Group B

FOIA/PA NO: 2015-0016

# RECORDS BEING RELEASED IN PART

The following types of information are being withheld:

Ex. 1: ☐ Records properly classified pursuant to Executive Order 13526
Ex. 2: ☐ Records regarding personnel rules and/or human capital administration
Ex. 3: ☐ Information about the design, manufacture, or utilization of nuclear weapons
☒ Information about the protection or security of reactors and nuclear materials
☐ Contractor proposals not incorporated into a final contract with the NRC
☐ Other_____
Ex. 4: ☐ Proprietary information provided by a submitter to the NRC
☐ Other_____
Ex. 5: ☒ Draft documents or other pre-decisional deliberative documents (D.P. Privilege)
☐ Records prepared by counsel in anticipation of litigation (A.W.P. Privilege)
☐ Privileged communications between counsel and a client (A.C. Privilege)
☐ Other_____
Ex. 6: ☒ Agency employee PII, including SSN, contact information, birthdates, etc.
☒ Third party PII, including names, phone numbers, or other personal information
Ex. 7(A): ☐ Copies of ongoing investigation case files, exhibits, notes, ROI's, etc.
☐ Records that reference or are related to a separate ongoing investigation(s)
Ex. 7(C): ☐ Special Agent or other law enforcement PII
☐ PII of third parties referenced in records compiled for law enforcement purposes
Ex. 7(D): ☐ Witnesses' and Allegers' PII in law enforcement records
☐ Confidential Informant or law enforcement information provided by other entity
Ex. 7(E): ☐ Law Enforcement Technique/Procedure used for criminal investigations
☐ Technique or procedure used for security or prevention of criminal activity
Ex. 7(F): ☐ Information that could aid a terrorist or compromise security

Other/Comments: OUT OF SCOPE

## Ferrante, Fernando

**From:** Karl Fleming <fleming@ti-sd.com>
**Sent:** Friday, June 07, 2013 11:46 PM
**To:** Ferrante, Fernando
**Subject:** Re: Papers on Site CDF & LERF
**Attachments:** DPP-SR-B.3-EE-MUS-PSA-R0(May 27 2013).docx; Notes on Multiunit IE frequencyRev3.docx; Multi-Unit PRA Paper.pdf

Fernando:

Thanks for your e-mail and good to see you in Albuquerque last month.

I have attached several files including a draft IAEA report and appendix which are a work in progress and I emphasize their "draft" status. We are meeting at the NRC next week to discuss this and several other related reports we are preparing and hope to finalize later this year. Nilesh Choksie is the NRC rep in this effort – and I have also been sharing this material with Nathan Siu.

Also attached is a paper based on this work to be presented at PSA 2013.

I would much appreciate any comments you may have on these materials as we hope to get good peer reviews of this before we complete the effort.

Best regards:

Karl

Karl N. Fleming
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**From:** Fernando Ferrante <Fernando.Ferrante@nrc.gov>
**Date:** Friday, June 7, 2013 10:22 AM
**To:** Karl Fleming <fleming@ti-sd.com>
**Subject:** Papers on Site CDF & LERF

Karl,

I very much enjoyed talking to you during the INEST Flooding Workshop. Hopefully, we will meet again in another venue.

Do you have papers written on the concepts of site-wide CDF/LERF and Conditional Probabilistic Multi-Accident metrics for multi-unit risk? I am interested in the topic and I would greatly appreciate it if you have any kind of literature available on your recent work in this area that you can point me to.



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Thank you,

Fernando Ferrante, Ph.D.
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## Comparison and Assessment of Aircraft Impact Methodologies

### Quasi-Probabilistic: NRC Standard Review Plan (NUREG-0800₁) Section 3.5.1.6

The goals of this section is to ensure that the probability of radiological hazards greater than 10 CFR Part 100 guidelines due to aircraft accidents is less than an order of magnitude of $10^{-7}$ per year.

NUREG-0800 begins by providing a proximity screening step. If a hazard source (airports, military training route, or federal airway) satisfies the screening calculation (based on either distance or level of activity), then the hazard can be screened from further consideration. The three screening calculations are as follows:

A. The plant-to-airport distance D is between 5 and 10 statute miles, and the projected annual number of operations is less than 500 $D^2$, or the plant-to-airport distance D is greater than 10 statute miles, and the projected annual number of operations is less than 1000 $D^2$

B. The plant is at least 5 statute miles from the nearest edge of military training routes, including low-level training routes, except for those associated with usage greater than 1000 flights per year, or where activities (such as practice bombing) may create an unusual stress situation

C. The plant is at least 2 statute miles beyond the nearest edge of a Federal airway, holding pattern, or approach pattern

Sources of hazards falling outside the above guidelines may be screened from further consideration.

If the criteria are not me then a more detailed review (generally more quantitative in nature) must be performed. Hazards that cannot meet the $10^{-7}$ criterion after more detailed review then fall under the requirements of General Design Criteria (GDC) 3 and 4.

Review procedures presented in Section 3.5.1.6 of NUREG-0800 include some techniques for evaluating the hazard probabilities presented by aircraft, given that the screening criteria above are not met. For evaluating the probability of civilian and military aircraft crashes into plant sites, the following general equation is presented:

$$P_A = \sum_{i=1}^{L} \sum_{j=1}^{M} C_j N_{ij} A_j \qquad (1)$$

$P_A$ probability per year of an aircraft crashing into the site
L number of flight trajectories affecting the site
M number of different types of aircraft using the airport
$C_j$ probability per square mile of a crash per aircraft movement, for the $j^{th}$ aircraft
$N_{ij}$ number (per year) of movements by the $j^{th}$ aircraft along the $i^{th}$ flight path
$A_j$ effective plant area (in square miles) for the $j^{th}$ aircraft

Suggested values for $C_j$ are provided:

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**Table 1.** Probability[1] of a fatal crash per square mile per aircraft movement

| Distance from end of runway (miles) | US Air Carrier[2] | General Aviation | USN/USMC[2] | USAF[2] |
|---|---|---|---|---|
| 0 - 1 | 16.7 | 84 | 8.3 | 5.7 |
| 1 - 2 | 4.0 | 15 | 1.1 | 2.3 |
| 2 - 3 | 0.96 | 6.2 | 0.33 | 1.1 |
| 3 - 4 | 0.68 | 3.8 | 0.31 | 0.42 |
| 4 - 5 | 0.27 | 1.2 | 0.20 | 0.40 |
| 5 - 6 | 0 | NA | NA | NA |
| 6 - 7 | 0 | NA | NA | NA |
| 7 - 8 | 0 | NA | NA | NA |
| 8 - 9 | 0.14 | NA | NA | NA |
| 9 - 10 | 0.12 | NA | NA | NA |

1. All probabilities given per 10 million movements (i.e., each should be multiplied by $10^{-8}$ to reflect a per-movement probability)
2. D.G. Eisenhut, "Reactor Siting in the Vicinity of Air Fields," American Nuclear Society, June 1973
3. D.G. Eisenhut, "Testimony on Zion/Wankegan Airport Interaction" (Docket No. 50-295)
4. NA indicates that data were not available for the given distance

The "effective area" term should include several relevant factors dependent upon the type of aircraft under consideration. First, the shadow area of the plant elevation upon the horizontal plane for those structures housing SSCs relevant to the risk assessment should be considered. That area will be dependent upon the assumed crash angle for the type of aircraft. Also, the skid area for each aircraft type is relevant.

For federal airways or aviation corridors, the following formula is provided:

$$P_{FA} = C \times N \times \frac{A}{w} \tag{2}$$

$P_{FA}$ probability per year of an aircraft crashing into the plant
C inflight crash rate per mile for aircraft using airway
N number of flights per year along airway
A effective area of plant in square miles
w width of airway in miles (plus twice the distance from the airway edge to the site when site is outside the airway)

Note that the 1/w term is sometimes defined as f, the lateral crash density ($mi^{-1}$). In that case, the equation is presented as $P_{FA} = C \times N \times A \times f$.

This formula can also be used for any holding patterns in the vicinity, with due consideration of traffic volume and orientation to the plant.

## Example use of Section 3.5.1.6

The following is a hypothetical presentation of an approach that might be taken by a licensee in an IPEEE. The first step would be performing a screening evaluation of nearby aircraft facilities.

**Table 2** Example application of Section 3.5.1.6 screening criteria

| Airport | Closest distance (mi) and orientation to site | Estimated number of operations per year | Allowable number of operations per Regulatory Guide 1.70 |
|---|---|---|---|
| Harris Municipal Airport | 11.2 W | 100,000 | 125,440 |
| Barton Airport | 5.3 NW | < 100 | 14,045 |
| Lewis AFB | 33 SSE | 300,000 | 1,089,000 |
| On site helipad | 0 | < 50 | N/A |

In this example, four nearby airports are evaluated. The third column presents estimates of the number of operations in a given year. The fourth column presents the calculation of the maximum operations allowable under the screening criterion given above. For airports between 5 and 10 miles from the plant site, the allowable number of operations is $500 \times D^2$ (where D is the distance). For sites more than 10 miles away, the number of allowable operations is $1000 \times D^2$. In this example, the first three sites are screened successfully.

The helipad would require further analysis, possibly utilizing Equation (1).

Airways near plants are evaluated with Equation (2). Table 3 shows a hypothetical example.

**Table 3** Example calculation of impact probability using Equation (2)

| Airway | Distance (mi) | Type of operation | Number of flights per year | Crash rate (per mi) | Effective impact area ($mi^2$) | Lateral crash density ($mi^{-1}$) | Impact probability |
|---|---|---|---|---|---|---|---|
| V-22 | 4.5 | Air carrier | 29,350 | 1.18E-09 | 0.05 | 6.0E-04 | 1.0E-09 |
| | | Military | 6,050 | 2.36E-08 | 0.04 | 5.6E-03 | 3.2E-08 |
| | | General | 104,600 | 1.41E-07 | 0.01 | 1.2E-04 | 1.8E-08 |
| V-513 | 3.2 | General | 200 | 1.41E-07 | 0.01 | 1.7E-03 | 4.8E-10 |
| IR298 | 12.6 | Military | 180 | 2.36E-08 | 0.04 | 1.7E-06 | < 1E-10 |

Using these results, and with some conservative assumptions, the licensee would likely conclude that the probability of exceeding radiological consequences greater than 10 CFR Part 100 guidelines was less than $10^{-7}$ per year.

## Data Sources

### Local Information

Certain data required for aircraft hazard calculations are most reliably obtained on a site-specific basis. Of course, airport distances and the location of nearby air corridors fall into this category. Data on air traffic volume should come directly from the airports identified near a given plant.

**FAA Accident/Incident Data System**
http://www.asias.faa.gov/portal/page/portal/asias_pages/asias_dbs/aids_db

The FAA Accident/Incident Data System (AIDS) database contains incident data records for all categories of civil aviation. Incidents are events that do not meet the aircraft damage or personal injury thresholds contained in the National Transportation Safety Board (NTSB) definition of an accident. For example, the database contains reports of collisions between aircraft and birds while on approach to or departure from an airport. While such a collision may not have resulted in sufficient aircraft damage to reach the damage threshold of an NTSB accident, the fact that the collision occurred is valuable safety information that may be used in the establishment of aircraft design standards or in programs to deter birds from nesting in areas adjacent to airports.

**FAA Air Traffic Activity System (ATADS)**
http://aspm.faa.gov/opsnet/sys/Airport.asp

This related site provides an easy way to look up the number of operations (take-offs and landings) at any FAA-controlled airport.

**NTSB Aviation Database**
http://www.ntsb.gov/aviationquery/index.aspx

The Bureau of Transportation Statistics (BTS) database contains traffic and capacity statistics on individual Air Carrier operations. BTS is an administration under the Department of Transportation (DOT), at a similar organizational level as the FAA. The NTSB aviation accident database contains information from 1962 and later about civil aviation accidents and selected incidents within the United States, its territories and possessions, and in international waters.

**FAA Accident & Incident Query (AviationDB)**
http://www.aviationdb.com/Aviation/AidQuery.shtm

Contains approximately 200,000 publicly available Accident and Incident reports filed with the FAA from 1973 to the present, and Aviation Accidents since 1962 as reported by the NTSB

## IPEEE Methodologies

A selected review of IPEEE approaches is presented in Table 4. None of the reviewed IPEEE submittals deviated appreciably from the methodology outlined in Section 3.5.1.6.

**Table 4. Selected IPEEE Aircraft Hazard Analyses**

(b)(3):42 U.S.C. 2161-2165

(b)(3):42 U.S.C. 2161-2165

(b)(3):42 U.S.C. 2161-2165

(b)(3):42 U.S.C. 2161-2165

(b)(3):42 U.S.C. 2161-2165

(b)(3):42 U.S.C. 2161-2165

## Additional Probabilistic Approaches to Aircraft Impact Assessment

Several other probabilistic models (References 2, 3, 4) employ the same basic concept as presented in Equation (1)5. For example, Solomon2 proposes a model similar to Equation (2) for analyzing airways. It separates the term C into two factors:

λ crash rate (per mile)
p(x) crash density function (per mile)

The model incorporates a negative exponential function for p(x):

$$p(x) = {}^{1}/_{2}\ \gamma e^{-\gamma|x|} \tag{3}$$

$\gamma$ crash density constant (per mile)
x orthonormal distance from the intended path to the site

This formulation extends the methodology in NUREG-0800 by assuming that the crash density is symmetrical and decays away from the intended flight path.

The following sections describe some other more complex models that have been presented in the literature.

## Method 1

Method 1 briefly examines the methodology used in July 2009 by Ove Arup & Partners Hong Kong Ltd in Environmental Impact Assessment (EIA) Report 24037-REP-125-01 to examine the aircraft impact frequency for the Airport Fuel Tank Farm from the Hong Kong International Airport. D. W. Phillips6 suggests the following expression for the distribution of aircraft crashes from flight in the vicinity of airports:

$$f(R,\theta) = 0.23e^{-\frac{R}{5}}e^{-\frac{\theta}{5}} \tag{4}$$

R = the radial distance in kilometers from the runway end
θ = the angle in degrees between the vector R and the runway centerline

Both R and θ are measured from the threshold at the departure end of the runway for aircraft taking off, and from the threshold at the arrival end of the runway for landing aircraft.

The aircraft crash frequency at the site of interest can then be estimated using the following equation:

$$F = (Crash\,rate) \times N \times f(R,\theta) \times (Proportion\ of\ flights\ in\ specified\ direction) \times (Proportion\ of\ flights\ using\ specified\ runway) \times (Target\ area) \quad (5)$$

F = aircraft crash frequency for specified runway
N = the number of aircraft movements per year at the airport

The frequency could be summed over all applicable runways and all applicable airports.

Data would be needed for aircraft movements and types of aircrafts moved and to determine the proportions.

## Method 2

Method 2 is by V. A. Kostikov7.

Determining the probability of damage to objects on the ground in an aviation accident is done by estimating the probability of striking an area of a given size from the characteristics of the aircraft, the characteristics of air motion in the region of the object, flight safety, and the ballistic characteristics of the falling aircraft or pieces of the aircraft.

Knowing that the probability that an aircraft or a fragment of an aircraft strikes an area of prescribed size within a prescribed time interval due to an aviation accident is determined according to the formula $P = 1 - exp(-\mu)$. The parameter μ of the Poisson distribution is the average number of events occurring in the time interval (0, τ) and is determined from the formula $\mu = \int_0^t \lambda(t)\,dt$, where λ is the intensity of events. The total intensity λ for non-coincident events, which aviation accidents are, is determined from the formula $\lambda = \sum_{i=1}^{N} \lambda_i P_i$, where $\lambda_i$ is the intensity of an event of type *i*; $P_i$ is the partial probability of damage to an object as a result of an aviation accident of the type *i*; and, N is the number of event types.

The partial probability for damage to an object resulting from an aviation accident of type i is determined according to the theorem of multiplication of probabilities $P_i = P_i^{a.e} P_i^d$, where $P_i^{a.e}$ is the probability of an aviation accident of the type *i* with an aircraft located near the object; $P_i^d$ is the conditional probability of damage (falling on an object of prescribed size) in an aviation accident of type *i* with aircraft near the object.

To determine the probability for an aviation accident of type *i* to occur in the accessibility zone of the object, it is necessary to have information about the intensity of flights in the zone adjoining the object. The accessibility zone is the flight zone from which an aircraft (or fragments of an aircraft) can strike an area of given size in the case of an aviation accident. It is also necessary to know estimated probabilities for the occurrence of events with the types of aircraft considered flying near the object.

The probability of an aviation accident with a specific type of aircraft near an object is determined by using the Poisson distribution $P_i^{a.e} = 1 - exp(-N_j P_i^b)$, where $N_j$ is the intensity of flights of type $j$ of an aircraft in the accessibility zone of the object over a definite period of time, h; $P_i^b$ is the probability of a type $i$ aviation accident within one hour of flight.

Presented below are the final formulas for estimating the conditional probability of damage to an object in the case when the aviation accident occurred within range of the object.

1. The probability of damage to an object on the ground in the case when the aircraft breaks up in flight is

$$P_1^d = \frac{S_{ob}}{S_{fall}^1} \frac{\sqrt{4S_{ob}/\pi}}{\pi(L_2 - L_1)} \tag{6}$$

   where $S_{ob}$ is the area of the object; $S_{fall}^1$ is the area over which the fragments of the aircraft are spread; $L_1$ and $L_2$ are the average distances over which the fragments (heavy and light) are carried, determined on the basis of statistical analysis of data on past catastrophes and ballistic calculation of fragment trajectories for different types of air masses.

2. The probability of damage to an object struck by an out-of-control aircraft is

$$P_2^d = \frac{S_{ob}}{S_{fall}^2} \frac{\sqrt{4S_{ob}/\pi}}{2\pi R_e} \tag{7}$$

   where $S_{fall}^2$ is the area of the possible fall zone; $R_e$ is the radius of the accessible range, determined from ballistic properties of the aircraft, the flight altitude, and possible deviations from the flight path.

3. Probability of damage to the object when an aircraft is disoriented under due to limited visibility conditions. A collision of an aircraft with the object in the case of spatial disorientation of the aircraft under conditions of limited visibility can occur when the aircraft becomes disoriented in the longitudinal direction (drops to too low an altitude) or lateral direction (wanders away from the flight path in the direction of the object). The damage probability to the object is determined in this case for independent events according to the theorem of multiplication of probabilities as the product $P_m P_p P_b$, where $P_m$ is the probability of limited visibility near the object, as determined from statistical analysis of meteorological data; $P_p$ is the conditional probability that the location struck by the aircraft coincides with the object in the longitudinal direction: $P_p = 2r/L_0$, where r is the size of the object; $L_0$ is the cruising distance under conditions of limited visibility near the object; $P_b$ is the conditional probability that the location struck by the aircraft coincides with the object in the lateral direction

$$P_b = \frac{1}{2}\exp(-D/\sigma)\,[1 - exp(2r/\sigma)] \tag{8}$$

where $r$ is the size of the object; $\sigma$ is the rms deviation of the aircraft from the flight path, chosen from the specific flight conditions along the path and aircraft characteristics; and, $D$ is the distance from the object to the flight path in the horizontal direction.

## Method 3

Method 3 is by H.-P. Berg8.

The plant-specific determination of the frequency for the occurrence of an aircraft crash is performed on the basis of flight accident statistics valid for the respective location, taking into account the types of aircrafts and the weight classes which can be set.

The following input information is needed:

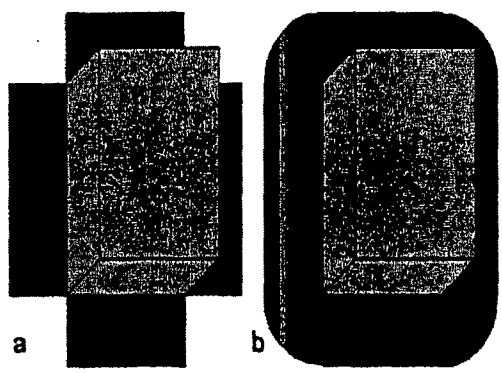
- the air traffic lanes in the near field of the plant,
- data concerning civil and military small and middle airports (in the range of about 50 km) and large airports (in the range up to 150 km) such as distance and adjustment of the starting and take-off runways.

The crash frequencies are determined separately in three different traffic categories:

- The landing and take-off phase,
- the air lane traffic and waiting loop traffic,
- the free air traffic

The aircrafts can be grouped into different weight classes. Furthermore, the weight classes can be correlated to accidents.

The annual frequency of impact for each weight class and flight phase is calculated based on global crash rates valid for Germany. Conservatively, crash angles are assumed 30° to the horizon. Silhouette areas result from projection of the crash angle over up-stations of the building onto the power plant surface (over the four directions). However, the definition of the silhouette area is not unique and allows




different calculations as shown below.

$$F_{Za} = 6\sqrt{3} + 2$$
$$F_{Zb} = 6\sqrt{3} + 2 + 3\pi \tag{9}$$

Further hits from the crash can result from wracked aircraft parts, even if the aircraft crashes outside the defined silhouette. An area of 1000 m outside the silhouette areas is assumed as a possible hazard range. The probability of a hit by a wracked part is estimated as 20%. More than one hit is not assumed.

Wracked parts are generated mainly by the heavier aircraft (from 2 mega grams to greater than 20 mega grams). The wracked part is treated like an aircraft weighing less than 2 mega grams.

Up to 10 km from the end of the runway, the crash rate decreases exponentially with distance R. Based on the weight and different angular segment (or sector) of flight directions, crash rate decrease has been calculated based on data of worldwide crash vectors.

The landing range before touchdown and the take-off range after the start are divided into three different sectors.

Sector 1: ±15º to the landing and take-off axis (30° sector angle)
Sector 2: outside of Sector 1 to ±45° to the landing and take-off axis (90° sector angle)
Sector 3: outside of Sector 2 to ±90° to the landing and take-off axis (180° sector angle)

The number of crashes $h_{i,j}$ within definite angular sectors are summed up for rings of ΔR = 1000 m and approximately expressed by the following formula

$$h_{i,j} = \frac{a_{i,j}}{c_i}\, exp(-b_{i,j}R) \tag{10}$$

i = Weight class
j = Annular segment with corresponding $C_j$ (correlation between sectors and aircraft weight classes) as portion of total sector.
$a_{i,j}$, $b_{i,j}$ = Constants to approximate the observed number of crashes in weight class i and sector j
$c_i$ = number of landing and take-offs in weight class i and within the observation time.

Based on the number of yearly flying operations (take-offs and landings) of the airport to be considered, the number of yearly crashes $H_{i,j}$ can be calculated for an impact area of the plant in a given annulus:

$$H_{i,j} = h_{i,j}\, \frac{d_i}{d_{global,i}\,\Delta t}\, \frac{F_{NPP}}{F_{\alpha,i}} \tag{11}$$

$h_{i,j}$ = Number of crashes within definite angular segment j at distance R (ΔR = 1000 m) and weight class i (Δt = 19 years)
$d_i$ = Number of flying operations per year at the airport considered
$d_{global,i}$ = Number of global flying operations per year
$F_{NPP}$, $F_{\alpha,i}$ = Area of the Nuclear Power Plant, Area of the annular segment
Δt = Time span analyzed (19 years)
$H_{i,j}$ = Number of theoretical yearly crashes within the Nuclear Power Plant area

Outside the distance of 10 km from the airport each flight direction becomes equally probable within the free air traffic. The number of crashes $H_i$ will be calculated by multiplying the global crash rate with the local flight density (the number of take-offs and landings of the considered airport):

$$H_{i,j} = \frac{a}{2\pi R_i} \frac{b}{v_j} d_{i,j} h_j \tag{12}$$

$H_{i,j}$ = Number of crashes per year within the Nuclear Power Plant (NPP) area of airplanes of airport i and in weight class j

a, b = Dimension of the NPP area

$R_j$ = Distance of NPP from airport i

$v_j$ = Average flying speed in weight class j

$h_j$ = Number of crashes of airplanes in the free air traffic (weight class j)

$d_{i,j}$ = Number of flying operations at the airport i considered (take-offs and landings, weight class j)

## References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6. Phillips, D.W., "Criteria for the rapid assessment of the aircraft crash rate onto major hazard installations according to their location," United Kingdom Atomic Energy Report SRD/HSE R 435, 1987
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U.S.NRC

United States Nuclear Regulatory Commission

*Protecting People and the Environment*

NUREG/CR-XXXX
Contractor Report Number

# Technical Basis for Flood Protection at Nuclear Power Plants

B3

Office of Nuclear Regulatory Research

U.S.NRC
United States Nuclear Regulatory Commission
*Protecting People and the Environment*

NUREG/CR-XXXX
Contractor Report Number

# Technical Basis for Flood Protection at Nuclear Power Plants

Manuscript Completed: May 2013
Date Published: To be Published

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Office of Nuclear Regulatory Research

## ABSTRACT

Current regulatory guidance on flood protection at nuclear power plants is contained in the Regulatory Guide 1.102, "Flood Protection for Nuclear Power Plants" (Nuclear Regulatory Commission, 1976). Regulatory Guide 1.102 requires that structures, systems, and components important to safety be designed to withstand the effects of natural phenomena such as floods, tsunami, and seiches without loss of capability to perform their safety functions. These requirements remain the same in 2013 as they did in 1976, however, the current technology, understanding and practice has changed. This report describes current (2013) flood protection feature types and applications for protecting safety-related structures, systems, and components. This document compiles and ties together information from multiple U.S. Army Corps of Engineers manuals in order to provide a resource of scientific and technical background concerning various flood protection structures and their associated risk and reliability. Several chapters and sections in this report summarize foundational U.S. Army Corps of Engineers documents. These include the "Draft Best Practices in Dam and Levee Safety Risk Analysis, version 3.0" (Bureau of Reclamation and U.S. Army Corps of Engineers, 2012), "U.S. Army Corps of Engineers Emergency Flood Fight Training Manual (U.S. Army Corps of Engineers, 2010), and several U.S. Army Corps of Engineers circulars, manuals, regulations, pamphlets, and technical letters (variously dated) available online at http://publications.usace.army.mil.

