# Cost–effective Numerical Modeling for Evaluation of Overtopping Protection Systems

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*ABSTRACT*

*Case studies that focus on exceedance of the design flood event and the resulting overtopping conditions may be used to provide cost-effective analysis of overtopping protection systems. For this case study, conditions of levee overtopping are evaluated for the Probable Maximum Flood (PMF), with a two dimensional flow model (FLO-2D) used to represent the flood event in a relatively flat area susceptible to flash flooding. The model calculates the duration of overtopping and the resulting flood depths and velocities. The model outputs are utilized to evaluate the effectiveness of riprap as potential overtopping protection.*

***Keywords:*** *dam, overtopping, erosion, modeling, FLO-2D*

## 1. INTRODUCTION

The area of the site considered in this case study is characterized by topographically flat sand, gravel and rock plains between mountains of boulder and gravels (Figure 1). Due to the sensitive nature of the site, exact specification of the site location is not provided. Even though annual rainfall totals are relatively low compared to other less arid locations, flash flood events can occur. Due to agricultural, transportation, residential, and industrial development, runoff in these desert areas may be more concentrated than for conditions prior to development.



Figure 1: Desert Valley Plain of Watershed near the Levee

The FLO-2D surface water flow model *(FLO-2D Software Inc., 2012)* is used to calculate potential flood flows at a levee, which provides flood protection. The model is used to represent conditions of the PMF, resulting from a Probable Maximum Precipitation (PMP) event. The PMP is the estimated theoretical maximum depth of precipitation that can occur at a time of year over a specified area. For this analysis, the time varying intensities of the six-hour PMP are applied uniformly over every active grid cell represented in the FLO-2D model.

Riprap design equations are used to size riprap as an erosion protection measure for the levee and compare the design riprap size to that of the existing levee. A Monte Carlo simulation is conducted to account for uncertainty in the overtopping flow rates, embankment conditions, and coefficients used in equations to size riprap.

For this analysis, the failure of the levee is considered from the perspective of erosion. Other potential failure modes, such as under cutting, slope stability or sliding, are not considered.


## 2. METHODS


### 2.1. Rainfall

The maps of the area-specific PMP depths were developed utilizing more recently reported extreme events than those reported in HMR-51 *(NOAA, 1978)*. The revised maps for PMP estimates were developed using methods of moisture maximization and transpositioning as described in HMR-51 *(NOAA, 1978)* and HMR-52 *(NOAA, 1987)*. The revised maps used in the estimates of the PMP depth for this analysis yield lower values than those previously recommended for this area based on the HMR reports.


### 2.2. Two Dimensional Flow Model

By using a two-dimensional (2D) flow model, such as FLO-2D, floodwater is routed in a natural manner without being "forced" to flow in predefined directions as would occur when specifying cross section locations for a one-dimensional (1D) model. FLO-2D is a physical based process model that routes rainfall-runoff and flood hydrographs over unconfined flow surfaces or in channels using the dynamic wave approximation to the momentum equation *(FLO-2D Software Inc., 2012)*. Additionally, the FLO-2D Basic model is approved by the Federal Emergency Management Agency for use in Flood Insurance Studies *(FEMA, 2013)*.

A model grid size of 25 feet (ft) by 25 ft is used in the 2D model analysis. Topographic data in the form of a Digital Elevation Model (DEM) are used in the model setup. For this project, a State Plane horizontal projection of 1983 and the National Geodetic Vertical Datum of 1929 (NGVD29) are used. The digital elevation raster data are provided with a 10 ft by 10 ft grid spacing. These data are imported into FLO-2D, and elevations of the raster data points falling within the larger model grid cells are averaged to obtain a representative topographic elevation for each cell.

The DEM is modified to effectively remove the levee from the topography. The levee is represented using the levee feature within the FLO-2D model. This provides a more precise representation of the levee crest elevations than would be approximated using elevations from the DEM.

In order to reduce the numerical effort required for this analysis, the watershed, which drains from north to south, is divided into two sections. HEC-HMS *(USACE, 2000)* is used to route runoff from the north portion of the watershed (which is represented using multiple subbasins) and FLO-2D is used to represent runoff through the southern portion of the watershed. This HEC-HMS runoff is used as a boundary condition for the FLO-2D model representing the south area of the watershed.


