of the floodwalls. All were caused by foundation failures induced by the formation of a gap along the canal side of the floodwall. All of these structures were built over a layer of marsh sediments, in two cases underlain by clays and in the other two underlain by relict beach sand deposits. Along the outfall canals, the subsurface conditions dictated the specific mechanics that, coupled with the high hydrostatic pressures introduced to depth by the gap along the face of the sheet pile, led to instability and failure. The sites underlain by sand experienced significant uplift pressures, seepage and, in one case, a massive piping of subsurface sand from under the levee to the protected side. This action undermined the floodwall.

Transitions between types and design levels and between flood risk reduction structures and other features created vulnerabilities to erosion and breaching and reduced the effectiveness of the flood risk reduction. Some of the transitions are associated with changes in the organization responsible for the structures, some are due to incompletion of the authorized construction, and others are associated with necessary penetrations through the levee/floodwall system.

GLOSSARY

Terms and Abbreviations

100-year flood (for NFIP levee system evaluation and FEMA mapping accreditation) **–** The median peak flood discharge having a 1% annual chance exceedance expressed as a return period.

Accredited levee – A levee that the Federal Emergency Management Agency (FEMA) has shown on the Flood Insurance Rate Map as having reasonable assurance of excluding the 1% annual chance exceedance flood from the leveed area. This determination is based on the submittal of data and documentation as required by Section 65.10 of the National Flood Insurance Program regulations. FEMA will accredit a levee that has been determined to be found in accordance with NFIP levee system evaluation requirements by federal agencies with levee design and construction competence, such as U.S. Army Corps of Engineers (USACE), or by a registered professional engineer.

Annual chance exceedance flood – The flood that has a (stated percent - %) chance of being equaled or exceeded in any given year, such as the 1% annual chance flood.

Annual exceedance probability (AEP) – The probability that a random variable (e.g. flood discharge or stage) will occur in any given year considering the full range of annual possible flood discharges**.**

Assurance – The probability that a target stage will not be exceeded during the occurrence of a specified flood. For example, USACE requires that, for a levee system to be found in accordance with NFIP levee system evaluation requirements, it must have at least a 90 percent chance of not being overtopped when subjected to the estimated 1% annual chance exceedance flood. Term selected to replace 'conditional non-exceedance probability'.

Base flood (FEMA BF) – The flood that has a median estimate of 1% annual chance of being equaled or exceeded in any given year.

Base flood elevation (BFE) – The water surface elevation of the median estimate of the 1% annual chance exceedance flood.

Community – Any state or area or political subdivision thereof, or any Indian tribe or authorized tribal organization, or Alaska Native village, or authorized native organization that has the authority to adopt and enforce floodplain management regulations for the areas within its jurisdiction.

Conditional non-exceedance probability – See definition for "Assurance."

Deterministic analysis – A technical analysis approach that is accomplished using single values for key variables as opposed to using a probability distribution of values for the key variables (which acknowledges and incorporates uncertainty).

Digital flood insurance rate map (DFIRM) – A Flood Insurance Rate Map (FIRM) that has been prepared as a digital product. Linkages are built into an associated database to allow users options to access the engineering backup material used to develop the DFIRM, such as hydrologic and hydraulic models, flood profiles, data tables, digital elevation models (DEMs), and structure-specific data, such as digital elevation certificates and digital photographs of bridges and culverts.

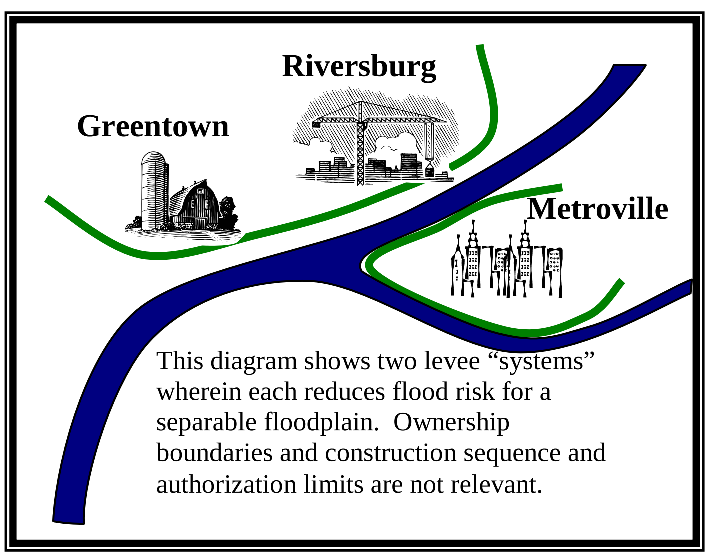
Federal Emergency Management Agency (FEMA) – The agency within the Emergency Preparedness and Response Directorate of the U.S. Department of Homeland Security. FEMA oversees the administration of the National Flood Insurance Program.

Flood damage reduction – The objective of flood-related projects, levee systems, structures, or measures. These include structural and non-structural measures taken to reduce flood damage. These may include implementation of reservoirs, detention storage, channels, diversions, levees, interior drainage systems, flood-proofing, raising, relocation, and flood warning and preparedness actions.