The U.S. Army Corps of Engineers recommends multiple layers of proven exterior structural barriers and interior pumping stations for reliable flood protection. Currently, adequate data and analyses do not exist in order for the U.S. Army Corps of Engineers to recommend the use of incorporated or temporary barriers if other proven exterior structural approaches are possible. Flood protection at nuclear power facilities requires structures that can reliably keep floodwaters from ever coming in contact with critical infrastructure. The use of incorporated barriers for flood protection at nuclear power facilities is inherently unreliable and inappropriate. Incorporated barriers may be able to supplement a complete flood protection strategy, but the reliability of the protection they provide is insufficient. Temporary barriers may be able to supplement a complete flood protection strategy, but are not a substitute for adequate exterior barriers, interior drainage systems, and pumping stations.

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# 1. INTRODUCTION

Current regulatory guidance on flood protection at nuclear power plants is contained in the Regulatory Guide 1.102, "Flood Protection for Nuclear Power Plants" (Nuclear Regulatory Commission, 1976). The US Army Corps of Engineers, Engineer Research and Development Center (ERDC), in cooperation with the Nuclear Regulatory Commission, Office of Nuclear Regulatory Research (NRC), developed this technical basis document as a foundation of reference information for the NRC, and to be used by NRC staff to update the existing Regulatory Guide 1.102 based on current understanding and practice related to flood protection at nuclear power plants.

Regulatory Guide 1.102 requires that structures, systems, and components important to safety be designed to withstand the effects of natural phenomena such as floods, tsunamis, and seiches without loss of capability to perform their safety functions. These requirements remain the same in 2013 as they did in 1976, however, the current understanding and practice has changed. This report describes a technical basis for the most commonly used flood protection structures and applications for protecting safety-related structures, systems, and components.

Flood protection methods for nuclear power plants fall into one of the following five categories: dry sites, exterior (primary) barriers, incorporated (secondary) barriers, temporary barriers, and interior drainage/pumping systems to accommodate local intense precipitation. Chapters 2–6 discuss the five categories of flood protection methods. Dry sites are located above the Design Basis Flooding Level (DBFL). The DBFL is the maximum water elevation attained by the controlling flood, including coincident wind-generated wave effects. At a dry site, because a site is above the DBFL, all safety-related structures, systems, and components are not affected by external flooding, but are subject to flooding from local intense precipitation. Exterior barriers are natural or engineered structures exterior to the immediate site. Examples of exterior barriers include earthen embankments, sea walls, floodwalls, revetments, and breakwaters. When properly designed and maintained, exterior barriers can produce a site with the flood risk approaching that of a dry site. Incorporated barriers are engineered structures located at the nuclear power plant site/environment interface. Examples of incorporated structures include floodgates, sealed doors, and pumping stations.

Depending on the location of a barrier, some can be categorized as either exterior or incorporated. In these cases, the barriers are included in the exterior (primary) barrier chapter. The distinction between primary (external) and secondary (incorporated) barriers is important because primary flood protection failures can be mitigated with a separate method prior to the floodwaters contacting the nuclear power plant site/environment interface. For example, at a wet site where pumping is required to control interior drainage during an external flood event, sandbag levees could be used to protect critical infrastructure within the site during periods when pumps must be taken off-line temporarily for maintenance. Secondary (incorporated) barriers, by definition, do not have this additional layer of external protection. For nuclear power plants, secondary or temporary barriers, in the absence of primary barriers, are not recommended under any circumstances. Flood protection at nuclear power facilities requires structures that can reliably keep floodwaters from ever coming in contact with critical infrastructure. Incorporated and temporary barriers may be able to supplement a complete flood protection strategy, but are not a substitute for adequate exterior barriers and interior pumping stations.

Chapter 5 covers temporary flood-fighting measures, summarizing the results of a program testing sandbags, in addition to three commercial measures. Chapter 6 covers flooding from locally intense precipitation, including interior drainage concerns. Chapter 7 summarizes the

most recent U.S. Army Corps of Engineers (USACE) guidance on flood fighting methods. Chapter 8 covers other issues, including climate change, resiliency for large storm events, and inspection and evaluation. The final chapter, chapter 9, provides a summary and recommendations.

# 2. DRY SITE

A dry site is defined as a location where all structures are built above the Design Basis Flooding Level (the maximum water elevation attained by the controlling flood, including coincident wind-generated wave effects), and therefore safety-related structures, systems, and components are not affected by external flooding. Exterior (primary) barriers are not applicable based on the definition of a dry site. Barriers intended to manage local intense precipitation become the primary barriers at dry sites. Incorporated barriers remain secondary. The qualitative reliability of a dry site is considered very reliable as long as the Design Basis Flooding Level does not increase causing a dry site to no longer be classified as a dry site.

# 3. EXTERIOR (PRIMARY) BARRIERS

This chapter includes an overview, design considerations, and a general reliability discussion for current (2013) flood protection methods. The categories covered in this chapter include: seawalls, bulkheads, revetments, breakwaters, levees and floodwalls. The qualitative reliability of external barriers alone depends on the design and maintenance of the barriers. Properly designed and maintained barriers alone are insufficiently reliable for protecting nuclear power facilities. However, coupled with properly designed and maintained internal drainage systems and redundant pumping stations, a flood protection system including the presence of external barriers can be very reliable.

## 3.1 Coastal Protection

Structures are often needed along shorelines to provide protection from wave action and/or to retain in situ soil or fill. Vertical structures are classified as either seawalls or bulkheads, according to their function, while protective materials laid on slopes are called revetments. For more detailed information on coastal protection, see the following Engineer Manuals from which this report is based: EM 1110-2-1614 "Design of Coastal Revetments, Seawalls, and Bulkheads", Technical Report CERC-93-19 "Engineering Design Guidance for Detached Breakwaters as Shoreline Stabilization Structures" (U.S. Army Corps of Engineers, 1993a), EM 1110-2-2502 "Retaining and Flood Walls", EM 1110-2-2503 "Design of Sheet Pile Cellular Structures Cofferdams and Retaining Structures", EM 1110-2-2504 "Design of Sheet Pile Walls", EM 1110-2-2906, "Design of Pile Foundations", EM 1110-2-1100 "Coastal Engineering Manual", and EM 1110-2-1617 "Coastal Groins and Nearshore Breakwaters".

### 3.1.1 Seawalls

Seawalls are defined as "structures separating land and water areas, primarily designed to prevent erosion and other damage due to wave action. They are frequently built at the edge of the water, but can be built inland to withstand periods of high water. Seawalls are generally characterized by a massive cross section and a seaward face shaped to dissipate wave energy" (EM 1110-2-2502 "Retaining and Flood Walls"). Seawalls may be either gravity- or pile-supported structures. Common construction materials are either concrete or stone. Seawalls can have a variety of face shapes (Figure 3-1).

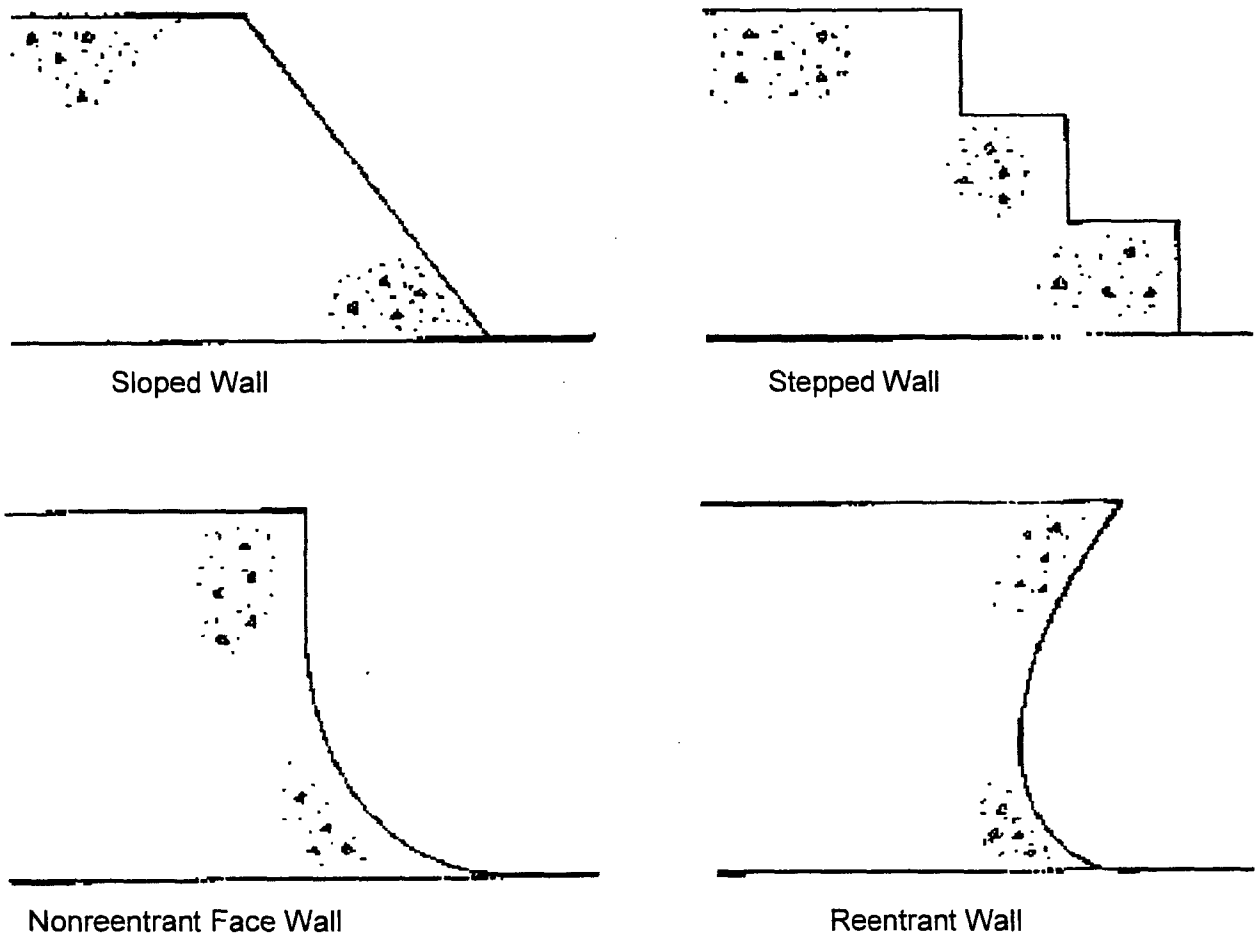


**Figure 3-1. Typical face shapes for concrete gravity seawalls.**

Concrete seawall structures are often pile-supported with sheetpile cutoff walls at the toe to prevent undermining. Additional rock toe protection may also be used to prevent scour. The seaward face may be vertical, sloped, stepped, or recurved. Rubble-mound seawalls are designed like breakwaters using a rock size or concrete armor unit that will be stable against the design wave (Figure 3-2, Figure 3-3). Critical design elements include a secure foundation to minimize settlement and toe protection to prevent undermining. The usual steps needed to develop an adequate seawall design follow (adapted from EM 1110-2-1614 "Design of Coastal Revetments, Seawalls, and Bulkheads").

1. Determine the water level range for the site.
2. Determine the wave heights.
3. Select suitable seawall configurations.
4. Design pile foundations using EM 1110-2-2906 "Design of Pile Foundations".
5. Select a suitable armor unit type and size (rubble seawalls and toe protection).
6. Determine the potential runup to set the crest elevation.
7. Determine the amount of overtopping expected for low structures.
8. Design underdrainage features if they are required.
9. Provide for local surface runoff and overtopping and runoff, and make any required provisions for other drainage facilities such as culverts and ditches.
10. Consider end conditions to avoid failure due to flanking.
11. Design the toe protection.

12. Design the filter and underlayers.
13. Provide for firm compaction of all fill and backfill materials. This requirement should be included on the plans and in the specifications, and due allowance for compaction must be made in the cost estimate.
14. Develop cost estimate for each alternative.

**Figure 3-2. Example of a rubblemound seawall on Wallops Island, VA in 2008.**

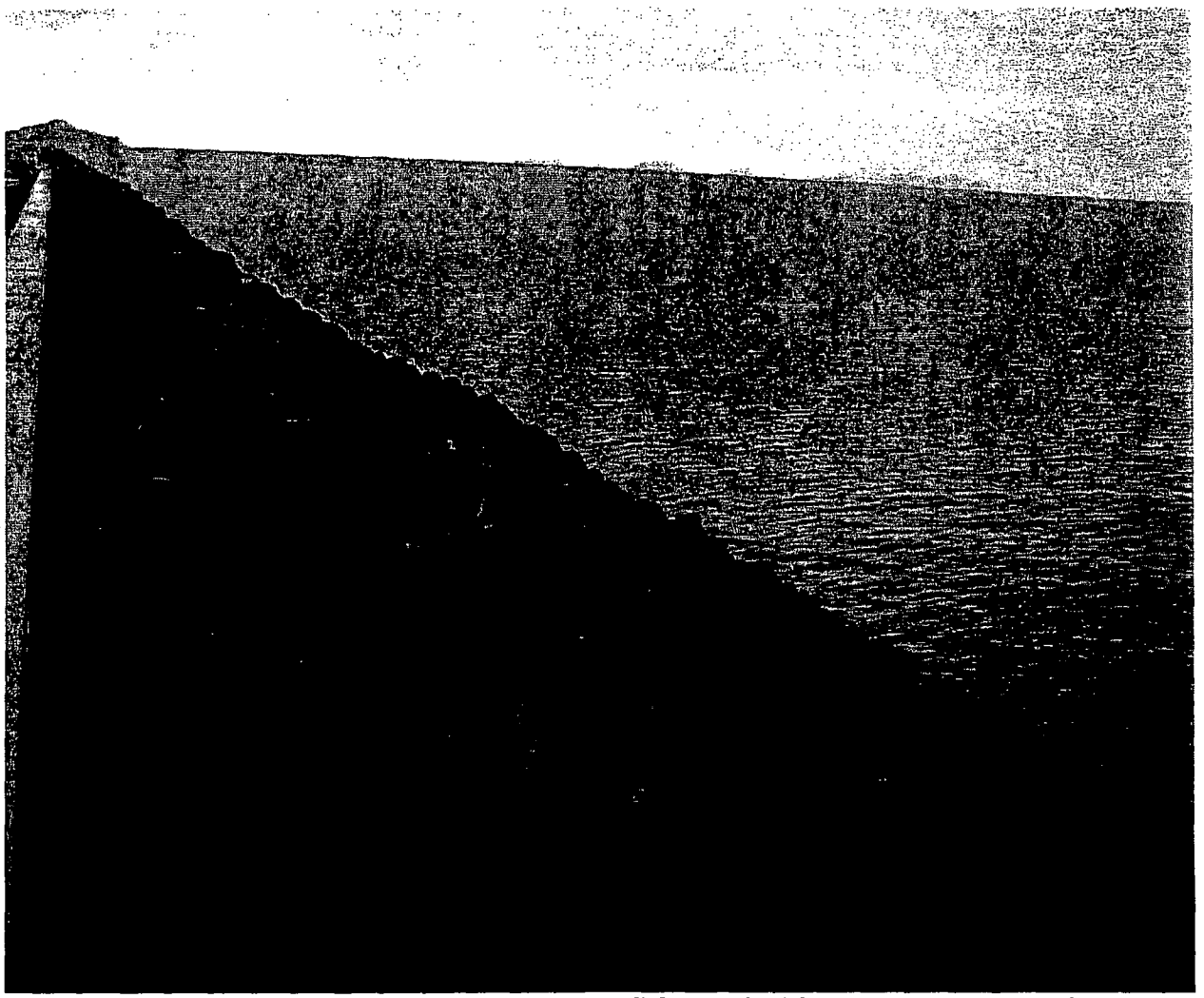

**Figure 3-3. Vertically-faced concrete seawall fronted with concrete armor units at Iwakuni, Japan in 2004.**

Seawalls are built parallel to the shoreline as a reinforcement of a part of the coastal profile. Quite often, seawalls are used to protect promenades, roads, and houses placed seaward of the crest edge of the natural beach profile. In these cases a seawall structure protruding from the natural beach profile must be built. Seawalls range from vertical face structures such as massive gravity concrete walls, tied walls using steel or concrete piling, and stone-filled cribwork to sloping structures with typical surfaces being reinforced concrete slabs, concrete armor units, or stone rubble.

Erosion of upland areas landward of a seawall might be stopped or abated by the structure. However, erosion of the seabed immediately in front of the structure will in most cases be enhanced due to increased wave reflection caused by the seawall. This results in a steeper seabed profile, which subsequently allows larger waves to reach the structure. As a consequence, seawalls are in danger of instability caused by erosion of the seabed at the toe of the structure, and by an increase in wave impact, runup, and overtopping. Because of their potential vulnerability to toe scour, seawalls are often used together with some system of toe protection such as groins and beach nourishment. Exceptions include cases of stable rock foreshores and cases where the potential for future erosion is limited and can be accommodated in the design of the seawall.

Seawalls fail for one or more of the following reasons: 1) design failure occurs when either the structure as a whole, including the foundation, or individual structure components cannot withstand load conditions within the design criteria; 2) load exceedance failure occurs because anticipated design load conditions were exceeded; 3) construction failure arises due to incorrect or bad construction or construction materials; 4) deterioration failure occurs as a result of structure deterioration and inadequate maintenance.

Common failure modes for gravity structures include:

1) Toe scour and undermining: because vertically-faced structures may increase the reflected wave energy, scour at the toe of the seawall is a common problem that can lead to undermining and instability.
2) Foundation failures: foundation failures include both settling and slip-plain failures.
3) Flanking: although a seawall may halt the erosion of the coastline directly behind the seawall, if the seawall is not properly tied into adjacent hard points erosion of the coastline will continue at both ends of the seawall leading to flanking and erosion of the land behind the seawall.
4) Erosion of backfill due to overtopping: unless a splash apron or other protective measure is applied, the lands behind the seawall may be eroded by overtopping. Sufficient drainage for the overtopping must also be considered.
5) Spalling or other deterioration of the structure: deterioration is inevitable and the design must include provision that the deterioration will not affect the functionality of the structure during its design life.

For rubblemound structures, failure modes also include:

1) Slope failure due to toe instability or insufficiently sized armor material: toe instability can lead to a slump-type failure of the armor layer, or insufficiently sized armor material will be removed from the matrix by wave action. In either case, the underlayer will be exposed leading to rapid degradation of the structure.
2) Leaching of the substrate through the armor stone: an improperly designed filter layer or tears in a filter fabric will allow the underlying material to leach through the armor layer through wave action. This will result in voids under the armor layer and eventual collapse of the armor.

Maintenance on concrete gravity structures includes sealing any cracks that may develop and repairing any broken sections. Logs and other debris that could be thrown against the structure by wave action should be removed. The toe protection and splash apron, if present, should be inspected and repaired as needed. Seepage drains should be inspected and repaired as needed. Any signs of flanking or backside erosion should be corrected. On rubblemound structures, any holes in the armor layer should be filled and any evidence of in situ material leaching through the filter layer should be corrected.

### 3.1.2 Bulkheads

The terms *bulkhead* and *seawall* are often used interchangeably. However, a bulkhead is a retaining wall with the primary purpose of holding or preventing the backfill from sliding while providing protection against light-to-moderate wave action (secondary importance). Bulkheads are used to protect eroding bluffs by retaining soil at the toe, thereby increasing stability, or by protecting the toe from erosion and undercutting. They are also used for reclamation projects, where a fill is needed seaward of the existing shoreline and for marinas and other structures where deep water is needed directly at the shoreline. Bulkhead use is limited to those areas

where wave action can be resisted by the bulkhead materials (EM 1110-2-1614 "Design of Coastal Revetments, Seawalls, and Bulkheads").

Bulkheads are typically either cantilevered or anchored sheetpiling or gravity structures such as rock-filled timber cribbing. Cantilevers require adequate embedment for stability and are usually suitable where wall heights are low. Toe scour reduces the effective embedment and can lead to failure. Anchored bulkheads generally are used where greater heights are necessary. Such bulkheads also require adequate embedment for stability but are less susceptible to failure due to toe scour. Gravity structures eliminate the expense of pile driving and can often be used where subsurface conditions hinder pile driving. These structures require strong foundation soils to adequately support their weight, and they normally do not sufficiently penetrate the soil to develop reliable passive resisting forces on the offshore side. Therefore, gravity structures depend primarily on shearing resistance along the base of the structure to support the applied loads. Gravity bulkheads also cannot prevent rotational slides in materials where the failure surface passes beneath the structure. A typical bulkhead section is presented in Figure 3-4. The bulkhead design procedure is similar to that presented for seawalls. In addition, toe protection should be designed using design geotechnical and hydraulic conditions, including wave action and scour potential.

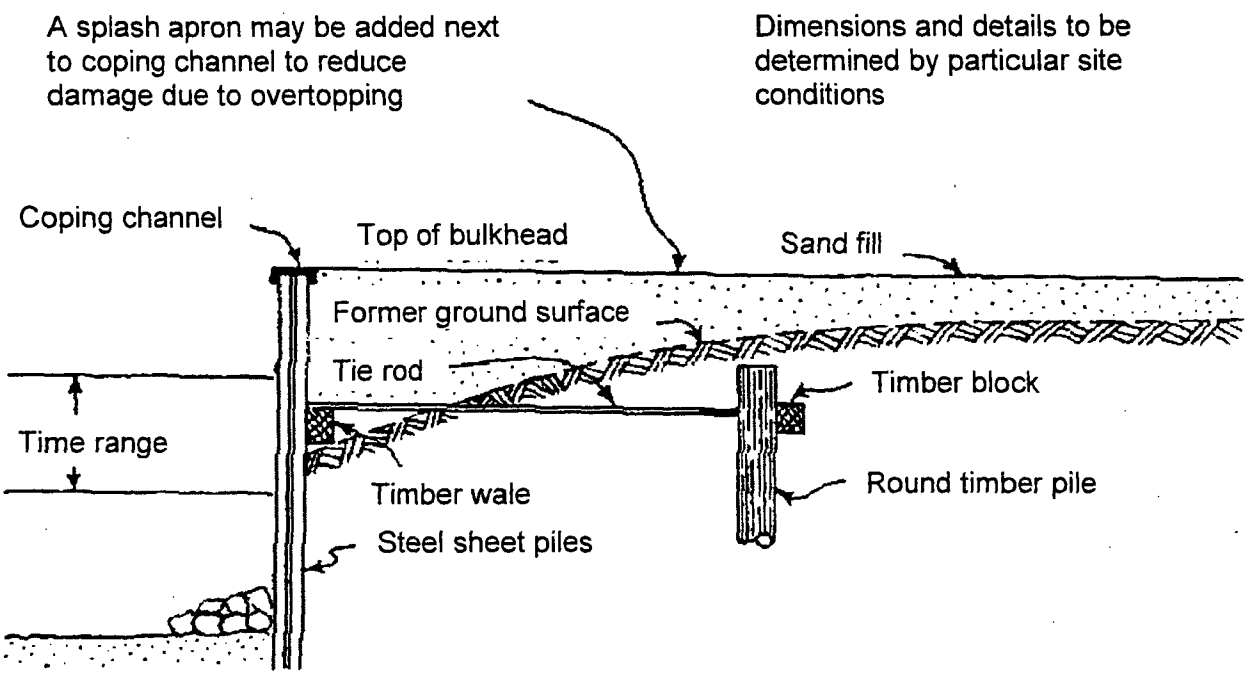


**Figure 3-4. Typical steel bulkhead section (from EM 1110-2-1614 "Design of Coastal Revetments, Seawalls, and Bulkheads").**

As with seawalls, bulkheads fail for one or more of the following reasons: 1) design failure occurs when either the structure as a whole, including the foundation, or individual structure components cannot withstand load conditions within the design criteria; 2) load exceedance failure occurs because anticipated design load conditions were exceeded; 3) construction failure arises due to incorrect or bad construction or construction materials; 4) deterioration failure occurs as a result of structure deterioration and inadequate maintenance. Common failure modes for gravity structures are the same as were listed for seawalls, i.e., toe scour and undermining, foundation failures, flanking, erosion of backfill due to overtopping, and spalling or

other deterioration of the structure. Failure modes of tie-back structures also include failure of the tie rod or anchor system. Maintenance on bulkheads is similar to the maintenance of seawalls covered in Section 3.1.1, plus any maintenance that may be required of tie-back or anchoring systems.

### 3.1.3 Revetments

A revetment is a facing of erosion resistant material, such as stone or concrete that is built to protect a scarp, embankment, or other shoreline feature against erosion. The major components of a revetment are the armor layer, filter, and toe (Figure 3-5). The armor layer provides the basic protection against wave action, while the filter layer supports the armor, provides for the passage of water through the structure, and prevents the underlying soil from being washed through the armor. Toe protection prevents displacement of the seaward edge of the revetment and prevents scour (EM 1110-2-1614 "Design of Coastal Revetments, Seawalls, and Bulkheads").

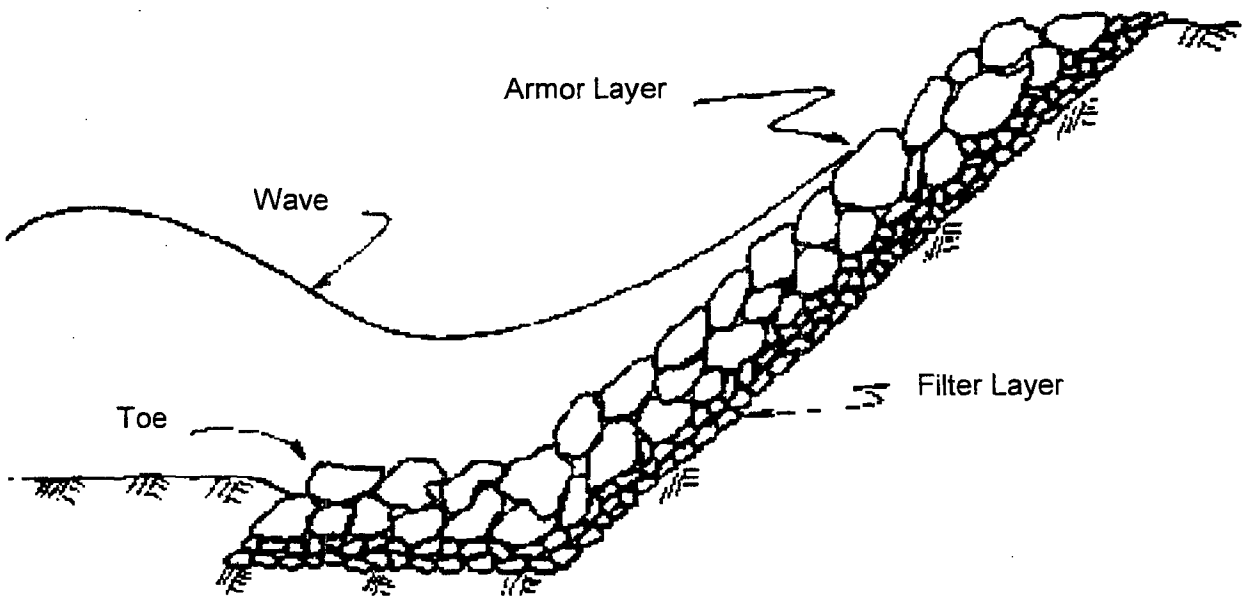


**Figure 3-5. Typical revetment section (from EM 1110-2-1614 "Design of Coastal Revetments, Seawalls, and Bulkheads").**

Coastal revetments are onshore structures with the principal function of protecting the shoreline from erosion. Revetment structures typically consist of a cladding of stone, concrete, or asphalt to armor sloping natural shoreline profiles. In the US Army Corps of Engineers (USACE), the functional distinction is made between seawalls and revetments for the purpose of assigning project benefits; however, in the technical literature there is often no distinction between seawalls and revetments.