### 2.2.1. Boundary Conditions

The watershed boundary is defined by topographic divides that are in some cases shallow, with the potential for runoff to cross these boundaries for the extreme conditions of the PMF. Outflow boundary conditions are specified along the eastern and western extents of the model domain. Along most of the east and west borders of the domain, runoff flows towards interior model cells. Runoff that flows towards the east or west boundaries represents excess runoff that crosses into adjacent watersheds. Discharge leaving the watershed does not flow along the levee of concern in

this investigation. A separate analysis was conducted, which determined that adjacent watersheds did not contribute runoff into the watershed containing the flood control levee.

At the northern extent of the model domain, an inflow hydrograph is specified in the FLO-2D model as previously described. The inflow hydrograph is developed independently using a HEC-HMS *(USACE, 2000)* simulation that includes rainfall losses, run-off transformation, and delays for stream routing.

The southern extent of the model domain is south of the levee and upstream of a river. A model outflow condition is specified along this boundary.

Rainfall over the model domain is specified in inches/hour and is based on the results of the PMP event analysis described in the Rainfall section.

Infiltration is calculated using the Green-Ampt method *(FLO-2D Software Inc., 2012)* and allows water to exit the model domain vertically.

For the arid conditions of this study area, zero base-flow conditions are assumed.

## 2.2.2. Roughness Coefficients

Manning's roughness coefficients are assigned using landform categories based on aerial photographs and site visit.

The following generalized landforms are identified:

- Unpaved, Bare Soil and Gravel area assigned a Manning's roughness of 0.016.
- Scattered Weeds and Shrubs are assigned a Manning's roughness of 0.070.
- Buildings and Pavement are assigned a Manning's roughness of 0.012.

The Manning's roughness coefficients listed above are within the expected range reported in the literature *(Brater and King, 1982; FLO-2D Software Inc., 2012)*,

Areas of standing water are also identified using aerial photographs and a site visit, and results of trial simulations; these areas are assigned a Manning's roughness of 0.065 to maintain numerical stability during the simulations.

## 2.2.3. Model Stability and Error Checks

After completion of the FLO-2D model simulation stability and error checks are undertaken to insure that the simulation results are reliable. These checks include the following:
- Volume conservation is checked to ensure that the error in the calculated water budget is less than 0.001 percent.
- Numerical surging is evaluated and addressed for the model simulations. Numerical surging is an instability caused by a mismatch between the flow area, slope, and roughness *(FLO-2D Software Inc., 2012)*. A reduction in model time step can resolve this problem. Alternatively, some adjustments to the topography, slope or Manning's roughness coefficients are applied at spot locations (i.e., single model cells) to reduce or eliminate flow surging.

## 2.3. Wind Wave Effects

The two-year maximum sustained wind speed (MSW) and maximum fetch are used to evaluate the available free-board during the PMF. The wind speed is calculated using procedures defined in Part II Chapter 8 of the United States Army Corps of Engineers (USACE) Coastal Engineering Model (CEM) *(USACE, 2008)*. The maximum fetch length and water depth are based on the FLO-2D model results.

Wind data obtained from the National Climatic Data Center (*NCDC, 2013*) is used. The Fisher-Tippett Type I and Weibull distributions are used to estimate the two-year-return annual wind speed as described in the CEM (*USACE, 2008*). The wind speed of the upper 95-percent interval of the worst case distribution is used to calculate run-up. The two-year MSW speed is used to calculate the drag coefficient and friction velocity using Equations II-2-36 of the CEM *(USACE, 2008).*

The significant wave height is calculated using the fetch and MSW with Equations II-2-36 of the CEM (*USACE, 2008*). Calculated wave periods are compared to the limiting wave periods calculated using Equation II-2-39 of the CEM *(USACE, 2008).* Should the calculated wave period exceed the limiting wave period, then subsequent calculations are done using the limiting wave period.

## 2.4. Wave Run-up

A measured slope of 34 percent along the levee embankment is considered a steep slope and the embankments have a riprap lining. For these conditions, the two-percent run-up ($R_{2\%}$) is calculated as described in the Part II, Chapter 4 and using Equations II-4-1, II-1-15, and VI-5-6 of the CEM *(USACE, 2008).* The run-up height is a function of the calculated wave conditions (Wind Wave Effects section) and the run-up elevation is the sum of the run-up height and still water levels calculated using FLO-2D.