Flood-frequency – A graph, table, or single tabulation showing the relationship of the flood variable of interest (peak flow, peak stage, 3-hour volume, etc.) to the probability of the variable being equaled or exceeded in any given year.

Flood insurance rate map (FIRM) – The insurance and floodplain management map produced by FEMA that identifies, based on detailed or approximate analyses, the areas subject to flooding during a 1% annual chance exceedance (100-year) flood event in a community. Flood insurance risk zones, which are used to compute actuarial flood insurance rates, also are shown. In areas studied by detailed analyses, the FIRM shows Base Flood Elevations (BFEs) to reflect the elevations of the 1% annual chance exceedance flood. For many communities, when detailed analyses are performed, the FIRM also may show areas inundated by 0.2% annual chance exceedance (500-year) flood and regulatory floodway areas.

Flood map modernization (Map Mod) program – The multiyear program undertaken by FEMA to perform flood hazard assessments and produce new or updated DFIRMs and Flood Insurance Study (FIS) reports for flood-prone communities throughout the United States.



This diagram shows two levee "systems" wherein each reduces flood risk for a separable floodplain. Ownership boundaries and construction sequence and authorization limits are not relevant.

Freeboard – The increment of levee height added to the design flood height to increase the likelihood of the design event being contained without the levee overtopping.

Incised channels – Incised channels are flood damage reduction channels wherein the design water surface is below the natural ground levels.

Levee system – A levee system comprises one or more components which collectively provide flood risk reduction to a defined area. Failure of one component within a system constitutes failure of the entire system. The levee system is inclusive of all components that are interconnected and necessary to ensure exclusion of the base flood from associated separable floodplain (or separable consequence area) – levee and floodwall sections, closure structures, pumping stations, culverts, and interior drainage works. This diagram is an example of how levee systems are to be interpreted for the purposes of this EC.

Leveed Area - the lands from which flood water is excluded by the levee system.

Level-of-protection associated with NFIP levee system evaluation– The recurrence interval of flooding that with a high degree of assurance will not result in levee failure or overtopping that subsequently would inundate the leveed area. As used herein, the full range of uncertainty in discharge or stage is considered but other elements of levee performance are based on deterministic analysis and criteria.

Life risk - The threat to loss of life from failure of a flood risk reduction system or feature. Life risk is often expressed as an annual exceedance probability vs. incremental life lost function or expected value of that function (annual lives lost), sometimes referred to as 'annual statistical lives lost.'

Monte Carlo analysis – A method that produces a statistical estimate of a variable of interest by drawing many random samples from a set of variables with associated uncertainty or relationships of interest with their associated uncertainty. The method is typically used when values for variables are uncertain and best described by appropriate probability distributions.

National Flood Insurance Program (NFIP) – Federal program under which flood-prone areas are identified and flood insurance is made available to the owners of the property in participating communities.

NFIP levee system evaluation – The purpose of an NFIP levee system evaluation is to determine how flood hazard areas behind levees are mapped on FIRMs. The resultant maps are used to determine flood insurance rates; federal, state, and local floodplain management requirements; and other floodplain management decisions. It should be noted here that the definition of 'NFIP levee system evaluation' for the purposes of USACE application under this EC is consistent with definitions in 44 CFR 65.10. If a positive finding is made in an NFIP levee system evaluation, FEMA will use this information to determine how the floodplain behind the levee system is mapped.

NFIP levee system evaluation determination – This is a technical finding by a registered professional engineer that, for the floodplain (or consequence area) in question, <u>there is, or is not</u>, a reasonable assurance that the levee system will exclude the 1% annual chance exceedance flood from the leveed area. A ‘there is’ answer leads to a positive finding, and an ‘is not’ answer means a negative finding for NFIP levee system evaluation thus, accreditation is not supported.

Non-structural measures – Non-structural measures reduce flood damages without significantly altering the nature or extent of flooding. Damage reduction from non-structural measures is accomplished by changing the use made of the floodplains, or by accommodating existing uses to the flood hazard. Examples are flood proofing, relocation of structures, flood warning and preparedness systems (including associated emergency measures), and regulation of floodplain uses.

Overtopping – A condition that occurs when the elevation of the still-water level and/or associated waves exceeds the top of the levee or system.

Periodic Inspection – A visual inspection, typically conducted every five years by a team of professional engineers, to verify proper operation and maintenance; evaluate operational adequacy and structural stability; identify features to monitor over time; and improve the ability to communicate the overall condition.

Probability – A measure of the likelihood, chance, or degree of belief that a particular outcome or consequence will occur. A probability provides a quantitative description of the likelihood of occurrence of a particular event. This is expressed as a value between 0 and 1.

Probabilistic analysis – The application of probability theory and statistical methods to make inferences about information, facility performance, and the associated uncertainty in inferences.