For revetments in tidal inlets or rivers, the stream velocities may be the factor to determine the revetment size and type. For additional discussion on revetments used for river and streambank protection, see section 6.3.1 "Streambank Protection". In most cases, the steepest recommended slope is one unit vertical for each two units horizontal. Fill material should be added where needed to achieve a uniform slope, but it should be free of large stones and debris and should be firmly compacted before revetment construction proceeds. Allowance should be made for conditions other than waves such as floating ice, logs, and other debris. Current velocities may also be important in some areas such as within tidal inlets where wave heights

are low. Properly sized filter layers should be provided to prevent the loss of slope material through voids in the revetment stone. If using filter cloth, an intermediate layer of smaller stone below the armor layer may be needed to distribute the load and prevent rupture of the cloth. Economic evaluation of rock revetments should include consideration of trade-offs that result between flatter slopes and smaller stone weights and the increased costs for excavation that usually result for flatter slopes. Planning and design procedure considerations for coastal projects are described in EM 1110-2-1100, Coastal Engineering Manual - Part V, "Planning and Design Process."- Part VI, "Coastal Project Element Design."

Revetment armoring may range from concrete slabs-on-grade (rigid) to riprap and quarrystone (flexible). Rigid armors tend to be more massive but are generally unable to accommodate settlement or adjustments of the underlying materials. Flexible armor is constructed with lighter individual units that can tolerate varying amounts of displacement and shifting.

The usual steps needed to design an adequate revetment are (adapted from EM 1110-2-1614 "Design of Coastal Revetments, Seawalls, and Bulkheads"):

1. Determine the water level range for the site.
2. Determine the wave heights.
3. Select suitable armor alternatives to resist the design wave.
4. Select armor unit size.
5. Determine potential runup to set the crest elevation.
6. Determine amount of overtopping expected for low structures.
7. Design underdrainage features if they are required.
8. Provide for local surface runoff and overtopping runoff, and make any required provisions for other drainage facilities such as culverts and ditches.
9. Consider end conditions to avoid failure due to flanking.
10. Design toe protection.
11. Design filter and underlayers.
12. Provide for firm compaction of all fill and backfill materials. This requirement should be included on the plans and in the specifications. Also, due allowance for compaction must be made in the cost estimate.
13. Develop cost estimate for each alternative.

Revetments are typically constructed as sloping-front flexible rubble-mound structures that are able to adjust to some toe and crest erosion. In the United States, pattern-placed block slopes are commonly found on revetments. The stability of the slope is dependent on an intact toe support. In other words, loss of toe support will likely result in significant armor layer damage, if not complete failure of the armored slope.

Common failure modes for rubblemound structures include slope failure due to toe instability or insufficiently sized armor material, leaching of the substrate through the armor stone due to improperly designed filter layer or tears in filter cloth, toe scour, flanking, and erosion landward of structure due to overtopping. Maintenance of rubble-mound structures includes filling any holes in the armor layer and ensuring that the filter layer is preventing leaching of the in situ material. The toe protection should be inspected and repaired as needed, and any problems with flanking or erosion landward of the structure from overtopping should be corrected.

### 3.1.4 Breakwaters

Breakwaters differ from seawalls, revetments, and bulkheads in that a breakwater will have water on both sides of the structure. There are numerous variations of the breakwater concept (see EM 1110-2-1617 "Coastal Groins and Nearshore Breakwaters). Breakwaters may be shore

attached, detached, submerged or emergent. Shore attached breakwaters are commonly used to form a protective barrier around mooring areas in ports and harbors. The breakwater forms a physical barrier to wave action providing an area of reduced wave action for mooring. Detached breakwaters are constructed at a significant distance offshore and are typically used for shoreline protection by reducing the amount of wave energy reaching the shoreline. Reef breakwaters are a type of detached breakwater designed with a low crest elevation and homogeneous stone size, as opposed to the traditional multilayer cross section. Low-crested breakwaters can be more suitable for shoreline stabilization projects due to increased tolerance of wave transmission and reduced quantities of material necessary for construction. Other types of breakwaters include headland breakwaters or artificial headlands, which are constructed at or very near to the original shoreline. A headland breakwater is designed to promote beach growth out to the structure, forming a tombolo or periodic tombolo, and tends to function as a transmissible groin. Another type of shore-parallel offshore structure is a submerged sill or perched beach. A submerged or semi-submerged sill reduces the rate of offshore sand movement from a stretch of beach by acting as a barrier to shore-normal transport. The effect of submerged sills on waves is relatively small due to their low crest elevation.

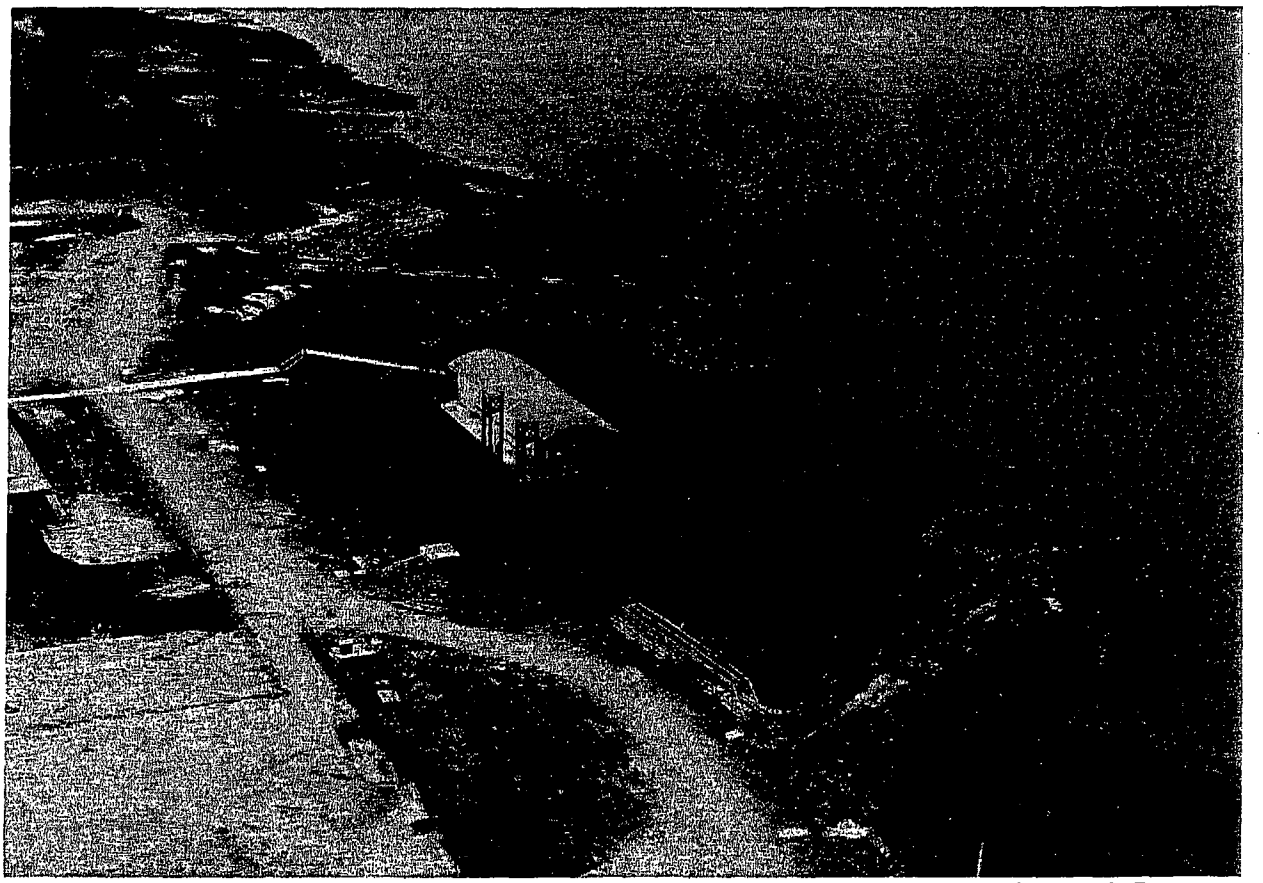

**Figure 3-6. Shore attached breakwaters protecting a small boat dock in Corozal, Panama in 2005.**

Detached breakwaters are generally shore-parallel structures that reduce the amount of wave energy reaching the protected area by dissipating, reflecting, or diffracting incoming waves. The structures dissipate wave energy similar to a natural offshore bar, reef, or nearshore island. The reduction of wave action promotes sediment deposition shoreward of the structure. Littoral material is deposited and sediment retained in the sheltered area behind the breakwater. The

sediment will typically appear as a bulge in the beach planform termed a salient, or a tombolo if the resulting shoreline extends out to the structure (Figure 3-7) (Chasten and others, 1993).

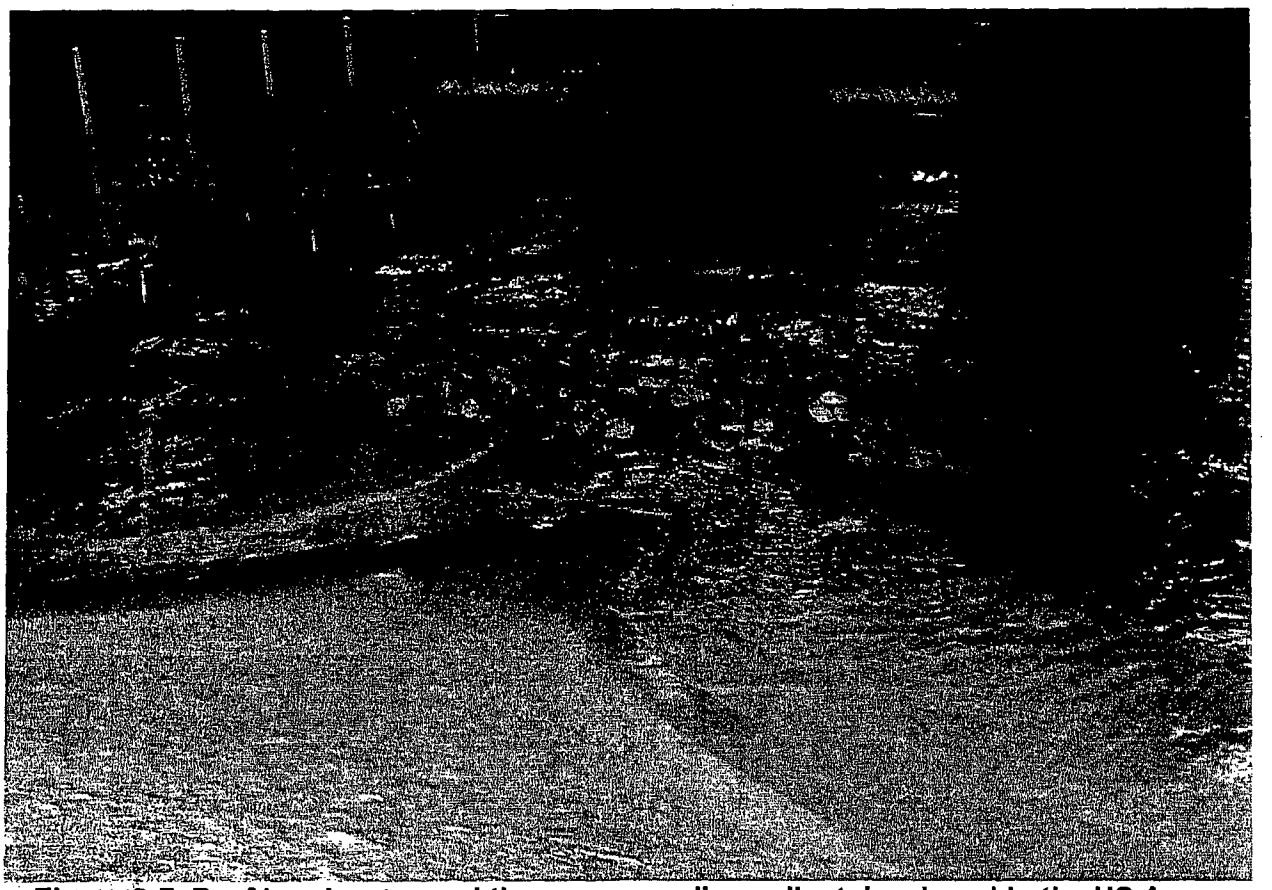

**Figure 3-7. Reef breakwater and the corresponding salient developed in the US Army Corps of Engineers, Large-Scale Laboratory Facility for Sediment Transport Research.**

Reef breakwaters are shore-parallel, submerged structures built with the objective of reducing the wave action on the beach by forcing wave breaking over the reef and dissipation of wave energy through turbulence within the reef. Reef breakwaters are normally rubble-mound structures constructed as a homogeneous pile of stone or concrete armor units. The breakwater can be designed to be stable or it may be allowed to reshape under wave action. Reef breakwaters might be narrow crested like detached breakwaters in shallow water or, in deeper water, wide crested with lower crest elevation. Besides triggering wave breaking and subsequent energy dissipation, reef breakwaters can be used to regulate wave action by refraction and diffraction. Reef breakwaters represent a non-visible hazard to swimmers and boats.

When used for shore protection, breakwaters are built in nearshore waters and usually oriented parallel to the shore. The layout of breakwaters is determined by the size and shape of the area to be protected as well as by the prevailing directions of storm waves, net direction of currents and littoral drift, and requirements for maneuverability of navigation vessels.

The cost of building a structure with sloping sides increases dramatically with increasing water depth. Cost of building a structure with rock or concrete armor units will increase with increasing wave climate due to the larger size of units required. For these reasons, breakwaters in deep

water are frequently constructed of concrete with vertical sides, either with sand-filled concrete caissons or stacked massive concrete blocks. The concrete caissons are often built on a high mound of quarry rock for economical reasons. These breakwaters are called *composite structures*. The upper part of the concrete structure might be constructed with a sloping front to reduce the wave forces. For the same reason the front wall might be perforated with a wave chamber behind to dissipate wave energy. Smaller vertical structures might be constructed of steel sheetpiling backfilled with soil, or built as a rock-filled timber cribwork or wire cages. In milder wave climates sloping reinforced concrete slabs supported by batter piles are also applied.

Rubblemound breakwaters are subject to the same failure modes as revetments with additional failure modes for the lee side due to overtopping or transmission through the structure. Vertically-faced breakwaters have the same failure modes as seawalls with the additional failure modes of overturning or sliding. Rubblemound breakwaters have similar maintenance requirements as rubblemound revetments discussed in Section 3.1.4. The concrete portions of composite breakwaters will have maintenance requirements similar to concrete seawalls discussed in Section 3.1.1.

### 3.1.5 Coastal Design and Reliability

All projects accept some level of failure probability associated with exceedance of design load conditions, but failure probability increases at project sites where little prototype data exist upon which to base the design. These cases may require a conservative factor of safety (for information on probabilistic design see EM 1110-2-1100, Coastal Engineering Manual Part V-1-3, "Risk Analysis and Project Optimization," and EM 1110-2-1100, Coastal Engineering Manual Part VI-6, "Reliability in Design"). Conventional design practice for coastal structures is deterministic in nature and is based on the concept of a design load that should not exceed the resistance (carrying capacity) of the structure. In most cases, the resistance is defined in terms of the load that causes a certain design impact or damage to the structure, and it is not given as an ultimate force or deformation. This is because most of the available design formulae only give the relation between wave characteristics and some structural response, such as runup, overtopping, armor layer damage, etcetera.

Almost all coastal structure design formulae are semi-empirical and based mainly on central fitting to model test results. The scatter in test results is not considered in general because the formulae generally express only the mean values. Consequently, the applied characteristic value of the resistance is then the mean value and not a lower fraction as is usually the case in other civil engineering fields. The only contribution to a safety margin in the design is inherent in the choice of the return period for the design load. The exception is when the design curve is fitted to the conservative side of the data envelope to give a built-in safety margin. It is now more common to choose the return period with due consideration of the encounter probability, i.e., the probability that the design load value is exceeded during the structure lifetime.

In addition to design load probability, a safety factor might be applied as well, in which case the method is classified as a Level I (deterministic/quasi-probabilistic) method. However, this approach does not allow determination of the reliability (or the failure probability) of the design; and consequently, it is not possible to optimize structure design or avoid overdesign of a structure. In order to overcome this problem, more advanced probabilistic methods must be applied where the uncertainties (the stochastic properties) of the involved loading and strength variables are considered.

Methods where the actual distribution functions for the variables are taken into account are denoted as Level III methods. Level II methods generally transform correlated and non-normally

distributed variables into uncorrelated and standard normal distributed variables, and reliability indices are used as measures of the structural reliability. More information about Level I, Level II and Level III methods, as well as planning and design procedure considerations for coastal projects, refer to EM 1110-2-1100, Coastal Engineering Manual - Part V, "Planning and Design Process." and EM 1110-2-1100, Coastal Engineering Manual - Part VI, "Coastal Project Element Design".

## 3.2 Riverine and Other Non-coastal Protection

Structures are often needed along rivers, streams, and other interior waterbodies (lakes, bays, etc.) to provide protection from flooding. Riverine structures are generally earthen embankments or flood walls. For more detailed information on riverine and other non-coastal protection, see the following Engineer Manuals from which this report section is based: EM 1110-2-1913 "Design and Construction of Levees", EM 1110-2-2502 "Retaining and Flood Walls", EM 1110-2-2503 "Design of Sheet Pile Cellular Structures Cofferdams and Retaining Structures", and EM 1110-2-2504 "Design of Sheet Pile Walls".

### 3.2.1 Earthen Embankments

Earthen embankments come in a variety of configurations that vary in design and construction details. A levee is defined as an "embankment whose primary purpose is to furnish flood protection from seasonal high water and which is therefore subject to water loading for periods of only a few days or weeks a year. For some large rivers, the period of flooding may exceed one month. Embankments that are subject to water loading for prolonged periods (longer than normal flood protection requirements) or permanently should be designed in accordance with earth dam criteria rather than the levee criteria given herein" (EM 1110-2-1913 "Design and Construction of Levees"). Illustrative sections of levees with engineered conduit penetrations (e.g. drainage structures) are shown in Figure 3-8. Engineer Manual 1110-2-1913 provides a comprehensive reference concerning the geotechnical design and construction of levees. The hydrologic and hydraulic design can be performed using references such as "Design Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America," (Nuclear Regulatory Commission, 2011) and EM 1110-2-1416 "River Hydraulics".

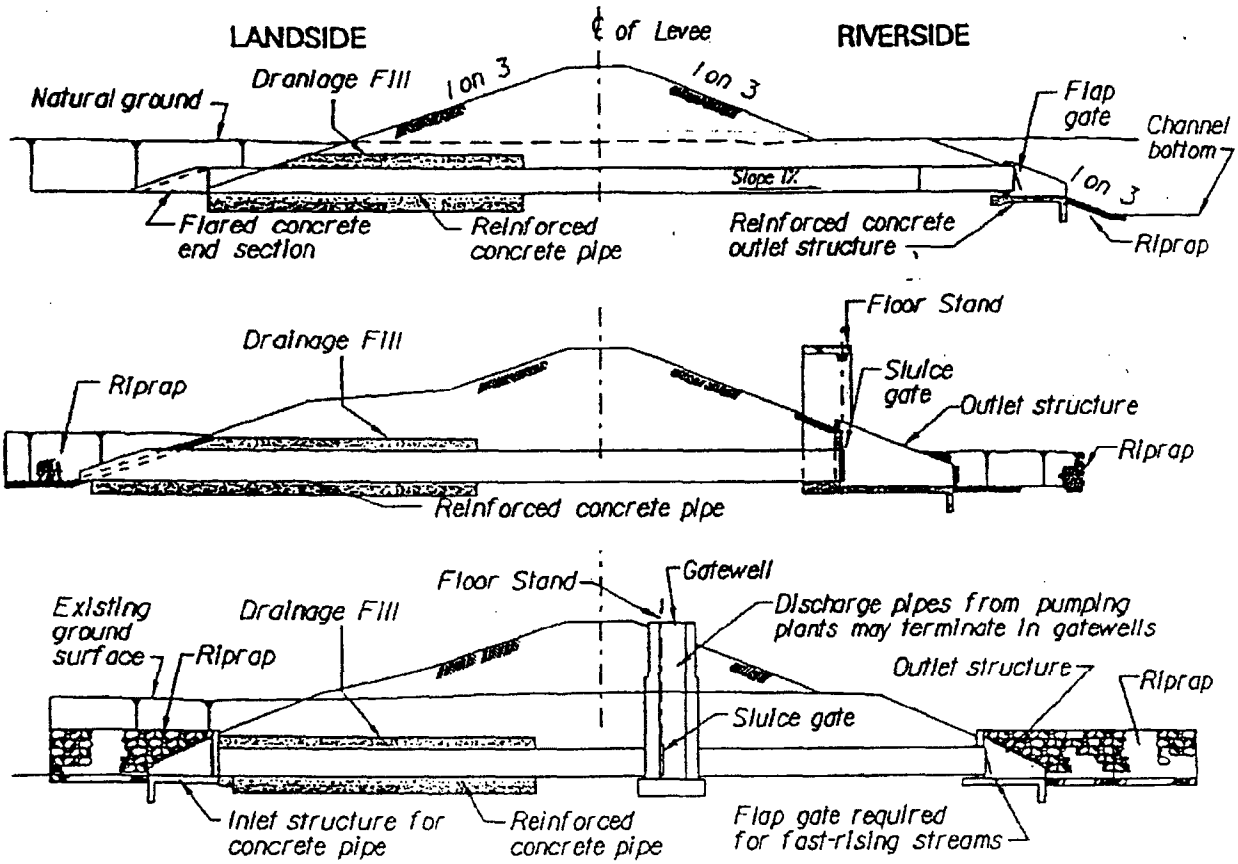


**Figure 3-8. Levee sections with conduit penetrations (from EM 1110-2-1913 "Design and Construction of Levees").**

Numerous local factors must be considered in levee design and no specific step-by step procedure covering details of a particular project can be established. However, general, logical steps based on successful U.S. Army Corps of Engineers past projects (adapted from EM 1110-2-1913 "Design and Construction of Levees") are listed below. These steps cover the geotechnical design, and use the top of levee profile determined by the hydrologic and hydraulic analyses.

1. Conduct geological study based on a thorough review of available data including analysis of aerial photographs. Initiate preliminary subsurface explorations.
2. Analyze preliminary exploration data and from this analysis establish preliminary soil profiles, borrow locations, and embankment sections.
3. Initiate final exploration to provide: additional information on soil profiles, undisturbed strengths of foundation materials, and more detailed information on borrow areas and other required excavations.
4. Using the information obtained in step 3, determine both embankment and foundation soil parameters and refine preliminary sections where needed, noting all possible problem areas, and compute rough quantities of suitable material and refine borrow area locations.
5. Divide the entire levee into reaches of similar foundation conditions, embankment height, and fill material and assign a typical trial section to each reach.

6. Analyze each trial section as needed for underseepage and through seepage, slope stability, settlement, and trafficability of the levee surface.
7. Design special treatment to preclude any problems as determined from step 6. Determine surfacing requirements for the levee based on its expected future use.
8. Based on the results of step 7, establish final sections for each reach.
9. Compute final quantities needed; determine final borrow area locations.
10. Design embankment slope protection.

The principal causes of embankment failure are overtopping and excessive seepage. Embankments should be designed to overtop at locations where the overtopping does not affect critical infrastructure (usually the downstream end). Most embankment failures are caused by excessive seepage, internal erosion or slope instability. Such failures tend to occur rapidly and with little or no warning – leaving little opportunity for evacuation prior to flooding. Failures caused by overtopping are often foreseeable and tend to progress more slowly, and in some cases can be prevented through aggressive flood fighting. Failures from overtopping provide much better opportunity to successfully evacuate the threatened area and to take steps to minimize damage. For more information on earthen embankment seepage principles and analysis, refer to EM 1110-2-1901 "Seepage Analysis and Control for Dams" and ETL 1110-2-569 "Design Guidance for Levee Underseepage".

Seepage is defined as the movement of water through the interstitial soil matrix located anywhere within an embankment, its foundation, or its abutments. It is differentiated from leakage, which is the unintentional flow of water through holes or cracks. As an example, a broken pipe (conduit) will leak, and the resulting flow of water into the surrounding soil will develop an interstitial flow path (i.e., seepage) of which the direction and quantity is directly proportional to the soil's hydraulic conductivity (permeability) and pore water pressure. Seepage discharge may vary in appearance from a wet surface area having slightly denser vegetation growth to one having measurable water flow rates and substantial water volume. In some cases, seepage may be harmless, but in others, it may be extremely serious and immediate remedial action should be taken to prevent a seepage erosion-induced breaching failure. Seepage must be considered in the design of interior drainage/pumping systems. This is discussed in section 6.1 "Interior Drainage Systems".