## 2.5. Potential Erosion

The potential for erosion and failure of the levee is evaluated by calculating the flowrate over the levee, determining the required design riprap size to prevent erosion and comparing this riprap size to the actual in-place riprap size. If the actual in-place riprap size exceeds the required design riprap size obtained using the design equations *(Robinson et al, 1998)*, then levee failure with some overtopping during the PMF flowrate is not anticipated.

For the first approximation of flow over the levee, a fixed stage, higher than the levee crest is considered based on the results of a previous FLO-2D simulation, wave height and wave run-up analyses. The broad crested weir equation is used to approximate the flow rate over the levee *(Brater and King, 1987)*:

$$q = 3.2 \cdot H^{1.5} \tag{1}$$

where $q$ is the flow per unit length of the weir (ft$^2$/sec = cfs/ft), and $H$ is the peak height of water above the levee crest (ft) of run-up ($R_{2\%}$). The weir crest is assumed to be flat for this evaluation, and a weir coefficient of 3.2 is selected for use in the calculation of flow over the weir crest based on values reported in *Brater and King (1987).* Note that the value of H is a function of the embankment slope.

An alternative estimate of flow over a levee is based on the equation of *EurOtop (2007, p 137, Equation 7.5)*, which considers that the overflow due to waves is periodic and not continuous. The mean overtopping rate, $q$ is:

$$q = (0.062 \pm 0.0062) * \sqrt{g * H_m^3} \tag{2}$$

where $g$ is the gravitational coefficient (32.2 ft/sec$^2$), and $H_m$ is the significant wave height above the weir crest (ft). Note that the value of $H_m$ as used here is independent of embankment slope. Also note that Eq. (2) provides an estimated q that is a little higher than that provided by *Hughes (2008, Equation 2).*

Overtopping rates estimated using Eq. (1) and Eq. (2) are set equal to maximum discharge rates ($q_{max}$) that maintain riprap integrity *(Robinson et al, 1998)* to solve for the $D_{50}$ providing riprap stability:

$$q_{max} = (0.03926) * D_{50}^{1.89} S_0^{-0.58} \quad \textit{For 0.1} \leq S_o \leq 0.40 \tag{3}$$

where $q_{max}$ is the highest stable unit discharge (cfs/ft), $D_{50}$ is the particle size for which 50 percent of the riprap sample is finer (inches), and $S_o$ is the decimal slope (dimensionless). Eq. (3) is used as recommended for slopes of between 0.167 and 0.4 (FEMA, 2014); the observed slope of 0.34 is within this range. Rearranging Eq. (3):

$$D_{50} = 5.546 * q_{max}^{0.53} S_o^{0.307} \tag{4}$$

$D_{50}$ of the existing levee is estimated to range between 5 and 11 inches based on photographs and field inspection.

Uncertainty in the estimated values of some parameters are considered in Monte Carlo simulations, which are conducted using Crystal Ball ™ (Oracle, 2016). Parameters included in the uncertainty analysis are shown in Table 1. A total of 5000 trials are included in the Monte Carlo simulation for each method of estimating overtopping flow rates (i.e., using Eq. (1) and Eq. (2)).

Standard deviations ($\sigma$) used in the Monte Carlo analysis are approximated from various sources, noting that approximately 95-percent of randomly selected values are expected to fall within the mean ($\mu$) $\pm 2\sigma$ range. Based on aerial photograph and field reconnaissance it is apparent that there is some variability in the embankment slope. The maximum value of the embankment slope is set to the maximum slope recommended by FEMA using Eq. (4) (FEMA, 2014). The standard deviation is set to one half the difference between the maximum and mean values within the range, and the minimum slope value (0.28) is set to $2\sigma$ below the mean. The variability in the broad crested weir coefficients is based on reported empirical results *(Horton, 1907; Tracy, 1957)*. The variability of the coefficient in Eq. (2) is based on the provided range in Eq. (2). That is, it is assumed that the range (0.0062) is within $\mu \pm \sigma$ as indicated in *EurOtop (2007)*. The estimated variability in the coefficient used in Eq. (4) is based on *Robinson et al. (1998, Table 1)*. For all cases the minimum and maximum values are selected to be at $\mu \pm 2\sigma$.