Probability function – A relationship between exceedance probability and a variable of interest. The function could be graphical, tabular, or analytical. For example, a discharge-exceedance or stage-exceedance probability relationship for a reach developed by traditional, site-specific, hydrologic engineering analysis procedures.

Project - A flood damage reduction project is made up of one or more flood damage reduction systems which are under the same authorization.

Public safety – Public safety involves the prevention of and protection from events that could endanger the safety of the general public from significant danger, injury/harm, or damage, such as natural and man-made disasters.

Reach – A levee reach is a portion of a levee system (usually a length of levee) that may be considered as a unit taken for analysis purposes to have uniform representative properties. A levee reach will be the unique entity having properties different than other reaches of the levee system and is used to determine the probability and uncertainty assessment of the levee system. No maximum length is associated with a reach.

Residual risk – The flood risk (probability of capacity exceedance or failure and the associated consequences) that remains after the levee system is implemented.

Return period – Alternate term 'recurrence interval.' The average time interval, usually expressed in years, between occurrences of an event of a certain magnitude. Normally computed as the reciprocal of the annual chance exceedance.

Risk – Measure of the probability and severity of undesirable consequences.

Risk analysis – Risk analysis is a decision-making framework that comprises three tasks: risk assessment, risk management, and risk communication.

Risk and uncertainty analysis – Risk analysis that explicitly, and analytically, incorporates consideration of uncertainty of parameters and functions used in the analysis to determine the undesirable consequences.

Routine Inspection – A visual inspection, typically conducted once a year, to verify proper levee system operation and maintenance based on the standard USACE inspection checklist. This inspection is also referred to as an annual inspection, initial eligibility inspection, or continuing eligibility inspection.

Separable Consequence Area – See definition for 'separable floodplain.'

Separable floodplain – The portion of a floodplain that is excluded from flooding from the base flood by its associated levee system. This is analogous to 'separable consequence area' - the part of an extensive floodplain that meets the definition of 'separable floodplain'. See above definition for 'levee system' and associated diagram for further clarification.

Special flood hazard area (SFHA) – The area delineated on a National Flood Insurance Program map as being subject to inundation by the FEMA BF.

Stage – Water height measured as the vertical distance in feet (meters) above or below a local or national elevation datum.

Stage-discharge function (alternatively 'Rating Curve') **–** A tabular or graphical relationship that yields the stage for a given discharge at a specific location on a stream or river.

Stage-discharge functions with uncertainty – Relationship of the water surface stage and discharge. Uncertainty is the distribution of the errors of stage estimates about a specific discharge.

Standard deviation – A statistical measure of the spread of the values of a probability distribution about the mean.

Structural (measures) – Those water resources project measures designed to modify the flow of flood waters.

Uncertainty – A measure of the imprecision of knowledge of variables and functions used in the risk analysis. Uncertainty may be represented by a specific probability distribution with associated parameters, or sometimes expressed simply as standard deviation.

Abbreviations

ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
AEP	Annual Exceedance Probability
ATR	Agency Technical Review
CFR	Code of Federal Regulations
CHL	Coastal and Hydraulics Laboratory
CMP	Corrugated metal pipe
CNP	Conditional non-exceedance probability
CRREL	Cold Regions Research and Engineering Laboratory
DFIRM	Digital Flood Insurance Rate Map
DM	Design Memorandum
DPR	Detailed Project Report
DSPMP	Dam Safety Performance Monitoring Program
EC	Engineer Circular
EM	Engineer Manual
EP	Engineer Pamphlet
ER	Engineer Regulation
ERDC	Engineer Research and Development Center
EST	Empirical Simulation Technique
FEMA	Federal Emergency Management Agency
FERC	Federal Energy Regulatory Commission
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
FLAC	Fast Lagrangian Analysis of Continua
FMA	Failure Mode Analysis
FPMS	Flood Plain Management Services (Program)
FRMP	Flood Risk Management Program
HEC	Hydrologic Engineering Center

HEC-FDA	Hydrologic Engineering Center – Flood Damage Reduction Analysis software program
HPS	Hurricane Protective System
ICW	Inspection of Completed Works
IHNC	Inner Harbor Navigation Canal
IPET	Interagency Performance Evaluation Team
JPM	Joint Probability Method
JPM-OS	Joint Probability Method Optimal Sampling
LaCPR	Louisiana Coastal Protection and Restoration
NLSER	NFIP Levee System Evaluation Report
LF	Load factor
LIDAR	Light Detection and Ranging
LSO	Levee Safety Officer
Map Mod	FEMA Flood Map Modernization Program
MsCIP	Mississippi Coastal Improvement Project
NFIP	National Flood Insurance Program
NLD	National Levee Database
O&M	Operation and Maintenance
PAR	Population at Risk
PGA	Peak ground acceleration
PFMA	Potential Failure Mode Analysis
PRB	Project Review Board
QA/QC	Quality assurance/quality control
RIP	Rehabilitation and Inspection Program
RIT	Regional Integration Team
SOP	Standard Operating Procedure
UFC	Unified Facilities Criteria
USACE	United States Army Corps of Engineers
USC	United States Code
USGS	United States Geological Survey