Water seepage only threatens the structural integrity of an embankment when the seepage begins to transport soil particles within the embankment or its subsurface proximity. When the soil particle movements are not prevented (either by design or by random chance), the subsequent sequence of events may eventually lead to unsatisfactory performance (i.e., breaching failure). The time-to-failure due to such uncontrolled soil particle movements can be rapid or it can be prolonged. Fell et al. (2003) estimated the elapsed time between first observing an internal soil displacement anomaly (usually evidenced by a muddy flow or an increase in seepage) to eventual failure occurred over time spans ranging from less than three hours to years. Historically, most seepage-induces failures happen rapidly (Charles 1997). These facts have great implications for properly conducting seepage detection, collection, measurement, monitoring, and evaluation aspects for any flood protection embankment.

If the embankment was constructed with an internal filter or drain system, any potential seepage water discharge should appear in the embankment downstream toe area or toe drain. Toe drains, chimney drains, and blanket drains are designed to intercept the through-seepage discharge, collect, and convey it for measurement. Properly-designed filters restrain particles from moving

with the seepage flow. If the embankment does not have a filter or drain system, seepage water may appear anywhere on the downstream face. In the absence of a properly designed and constructed filter or drain system, the seepage has the potential to erode embankment or foundation materials through internal erosion or piping. Seepage through embankments must always be monitored, measured, and evaluated to verify performance of the core, filter, and drain systems.

Internal erosion can occur as water flows through the internal fractures, cracks, and voids of the earthen embankment. If the water physically removes soil material from within the embankment or foundation, there is a potential for an erosion-induced embankment failure. Internal erosion is the primary mechanism responsible for seepage erosion-induced damage. Seepage incidents are reported more often than internal erosion failure incidents because for seepage incidents the erosion progression is internally terminated before a breach mechanism can develop (Federal Emergency Management Agency 2000).

When an earthen embankment is encountered by floodwaters, storm surge, or wave action, the protected side (inner slope) may erode, and the progressive soil loss may eventually cause a breaching failure. This overtopping erosion process has been empirically observed during natural disasters such as floods and hurricanes. There is also the risk that the embankment wave-side (outer) slope will erode and cause breaching due to wave action, based on historical observations. Whether resisting outer slope wave attack or inner slope overtopping forces (or combinations thereof), the embankment structure resilience depends in part on the likelihoods of erosion initiation, progression, and subsequent breaching failure.

The most common slope stability failure mechanisms include shear failure, surface sloughing, excessive deformation, and seismically-induced liquefaction. A shear failure involves sliding of a portion of an embankment, or an embankment and its foundation, relative to the adjacent mass. A shear failure is conventionally considered to occur along a discrete surface and is so assumed in stability analyses, although the shear movements may in fact occur across a zone of appreciable thickness. Failure surfaces are frequently approximately circular in shape. Where zoned embankments or thin foundation layers overlying bedrock are involved, or where weak strata exist within a deposit, the failure surface may consist of interconnected arcs and planes. Surface sloughing is considered a maintenance problem, because it usually does not affect the structural capability of the embankment. However, repair of surficial failures can entail considerable cost. If such failures are not repaired, they can become progressively larger, and may then represent a threat to embankment safety.

To avoid excessive deformations, particular attention should be given to the stress-strain response of cohesive embankment and foundation soils during design. When strains larger than 15 percent are required to mobilize peak strengths, deformations in the embankment or foundation may be excessive. If cohesive soils are compacted too dry, and they later become wetter while under load, excessive settlement may occur. Also, compaction of cohesive soils dry of optimum water content may result in brittle stress-strain behavior and cracking of the embankment. Cracks can have adverse effects on stability and seepage. When large strains are required to develop shear strengths, surface movement measurement points and piezometers should be installed to monitor movements and pore water pressures during construction, in case it becomes necessary to modify the cross section or the rate of fill placement. Engineer Technical Letter (ETL) 1110-2-556 "Risk Based Analysis in Geotechnical Engineering for Support of Planning Studies" describes techniques for probabilistic analyses and their application to slope stability studies.

Seismically-induced soil liquefaction, or a significant reduction in soil strength and stiffness as a result of shear-induced increase in pore water pressure, is an earthquake damage concern for earthen embankments. Most instances of liquefaction have been associated with saturated loose sandy or silty soils. Loose gravelly soil deposits also are vulnerable to liquefaction. Cohesive soils with more than 20 percent of particles finer than 0.005 mm, liquid limit of 34 or greater, or with the plasticity index of 14 or greater are generally considered not susceptible to liquefaction. Evaluation and mitigation for seismic performance of earthen embankment systems has generally had low priority in the past, except for earthen embankments with a high likelihood of having coincident high water and earthquake loading, such as many earthen embankments in the California Delta. The current approach for earthen embankments with infrequent high water is for seismic performance evaluation to occur at typical water surface elevations. Flood risk coincident with seismic performance has typically been addressed with emergency response, interim and long-term repairs following the earthquake, and/or seismic remediation prior to the earthquake.

Several other types of slope movements, including rock falls, topples, lateral spreading, flows, and combinations of these, are not controlled by shear strength. Earthen embankments are generally not designed for these types of mass movements, but the possibility of their occurrence should not be ignored. Earthen embankment design guidance is provided in EM 1110-2-1913 "Design and Construction of Levees", ETL 1110-2-569 "Engineering and Design: Design Guidance for Levee Underseepage", ETL 1110-2-570 "Certification of Levee Systems for the National Flood Insurance Program", and the Geotechnical Levee Practice Standard Operating Procedure for US Army Corps of Engineers District offices. Successful design requires consistency in the design process. Appropriate safety factors are inseparable from the procedures used to measure shear strengths, analyze stability, and evaluate seepage.

A deterministic design approach based on the expected water surface elevation for a given flood frequency event is typically used to certify earthen embankments for accreditation by the Federal Emergency Management Agency. The earthen embankment must be analyzed for erosion, stability, seepage, and settlement based on this water surface and a minimum amount of freeboard (typically three feet) provided above this water surface elevation. As little as two feet of freeboard may be allowed if the uncertainty in flow and stage is characterized and justifies less than three feet of freeboard. In recent years the US Army Corps of Engineers has been developing and transitioning toward a semi-probabilistic approach. The semi-probabilistic approach is a risk-based geotechnical analysis method to replace the deterministic geotechnical analysis method contained in guidance documents, including EM 1110-2-1913 "Design and Construction of Levees", ETL 1110-2-569 "Engineering and Design: Design Guidance for Levee Underseepage", and ETL 11102-570 "Certification of Levee Systems for the National Flood Insurance Program".

The presence of non-grass vegetation such as trees and shrubs may inhibit satisfactory performance of an earthen embankment's functions. Tree roots, for example, penetrating into a levee structure may increase the probability of levee underseepage. ETL 1110-2-571 "Engineering and Design: Guidelines for Landscape Planting and Vegetation Management at Levees, Floodwalls, Embankment Dams, and Appurtenant Structures" provides guidelines for vegetation management at embankments and floodwalls. A minimum non-grass vegetation free zone beyond the levee toe (or distance to edge of normal water surface) of 15 feet is codified in that document.

Maintenance issues on earthen levees include maintaining the grass covering while inhibiting and removing non-grass vegetation. Drainage channels or structures need to be inspected and

repaired as necessary. The levee must be regularly inspected for signs of seepage and corrective measures taken as needed. Damage from overtopping or weather should be repaired. Animal burrows need to be filled and burrowing animals relocated.

### 3.2.2 Flood Walls

There are numerous floodwalls in place across the nation's system of levees. In general, floodwalls are used when there is insufficient land to place an earthen levee up to the required level of protection. They are more prevalent in urban areas where real estate is at a premium, but they may have limited use in some rural areas as well. There are a wide variety of floodwalls, but the overwhelming majority of these are I-walls and T-walls. Other less common floodwall types include L-walls, buttress/counterfort walls, and gravity style walls. L-walls can be assessed with similar methods as those outlined in this T-wall section of this report. Gravity walls can be assessed for stability using the general wedge methodology as EM 1110-2-2100 "Stability Analysis of Concrete Structures". Buttress (counterfort) walls are essentially T-walls with a structural member on intervals to help support the stem of the wall. These are more difficult to analyze than traditional T-walls because they have different failure mechanisms such as moment and shear failure of the buttress (counterfort) section.

T-walls are one of the predominant types of floodwall in use. As noted earlier, T-walls get their name from the fact the cross-sectional area takes the general shape of an inverted "T". T-walls are generally used in lieu of I-walls when the heights required for flood protection become larger than an I-wall can safely handle. Only a review of the as-built plans will allow one to determine whether a wall is a T-wall or an I-wall. One cannot tell by simply looking at it from the ground. When the foundation conditions are undesirable, T-walls are often pile founded for stability purposes. The piles transfer the load to better soil/rock conditions founded below the unsuitable foundation soils near the surface. In addition, many T-walls have sheetpile cutoff walls located on the riverward (heel side) to improve underseepage performance. Some T-walls may have sloped base slabs to improve global stability. Relief wells and/or toe drains on the protected side may also be present to help control underseepage. Examples of three different T-wall cross-sections, taken from EM 1110-2-2502 "Engineering and Design - Retaining and Flood Walls", are shown in Figure 3-9 for reference. The external loads acting on most flood protection T-walls are usually relegated to earth and water pressures. The weight of the concrete is also considered in the global stability analysis.

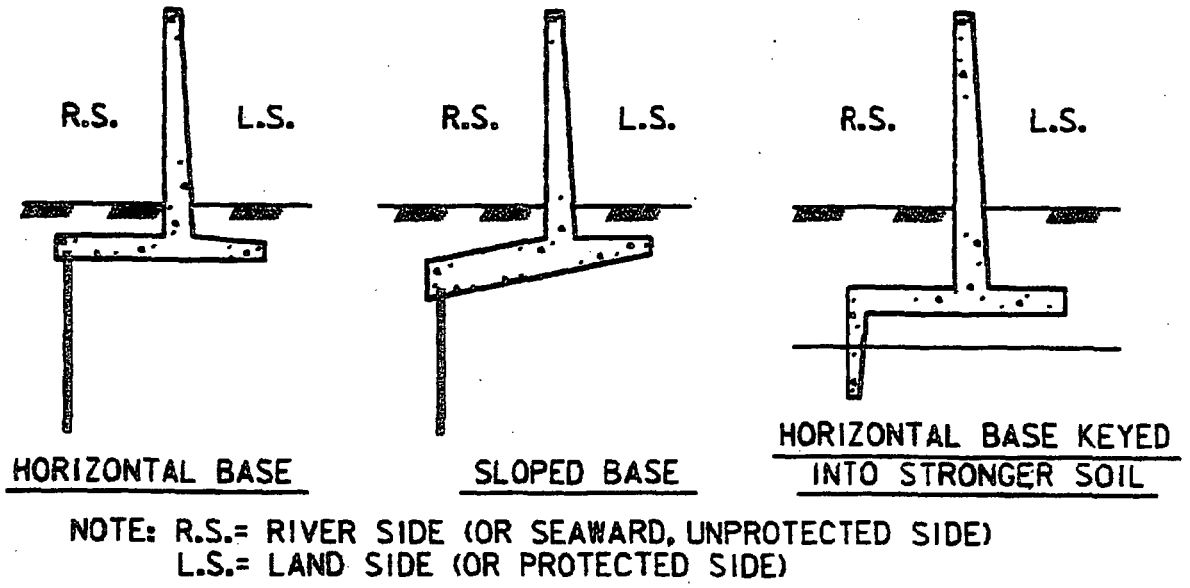


**Figure 3-9. Inverted T-type cantilever flood walls (from EM 1110-2-2502 "Engineering and Design - Retaining and Flood Walls").**

Several different water levels will likely have to be evaluated in order to develop a system response curve (probability of wall failure vs. water level) for a risk analysis. There are several important considerations for determining the water levels to evaluate as part of the risk analysis. An estimate of the water surface profile compared to an accurate top of levee/floodwall along the line of protection will help one determine how high the water likely will rise against the floodwall section being evaluated before incipient overtopping possibly occurs at another location along the line of protection. A few other key points regarding selecting water elevations for risk analysis purposes include:

1. Ensure the datum being used for water surface elevation estimate is consistent with the top of levee/floodwall profile. Different datums have been used throughout the U.S.
2. Ranges of loading will most likely be required for the risk analysis. A good starting point is 25 percent, 50 percent, 75 percent, and 90 percent of the wall overtopping.
3. Evaluate for the mid-point of the range since the failure probability that is developed is used to represent the entire range. The frequency of this loading also needs to be taken into account from a risk analysis perspective. It is important to have tighter ranges at water elevations that are likely to be critical from a performance standpoint. For example, if one is assessing the performance for the 50 percent to 75 percent exposed height range, then one should assess it for the 62.5 percent exposed height and use those results for the entire range. This will be done for each range evaluated.
4. The water levels used to develop failure probabilities for the wall section need to be consistent with the levels used for the consequence estimates. A relation between consequences and water elevation should be developed. The analysis for both the wall performance and consequences needs to cover the entire range of water elevations considered for the risk analysis.

There are several failure modes that are considered viable for levee T-walls, but they can generally be separated into three broad categories: global instability, structural performance, and underseepage/piping. Global instability refers to overturning, sliding, and bearing capacity. Global instability failures can occur before or after overtopping of a floodwall. If the floodwall

holds and then overtops, passive resistance on the protected side can be eroded away leading to global instability. Structural performance relates to excessive moment and shear forces failing the structural wall section. Underseepage and piping involves the movement of foundation soils below the wall causing a loss of wall foundation support and subsequent stability failure.

The presence of trees and significant vegetative growth immediately adjacent to floodwalls has the potential to adversely affect stability of floodwalls in a variety of ways. This could be vegetation on either side of the wall. A 'safe distance' needs to be provided from the foundation of the wall to any significant vegetation; unfortunately, there is no 'preset' safe distance that will account for all situations and each must be judged in the context of how a tree might adversely affect floodwall stability in its given environment. The 15-ft vegetation free zone within USACE guidance is specific to maintenance and inspection requirements. This distance is not necessarily indicative of how vegetation may affect floodwall stability and should not be taken as such. There are instances where certain types of vegetation within 15 feet may not be harmful to the performance of the floodwall, just as there are instances where vegetation greater than 15 feet away from the floodwall's foundation could potentially fail the wall. Careful engineering judgment is required to evaluate each situation on its own merits. When large trees and/or trees with significant root systems are located in the vicinity of floodwalls, one should carefully consider how they might adversely affect performance. A few situations to consider include:

1. Trees with large root systems extending below floodwalls have the potential to 'jack' or lift the wall potentially causing a wide range of failure issues such as cracking, separation of joints, or wall failure.
2. Large trees adjacent to walls can topple over and structurally damage a wall particularly when surrounding soils are already saturated from heavy rains and flooding.
3. Floodwalls with toe drainage systems in place to relieve uplift pressures for wall stability can be damaged either by tree roots penetrating the toe drain system or having an uprooted tree dislodge the drainage system rendering it ineffective.
4. Floodwalls requiring passive resistance for stability can also fail if a large soil mass on the protected side is removed by an overturned tree.

Inland flood walls typically are installed along a riverbank and are subjected to design loadings (pool to freeboard line) for periods of hours or days (long-term loadings). Coastal flood walls are primarily subjected to short-term loadings (waves from hurricanes along with wind/tide high water surges). The wave loadings are dynamic in nature and act upon the structure for only a few seconds each. Concurrent high winds can prevent any emergency maintenance during a storm. Utility line crossings through a flood wall require careful attention to allow for independent movement of the utility lines and the wall, which requires special expansion joint details.

Water-retaining structures are subject to through-seepage, underseepage, and seepage around their sides or ends. Seepage control is a primary consideration of flood wall design. Uncontrolled seepage may result in water pressures and uplift forces on the wall base in excess of design assumptions and consequent structural instability. Excessive porewater pressures in foundation materials near the landside toe of a wall may create "quick" conditions evidenced by sand boils or heaving. Emerging seepage may have sufficient velocity to move cohesionless foundation materials and erode the wall foundation (piping). Seepage control entails the design of measures to ensure that seepage pressures and velocities are maintained below tolerable values. Properly controlled seepage, even if quantities are large, can present no hazard. Flood walls in congested areas often require seepage to be pumped out of the protected area. While the seepage quantity is often small compared to other sources, it is occasionally appropriate to consider seepage control measures for the purpose of reducing seepage quantities.

Inadequate seepage control may jeopardize the stability of a flood wall. In flood walls, control of through-seepage is provided for by water stops. Seepage around the wall is controlled by specially designed and constructed levee wrap-around sections. Flood walls are usually provided with a toe drain to control local underseepage along the flood wall base. As flood walls are usually founded on alluvial materials, pervious zones of significant thickness are often present at some depth below relatively impervious top stratum materials and may be hydraulically connected to the river. Because of the horizontal stratification of alluvial deposits, the horizontal permeability may be greatly in excess of the vertical permeability.

The combination of these conditions may allow seepage to be readily conducted landward beneath the flood wall. Where flood walls are underlain by such pervious strata (the usual case), analysis may indicate the need for underseepage controls in addition to the toe drain. Underseepage control measures vary because the selection and design of an appropriate control scheme is highly dependent on site-specific conditions, particularly the stratification and permeability of foundation materials, availability of right-of-way, and local construction practices and costs.

Careful attention must be given to wall monoliths that have loading, support, or other conditions that vary along the length of the monolith. These monoliths, which may include closure structures, pipeline crossings, corner structures, etc., must be analyzed as complete three-dimensional entities instead of the usual two-dimensional unit slices. Planning and design procedure considerations for floodwall projects are described in EM 1110-2-2502 "Engineering and Design – Retaining and Flood Walls" and in EM 1110-2-2102 "Engineering and Design – Waterstops and Other Preformed Joint Materials for Civil Works Structures".

EM 1110-2-6053 "Engineering and Design – Earthquake Design and Evaluation of Concrete Hydraulic Structures" covers requirements for the seismic design and evaluation of plain and reinforced concrete hydraulic structures. The types of concrete hydraulic structures addressed in this manual include dams, U- and W-frame locks, gravity walls, and intake/outlet towers. The guidelines are also applicable to spillways, outlet works, hydroelectric power plants, and pumping plants. The structures may be founded on rock, soil, or pile foundations and may or may not have back-fill soil.

Potential failure modes of cantilever sheet piling walls are discussed in EM 1110-2-2504 "Design of Sheet Pile Walls". An I-wall is a slender cantilever wall, embedded in the ground or in an embankment that rotates when loaded and is thereby stabilized by reactive lateral earth pressures. Lessons learned from Hurricane Katrina indicate that formation of a flood side gap between the sheet piling and the foundation soils can contribute to poor performance of I-walls in global stability, may increase seepage and uplift problems, and can increase lateral loads on the I-wall requiring greater piling tip penetration to ensure stability. These conditions lead to failure modes that should be included during design. Design and evaluation of I-walls are covered in EC 1110-2-6066, (2011), "Engineering and Design – Design of I-Walls" and ETL 1110-2-575, (2011), "Engineering and Design: Evaluation of I-walls".

Depending on the application, I-walls are subject to varying global stability failure mechanisms. Failure of an I-wall and embankment slope towards a river or canal may be a concern during low water levels, but failure toward the protected side must be considered during high water levels. Global stability analyses should be conducted without using a gap between sheet piling and soil. When I-wall/earthen embankment composite systems are proposed, global stability of the levee and riverbank must also be addressed. Sheet piling associated with I-walls can act as reinforcement within the embankment

and enhance global stability. Until further research is done to quantify the additional stresses imposed on I-wall sheet piling within embankments, designs for embankment portions of I-wall/earthen embankment composite systems should follow in guidance from EM 1110-2-1913 "Design and Construction of Levees", excluding any reinforcing effect of the sheet piling.

Rotation failure due to inadequate piling penetration also is possible. Classical earth pressure theories are typically used to estimate lateral earth loads and required piling penetration for cantilever walls. This type of failure is prevented by adequate penetration of the piling for the cantilever wall. If a gap forms on the flood side, then both the force and resistance are reduced and therefore the overturning moment increases, not decreases.

The loads governing the design of an I-wall arise primarily from the water loads applied to the I-wall stem, buried sheet piling and foundation soils. Other loads applied to I-wall systems include impact, ice, and wind forces. Current methodologies for determination of these loads are discussed in EM 1110-2-2504 "Design of Sheet Pile Walls" and ETL 1110-20575 "Engineering and Design: Evaluation of I-walls". The most recent earthquake guidance is given in ER 1110-2-1806 "Earthquake Design and Evaluation for Civil Works Projects".

The Onset of Overtopping loading condition represents a rising river with the water elevation at or above the top of the wall. The water level or saturation level on the protected side should be based on project specific hydrology and hydraulics, and existing interior drainage features and projected overtopping flows, but will likely be at the top of ground. This loading condition is usually the maximum differential loading condition.

The Design Overtopping Level loading condition is a resilience and/or toughness analysis and corresponds to a water level at or above the top of the wall. The Design Overtopping Level water level is based on projected overtopping flows during unusual or extreme events. The water level or saturation level on the protected side should be based on project specific hydrology and hydraulics, and existing interior drainage features and projected overtopping flows, and will likely be above the top of ground.

EM 1110-2-1902 "Slope Stability", provides criteria to be used with methods of stability analysis that satisfy all conditions of static equilibrium for flood walls. Finite element analyses may also be used to solve for global stability. Rotational stability is satisfied when minimum required safety factors are applied to Mohr-Coulomb shear strength properties prior to analyzing tip penetration. The use of effective shear strength properties is discussed in Chapter 5 of EM 1110-2-2504 "Design of Sheet Pile Walls". Projects with past seepage erosion concerns should be analyzed on an individual basis relating past to expected performance. EC 1110-2-6067 "Engineering and Design: USACE Process for the National Flood Insurance Program (NFIP) Levee System Evaluation" discusses the potential for increased seepage erosion along pre-existing seepage paths where piping occurred. The selection and application of material properties for analyzing the stability of walls and slopes is detailed in EM 1110-2-1902 "Slope Stability" and EM 1110-2-1913 "Design and Construction of Levees". Failure towards the flood side also is covered in these two engineer manuals.

### 3.2.3 System Components

Riverine flood protection systems typically contain many component structures in addition to levees and floodwalls. These include gravity drainage systems, pumping stations, closure structures and other items. These items are not covered in detail in this document, but their design, maintenance, and operation during flood events is critical to the function of the flood protection system. Closure structures, in particular, must be tested periodically, and personnel

must be available for operation during a flood event. Design guidance for closure structures for openings in earthen embankments and flood walls of inland local flood protection projects is located in EM 1110-2-2705 "Structural Design of Closure Structures for Local Flood Protection Projects". Closure structures are required at openings in earthen embankment and floodwall systems when facilities such as railroads, roadways, and pedestrian walkways pass through earthen embankment and floodwall systems at elevations below the level of protection provided by the project. Closure structures for openings in earthen embankment and floodwall systems include various gate mechanisms, such as stop logs (made of aluminum or steel), swing gates, miter gates, trolley gates, and various types of rolling gates.

### 3.2.4 Riverine and Other Non-coastal Considerations

The performance of riverine flood protection systems may be compromised by river processes. Channel migration or incision may erode earthen embankments. Channel sedimentation (deposition or aggradation) may cause higher stages than designed; this may result in embankment overtopping at a more frequent event than designed, or it may result in overtopping at an unplanned location. River morphology and sedimentation processes need to be considered in design, and also need to be evaluated during the life of the project. Changes are often gradual, and may take place at a distance from the embankment footprint, resulting in changes to the water surface profile that are not evident until a flood event occurs. Engineer Manual 1110-2-1418 "Channel Stability Assessment for Local Flood Control Projects" provides guidance.

Ice effects are addressed in several references. Current USACE guidance on the development of ice-affected stage frequency relationships is covered in ETL 1110-2-576 "Engineering and Design: Ice-Affected Stage Frequency ". Engineer Manual 1110-2-1612, "Ice Engineering", is a comprehensive reference covering every aspect of ice-related design and analysis. "Design-Basis Flood Estimation at Nuclear Plants in the United States of America" (NRC, 2011) discusses mechanisms of ice-induced flooding.

## 3.3 Risk Analysis

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# 4 INCORPORATED (SECONDARY) BARRIERS

Incorporated protection is provided by special design of walls and penetration closures. Walls are usually reinforced concrete designed to resist the static and dynamic forces of the DBFL and incorporate special waterstops at construction joints to prevent leakage. Penetrations include personnel access, equipment access, and through-wall piping. Pipe penetrations are usually sealed with rubber boots and flanges. Personnel access closures include submarine doors and hatches. Penetrations that are too large to close with a single door generally require stop logs or flood panels for closure. The maritime industry should be consulted for detailed information concerning the closure structures for incorporated barriers.

Design, construction, performance, and reliability standards of incorporated barriers are limited. The National Flood Barrier Testing and Certification Program (http://nationalfloodbarrier.org) is implementing a national program of testing and certifying flood barrier products used for flood proofing and flood fighting. This program currently tests barrier products in two broad categories, temporary flood barriers and closure devices. The purpose of the program is to provide an unbiased process of evaluating products in terms of resistance to water forces, material properties, and consistency of product manufacturing. This is accomplished by testing the product against water related forces in a laboratory setting, testing the product against material forces in a laboratory setting, and periodic inspection of the product manufacturing process for consistency of product relative to the particular product that received the original water and material testing. Upon products meeting the consistency of manufacturing criteria and meeting the established standards for the material and water testing, the certification part of the program becomes available to the product.

The USACE does not have design guidance on the closure structures for secondary barriers. However, design guidance for closure structures for openings in earthen embankments and flood walls of inland local flood protection projects is located in EM 1110-2-2705 "Engineering and Design – Structural Design of Closure Structures for Local Flood Protection Projects". Examples of flood proofed structures in the United States are included in "Flood Proofing Systems and Techniques" (U.S. Army Corps of Engineers, 1984). This document notes that plastic, marine paints, water proofing compounds, and other sealants can be applied to structures, but it is extremely difficult to make closures completely watertight, and many systems using this technique employ pumps to evacuate leakage.