Epistemic uncertainty in the type of probability distribution function (pdf) representing the parameter variability is addressed considering three of the most common pdfs: normal, lognormal and uniform distributions.

Table 1. Parameter of Uncertainty Included in Monte Carlo Simulation

| Parameter | Mean | Standard Deviation | Minimum | Maximum |
|---|---|---|---|---|
| Embankment Slope | 0.34 | 0.03 | 0.28 | 0.4 |
| Broad Crested Weir Coeff. Eq. (1) | 3.2 | 0.05 | 3.1 | 3.3 |
| Wave Overtopping Coeff. Eq. (2) | 0.062 | 0.0062 | 0.0496 | 0.0744 |
| $D_{50}$ Eq. (4) Coeff. | 5.546 | 0.063 | 5.421 | 5.672 |

Note: The value of $H$ in Eq. (1) is a function of the embankment slope.

Potential alternatives for actions related to the results from the flooding and overtopping analysis are considered. A full analysis is not provided; however some aspects of the analysis are discussed at the end of the Results section.


## 3. RESULTS

The calculated six-hour PMP distribution with a cumulative depth of 10.09 inches is illustrated in Figure 1.

For the watershed area represented using FLO-2D:
- Topographic elevations are highest north of the site and decrease progressing south with final watershed discharges to the river, and
- The average slope of the subbasin (running north to south) is approximately 0.0038 ft/ft.

Results from the FLO-2D simulations indicate that the levee would not be overtopped during the PMF event. However, the lowest freeboard for the conditions simulated is 0.42 ft and the floodplain adjacent to the levee is sufficiently wide that further consideration is given to potential wind-wave effects that may cause overtopping.
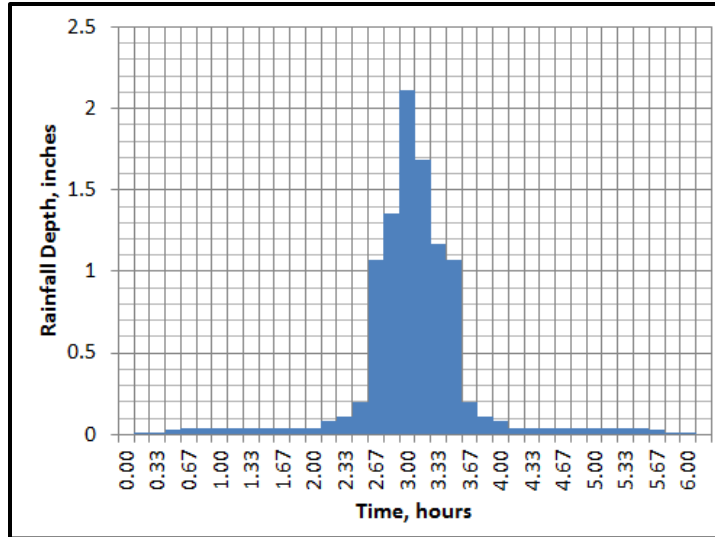
Figure 1.  Temporal distribution of the PMP
Note:  Total cumulative rainfall depth for the event is 10.09 inches.

The runoff discharge hydrograph calculated using HEC-HMS for the watershed area north of that represented by the FLO-2D model has two peaks (Figure 2).  The two peaks are due to variations in time of concentration for the watershed subbasins.  The HEC-HMS simulation runoff is used to describe inflow along the North boundary of the FLO-2D.
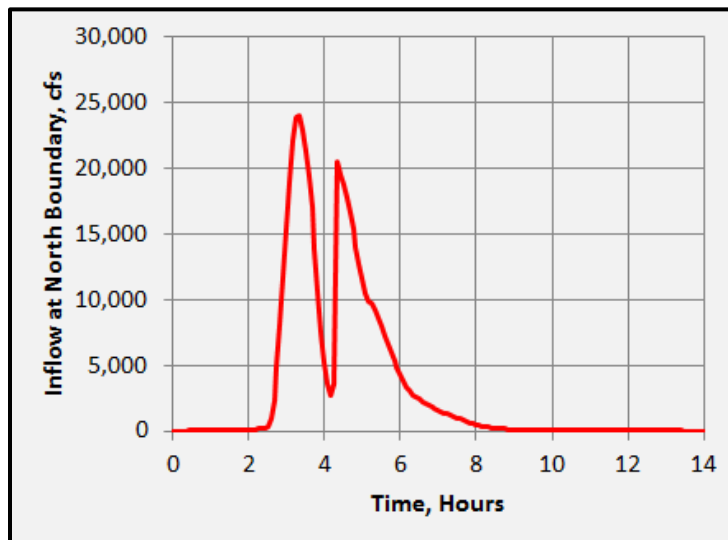