There are two types of flood proofing of incorporated barriers, "wet flood proofing" and "dry flood proofing". The first technique allows floodwater to enter the structure. Vulnerable items such as utilities appliances and furnaces are relocated or waterproofed to higher locations. By allowing floodwater to enter the structure hydrostatic forces on the inside and outside of the structure can be equalized reducing the risk of structural damage. The second technique is known as "dry flood proofing." With the dry flood proofing technique, a building is sealed so that floodwaters cannot get inside. The intent of a nuclear power plant incorporated barrier is to be a dry barrier with an internal system of drainage and pumping for leakage inside the incorporated barrier. Dry flood proofing is applicable in areas of shallow, low velocity flooding. All areas below the flood

protection level are made watertight. Walls are coated with waterproofing compounds or impermeable sheeting. Openings such as doors, windows, sewer lines, and vents are closed with permanent closures or removable shields, sandbags, valves, etc. Dry flood proofing maximum protection level is three feet and is not for buildings with basements since those structures are difficult to protect from underseepage. Some of the disadvantages of this technique are that many waterproofing compounds are not made to withstand the pressures of the water and will deteriorate over time. Also, closures on windows and doorways are dependent on adequate warning time for installation, as well as the presence of someone to install them correctly.

Much research and documentation of flood proofing has been conducted and/or compiled by the U.S. Army Corps of Engineers, National Nonstructural Flood Proofing Committee (NFPC). The NFPC website (http://www.usace.army.mil/Missions/CivilWorks/ProjectPlanning/nfpc.aspx) contains flood proofing information and links to online reports, including the following that may be applicable for a nuclear power plant:

- "Flood Proofing Tests - Tests of Materials and Systems for Flood Proofing Structures" (U.S. Army Corps of Engineers, 1988) addresses closures, materials, and systems that were tested to determine the effectiveness in protecting structures from floodwaters.

- "Flood Proofing - How to Evaluate Your Options" (U.S. Army Corps of Engineers, 1993b) is a layperson's guide to evaluating and selecting flood proofing alternatives. It includes simplified damage, cost, and performance analyses.

- "In the Tug Fork Valley: Flood Proofing Technology" (U.S. Army Corps of Engineers, 1994) summarizes and provides technical details, photos, and information regarding one of the largest Federal nonstructural flood proofing projects ever completed within the United States.

- EP 1165-2-314, "Flood Proofing Regulations" is a source for flood proofing regulations and technical schematics depicting various structural components of flood proofing measures.

- "Flood Proofing Performance - Successes and Failures" (U.S. Army Corps of Engineers, 1998) documents the successes and failures of various nonstructural flood proofing measures from post storm events throughout the United States.

- "Flood Proofing: Techniques, Programs and References" (U.S. Army Corps of Engineers, 2000) addresses the approaches to flood proofing and government flood proofing programs, references, and terminology. It presents a general overview of flood proofing techniques and provides the reader information on government agencies that offer more specific assistance and publications containing detailed flood proofing information.

Most wall materials, except for some types of high-quality concrete, will leak unless special construction techniques are used. The most effective method of sealing a brick faced wall would be to install a watertight seal behind the brick when the building is constructed. For flood proofing existing structures, the best way to seal a wall is to add an additional layer of brick with a seal "sandwiched" between the two layers. It is possible to apply a sealant to the outside of a brick or block wall. Cement or asphalt based coatings are the most effective materials for

sealing a brick wall, while clear coatings such as epoxies and polyurethanes tend to be less effective. As a result, the aesthetic advantages of a brick wall are lost with the use of better sealant coatings.

Personnel access watertight doors are very similar to sliding or hinged flood shields in purpose, yet they are designed to function as actual doors that are used during normal operating conditions. This type of door can be closed and sealed by a latch mechanism without the use of bolts that are normally used to secure a flood shield. These type doors must be capable of resisting flood-related forces. These are the forces that would be exerted upon the building as a result of floodwaters reaching the DBFL and include the hydrostatic force, buoyancy, hydrodynamic and backflow force, and debris impact forces (Federal Emergency Management Agency, 1993). The building's utilities and sanitary facilities, including heating, air conditioning, electrical, water supply, and sanitary sewage services must be located above the DBFL, completely enclosed within the building's watertight walls, or made watertight and capable of resisting damage during flood conditions (Federal Emergency Management Agency, 1993).

Design guidance for closure structures for openings in levees and flood walls of inland local flood protection projects is located in EM 1110-2-2705, "Engineering and Design – Structural Design of Closure Structures for Local Flood Protection Projects". Closure structures are required at openings in levee and floodwall systems where facilities such as railroads, roadways, and pedestrian walkways pass through levee and floodwall systems at elevations below the level of protection provided by the project. Closure structures for openings in levee and floodwall systems are usually either stop log or gate type closures.

Currently, adequate data and analyses do not exist in order for the U.S. Army Corps of Engineers to recommend the use of incorporated barriers as a reliable flood protection barrier at a nuclear power plant. Flood protection at nuclear power facilities requires structures that can reliably keep floodwaters from ever coming in contact with critical infrastructure. Incorporated barriers may be able to supplement a complete flood protection strategy, but are insufficiently reliable to be considered part of the complete flood protection strategy.

## 4.1 Mechanical or Electrical System Penetrations

Although the U.S. Army Corps of Engineers expertise in mechanical and electrical systems in flood protection systems is primarily through dams and levees, the principles generally apply to using mechanical and electrical systems in incorporated barriers. This chapter summarizes applicable information that can be found in "Best Practices in Dam and Levee Safety Risk Analysis" (Bureau of Reclamation and U.S. Army Corps of Engineers, 2012).

To control operation of an item manipulated by mechanical or electrical systems in an incorporated barrier, three things must be provided: power to move the item, machinery to operate the item, and the structural item itself. Before one can evaluate the risk and reliability of operating the item, various components that make up the system and the probability of each component's failure must be defined. To do this, the most common statistical formula used is the Weibull Distribution formula (equation 1)

$$R(T) = e^{-\left(\frac{T-\gamma}{\eta}\right)^{\beta}} \tag{1}$$

where
$R(T)$ = the reliability,
$e$ = 2.718,
T = time,
$\gamma$ = the location parameter,

$\eta$ = the characteristic life, and
$\beta$ = the shape parameter.

The Weibull formula does not take into account time when components are not in use. Therefore a modified version of the formula called the Dormant-Weibull Formula (equation 2) is used

$$Q_n = 1 - \exp(\frac{(n-1)\tau-\gamma}{\eta})^{\beta} . \exp[-\left(\frac{n\tau-\gamma}{\eta}\right)^{\beta}] \quad (2)$$

where
$Q_n$ = the probability of failure over the entire interval n,
$\eta$ = the characteristic life parameter,
$\beta$ = the shape parameter,
$\gamma$ = the location parameter,
t = the inspection interval or time since last operated, and
n = the number of times the component operated in its life.

The Dormant-Weibull model is a failure model that allows the user to model a component or system that undergoes periodic inspection, but is also subject to aging; i.e. the failure rate increases with time. This model also represents a component whose failure will be revealed due to periodic usage during normal operations. "Best Practices in Dam and Levee Safety Risk Analysis" (Bureau of Reclamation and U.S. Army Corps of Engineers, 2012) contains details and analysis demonstrating the use of these equations in defining the risk and reliability of structures utilizing mechanical and electrical systems.

### 4.1.1 Utilities

Deteriorated culverts, pipes, and utility lines below the foundation of flood barriers may result in underseepage and piping, compromising the structural foundation. This is particularly true for pipes that are constructed of materials likely to degrade over time and are not routinely inspected to determine their actual condition. A defect through a pipe below a flood barrier such as a floodwall can lead to a preferred seepage path and depending upon the conditions (surrounding soil, loading duration, and etcetera) can cause piping of foundation materials through the defect. This can cause a loss of foundation support, wall instability, and failure of the structure. Defects can occur either through the body of the pipe (perforations) or at separated joints. A thorough review of the as-built plans and permits needs to be done in order to determine if and where pipes cross below project flood barriers. Deteriorated pipes running parallel to flood barriers can also be an issue if they are close enough to adversely affect performance from an underseepage or stability standpoint. The Levee Screening Tool Technical Manual (U.S. Army Corps of Engineers, 2011) provides a detailed narrative on adverse environments for various types of pipes and is a good resource to determine if a pipe may affect flood protection performance. Sewer lines should be fitted with cutoff or check valves that close when flood waters rise in the sewer, to prevent backup and flooding inside the building. Emergency power is vital to the operation of a flood protection system for a nuclear power plant. Every aspect of the system must have protection, including protecting the fuel used to operate emergency generators.

## 4.2 Personnel Penetrations

Dry flood proofing involves sealing building walls with waterproofing compounds, impermeable sheeting, or other materials and using shields for covering and protecting openings from floodwaters. In areas of shallow, low velocity flooding, shields can be used on doors, windows, vents, and other building openings. The first step with the use of shields placed directly on buildings is to be certain that both the shield and the building are strong

enough and sufficiently watertight to withstand flood forces. Generally, dry flood proofing should only be employed on buildings constructed of concrete block or brick veneer on a wood frame. Weaker construction materials, such as a wood frame, will fail at much lower water depths from hydrostatic forces. Even brick or concrete block walls should not be flood proofed above a height of three feet, due to the danger of structural failure from hydrostatic forces.

Some waterproofing compounds cannot withstand significant water pressure or may deteriorate over time. For effective dry flood proofing, a good interior drainage system must be provided to collect the water that leaks through the sealant or sheeting and around the shields. These systems can range from small wet-vacs to a group of collection drains running to a central point from which water is removed by a sump pump. In many cases, flooded sites are isolated during a flood event. Once barriers, shields, closures, etc. are installed at a site, the normal site egress paths are often obstructed or removed. Attention must be given to this safety issue.

The difficulty and complexity of sealing a structure also depends on the type of foundation, since all structural joints, such as those where the walls meet foundations or slabs, require treatment. For very low flood levels, such as a few inches of water, a door can be flood proofed by installing a waterproof gasket and reinforcing the door jamb, hinge points, and latch or lockset and coating it with a waterproof paint or sealant. If there is a chance of higher flood levels, some type of shield will be needed. If the expanse across the door is three feet or greater, the shield will have to be constructed of heavy materials, such as heavy aluminum or steel plate. The resulting weight may require the shield to be permanently installed, using either a hinged or slide-in design. The frame for such an installation must be securely anchored into the structure. When windows are exposed to flooding, some form of protection is needed because standard plate glass cannot withstand flood forces. One solution is to brick up all or part of the window. It may also be possible to use glass block, instead of brick, to admit light.

For normal-sized windows, shields can also be used. They should be made of materials such as heavy Plexiglas, aluminum, or framed exterior plywood. These can be screwed in place, or slid into pre-designed frame slots. Another alternative is to replace the glass with heavy Plexiglas; however, the window must be sealed shut and waterproofed using water resistant caulking.

The specifications for materials, fabrication, installation, and quality assurance of the various components of closure structures are provided in the Unified Facilities Guide Specifications for Waterway and Marine Construction (U.S. Army Corps of Engineers, 2008). More detailed information is available in the Unified Facilities Guide Specifications library (http://www.wbdg.org/ccb/browse_cat.php?c=3).

## 4.3 Warning Systems

Warning systems may be able to reduce the risk of danger to people and structures. However, the warning system itself is subject to potential operational failure. The following steps are required for a warning system is to be successful:

1. Failure is detected by the system and the alarms are triggered
2. A decision is made to initiate an action
3. The population at risk is notified of the impending failure
4. The population at risk is successfully prepared prior to the failure

The following two factors make a warning system "more likely" to detect a flood failure:

1. There are multiple independent platforms to collect and transmit data. This provides redundancy in transmitting data.

2. There are numerous independent instruments that provide for possible verification of a flood failure.

The following two factors would make the warning system "less likely" to detect a flood failure:

1. A false alarm has already occurred. The instrumentation is not 100 percent reliable.
2. A major seismic event near the site capable of failing the flood control structures could wipe out all communications platforms at the site. While this could be an indication of a flood failure, it could also be interpreted as something else.

Given a flood failure and that the early warning system successfully detects the failure through the alarm systems in place; two factors that make the decision to initiate an action "more likely" include the following:

1. Operating personnel have taken part in an exercise related to flood failure and the need to secure facilities and staff.
2. Operating personnel have been given the authority to initiate the action. The notice to begin the action can be given directly without going through other offices for approval.

# 5 TEMPORARY BARRIERS

Flood protection for critical infrastructure should not rely on temporary structures, and permanent structures capable of withstanding floods of a desired probability should be constructed. However, in cases of an extreme event or combination of events that threaten to overtop the primary flood defenses, temporary measures may be deployed. Traditionally, sandbags have been used as temporary flood barriers to raise the effective crest of a levee or as a barrier encircling a building(s). Numerous commercial products are available that are capable of raising the effective crest of an earthen barrier on a temporary basis in far less time than is required if sandbags are used. Most of these temporary barriers may be classified as sand-filled, water-filled, frame with skirt, or other. Temporary flood protection structures are insufficiently reliable to be included in a complete flood protection system. However, the use of temporary barriers can supplement a complete flood protection system of external barriers, an interior drainage system, and redundant pumping stations.

The US Army Corps of Engineers has a Flood Fighting Structures Demonstration and Evaluation Program intended to devise real world testing procedures for promising flood fighting technologies (http://chl.erdc.usace.army.mil/chl.aspx?p=s&a=PROGRAMS;16). As part of this program, in 2004 the Engineer Research and Development Center (ERDC) tested a few temporary barrier-type flood fighting structures. This chapter summarizes the testing and evaluation of sandbags as well as three commercial flood-fighting products. The full analysis can be found in ERDC TR-07-3 "Flood-Fighting Structures Demonstration and Evaluation Program: Laboratory and Field Testing in Vicksburg, Mississippi" (U.S. Army Corps of Engineers, 2007).

Laboratory and field testing were conducted from March to August 2004. The laboratory testing was completed in a wave research basin at ERDC, Vicksburg, MS. Field testing was accomplished at a site north of Vicksburg, on the southern bank of the turning basin of the Vicksburg Harbor. Summary results for the laboratory test are shown in Table 4-1 and for the field test in Table 4-2. Both the laboratory and field testing show conclusively that a Portadam, Hesco Bastion, and Rapid Deployment Flood Wall (RDFW) structure can be constructed much faster and with much less labor force than a comparable sandbag structure. All three products performed well for most all of the testing parameters.

**Table 5-1. Laboratory test summary.**

| Item | Portadam | Hesco Bastion | Sandbags | RDFW |
|---|---|---|---|---|
| Construction time (hours) | 4.8 | 3.5 | 11.5 | 5.5 |
| Construction effort (man-hours) | 24.4 | 20.8 | 205.1 | 32.8 |
| Removal time (hours) | 1.1 | 2.7 | 4.5 | 7.0 |
| Removal effort (man-hours) | 4.4 | 13.4 | 9 | 42 |
| Seepage (gal/min/ft) for static water at 95% of structure height (average) | 0.14 | 1.81 | 0.54 | 0.10 |
| Damage from overtopping | None in 1 hour | No damage in 1 hour | Failed in 1 hour | None in 1 hour |
| Damage from log impact | Vinyl tarp puncture | No damage | No damage | No damage |
| Repairs concern | Minor | Minor | Major | Minor |

**Table 5-2. Field test summary.**

| Item | Portadam | Hesco Bastion | Sandbags | RDFW |
|---|---|---|---|---|
| Construction time (hours) | 5.1 | 8.9 | 30.5 | 7.5 |
| Construction effort (man-hours) | 26.2 | 57.5 | 453.1 | 48.4 |
| Removal time (hours) | 2.9 | 8.7 | 2.6 | 17.3 |
| Removal effort (man-hours) | 12.6 | 36.3 | 3.5 | 113.4 |
| Seepage (gal/hr) for 400 $ft^2$ wetted area | 550 | 6,000 | 300 | 900 |
| Repairs | minor | minor | minor | minor |
| Reusability (percent) | 100 | > 95 | 0 | > 90 |

## 5.1 Sandbags

Sandbag barriers have traditionally been the method of choice to raise earthen embankment heights and to protect infrastructure against rising floodwaters. Sandbag levee construction protocol calls for a width three times that of the height as the minimum width criteria. The strengths of a sandbag structure include low product cost. Sandbags also conform well to varying terrain. In both the laboratory and field tests, the sandbag structure had low seepage rates. Also, sandbag structures can be raised if needed by simply placing additional sandbags. The weaknesses of a sandbag structure are that they are labor intensive and time consuming to construct. Also, sandbags are not reusable. During the laboratory testing (Figure 5-1), the sandbag structure was damaged during the wave impact tests and failed during the overtopping tests. The sandbags began to deteriorate during the field tests. For additional information on sandbags, materials, tools and equipment, see section VI of the USACE Flood Fight Manual (U.S. Army Corps of Engineers, 2010).

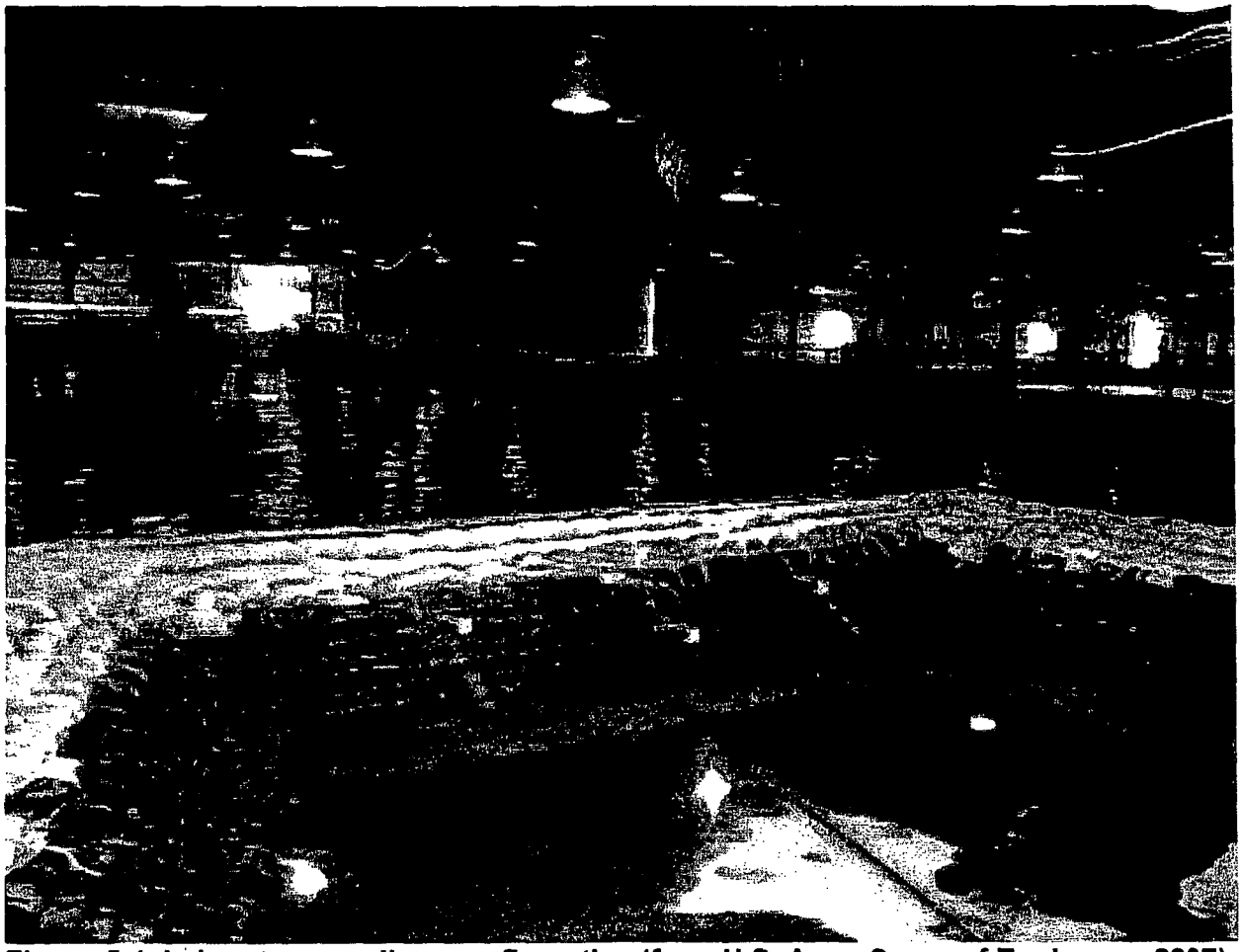

**Figure 5-1. Laboratory sandbag configuration (from U.S. Army Corps of Engineers, 2007).**

## 5.2 Hesco Bastion Concertainer®

Hesco Bastion's strengths include ease of construction and removal for both time and manpower. The Hesco Bastion structures were constructed much faster and with much less labor force than the sandbag structures. The Hesco Bastion product is relatively low cost, and a Hesco Bastion structure can be raised if required by placing a second row of units to the top of the structure. Stability can become an issue for increased height due to the narrow width of the Hesco units. If stability is an issue, a pyramid structure (two units wide on bottom row topped with a single row of units) should be constructed. Hesco Bastion units proved to have a high degree of reusability. During the laboratory and field testing, the Hesco Bastion structures suffered only minimal damage. Weaknesses of the Hesco Bastion product include the need for significant right of way due to the addition of granular fill with machinery perpendicular to the structure and high seepage rates. Since completion of the testing, Hesco Bastion has evaluated their high seepage rates. Their evaluation concluded that in both the laboratory and field testing, the Hesco Bastion units were installed incorrectly. When re-tested in the laboratory, the seepage rate for a properly-installed barrier at a depth of one foot was reduced by 75 percent from the original test results.

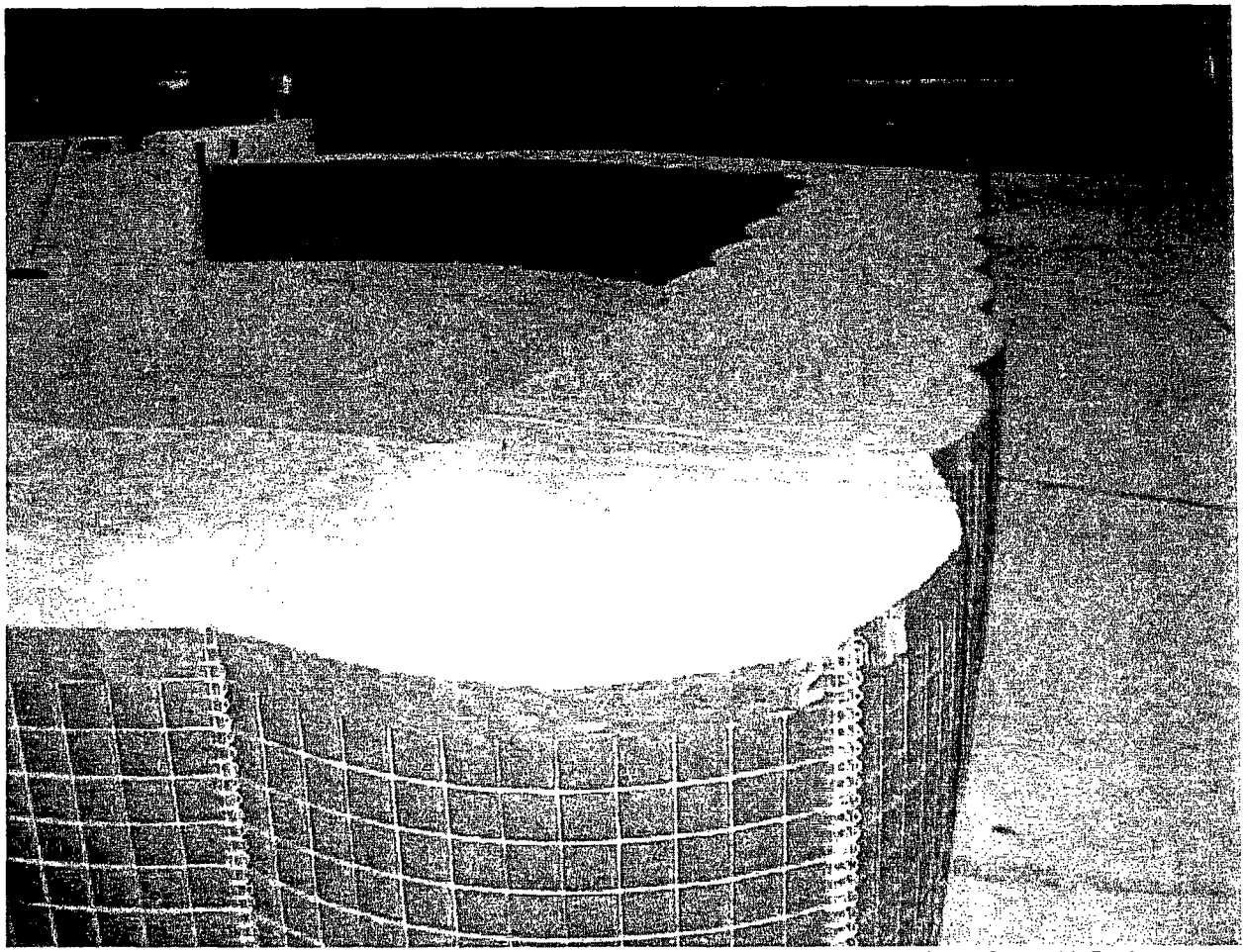

**Figure 5-2. Laboratory Hesco Bastion Concertainer® configuration (from U.S. Army Corps of Engineers, 2007).**

## 5.3 Geocell Systems, Inc., Rapid Deployment Flood Wall® (RDFW)

Geocell Systems Rapid Deployment Flood Wall's strengths include ease of construction for both time and manpower. In both the laboratory and field testing, the RDFW structures were constructed much faster and with a much smaller labor force than the sandbag structures. Additional strengths of the RDFW structures included low seepage rates, high degree of reusability, a RDFW structure can be raised as needed by placing additional rows of units to an existing structure, and since the RDFW units are 8 in. high, an RDFW structure provides various height options. For instance, if a user purchased a quantity of RDFW to construct a 4-ft high flood-fighting structure 1,000 ft long and in a particular flood only needed a 2-ft high structure, then this user would have sufficient product to construct a 2,000-ft-long structure. RDFW's weaknesses include significant right of way required due to the placement of granular fill with machinery perpendicular to the structure, high cost of the product, and in both the laboratory and field testing, the RDFW structures were difficult and time consuming to remove. Although the units were easy to fill with clean granular material during the tests, the small size of the grid openings (8 inch by 8 inch) may be a problem for construction with unsorted local fill material.