Figure 2.  Runoff Hydrograph for North Portion of Watershed
Note:  This runoff is specified as inflow hydrographs along the north boundary of the FLO-2D model.  The inflow is equally distributed over a width of 1,250 ft centered across an intermittently flowing creek.

The MSW is calculated to be 39.35 miles per hour (mph).  The maximum fetch length used in the calculations is between 1.67 and 1.79 miles.  These lengths are based on the inundation area determined using FLO-2D to represent the PMF.  Table 2 lists the calculated conditions of wave height and period along the levee. Note that the calculated wave periods are less than the limiting wave period, which has a value of 7 to 8 seconds.  In addition, the calculated wave heights do not exceed 0.6 times the water depth at the levee for peak flow conditions of the PMF.  This indicates that wave heights are not limited by water depth adjacent to the levee.

Table 2. Wave Height and Wave Period

| Location | Fetch (miles) | Drag Coefficient | Friction Velocity (mph) | Wave Height (ft) | Wave Period (sec) | Wavelength (ft) |
|---|---|---|---|---|---|---|
| A | 1.67 | 0.00184 | 2.02 | 2.03 | 2.20 | 24.88 |
| B | 1.79 | 0.00184 | 2.02 | 2.10 | 2.26 | 26.06 |

Table 3 summarizes the results of wave run-up calculations. At location B, $R_{2\%}$ run-up heights exceed the levee crest elevation by 1.7 feet. This indicates that overtopping is a potential at the specified location. The duration of conditions during the PMF conducive to run-up overtopping is approximately one hour.

Table 3. Wave Run-up

| Location | Levee side slope [1] | Significant Wave Height/Wave Length | Surf-Similarity [1] | $R_{2\%}$ Wave Run-up Height (ft) [1] | Levee Freeboard (ft) [1] |
|---|---|---|---|---|---|
| A | 34% | 0.08 | 1.17 | 2.0 | 0.37 |
| B | 34% | 0.08 | 1.18 | 2.1 | -1.7 [2] |

Notes: 1. This is the mean value, which varies from one iteration to the next of the Monte Carlo simulation.
2. A negative value indicates the water levels exceed the levee crest elevation.

The results of the Monte Carlo simulation (for each of the three pdf: normal, lognormal, or uniform) show that the required $D_{50}$ to maintain riprap stability is most sensitive to the embankment slope, which contributes about 47 to 99 percent of the variation in $D_{50}$ depending on whether Eq. (2) or Eq. (1), respectively, is considered. The coefficient in Eq. (4) accounts for approximately 1 percent of the variability in the required $D_{50}$ for overflow rates based on Eq. (1). The coefficient of Eq. (2) accounts for approximately 18 to 47 percent of the variability in the required $D_{50}$ for the wave overtopping case.

Figure 3 shows the calculated $D_{50}$ required to maintain riprap stability using Eq. (1) and Eq. (4). Figure 4 shows the required $D_{50}$ to maintain riprap stability using Eq. (2) and Eq. (4). As expected, there is very little difference in the calculated $D_{50}$ assuming normal or lognormal distribution of the parameters listed in Table 1. However, assuming a uniform distribution for the parameters listed in Table 1, results in a significant difference in the shape of the $D_{50}$ probability density curve, which is more uniform in shape as illustrated in Figures 3 and 4.

Several alternatives are considered to address the flooding issue using the results of the riprap analysis:
1. Do nothing,
2. Increase riprap size in area of potential overtopping,
3. Modify levee geometry and redesign riprap in area of potential overtopping, and
4. Consider secondary methods of routing flood waters away from areas of special concern.

In order to better select from the alternatives listed above, the consequences associated with failure of the levee need to be determined. A conservative approach towards a determination of consequences associated with levee failure includes a FLO-2D model simulation that removes the levee in the area of potential overtopping. For this simulation, the resulting area of flooding, flooding depths and warning times can be evaluated. If there is no consequential damage, loss of life, or significant delays in normal activities then the first alternative (i.e., do nothing), the least costly action, might be the most acceptable alternative. If the consequences associated with levee failure are considered significant then alternatives 2, 3 and 4 listed above can be considered.

Regardless of which of the listed alternatives is selected, further analysis to determine the potential for undercutting, sliding and slope stability and other potential failure modes should be considered. Results of these additional analyses may indicate a potential for levee failure at other locations that the overtopping analysis did not identify.
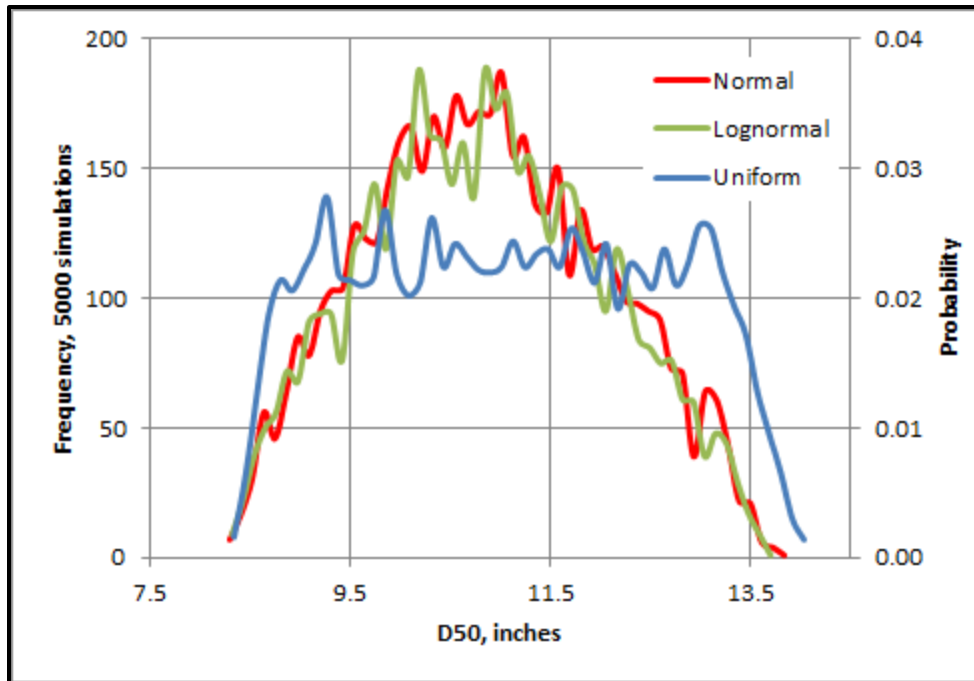
Figure 3.  Monte Carlo Simulated $D_{50}$ Using Weir Equation
Notes:  Estimated using Eq. (1) and Eq. (4) with conditions listed in Table 1. Parameters are assumed to have a normal, lognormal or uniform distribution. The 5000 simulated values are sorted into 50 bins.
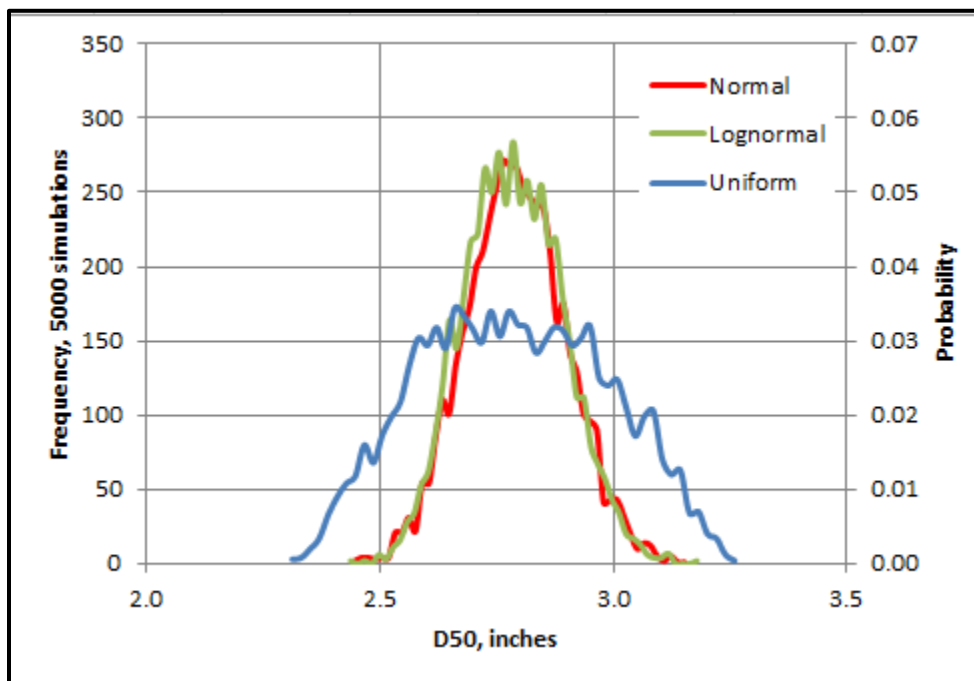