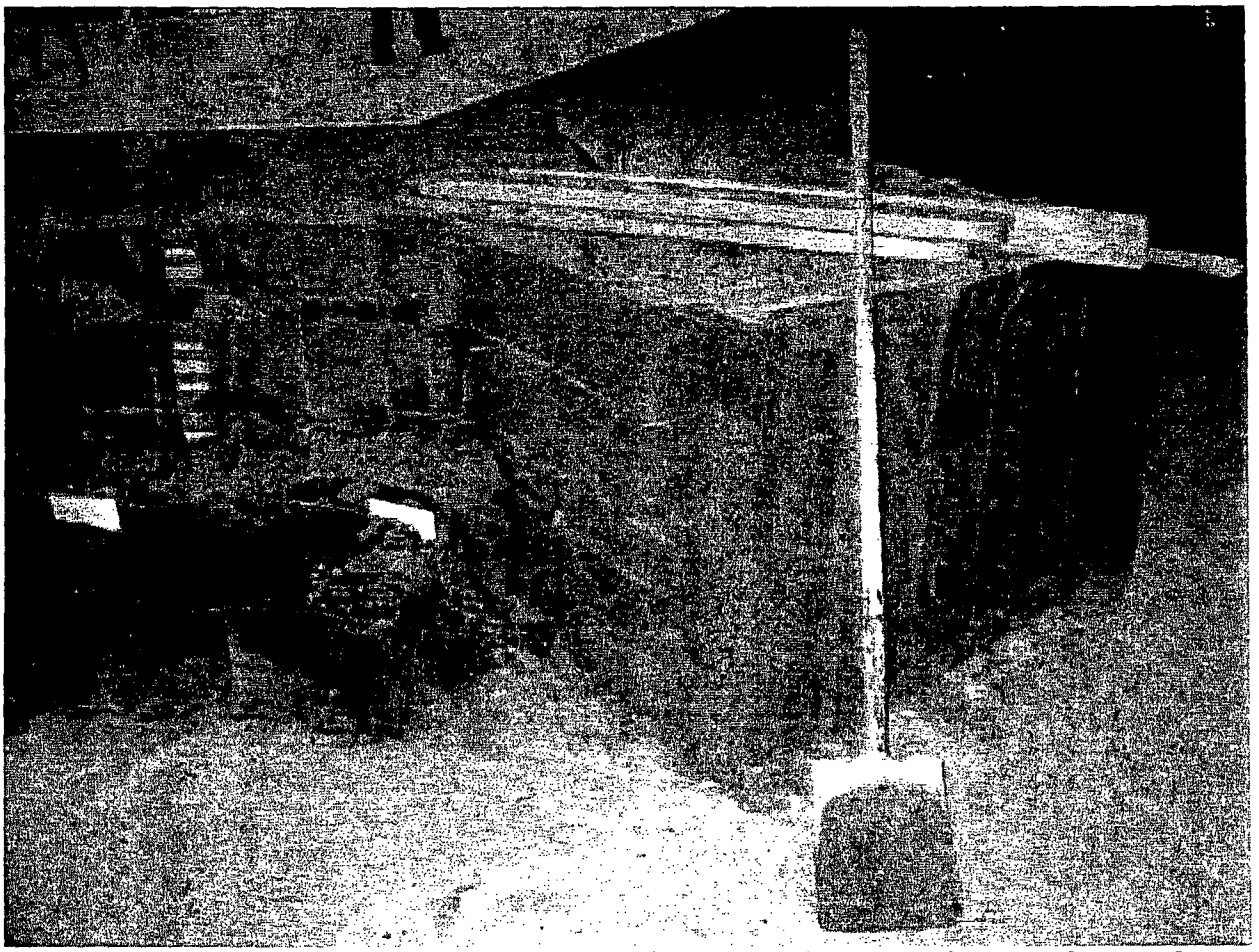

**Figure 5-3. Laboratory Rapid Deployment Flood Wall® (from U.S. Army Corps of Engineers, 2007).**

## 5.4 Portadam®

Portadam's strengths include ease of construction and removal (time, manpower, and equipment). The Portadam structures were constructed in less time and with a much smaller labor force than the sandbag structures. Also, the Portadam structure was constructed without the use of heavy machinery. The Portadam structure proved easy to remove. The Portadam structure had low seepage rates in both the laboratory and field tests. Portadam structures require no fill except for some sandbags that are used to help seal the leading edge of the membrane liner and to add weight to prevent wind damage. Portadam structures have a high degree of reusability. For the field test, the Portadam structure was 100 percent reusable. Since no heavy machinery is required to construct a Portadam structure, only limited right of way is required. However, Portadam does have the largest footprint of the products tested. Portadam's weaknesses include that the membrane liner punctured during the laboratory debris impact tests and a Portadam structure may not be applicable for high wind use unless the structure is anchored or weighted with sandbags. Additionally, it is not possible to raise the height of a barrier once erected if additional height becomes necessary.

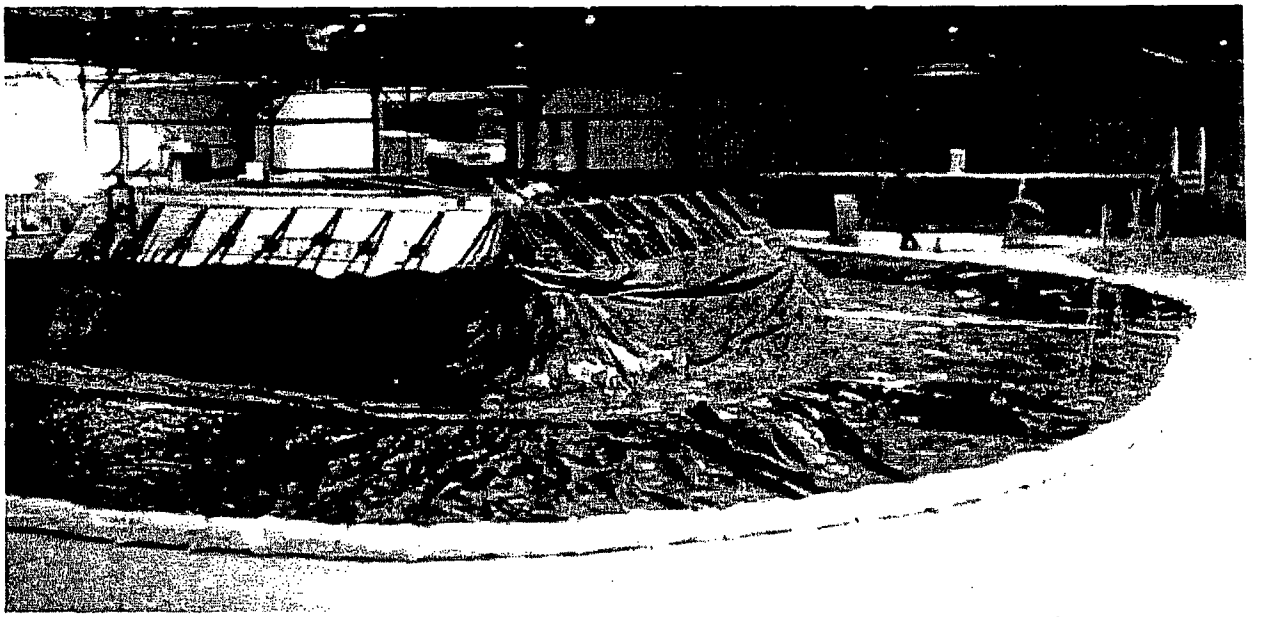

**Figure 5-4. Laboratory Portadam® configuration (from U.S. Army Corps of Engineers, 2007).**

## 5.5 Generalizations

Although the government-funded study described above ended in 2007, ERDC has maintained the laboratory testing basin and continues to test temporary flood-fighting products using the same standardized testing protocol on a vendor-funded basis. Based on these tests, the following generalizations are offered.

Most of the sand-filled barriers, such as the Hesco-Bastion Concertainer (section 5.2) and Geocells System RDFW (section 5.3) that were tested were developed as force protection measures designed to stop bullets rather than hold back floodwaters, but are being deployed as a replacement for sandbags to raise the effective crest elevation of earthen barriers. Various products were tested at water depths up to around three feet and generally performed well with moderate seepage rates. Products tested ranged from low-cost disposable geotextile units to higher priced reusable units with frameworks of plastic, sheet metal, or wire grid.

Failure modes of sand-filled barriers include overturning, sliding, undermining, loss of fill material, and damage to the structure framework. Overturning may become a problem if the barrier is built multiple layers in height without widening the base of the structure, or when the structure is undermined either on the river side or protected side. Protection against overturning may be possible by using multiple rows of units to make a wide base, then fewer rows on top like a pyramid. Sliding has not been found to be a problem in the laboratory tests but has been shown to be a concern with barriers built on frozen ground. Undermining is not a problem with static water, but may be an issue with waves or currents undermining the river side toe of the barrier, or overtopping or seepage flow undermining the protected toe. Undermining may lead to overturning of the structure or, if the structural units are open-bottomed, to loss of fill material. Fill material may also be lost from the top of the units due to wave action or overtopping, or from the bottom of open-bottomed units due to rocking of the barrier during wave action causing the toe to lift and fill to be washed out. Damage to the framework may occur from debris impact or wave action causing tears in fabrics or breaks in solid barriers.

The most common maintenance issue in the laboratory tests was a need to replace sand fill that was washed out by wave action or overtopping. These temporary barriers need to be inspected regularly to correct any observed problems of leaning, sliding, or scour. If the barrier is going to be re-used, it must be cleaned thoroughly, allowed to dry, and any damaged components replaced. Fabric sections, if present, should be inspected for areas weakened by ultra-violiet rays.

The advantage of water-filled barriers is that there is always plenty of water during a flood, so the fill material does not need to be transported to the site. Water-filled barriers include tubes made of reinforced plastic, rubber, or other materials, or hard shell cases (typically plastic or fiberglass) that are fastened together in a row(s) and filled with water. One type of water-filled tube is shown in Figure 5-5.

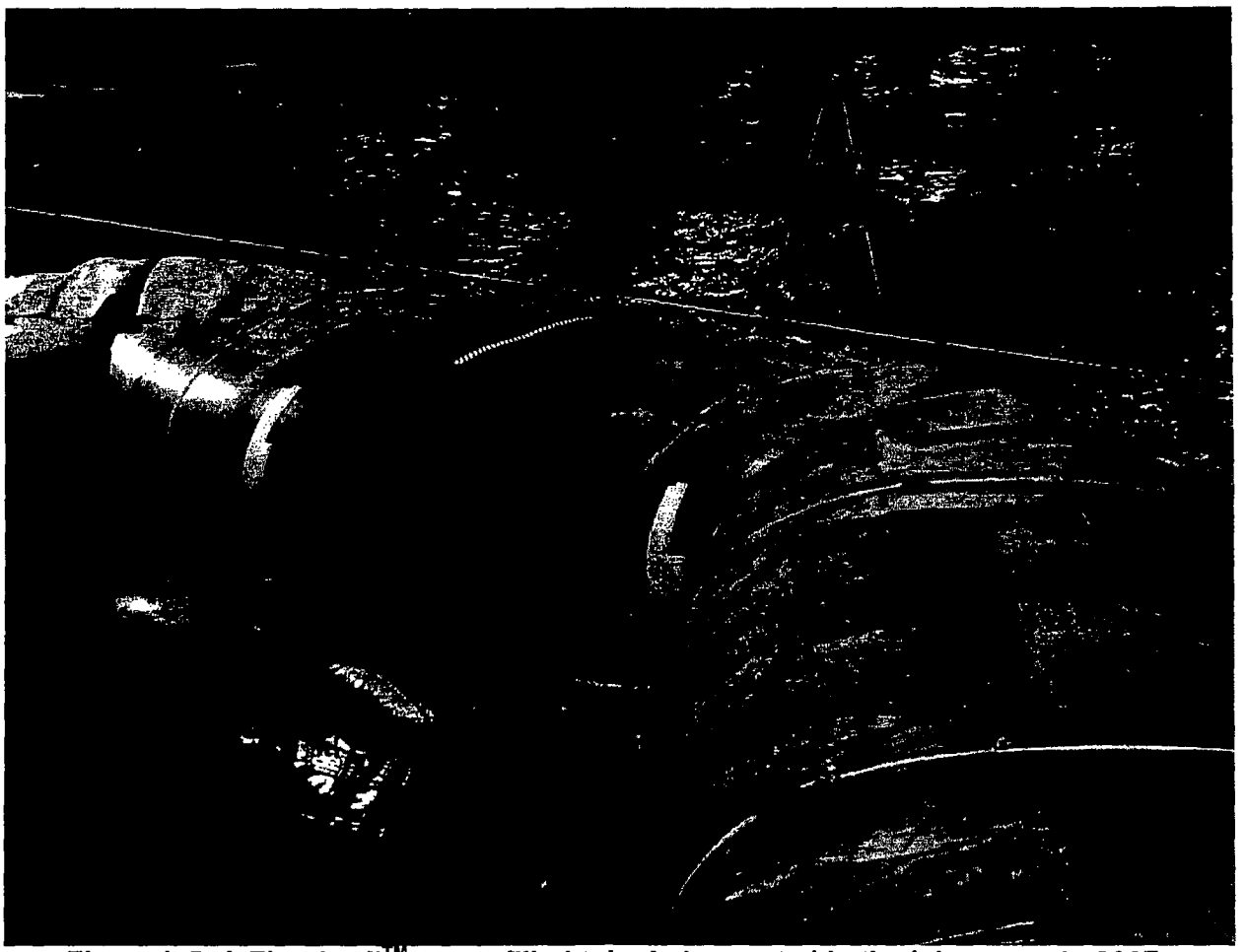

**Figure 5-5. A Floodwall™ water-filled tube being tested in the laboratory in 2007.**

A disadvantage of water-filled tubes is that they roll easily if water is raised on one side of the barrier. Commercially-available tubes for flood fighting use one of three methods to prevent rolling: internal baffles, multiple tubes fastened together with straps, or anchors. Because water-filled tubes are not round but more like a flattened oval when filled on dry land, an internal baffle spanning the short axis of the oval will prevent rolling as it prevents the short axis from rolling over to the long axis. If two or more tubes laying side-by-side on the ground are fastened together with straps, friction between the tubes will prevent the barrier from rolling. By fastening multiple barriers together in a pyramid shape, it is also possible to raise the height of the barrier.

The third option is to fasten a tube to ground anchors on the riverside of the barrier for static water or currents or on both sides of the barrier if wave action is expected.

A second disadvantage of water-filled barriers is that the water fill has no weight when submersed in water. The water-filled barrier gets its stability by the weight of the water inside the barrier that is higher than the outside water level. As a rule of thumb, water-filled barriers should not be used if the river depth at the structure toe is greater than two-thirds the height of the barrier. At deeper water levels, the tubes tend to not seal well along their bottom, seepage rates increase, and the barrier may become unstable.

Failure modes for water-filled barriers include rolling, undermining, punctures, and leaking. Rolling was discussed above. Undermining may be an issue with waves or currents undermining the river side toe of the barrier, overtopping may lead to undermining of the protected side toe, or seepage flow under the barrier may cause undermining. An advantage of a flexible barrier such as a fabric tube is that it may flex into a scour hole or area of undermining and prevent the hole from enlarging. Punctures caused by debris impact, vandalism, or construction mishaps are serious in that an entire unit may drain from a single hole unless it is repaired. Because a tube may be 100 ft or more in length, draining an entire tube could cause a serious breach in the barrier. Leaking at seams or ports may also drain a unit, lowering the crest elevation and weakening the barrier.

The most common maintenance issue for water-filled barriers is repair of leaks. Other issues include repair of scouring or undermining, sliding, and separation at joints between adjacent units. As most water-filled barriers are re-usable, cleaning and drying the units for storage is critical. Because the units were probably filled with water from the flooding river, there will be mud and debris inside the tube that can be difficult to remove.

Frame and skirt barriers, such as the Portadam (section 5.4) consist of sheet of plastic or other flexible material that is fastened to a framework to provide elevation to the barrier while the other end of the sheet is spread out on the ground on the river side of the barrier and extends outward several feet or tens of feet from the barrier as a skirt. The outer toe of the skirt may be trenched or held in place with sandbags. As the flood waters rise up over the skirt, the weight of the water holds the skirt against the ground providing a seal.

Failure modes include tearing of the fabric, seepage under the skirt, or failure of the frame. The fabric draped over the framework may be cut by water-borne debris, vandalism, or installation errors. The skirt can be partially lifted by wave action, currents, or debris which can lead to water flowing under the barrier. Properly designed and installed, the frame should be able to withstand static water pressures but may be damaged by debris impact or wave action. Settling of the frame in mud or soft soil may be a problem.

Maintenance issues include repair of tears in the fabric and repair of damaged frame components. Issues with settling or sliding may need to be corrected. It may be necessary to add sand bags to the skirt to improve the seal or keep it from being moved by wind (pre-flood) or waves and currents. Most frame and skirt barriers are re-usable, so the units must be thoroughly cleaned and dried, inspected and repaired before returning to storage.

There are many other types of temporary barriers, including interconnecting blocks of various shapes, impervious panels in different configurations, gated structures, self-rising flood barriers, and more. Some require pre-installation of a portion of the barrier such as a base or anchoring system ("semi-permanent" flood barriers). All types have their own advantages and

disadvantages, storage requirements, installation and removal requirements. Summaries of tests conducted by the US Army Corps of Engineers may be found at http://chl.erdc.usace.army.mil/chl.aspx?p=s&a=Projects;182 .

# 6 LOCALLY INTENSE PRECIPITATION

While many nuclear power plants face external flood mechanisms such as nearby flooding rivers or storm surge in a coastal area, an internal flood mechanism for nuclear power plants is locally intense precipitation. If not properly accounted for and managed, locally intense precipitation can turn into a significant flooding threat. For example, recent hurricanes and major named storms have produced 10-20 inches of rainfall over significant areas in just a few hours. Properly designed and maintained interior drainage systems and pumping stations alone are insufficient for flood protection at nuclear power facilities. However, coupled with properly designed and maintained external barriers, the flood protection is very reliable.

Interior drainage in a local flood protection project is caused by local precipitation, seepage from temporary flood fighting structures, levees, floodwalls and other flood protection structures. Engineer Manual 110-2-1413 "Engineering and Design – Hydrologic Analysis of Interior Areas" provides an overview of the concepts and strategy for examining the potential for interior flooding. The two concerns for managing locally intense precipitation are conveyance and containment. Conveyance includes interior drainage systems, channels, pipes, culverts, and floodwall closing structures. Containment includes structures such as outlet works, sump works, and pumping stations.

The probable maximum flood (PMF) has flood characteristics of peak discharge, volume, and hydrograph shape that are considered to be the most severe reasonably possible at a particular location, based on relatively comprehensive hydro-meteorological analyses of critical runoff-producing precipitation, snow melt, and hydrologic factors favorable for maximum flood runoff. The PMF load condition represents the most severe hydraulic condition, but because of possible overtopping effects, it may not represent the most severe structural loading condition, which is represented by the onset of overtopping. Therefore, the PMF condition will not necessarily be examined for design.

## 6.1 Interior Drainage Systems

In designing a conveyance system to protect critical facilities one must understand the flow patterns in both the natural topography and the engineered topography. This requires accurate topographic data. Engineer Manual 1110-1-1005 "Engineering and Design: Control and Topographic Surveying" discusses accuracy requirements, reference systems, survey procedures, and other key points related to producing accurate topographic data. There are publicly available sources of topographic data, such as the USGS National Map topographic data (http://www.nationalmap.gov), but care should be taken to ensure that products used have sufficient vertical accuracy for use in critical protection efforts.

**Figure 6-1. Example natural flow directions, contours, and streams from publicly available topographic data.**

Interior drainage in a local flood protection project is caused by local precipitation, seepage from temporary flood fighting structures, levees, floodwalls and other flood protection structures. Engineer Manual 110-2-1413 "Engineering and Design – Hydrologic Analysis of Interior Areas" provides an overview of the concepts and strategy for examining the potential for interior flooding.

## 6.2 Conveyance Structures: Channels, Pipes, and Culverts

Natural and engineered structures work as a system to convey rainfall. Storm drainage conveyance structures include channels, culverts, pumps, and pipe networks. The channels, culverts, pumps, and pipe networks in a system should all be designed to convey the required water quantities as well as to not be subject to erosion or deposition or debris blockage that would degrade the conveyance or reduce the capacity. Engineer Manual 1110-2-1205 "Engineering and Design – Environmental Engineering for Flood Control Channels", EM 1110-2-1418 "Engineering and Design – Channel Stability Assessment for Flood Control Projects", and EM 1110-2-1601 "Engineering and Design – Hydraulic Design of Flood Control Channels" can be consulted for design guidance, ensuring its long-term stability, and functional operation during flood-fighting situations.

Engineer Manual 1110-2-2902 "Engineering and Design – Conduits, Culverts, and Pipes" covers the design and application of pipes through levees to create outflow mechanisms. Engineer Manual 1110-3-136 "Engineering and Design – Drainage and Erosion Control Mobilization Construction" discusses designing pipe networks, culverts, inlets and other internal drainage conveyance structures. Pipe networks that operate as outlet works through a levee or floodwall should have appropriate closing mechanisms to prevent backflow.

### 6.2.1 Streambank Protection

Designs of conveyance and containment structures for the long-term protection of critical structures must account for the effects of changing hydraulics on the natural system. In the

natural environment, streams operate as sediment conveyor belts. This sediment conveyor belt system creates a dynamic equilibrium with the channel shape, slope, and bed material. Engineered channels that are under-designed for the sediment transport requirements of the natural channel will develop sedimentation problems that will significantly reduce the capacity of the channel and hence the protective ability of the channel. Engineered channels that are over-designed for sediment transport will increase the erosion potential either in the channel or downstream of the engineered portion of the channel. Erosion can cause flood protection structures to fail or it can alter the course of the channel towards an undesired location, resulting in an increased threat to the facility.

The WES Stream Investigation and Streambank Stabilization Manual (U.S. Army Corps of Engineers, 1997) discusses the geomorphologic and hydraulic aspects of channel design for streambank protection, various structural and non-structural protection mechanisms, lifecycle management of the engineered channel, as well as maintenance and monitoring requirements for engineered channels.

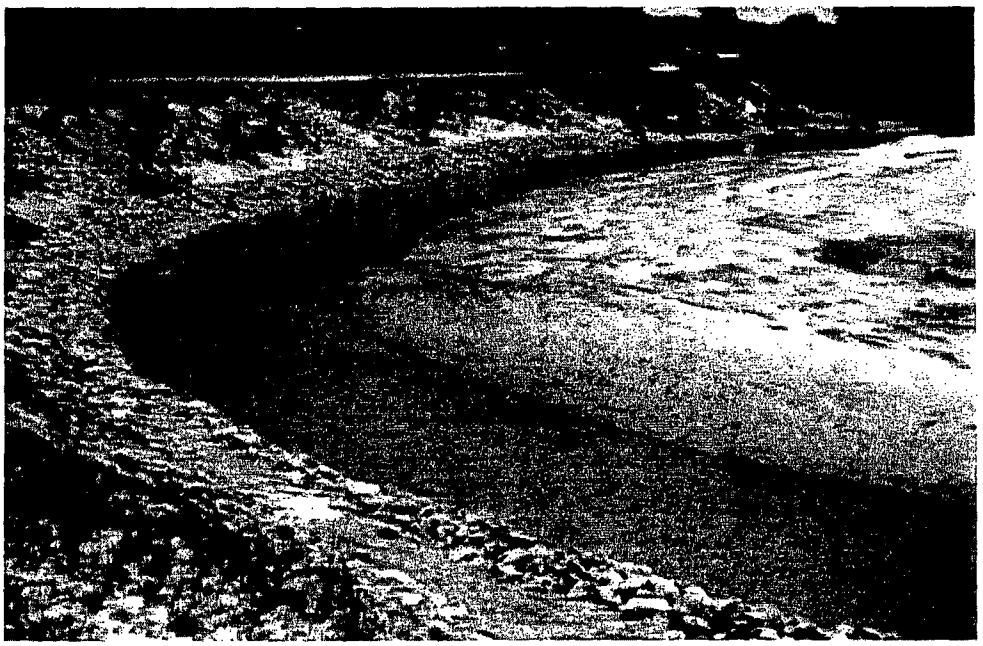

**Figure 6-2. Rip-rap protection placed along the stream bank to protect against erosion (from the WES Stream Investigation and Streambank Stabilization Handbook).**

## 6.3 Floodwall Closing Structures

Frequently floodwalls have openings that facilitate transfer of vehicles, materials, and personnel in and out of the contained area. These breaks in the flood defenses need to have closing structures that perform to the required level of reliability. Engineer Manual 1110-2-2705 "Engineering and Design – Structural Design of Closing Structures for Local Flood Protection Projects" discusses various flood gate mechanisms, including stop logs (made of aluminum or steel), swing gates, miter gates, trolley gates, and various types of rolling gates.

## 6.4 Containment Structures

During flood fights that include interior drainage as well as high water on the outside of the containment area, the locally intense precipitation must be contained or pumped beyond primary barriers. Containment systems include sump basins and detention basins. Engineer Manual 1110-2-1420 "Engineering and Design – Hydrologic Engineering Requirements for

Reservoirs" discusses the role and design principles of reservoirs and detention basins. Design guidance for detention basins of a permanent nature can be found in Engineer Manual 1110-2-2300 "Engineering and Design – General Design and Construction Considerations for Earth and Rock-Fill Dams". The design capacity of the basin or sump should be for a rainfall event that takes into consideration the frequency of rainfall along with the associated (correlated) frequency of high water precluding discharge of the basin. The volume calculations also need to include seepage through other flood-fighting defenses.

### 6.4.1 Outlet Works

Outlet works serve to regulate or release water impounded by a dam or detention basin. It may release incoming flows at a reduced rate, as in the case of a detention dam; divert inflows into canals or pipelines, as in the case of a diversion dam; or release stored-water at such rates as may be dictated by downstream needs, evacuation considerations, or a combination of multiple-purpose requirements. Engineer Manual 1110-2-1420 "Engineering and Design – Hydrologic Engineering Requirements for Reservoirs" describes spillways and outlet works concepts for reservoir design.

In reservoir design there are two purposes of the outlet works. The first is to control the outflow and the second is to prevent the rapid failure of the dam by overtopping. Designing these two levels of outlet works requires an understanding of the potential inflows and total volume of rainfall to be impounded. The design of the regular use spillways is generally dependent upon the flow rates expected to be input to the system during regular and emergency events. The design of the outlet works also calls for an emergency outlet that is sufficient to pass (not contain) the Probable Maximum Flood (PMF) in order to prevent the immediate failure of the dam from erosion due to overtopping flows. Engineer Manual 1110-2-2400 "Engineering and Design – Structural Design and Evaluation of Outlet Works" covers the design of outlet works, including seismic requirements and various gates, valves, and other equipment.

### 6.4.2 Sump Works

The American National Standards Institute/Hydraulic Institute (ANSI/HI) Pump Standards (http://www.pumps.org/content_detail.aspx?id=1412) are the primary source for current information on pump standards and design. The American National Standards Institute/Hydraulic Institute Pump Standards include definitions, industry terminology, design, application, installation, operation and maintenance guidelines, plus the widely-accepted Hydraulic Institute Test Standards.

The U.S. Army Corps of Engineers has several manuals related to sump works, but when the same content from these manuals is covered in the ANSI/HI Pump Standards, the general practice is to use the ANSI/HI Pump Standards. Engineer Manual 1110-2-3102 "Engineering and Design – General Principles of Pumping Station Design and Layout." discusses the design of pumping stations, electrical power requirements, and equipment selection. Structural and architectural design of pumping stations are described in Engineer Manual 1110-2-3104 "Engineering and Design – Structural and Architectural Design of Pumping Stations". Mechanical and electrical design of pumping stations is described in Engineer Manual 1110-2-3105 "Engineering and Design – Mechanical and Electrical Design of Pumping Stations". Pump outlets must be designed to avoid damage to containment barriers. For any non-dry site, a reliable pumping system with built-in redundancy is recommended.

# 7 U.S. ARMY CORPS OF ENGINEERS FLOOD FIGHTING METHODS

The USACE has decades of experience fighting floods. The methods developed from these experiences are documented in detail in the USACE Flood Fight Manual (U.S. Army Corps of Engineers, 2010). This chapter summarizes the principle methods from the USACE Flood Fight Manual associated with maintenance and repair of levees and floodwalls during a flood event. The exact steps described herein may not apply to all flood fight situations, and site specific maintenance and flood fight plans should be developed that consider the issues discussed herein. The USACE routinely conducts flood fight exercises to ensure that personnel are properly trained and that required resources are available for actual flood fights.

A plan for maintenance activities should be developed well in advance of the normal flood season and updated as necessary to account for changes in personnel responsible for maintenance activities. The plan should include an adequate warning system and a well thought out evacuation plan to be followed should the need arise during the life of the project. Upon receipt of official information forecasting the possibility of high water, the facility staff should immediately mobilize a skeleton organization capable of rapid expansion. Definite reaches of the project should be assigned to individuals (section leaders). Each section leader should immediately go over the entire assigned project reach and make a detailed inspection giving special attention to the following:

1. Section Limits: Ascertain that dividing lines between section responsibilities are clearly defined and if necessary marked.
2. Condition of the drainage ditches, levee, and any recent repairs.
3. Conditions of drainage structures with special attention given to flap gates.
4. Water conditions and any accumulations of trash, debris, ice, etc.
5. Transportation Facilities: Vehicular roads and access.
6. Material Supply: Location, item, quantity and conditions.
7. Communications: Locate and check all necessary two-way radios and telephones.

After the initial inspection, each Section Leader should recruit a group and perform the following work as required:

1. Fill holes, gullies and washes in the levee crown and slopes. Farm equipment can be used in repairing small deficiencies.
2. Repair all gaps or depressions that have degraded or are lower than the original levee grade. Filling such depressions may necessitate using material from borrow pits in which case excavation for the material should be kept at 500 feet from the toe of the levee. This type of filling should be tamped in place and, if subject to wave wash, the new section should be faced with sandbags.
3. Check all flap gates to see that they will set properly.
4. Ascertain that necessary access roads along the levee are usable or will be satisfactorily conditioned.
5. Locate necessary tools and materials (sacks, bags, brush, lumber, lights, etc.) and distribute and store same at points where active maintenance is anticipated.
6. Check all needed telephone lines for proper functioning. Obtain lists of all team forces, construction equipment, motorboats, motor cars and truck transportation that can be made available.
7. Arrange with local citizens for supply, transportation, subsistence and shelter for labor force.

8. Examine all drainage ditches on the land side of the levee and remove any obstructions.
9. Remove all dynamite and explosives from the vicinity of the levee.

A maintenance inspection should be made of all drainage structures any time high water stages are forecast. No structure should be omitted from such inspection because of adequate performance during past high water events. If any condition is found that would indicate that the flap gate will not properly operate, the gate should be trial operated at once. Most drainage structures are situated to convey interior drainage from low points of the protected area through the levee by gravity flow. Because of location, drainage structures are generally subject to inundation at lower stages than most other project features. If possible, sluice gates should be inspected before the outlet end of the structure becomes submerged and any trash, debris or other potential obstruction present should be removed. If, for any-reason, the gate system provided on a drainage structure fails to operate and cannot be repaired because of high water, immediate consideration should be given to blocking the structure opening by other means.

Blocking the outlet end of the structure by sandbags is a suggested method of providing an effective temporary closure. If the efforts to plug the outlet structure fail, immediate action should be taken to build a sandbag or earth ring around the inlet structure. While it is of the utmost importance that the structures are blocked to prevent high stages of the river from flowing into the protected area, such emergency closures should be such that they can be readily removed after high river stages recede.

An earthen levee is in potential danger whenever there is water against it. The danger increases with the height of water, the duration of the flood stage and the intensity of either the current or wave action against the levee face. A well constructed levee of correct cross section should, if properly maintained and not overtopped, hold throughout any major flood. Potential failures due to sand boils, sinking levees, slides or sloughing may be prevented if prompt action is taken and proper methods of treatment are employed.

## 7.1 Overtopping

Overtopping is the rush of flood waters over the top of the levee section. The practice of increasing the height of a levee by placing material on the crown to prevent overtopping is called capping or topping. In any high-water situation, sound practice requires that immediate consideration be given to the levee grade line. Although grade lines or profiles should be kept current, a new line of levels (survey) should be run over any reach that appears to be below the predicted flood crest. The grade, in general, is based upon a freeboard approximately 2 feet above the anticipated elevation of water.

Field supervisors should use a certain amount of judgment in determining the type and extent of capping. For example, if the profile shows that a stretch of levee requires less than ½ foot of capping to provide the desired 2 feet of freeboard, the capping may be temporarily omitted. If, however, 2-½ feet of capping is necessary and only 12-inch boards are available, three boards should be used, although the earth needs to be built up to a height of only 2-½ feet. Since capping should be as nearly watertight as possible, care should be taken in preparing the portion of the crown of the levee upon which capping rests. All depressions, such as paths or ramps, should be restored to the natural levee grade, with adequate cross section. Sandbags are frequently used to bring low ramps up to grade. The levee crown should be thoroughly scarified to a minimum depth of 2 inches by plowing, or other similar means, in order to obtain a watertight bond between the capping and the levee crown (levee surface). There are generally four types of capping: earth-fill, sandbag, flashboard, and mud-box or box levee.

The type of capping required is governed by local conditions. Earth-fill capping is the simplest type and quickest to construct. In areas where cohesive materials are unavailable and where the capping would not be exposed to severe wave wash, earth filled capping can be used to a height of approximately 1.5 feet. In areas where cohesive materials (such as clay) are available, greater heights can be achieved, depending on wave action and current velocities. If the levee crown width is 20 feet or more, the height to which earth levee capping can be placed may exceed 1.5 feet.

Under usual conditions where the height capping exceeds 1.5 feet, and is less than 3 feet, or where wave action is anticipated, sandbags or flashboards can be used to raise the level of protection. Capping in excess of 3 feet in height usually requires mud-box or box levee construction, depending on the width of the levee crown and the nature of the material used for capping. Under conditions where capping is necessary over previous high water capping, care should be taken to provide adequate base width for the new work. The height controlling the type of structure should include the height of previous capping. That is, if the combined height of new and old capping exceeds 1.5 feet, flashboard capping should be used. In such cases, it is often necessary to resort to the mud-box or box levee type structure in order to provide adequate stability. The construction methods for each type of capping will be described in the order mentioned above.

The usual sources of earth-fill for capping are from the farm fields on the landside of the levee, from the banks of drainage ditches, or from the landside edge of the crown when the levee has a crown in excess of 15 feet in width. Ordinarily, material should not be taken from within 100 feet of the landside toe of the levee. It is customary to take a cut only about one spade deep over a relatively large area. The method of placing material for capping is important in order to minimize the amount of seepage through the capping. The material adjacent to the flashboards should be free from clods and stubble and should be thoroughly tamped. All additional material should be compacted as well as conditions permit.

The supervisors in charge of capping should organize crews so that the work will proceed in a regular order, each crew of people executing a particular phase of the work, such as preparing and distributing lumber, plowing, setting posts, nailing boards, placing burlap or other materials, and placing the earth-fill. If long stretches of levee are to be capped to a given height in the face of a rapidly rising river, it is well to set the posts to the required height and place the bottom boards only. Succeeding boards and fill are placed after the first boards have been placed throughout. Capping work should be laid out so that the low places are concentrated on and a uniform freeboard provided, parallel to the anticipated flow line, throughout the entire length of the job. The exact method of conducting this kind of work depends upon local conditions and upon the best judgment of those in charge of the work.

These methods of capping are fairly labor intensive and costly. They are also very susceptible to wave erosion if the waves break at the intersection of the flashboard and the levee. If this case arises, protective measures should be executed to ensure that the flashboard or mud box is not undercut.

## 7.2 Wave-wash and Ice Attack

The type of wave-wash protection to be constructed depends upon local conditions, whether or not the levee is exposed to severe wave-wash, the materials of which the levee is constructed, the type and quantity of trees and protective vegetation which may be expected and the existing and predicted stages of the river. The types of wave-wash protection generally used are vertical board revetment, horizontal board revetment, and earth-filled sack revetment. Each of these

types is described in more detail in the USACE Flood Fight Manual (U.S. Army Corps of Engineers, 2010). Sometimes ice conditions are such that protection provided by the methods outlined above will not be totally effective. A boom of logs, driftwood, or any available timber fastened together, strung along the levee slope and anchored about 15 feet from the water's edge has proven particularly effective against ice attack.

Rock riprap is a very popular method to prevent wave-wash erosion and current scour. Depending on the haul distance, this method can, however, be very costly. If site conditions and time permits, the use of filter fabric and / or bedding and spalls placed prior to riprap should be considered to prevent soil material from being pulled through the riprap layer. Straw bales wrapped with polyethylene sheeting on the waterside can be used to provide some wave-wash protection. Sand bags should be used to weigh the bales down so they don't float away.

## 7.3 Current Scour

The methods to be used in protecting a levee against current scour depend entirely upon local conditions. In some cases, the current attack is so severe, and the scour is of such serious nature, that it requires specially designed structures that cannot be constructed with the ordinary high-water-fighting equipment and personnel. Ordinarily however, current scour can be prevented or stopped by relatively simple techniques. The methods that can be used to prevent current scour are widening of waterway gaps in abandoned levees, protecting the riverside slope of the levee with riprap or wave-wash fences, or the construction of brush dikes, each of which is described in detail in the USACE Flood Fight Manual (U.S. Army Corps of Engineers, 2010).

## 7.4 Throughseepage

Drainage of the landside slope of the levee is one of the most important high-water maintenance operations. Consequently, the function must be fully understood and appreciated. Drainage of the adjacent terrain is also highly important. The methodology utilized in draining the slope is to concentrate the flow of seepage into directed channels that carry it rapidly down the slope and away from the levee. The result is that the slope will often become dry and firm between the drains. The drains themselves sometimes never stop flowing. Drainage alone sometimes will not stabilize a wet slope and the slope could become unstable. If this happens, watch the slope carefully for signs of sliding or sloughing and be prepared to construct a mattress (described later) immediately.

Water seeping through a levee may first appear as a wet spot on the slope. As the seepage increases, the wet spot spreads in size until the whole slope is wet and the seep water slowly flows down in a sheet. Continued exposure will cause the slope to become more and more saturated and soggy until it is liable to slide or even flow out resulting in a levee failure or requiring extreme measures to prevent a failure. To prevent sloughing of the levee where the slope is steep and saturated, small "V"-shaped seep drains should be cut in the landside slope to remove the seepage water. These drains may be cut diagonally down the levee slope and should not be more than 4 inches in depth. Several diagonal drains may be led into one drain running straight down the levee. Horizontal drains should not be used, and extreme care should be taken not to disturb the sod unnecessarily outside of the seep drains.

The work consists of opening and clearing the various ditches so that seep water or rainwater will have a free flow from the levee into drainage ditches which convey the water to the drainage structures through the levee. If drainage is perfected prior to high water, the effectiveness of the drainage system will be far greater than if the work is attempted after the ground has become saturated. During flood events, the gates on the drainage structures should be closed to prevent

floodwaters from inundating the protected area landward of the levee. This condition may cause runoff water to pond behind the levee until the floodwaters recede. If the water behind the levee begins to cause damages it should be pumped across the levee to the riverward side. The first drains should be cut 12 to 15 feet apart, V-shaped, no more than 4 inches deep. The drains should originate at the upper or highest limit of seepage and run straight down the slope and lead across the landside berm into a drainage system. To secure better coverage of the seeping area, additional drains spaced 4 feet to 6 feet should be cut between the first drains.

The above-described method of drainage is applicable to clay and other fine-grained soils on levee surfaces. It should not be used as a means of drainage on sand levees, or where the foundation supporting the levee consists of sand. On sand levees, the seep drains should be omitted and the seepage allowed to "trickle" down the landside slope to the seep ditch paralleling the levee toe. If seepage through a sand levee is excessive, a blanket of clayey material should be placed on the riverside slope. If additional excavation is necessary to provide adequate drainage, the general plan described in the above paragraph, should be followed as closely as practicable. The material excavated from the seepage ditches should be deposited on the side away from the levee, and material excavated from the off-take ditches should be deposited in such a manner that it can later be used as material for capping, if necessary. In no case should an attempt be made to cut slope drains until seepage actually appears. All traffic, animals and personnel should be kept off seeping side slopes.

In the event of a sudden draw down failure, loading the toe of the levee similar to the techniques described for through-seepage control can be used. If underwater placement becomes a problem, a temporary earth-filled setback levee may be the only solution.

## 7.5 Sloughs and Slides

Where seepage appearing high on the levee slope cannot be controlled by seep drains, and the condition grows progressively worse, there is danger that a slough or slide may develop. A slough is a condition in which the slope is excessively wet and soggy and is inclined to flow or fall away from the slope and heave or pile up at the toe. A slide is more apt to occur on steep slopes even when the soil does not appear to be extremely wet. In a slide, the slope breaks away in a clearly defined crack or cleavage plane and moves outward taking the toe of the embankment. In any case, where it appears that slope failure is likely or has occurred, the recommended treatment is reinforcement in the form of a buttress on the berm below the slide, tapering up over the failure. A brush or board mattress is always placed under the buttress and constructed in such a manner that it will permit drainage, provide a stable but flexible base for distribution of uniform pressure, bridge the failure, and anchor it against further movement.

## 7.6 Under-seepage

Excessive underseepage can result in what is known as a sand boil. The following is a discussion of methods to treat sand boils. Piping is an extreme condition caused by excessive underseepage in which foundation materials (soil) are transported from beneath the levee. Unless corrective actions are taken, a solution channel or "pipe" may develop and enlarge to the point where the levee could fail. Early treatment of sand boils found to be transporting soil materials is the best insurance against a piping condition from developing.

The most effective method of controlling a sand boil is to reduce the head of water on the riverside of levee. This method; however, is not normally practical because it would take construction of a set back levee to eliminate or lower the river elevation.

The most widely accepted emergency method of treating a sand boil is to construct a ring of sacked earth/sand around the boil, building up a head of water within the ring sufficient to check the velocity of flow and prevent further erosion of sand and silt. The ring should not be built to a height that stops the flow of water because of the probability of building up an excessive local pressure head, causing additional failures and boils nearby.

The accepted method of ringing or sacking (i.e. sand-bagging) a sand boil is described below:

1. The base of the sack ring is prepared by clearing the adjacent ground of debris, vegetation, or other objectionable material, to a width sufficient for the base of the ring. The base should then be thoroughly scarified to provide a watertight bond between the natural ground and the sack ring (a very important step).
2. The sacks are laid in a general ring around the boil, with joints staggered and with loose earth as mortar between all sacks. In general, it has been found that the best results can be obtained by commencing construction of the sack ring at its outer edge and working toward the center.
3. The ring is carried to a sufficient height to stop the flow of soil from the boil. Work is stopped when clear water only is being discharged.
4. A V-shaped drain constructed of two boards or a piece of sheet metal should be inserted near the top of the ring to carry off the water. A spillway made of sandbags can also be used to discharge water from the sandbag ring.

It is impossible to establish exact dimensions for a sack ring. Field conditions in each situation will govern. The diameter of the ring, as well as its height, depends upon the size of the boil and the flow of water from it. Field forces should determine the size of the ring upon consideration of the following:

1. The sack ring should have sufficient base width to prevent side failure. The width should be determined by the contemplated height of the ring, and should be not less than 1-1/2 times the height.
2. The enclosed basin should be of sufficient size to permit the sacking operations to keep ahead of the flow of water. If ground weakness is indicated close to the sand boil, it is well to include the weak ground within the ring, thereby avoiding the possibility of a breakthrough later.

Sand boils at the toe of the levee are sacked in the same manner as those away from the levee, using the levee slope as one side of the enclosure. The seep drains on the levee slope should be constructed to drain the water from the sack ring. If several sand boils appear within a relatively small radius, it is better to enclose the entire group in a sub-levee or single sack ring. If sand boils break out in very low ground or deep ditches, it may be necessary to step down the head of water within the enclosure in two or three steps, by means of outside concentric rings, to avoid a "blowout" near the ring.

An inverted filter is an expedient and economical means to control excessive seepage such as sand boils. A fine sand and/or filter fabric is normally placed over the seepage area with successively larger granular material placed on top. The section will allow the seepage water to be safely removed while holding down or trapping the fine soil material preventing the development of a piping situation. An alternate method of ringing sand boils is by the use of corrugated sheet-steel piling or pipe culverts. Using sheet-steel piling or pipe accomplishes the same task faster than sandbagging but is limited in use by the availability of material, equipment and location of boils. However, circumstances will dictate the system or method most applicable.

There are generally two methods used to control levee failure caused by water flowing through holes in the levee created by burrowing animals: ring the landside opening with sacks the same as for a sand boil, and plug the opening on the riverside with sandbags or plastic sheeting. When a leaking burrow is first observed, effort should be made to first stop the flow from the riverside by spreading a tarpaulin or plastic sheeting on the riverside slope and weighting it down with sandbags. A single sack over the riverside opening of the burrow may stop the burrow from leaking if the opening can be found, but the tarpaulin or plastic sheeting has the advantage of covering a larger area since the intake opening might not necessarily be exactly opposite the discharge opening. The tarpaulin or plastic sheet would probably be more impervious than a sandbag and would therefore provide a better seal.

If the burrow hole is high up on the landside slope with minimal hydraulic head, sacks tamped directly into the outlet will effectively stop the flow. It would be necessary to cut a small notch or bench at the opening to seat the sacks into place. Landside treatment, which may be required if the riverside opening cannot immediately be stopped, is to build a sack ring similar to a boil ring around the landside opening with a sufficient base width to support a ring to a height sufficient to stop the flow of water. This ring differs from a boil ring in that it is required to stop the flow of water. The time, material, and labor required for a ring emphasizes the importance of first attempting to stop the flow from the riverside of the levee structure.

## 7.7 Levee Breaks

Where it is practical and desirable to do so, closure of a break in a levee will reduce the period of inundation of the property inside of the levee, prevent the break from widening, and reduce the damage caused by subsequent rises that may occur before the levee can be repaired. Generally, a break closure should not be attempted on a rising river stage or on an extremely high stage. Conditions could develop such that it would become impossible to accomplish the closure. The time to attempt a closure is on a falling river stage when the velocity and turbulence of the flow through the break has decreased sufficiently to assure complete success of the effort.

There are undoubtedly several acceptable methods of making a closure. However, each closure must be considered as a special case depending on the general location, size, river stage, economics, and the health and safety of the general public. Seepage through and under a levee may be controlled to prevent a levee failure from occurring, however, a significant quantity of water may pond on the landward side of the levee with no place to drain to. In this situation, pumping may be used to prevent damages caused by seep water.

One levee closure plan, which has been developed and successfully used by the Corps of Engineers, is detailed in the following paragraphs. It should be considered for use only under specific situations where the plan and general conditions are complementary and not as a standard procedure for all closures.
The structure is composed of two parts: a timber trestle filled with sandbags to shut off the free flow of water, and an earth-filled mud box landward of the trestle to reinforce and make the structure watertight.

A scour hole usually forms in the break slightly landward and enlarges to the landside. The closure structure should be located far enough away from the edge to allow for enlargement of the scour hole and the structure may be placed either on the landside or the riverside of the crevasse depending on which has the shallower water and the least amount of obstructions. The ends of the structure should join the existing levee well back from the edges of the break to allow for caving while the closure is being built. Trees should be cut off just above the water

surface to prevent any movement of sandbags caused by trees swaying in the wind. The closure should never be started until all required labor and material are available at the site so that closure can be made without interruption. The delay of a few minutes at a critical time may mean the loss of the closure.

Closing a levee break entails considerable danger to personnel working on the closure. Handrails should be installed where needed, the project should be well lighted and employees should wear life vests when working near water. At least two boats, equipped with oars and ring buoys with hand-lines, and manned at all times by experienced operators, should be anchored just below the levee break. An experienced first-aid team equipped with first-aid equipment should be available at all times. In areas where soil material and earthmoving equipment are available, the levee closure can be constructed of earth.

# 8 OTHER CONSIDERATIONS

The topics below are general considerations that apply to all flood risk reduction projects. The reliability of flood protection can change if external variables change (such as climate and land use change increasing the DBFL), design flaws exist (such as inadequate stormproofing of pumping stations), or insufficient maintenance is performed (such as failing to inspect and repair all aspects of the flood protection system).

## 8.1 Climate Change

Climate change should be addressed in the design and evaluation of flood risk reduction projects. Recent USACE guidance addresses the incorporation of sea-level change into project planning and designs (EC 1165-2-212 "Sea-level Change Considerations for Civil Works Programs"). The recently completed Greater New Orleans Hurricane and Storm Damage Risk Reduction System (U.S. Army Corps of Engineers, 2013) incorporated sea-level rise into the design of the levee system (along with subsidence and settlement, and other factors). Changes in flood magnitudes and frequencies are discussed in "Design-Basis Flood Estimation for Site Characterization at Nuclear Plants in the United States of America" (Nuclear Regulatory Commission, 2011).

## 8.2 Large Storm Event Resiliency

Flood risk reduction systems need to perform during large storm events. This means that every aspect of system performance must be considered in advance. Operational procedures that work well under smaller or localized flood events may not work at all when events are larger and cover an entire region. Hurricane events can be used as illustrations. Utilities such as power, phone and internet service may be unavailable for days or weeks. Gasoline may not be available. Highways, rail lines, and public transportation may be unusable due to debris, flooding, landslides or other reasons. It may be impossible to get key personnel, equipment and supplies on site. In short, sources of assistance that are often taken for granted may be totally unavailable. The GNO HDRRS (USACE, New Orleans District website) was built with multiple features to ensure continued operation during future storm events. For example, the pump stations have been stormproofed with backup power and fuel sources. Critical electronic equipment has been raised to avoid submersion. Safe rooms have been provided and strengthened to withstand hurricane-force winds. Emergency food and water are located on-site so that station operators can stay on the job during a storm event. In some cases, the stormproofed area in the pump station includes windows so that operators can view gate operation and water levels from inside during the storm event. Preparation for large storm events must consider a lack of services and supplies on a regional scale, for an extended time period.

## 8.3 Inspection and Evaluation

Flood risk reduction systems require periodic inspection, maintenance, rehabilitation and repair. Many existing levee systems have serious deficiencies, and may fail or require heroic floodfighting measures during a flood event. The level of protection should be re-evaluated periodically, since hydrologic and hydraulic conditions may change over time. These changes often increase flood risk. For example, changes in land use may cause increased peak flows. Additional vegetation or structures in the floodplain may cause higher stages and increase the slope of the water surface profile, leading to an increased risk of seepage and overtopping. Conditions downstream of the project may increase flood levels or erosion. Sedimentation (deposition in the channel or floodplain) may cause increases in stage, for either the entire project or in a localized area. These conditions may cause overtopping at events smaller than the design event, or they may cause overtopping at an unplanned location. Channel migration or channel incision may erode earthen embankments, with the potential to cause breaches during floods. Settlement and subsidence may lower the top of protection, and many factors may change the water surface profile. Sites that were evaluated as "dry" during the planning phase of a project may be subject to flooding under changed conditions at a later date. In summary, projects are not static. Both the project components and activities in the watershed upstream and downstream may change over time. These changes are seldom positive from a flood risk reduction standpoint, and may diminish the reliability of the project. A project should be inspected and evaluated periodically to ensure that the design level of flood risk reduction is maintained.

# 9 SUMMARY AND RECOMMENDATIONS

Flood protection methods for nuclear power plants fall into one of the following five categories: dry sites, exterior (primary) barriers, incorporated (secondary) barriers, temporary barriers, and interior drainage to accommodate locally intense precipitation. Dry sites are located above the Design Basis Flooding Level (DBFL). The DBFL is the maximum water elevation attained by the controlling flood, including coincident wind-generated wave effects. At a dry site, because a site is above the DBFL, all safety-related structures, systems, and components are not affected by external flooding, but are subject to flooding from local intense precipitation. Exterior barriers are natural or engineered structures exterior to the immediate site. Examples of exterior barriers include earthen embankments, sea walls, floodwalls, revetments, and breakwaters. When properly designed and maintained, exterior barriers can produce a site with the flood risk approaching that of a dry site. Incorporated barriers are engineered structures located at the nuclear power plant site/environment interface. Examples of incorporated structures include waterproof walls and sealed hatches.