Figure 4.  Monte Carlo simulated $D_{50}$ Using Wave Overtopping Equation
Notes:  Estimated using Eq. (2) and Eq. (4) with conditions listed in Table 1.  Parameters are assumed to have a normal, lognormal or uniform distribution. . The 5000 simulated values are sorted into 50 bins.

## 4. LIMITATIONS

The Monte Carlo simulations are not intended to be comprehensive and do not for example consider uncertainty in parameters used to calculate runoff using HEC-HMS or FLO-2D or uncertainty in wind speeds in the wave height calculations. In addition, there are other aspects of riprap design that might be considered, such as riprap layer thickness, riprap bed materials and potential concentrated flow at the toe of the embankment.

It seems possible that the original design conditions for the levee may have been highly risk adverse. However, the failure of the levee for the analysis provided here is considered from the perspective of erosion, and other potential failure modes such as undercutting, slope stability or sliding, are not considered. Also, additional study is required to more fully evaluate the suitability of the existing riprap.

The present study does not fully identify potential costs, as it does not address consequences and costs associated with levee failure.


## 5. CONCLUSIONS

For this investigation, still water levels during the PMF do not exceed the levee crest elevation. However, potential run-up associated with the two-year return MSW overtops the levee crest elevation at one location along the levee. Once a breach occurs, flows are diverted through the breach and water levels along the levee downstream of the breach will be lower than for conditions without a breach. The 2D model approach using FLO-2D allows the location of embankment modifications to be more focused providing cost savings.

The literature indicates some uncertainty in the rates of erosion associated with overtopping by waves and run-up *(USBR and USACE, 2015)*. It is assumed for a first most-conservative scenario that water levels along the levee crest are at an elevation equal to the run-up elevation to calculate flow rates over the levee crest using the broad crested weir equation. For a second less-conservative assumption, the overtopping rate for waves is calculated using the equations of *EurOtop (2007)*.

A range of possible riprap sizes is estimated based on the equations of *Robinson et al. (1998),* to match the overflow rates for conditions of weir flow and wave overtopping. These estimates are incorporated in Monte Carlo simulations to address some uncertainty in parameters used to calculate the $D_{50}$ required to maintain riprap stability for the conditions of potential overtopping. These simulations indicate a riprap $D_{50}$ of up to 14 inches may be required to maintain levee integrity, which indicates that without further analysis the in-place riprap $D_{50}$ of between 5 and 11 inches based on a field visit (not by thorough sample collection and analysis) may be inadequate.

The multistep approach illustrated in this case study utilizes numerical methods that can be reasonably implemented (i.e., provide a cost effective analysis) to refine the need for further analysis, data gathering, and potential modification to the flood control levee. With some engineering judgment, it might be concluded that the levee will not fail for conditions of the PMF simulated using FLO-2D that indicate wave overtopping for a period of approximately one-hour during a 6-hour PMP event. However, further analysis that more fully considers risk and addresses identified limitations is suggested.


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