The U.S. Army Corps of Engineers recommends multiple layers of proven exterior structural barriers for flood protection. Flood protection at nuclear power facilities requires structures that can reliably keep floodwaters from ever coming in contact with critical infrastructure. Currently, adequate data and analyses do not exist in order for the U.S. Army Corps of Engineers to recommend the use of incorporated or temporary barriers as part of a complete flood protection system. The reliability of incorporated and temporary systems are insufficient. However, incorporated and temporary barriers may be used to supplement a complete flood protection system that includes a properly designed and maintained external barrier, an internal drainage system, and redundant pumping stations.

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(b)(6)

April 11, 2013

Case No.: FOIA/PA-2013-0009A - Amended
Date Rec'd: 4/12/13
Specialist: Kilgore
Related Cases:

Bill Borchardt, Executive Director for Operations (EDO)
Darren Ash, Chief Freedom of Information Act Officer
United States Nuclear Regulatory Commission
Washington, DC 20555-0001

SUBJECT: Update to FOIA Appeal 2013-009A

Dear Mr. Borchardt and Mr. Ash:

This letter is an update to a FOIA appeal the NRC acknowledged on March 29, 2013 concerning FOIA/PA 2013-0126. The NRC's acknowledgment letter to that appeal is included as Enclosure 1. Today (2013-04-11), I received a response to FOIA 2013-0126, which I have included as Enclosure 2. Note that this response has come 40 working days after my initial request and 9 working days after I submitted an appeal in accordance with 10 CFR §9.25.

On Tuesday, February 12, 2013 I requested five records from the NRC:

1. ML103490330, Oconee Nuclear Site, Units 1, 2, and 3. Oconee Response to Confirmatory Action Letter (CAL) 2-10-003, dated Nov. 29, 2010
2. ML111460063, Oconee Nuclear Site, Units 1, 2, and 3. Response to Confirmatory Action Letter (CAL) 2-10-003, dated April 29, 2011
3. ML100780084, Generic Failure Rate Evaluation for Jocassee Dam Risk Analysis
4. ML101610083, Oconee Nuclear Station, Units 1, 2, and 3, - External Flood Commitments
5. ML101900305, Identification of a Generic External Flooding Issue Due to Potential Dam Failures

My incoming FOIA request can be found in ADAMS as ML13044A487.

Today, I was provided the following documents in your response to FOIA 2013-0126:

1. ML103490330 (released without redactions so not part of FOIA Appeal 2013-009A)
2. ML13099A247 instead of ML111460063 (included with this appeal as Enclosure 3)
3. ML13039A084 instead of ML100780084 (included with this appeal as Enclosure 4)
4. ML101610083 (released without redactions so can be removed from FOIA Appeal 2013-009A)
5. ML13039A086 instead of ML101900305 (included with this appeal as Enclosure 5)

For the records denied, Exemption 7F of the Freedom of Information Act is claimed. I disagree with this decision and in this letter am providing you the reasons for that disagreement so that, if you chose, you can take this information into account when evaluating FOIA Appeal 2013-009A.

I see nothing in the records requested which indicate they were compiled for law enforcement purposes nor do I see anything which would indicate to me that disclosure could reasonably be expected to endanger the life or physical safety of an individual. It appears to me that the NRC is using Exemption 7F as a means to withhold information which it believes may be beneficial to terrorists or saboteurs yet none of the information withheld pertains to security processes or hardware. The information withheld

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merely pertains to the nuclear safety hazards which deficiencies in the Oconee Station's flooding defenses pose to the American public. These safety risks are present due to the risks of natural disasters and latent engineering/construction flaws and have nothing specifically pertaining to terrorist activities.

As a specific example, consider ML100780084, the *Generic Failure Rate Evaluation of Jocassee Dam Risk Analysis*, submitted by James Vail, Fernando Ferrante, and Jeff Mitman on March 15, 2010. This document was a formal write up of analyses done by NRR in 2007/2008 to support Region II's efforts to get Duke Energy to address safety concerns regarding flooding protection at Oconee. In this document, the Reliability & Risk Analysts at NRR estimated the failure frequency of the Lake Jocassee Dam to be 2.8E-4/yr which equates to a 1 in 3600 chance of failing in any given year. Given that the catastrophic failure of the Lake Jocassee Dam would likely lead to a meltdown of the three reactors at the Oconee Nuclear Station, the dam failure rate calculated by Vail, Ferrante & Mitman suggests that the probability of a nuclear accident at Oconee Nuclear Station is ten times what it is at a typical US reactor plant. This is the type of important information which President Obama expects us to share with the American public (see the President's 2009-01-19 memo on Open Government). Yet the NRC did not share this information with the public. Instead, we stamped the Vail et. al. analysis as "Official Use Only – Security-Related Information" and for years never publicly mentioned its existence. Then, in response to a FOIA request by Dave Lochbaum, we released a redacted version of this supposed "Security-Related" report as ML13039A084. The only redaction in this 15 page report was a figure on page 1 showing the generic construction of Jocassee Dam – a figure very similar to what one can find in any Civil Engineering text book. I have included similar publicly available figures as Enclosure 6. Despite the fact that this figure did not provide any insight to terrorists, it apparently kept this important report from the public for nearly three years. On March 25, 2013 I attended a public meeting with Duke Energy in which this very same figure was presented by Duke Energy as a slide (see Enclosure 7). The slide show from this meeting was forwarded to me by Jim Riccio of Greenpeace and was posted by the NRC on their public website (ML13084A072). So this supposedly "Security-Related" figure, which caused NRR to keep the Vail et. al. analysis from the public for nearly three years and which NRR had redact from Dave Lochbaum's FOIA response in February 2013, was by March 2013 being emailed by NRR to Greenpeace and being posted by NRR on the world-wide web.

The world-wide web gets that name for a reason: it is truly world-wide. Iran, North Korea, Pakistan, Saudi Arabia, and the host of other countries which sponsor terrorist activity against the United States have access to this world-wide web. So why can NRR email this generic drawing to Greenpeace and post it on the web for our enemies to see yet must redact it from the version of the Vail et. al. analysis that it sent to David Lochbaum, Tom Zeller and Paul Blanch in response to their separate FOIA requests? Is this figure "Security-Related" or not? If it is, why are we sharing it on the world-wide web? If it is not, why did we keep the Vail et. al. report from the public for nearly three years and why do we still refuse to release it in its entirety? These are rhetorical questions. Please do not delay answering FOIA Appeal 2013-009A due to these questions. I merely wish to point out to you some inconsistencies in your control of information in the event you would like to consider those inconsistencies while addressing FOIA Appeal 2013-009A.

Additionally, information redacted from the documents supplied to me today has already been publicly release to Greenpeace in our 2013-02-06 partial response (ML130520858) to FOIA 2012-0325 (ML12263A087).

Under FOIA Appeal 2013-009A, please release the following three records to me with no redactions.

- ML111460063, Oconee Nuclear Site, Units 1, 2, and 3. Response to Confirmatory Action Letter (CAL) 2-10-003, dated April 29, 2011
- ML100780084, Generic Failure Rate Evaluation for Jocassee Dam Risk Analysis
- ML101900305, Identification of a Generic External Flooding Issue Due to Potential Dam Failures

Again, this letter is an update to FOIA Appeal 2013-009A in response to documents I received today from the NRC. The information I received came in response to FOIA Request 2013-0126 and not FOIA Appeal 20013-009A. I expect FOIA Appeal 2013-009A to be answered within 30 working days from March 29, 2013 (i.e. by May 10, 2013). I am providing the information in this letter for you to consider if you so choose.

Although I live in (b)(6) I work in (b)(6) Please do not send documents to my home in (b)(6) as I will not get them in a timely manner. Please send all written correspondence to me via email at (b)(6) If your processes will not allow you to do this, then please contact me via phone or email and I will come by the FOIA desk to pick up the correspondence.

Very respectfully,

Lawrence S. Criscione

Lawrence S. Criscione, PE
(b)(6)

Enclosures (7)

Cc: Billie Garde, Esq., Clifford & Garde
Louis Clark, The Government Accountability Project
Fernando Ferrante, NRC/NRR/DRA
Jeff Mitman, NRC/NRR/DRA
Dave Lochbaum, Union of Concerned Scientists
Jim Riccio, Greenpeace
Tom Zeller, Huffington Post
Paul Blanch, consultant

## Ferrante, Fernando

**From:** DePaula, Sara
**Sent:** Wednesday, June 12, 2013 11:24 AM
**To:** Ferrante, Fernando; Mitman, Jeffrey
**Subject:** FW: ACTION: FOIA APPEAL 2013-0009A
**Attachments:** ACTION: FOIA APPEAL 2013-0009A; 2013-0009A_DRA_Response.docx

Fernando, Jeff,

Thanks for discussing this with me this morning. This is just FYI on our recommended resolution.

-Sara

**From:** DePaula, Sara
**Sent:** Wednesday, June 12, 2013 11:23 AM
**To:** Craver, Patti; Weerakkody, Sunil
**Cc:** Chung, Donald; Lee, Samson; Boska, John; Wilson, George; Albert, Ronald; Giitter, Joseph
**Subject:** RE: ACTION: FOIA APPEAL 2013-0009A

Hi Patti,

(b)(5)

(b)(5)

Thanks.
-Sara

**Sara DePaula, PE,** Technical Assistant
US NRC – NRR/DRA, Mailstop: OWFN/ 10 C15, Washington, DC 20555
Office: (301) 415-2861 Cellular: (b)(6)
Sara.DePaula@nrc.gov

Outside of Scope



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Outside of Scope

## Chidichimo, Gabriele

**From:** Sancaktar, Selim
**Sent:** Wednesday, June 05, 2013 9:02 AM
**To:** Ferrante, Fernando
**Cc:** Soli T Khericha; Valos, Nicholas
**Subject:** RE: DC Cook - SPAR AHZ Model Questions

Hi Fernando,

The latest excel workbook containing separate worksheets for each fire area modeled shows those three sequences lumped into FRI-FZ6N-T00, which is good, since they have almost the same CCDP as the other sequences in the same scenario.

On the other hand, the draft SPAR-AHZ model has a separate scenario named FRI-FZ-6S-2A, which has these three sequences. Unfortunately, the SPAR-AHZ CCDP of this scenario is lower than the corresponding CCDP from the plant model. Moreover, these sequences have a considerably high total initiating event frequency compared to other sequences currently in the SPAR-AHZ scenario FRI-FZ6N-T00.

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However, I expect that INL will correct it, simply by implementing the mapping shown in the INL workbook I mentioned above (as shown in Table C-2 also), deleting FRI-FZ-6S-2A and adding the initiating event frequency of those three sequences to FRI-FZ6N-T00.

Murphy's hand directed Nick to pick this example, which is actually beneficial to our effort. It shows to me that Tables C-2/C-3 are essential for verifying the mapping, since the CCDPs and CDFs pop out.

There are other little mapping enhancements I think INL is also going to implement.

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Does this explanation make sense to you?

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**From:** Rodriguez, Veronica
**Sent:** Monday, June 10, 2013 1:33 PM
**To:** Ferrante, Fernando; Quinones, Lauren
**Cc:** Weerakkody, Sunil
**Subject:** RE: TAC for International Activities

Fernando – you should use the TAC in yellow to support your meetings and travel arrangements. Time spent in IAEA should also be charged to that no.

Any document that is sent to you for review should be sent out to Lauren. At that point, she will determine if we need to open a TAC or if the one in green should be used. Lauren tracks all actions related to document reviews. This is reported to the LT as part of NRR operating plan for intl activities. Please keep her in the loop.

**From:** Ferrante, Fernando
**Sent:** Friday, June 07, 2013 2:44 PM
**To:** Quinones, Lauren; Rodriguez, Veronica
**Cc:** Weerakkody, Sunil
**Subject:** TAC for International Activities

Lauren,

Sorry to pepper you with questions, but I am starting to support the IAEA effort on tsunami PRA and I was wondering which TAC to use. Should I create one or should I stick with either ME4239 or [redacted]? I would greatly appreciate it if you could let me know.

Thank you,

Fernando Ferrante, Ph.D.
Office of Nuclear Reactor Regulation (NRR)
Division of Risk Assessment (DRA)
PRA Operational Support Branch (APOB)
Mail Stop: 0-10C15
Phone: 301-415-8385
Fax: 301-415-3577

## Chidichimo, Gabriele

**From:** Luis Francisco Ibarra <luis.ibarra@utah.edu>
**Sent:** Friday, June 07, 2013 5:33 PM
**To:** Ferrante, Fernando
**Subject:** RE: Waste Confidence Link + Dam Failure Frequency Papers
**Attachments:** (b)(6)

(b)(6)

(b)(6)

(b)(6)

Luis

**From:** Ferrante, Fernando [mailto:Fernando.Ferrante@nrc.gov]
**Sent:** Friday, June 07, 2013 2:24 PM
**To:** Luis Francisco Ibarra
**Subject:** RE: Waste Confidence Link + Dam Failure Frequency Papers

Luis,

(b)(6)

Thank you,

Fernando Ferrante, Ph.D.
Office of Nuclear Reactor Regulation (NRR)
Division of Risk Assessment (DRA)
PRA Operational Support Branch (APOB)
Mail Stop: 0-10C15
Phone: 301-415-8385
Fax: 301-415-3577

**From:** Luis Francisco Ibarra [mailto:luis.ibarra@utah.edu]
**Sent:** Friday, June 07, 2013 4:23 PM
**To:** Ferrante, Fernando
**Subject:** RE: Waste Confidence Link + Dam Failure Frequency Papers

Fernando



B8

(b)(6)

Luis

**From:** Ferrante, Fernando [mailto:Fernando.Ferrante@nrc.gov]
**Sent:** Friday, June 07, 2013 10:58 AM
**To:** Luis Francisco Ibarra
**Subject:** Waste Confidence Link + Dam Failure Frequency Papers

Luis,

Aqui te mando un link sobre Waste Confidence. A partir de ahi tenes links a toda una série de documentos.

http://www.nrc.gov/waste/spent-fuel-storage/wcd.html

Tambien estan los papers sobre dam failures.

Thank you,

Fernando Ferrante, Ph.D.
Office of Nuclear Reactor Regulation (NRR)
Division of Risk Assessment (DRA)
PRA Operational Support Branch (APOB)
Mail Stop: 0-10C15
Phone: 301-415-8385
Fax: 301-415-3577

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**From:** Lawrence Criscione (b)(6)
**Sent:** Thursday, June 13, 2013 11:23 PM
**To:** Hirsch, Pat; Albert, Michelle
**Cc:** Sealing, Donna; Vietti-Cook, Annette; CHAIRMAN Resource; Borchardt, Bill; Ash, Darren; Bell, Hubert; Doane, Margaret; Boska, John; Wilson, George; Tom Zeller; paul@times.org; Jim Riccio; Dave Lochbaum; jruch@peer.org; Louis Clark; tomd@whistleblower.org; Billie Garde; sshepherd@cliffordgarde.com
**Subject:** FW: Response to your letter

Pat,

Thank you for your June 13, 2013 reply to my May 24, 2013 letter.

First, I recognize that the NRC is, of late, putting forth significant efforts to respond to the many related FOIA requests and appeals that I and others have submitted regarding the Jocassee/Oconee issues. I recognize these efforts require coordinating among multiple offices within the NRC to ensure that we are taking a consistent approach. I appreciate these efforts and I would like to see us as an agency put the time forth to get these issues

resolved correctly, even if that means not meeting some of the deadlines prescribed by the Freedom of Information Act.

Not all FOIA requests are equal. Some (e.g. FOIA 2012-0325) are impossible to answer in the 30 working day window prescribed by the FOIA. However, others (e.g. 2013-0126, 2013-0127, 2013-0128) can readily be answered, especially when they consist of documents already redacted and release.

I don't know if you have noticed a pattern (I have certainly not tried to hide it), but most of the documents I have been requesting have already been requested - and released - by others (e.g. Koberstein, Zeller, Riccio, Lochbaum). I have been doing this because I do not agree with the redactions we have made to the documents released BUT under the law have no authority to challenge redactions made to the FOIA responses to others. Therefore, in order to appeal these redactions, I must first request the documents myself (since I can only appeal my own FOIA responses). It has been frustrating to me that my FOIA requests cannot be responded to within 20 working days when they consist entirely of documents that have already been reviewed and released under earlier FOIA's. It would help the overburdened NRC technical staff immensely if the FOIA office had a process for easily flagging documents that have already been released in redacted form and immediately responding to those FOIA requests without burdening the technical offices.

Although I am somewhat sympathetic to the challenges facing the technical and FOIA staffs in meeting the time commitments prescribed in the Freedom of Information Act, please note that much of the reason these time limits cannot be met are because of our own flawed processes:

1. The woefully disjointed guidance that exists for the determination of SUNSI.
2. The decision to not require portion marking on "Official Use Only" records
3. The decision to fail towards secretive withholding instead of fail towards transparent release
4. The decision to limit the types of documents routinely released

**With regard to item 1**: Management Directive 12.6 is from 1999 (i.e. two years prior to the drastic information handling changes resultant from the 2001-09-11 attacks) and is woefully out of date as evidenced by the need to sort through conflicting guidance in SECY papers, policy statements and intranet announcements to resolve significant questions. On October 26, 2012 I wrote an 8 page email to my union representation advising them of the poor condition of the guidance for Official Use Only information. I also wrote a two page email on October 25, 2012 to NRC Facilities Security (the program owner for MD 12.6) detailing some of this conflicting guidance. Both these emails were captured in internal NRC ADAMS as ML12313A059. These emails had been meant to point out a problem in the hopes of reaching a dialogue to produce solutions; they were not merely meant to be finger pointing. However, thus far no dialogue has ensued and instead the NRC has labeled ML12313A059 as "Allegation Material". As typical of the so-called "allegations" which others have submitted to the Inspector General in my name, no one investigating it has yet engaged me to discuss it. Since my Office Director and my Union President have been unreceptive to my concerns, I do not expect you to engage me to address them either. But if anyone is interested, my concerns regarding the marking and handling of SUNSI are provided in ML12313A059.

**With regard to item 2**: Secret and Top Secret documents must be portion marked. When looking at a Top Secret document, it is readily apparent which paragraphs cannot be released (they are marked with either a "(TS)", "(S)", or "(C)") and it is readily apparent which paragraphs can be released (they are marked with "(U)" for "Unclassified"). This methodology was not prescribed to make FOIA releases easier; it was prescribed in order to protect classified information. Consider an environment in which

Top Secret documents were not portion marked. In such an environment, individuals working with the documents would not be definitively certain what exactly was classified and what was not. If an individual was attempting to prepare a power point presentation with an overall classification of "Secret" and wanted to ensure there was no "Top Secret" information included, without portion markings he would need to use his own individual judgment and individual interpretation of the classification guidelines when reviewing his Secret and Top Secret references. Not only is this tedious, but it is ripe for error. Having a trained classifier portion mark the paragraphs when the document is written will ensure the individuals utilizing the information definitively understand the classification level of the various pieces and do not need to rely on individual judgment and interpretation.

If the Jocassee/Oconee documents which I requested under the Freedom of Information Act had been portion marked, then they could not only have been readily redacted for release (and thus ensure the agency meets its time commitments under the Freedom of Information Act) but there would have been none of the inconsistencies that have been rampant between the information provided to Green Peace (Riccio) yet withheld from the Union of Concerned Scientists (Lochbaum) and the *Cascadia Times* (Koberstein). The reason for the inconsistencies which have been bogging down your OGC staff of late is because the varied technical staffers in NRR have had to individually use their judgment and interpretation of the highly disjointed SUNSI guidance to decide what can and cannot be released instead of relying on portion markings supplied by a trained SUNSI designator.

Requiring portion marking places the burden upon those who wish to withhold information in that in order to withhold information the individual desiring secrecy must specifically state what portions of a document are "Official Use Only" and why. Not requiring portion marking places the burden upon those who wish to transparently share information in that once an entire document or even an entire issue (e.g. the flooding concerns at Oconee) has been designated "Official Use Only" the individual desiring "Open Government" must specifically justify - often to several concerned parties any one of which can un-informedly veto the decision - why a particular piece of information can be shared with the public.

**With regard to item 3**: We are no longer the AEC. We do not build nuclear weapons and run enrichment facilities - those functions of the AEC were relegated to the Department of Energy which, by the nature of its mission, must be secretive. We are the NRC. We regulate the commercial nuclear industry. The public must be able to trust our ability to be an impartial and competent regulator. Secrecy is as fatal to that public trust as transparency is vital to it. If a mid-level bureaucrat (e.g. George Wilson or John Boska) believes an important safety vulnerability (e.g. a potential Fukushima-style scenario in South Carolina due to a dam break) must be kept from the public due to concerns regarding dam security, then he needs to be challenged. We need to make sure that, not only is the security threat real (i.e. it is not mere "*speculative or abstract fears*"), but also that its secrecy takes precedence over our vital mission of transparently informing the American public (which includes elected decision makers, emergency responders, concerned homeowners, etc.) of potential safety concerns arising at nuclear facilities we regulate. Controlling security-sensitive information is important, but it is not of such importance that it must be our conservative default position. Much harm can be done by secrecy to not only our public confidence but also to our ability to proactively stir internal and external debate regarding important safety topics such as flooding due to dam failures. A mid-level bureaucratic should not be able to squelch our mission of transparency by taking an overly conservative stance on what can and cannot be publicly released. At the NRC we need to default to transparency and require those desiring secrecy to rigorously make their case.

With regard to item 4: In 10 CFR § 9.21 we list 6 records of NRC activities that are available for public inspection and copying. We need to expand that list to include:

(7) all correspondence between the NRC and its licensees concerning inspections, including correspondence following through on issues which arise during inspections

(8) all correspondence between the NRC and its licensees concerning license amendments, including correspondence following through on issues which arise during the evaluation of license amendments

(9) all correspondence between the NRC and its licensees concerning allegations, including correspondence following through on issues which arise as the result of the investigation of an allegation

It is unconscionable that our correspondence with a licensee (Duke Energy) regarding a significant safety concern (the Jocassee/Oconee flooding issues) has been kept from the public for six years. Even if Wilson and Boska are right and all information regarding "*dam failure probabilities, specifics of nuclear power events caused by dam failure, and flood elevations resulting from dam failure*" must be withheld from the public due to security concerns, that does not justify withholding six years worth of correspondence on the issue. All the documents I have requested under the FOIA should have been portion marked and the non-Official Use Only portions should have been voluntarily released by the NRC so that the American public would have at least known about the non-security sensitive aspects of the issue. The American public deserves to be aware of our correspondence with a licensee regarding a significant safety concern. It is my opinion that NRR withheld this correspondence for malicious reasons. It is my position that NRR found it embarrassing that the NRC did not have a ready solution to the flooding concerns at Duke Energy and welcomed the "Official Use Only" designations - required by the supposed security concerns - which prevented the voluntary release of this information to the public. I am not stating this to you as an allegation; I am merely informing you of my position. Were 10 CFR § 9.21 to include official correspondence with licensees as documents routinely made public, it would have gone a long way to ensure transparency on the Oconee/Jocassee flooding issue.

I do not need a response from you on the above four items. They are merely my observations to you and you can take them for what they are worth. However, I would like you or Ms. Albert to provide me the following:

- A date when I can expect to receive a response to FOIA request 2013-0129 and FOIA appeal 2013-013A concerning emails between the NRC and other federal agencies (USACE, FEMA, FERC, DHS, TVA) regarding redactions to the GI-204 Screening Analysis Report.
- A date when I can expect to receive a response to FOIA request 2013-0127 and FOIA appeal 2013-010A concerning a copy of my 2012-09-18 email and attached letter to Chairman Macfarlane
- A date when I can expect to receive a response to FOIA appeal 2013-004A concerning redactions to ML081640244 contained in ML12363A132, redactions to ML082750106 contained in ML12363A129, redactions to ML090570779 contained in ML12363A133, redactions to ML091380424 contained in ML12363A134, and redactions to ML092020480 contained in ML12363A135
- A date when I can expect to receive a response to FOIA appeal 2013-006A concerning redactions to ML110740482 contained in ML12188A239
- A date when I can expect to receive a response to FOIA appeal 2013-009A concerning redactions to ML111460063 contained in ML13099A247, redactions to ML100780084 contained in ML13039A084, and redactions to ML101900305 contained in ML13039A086
- A date when I can expect to receive a response to FOIA appeal 2013-011A concerning redactions to ML091170104

I am at the stage in the process where my next step is to sue in federal court to obtain your responses to my FOIA appeals and to contend any disagreements I have with those responses. I was planning on preparing such a suit this weekend and filing it next week. However, in light of the agency's recent engagement with me (i.e. your June 13, 2013 letter) and in light of your stated efforts regarding "*coordinating among multiple offices within the NRC to ensure that we are taking a consistent approach*". I am willing to forgo the filing of a FOIA suit provided you can provide me with reasonable dates for the completion of responses to my FOIA appeals and requests mentioned in the bullet-ed items above. Please note that I consider reasonable dates as dates which fall within June or July of 2013. The appeals mentioned above are already fourteen to six weeks old and I believe expecting an answer within the next 30 working days (i.e. six weeks) is wholly reasonable on my part.

In writing this response to you, I noted some errors in my May 24, 2013 letter. Attached is a revised copy with changes to pages 3, 5 and 6. I apologize for any confusion my errors may have caused.

Very respectfully,

Larry Criscione
(b)(6)

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From: Pat.Hirsch@nrc.gov
To: (b)(6)
CC: Donna.Sealing@nrc.gov; Michelle.Albert@nrc.gov
Date: Thu, 13 Jun 2013 12:47:58 -0400
Subject: Response to your letter

Pat Hirsch

Assistant General Counsel for Legal Counsel,

Legislation and Special Projects

Alternate Agency Ethics Official

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Nuclear Regulatory Commission

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U.S.NRC

Protecting People and the Environment