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Key Points:
- A feasible risk-based optimization model is developed for single levee planning
- Intermediate levee failure from through-seepage is more frequent than overtopping levee failure
- Optimal levee crown width is more sensitive to varying parameters than optimal levee height

Correspondence to:
R. Hui,
rhui@ucdavis.edu

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Risk-based planning analysis for a single levee

Rui Hui[1], Elizabeth Jachens[2], and Jay Lund[1]

[1]Center for Watershed Sciences, Department of Civil and Environmental Engineering, University of California-Davis, Davis, California, USA, [2]Biological and Ecological Department, Oregon State University, Corvallis, Oregon, USA

Abstract Traditional risk-based analysis for levee planning focuses primarily on overtopping failure. Although many levees fail before overtopping, few planning studies explicitly include intermediate geotechnical failures in flood risk analysis. This study develops a risk-based model for two simplified levee failure modes: overtopping failure and overall intermediate geotechnical failure from through-seepage, determined by the levee cross section represented by levee height and crown width. Overtopping failure is based only on water level and levee height, while through-seepage failure depends on many geotechnical factors as well, mathematically represented here as a function of levee crown width using levee fragility curves developed from professional judgment or analysis. These levee planning decisions are optimized to minimize the annual expected total cost, which sums expected (residual) annual flood damage and annualized construction costs. Applicability of this optimization approach to planning new levees or upgrading existing levees is demonstrated preliminarily for a levee on a small river protecting agricultural land, and a major levee on a large river protecting a more valuable urban area. Optimized results show higher likelihood of intermediate geotechnical failure than overtopping failure. The effects of uncertainty in levee fragility curves, economic damage potential, construction costs, and hydrology (changing climate) are explored. Optimal levee crown width is more sensitive to these uncertainties than height, while the derived general principles and guidelines for risk-based optimal levee planning remain the same.

1. Introduction

Levees partially protect land from flood damage by restraining water from entering the protected area. Globally, levee investment decisions significantly affect public safety and economy. The National Levee Database of the U.S. Army Corps of Engineers (USACE) includes more than 2500 levees, with a total length exceeding 14,500 mi (1 mi/mile = 1609.34 m). The complementary Federal Emergency Management Agency (FEMA) Mid-Term Levee Inventory database includes 29,800 mi of levee [*National Research Council (NRC)*, 2013]. However, even the best levees cannot guarantee protection, given levee failures under various conditions.

Flood risk to economic activity is the likelihood of losing property due to flooding (loss of life is not considered here), and is measured by economic metrics such as direct and indirect costs [*Traver et al.*, 2014]. Flood risk is calculated by multiplying the probability of each failure by its consequences, summed over all events [*Van Dantzig*, 1956; *Arrow and Lind*, 1970; *Samuels et al.*, 2008; *Eijgenraam et al.*, 2014]. Levees reduce, but do not eliminate flood risk.

Risk-based analysis became common for evaluating flood consequences in the twentieth century. In 1960, the Netherlands Delta Plan first used return period (or exceedance frequency) to establish optimal design water levels to protect against flooding, based on a risk-based cost-benefit analysis to identify optimal return periods for overtopping individual dike rings [*Van Dantzig*, 1956; *Van Der Most and Wehrung*, 2005; *Eijgenraam et al.*, 2014; *Kind*, 2014]. Acceptable average return periods for the design of levees and dikes are stated in Dutch law, including four common safety classes: 1250, 2000, 4000, and 10,000 years [*Van Manen and Brinkhuis*, 2005] and two rare classes: 250 (for upstream part of Meuse) and 500 (only for one stretch along the river Rhine). Flood risks considering probabilities and consequences have been established as a preferred basis for levee planning and safety. Starting from 1992, the Technical Advisory Committee for Flood Defence initiated the development of a flood risk approach to more comprehensively calculate probabilities of flooding of dike ring areas [*Van der Kleij*, 2000] and has been widely used [*Baan and Klijn*, 2004; *Klijn et al.*, 2004; *Jonkman et al.*, 2008; *Klijn et al.*, 2008; *Zhu and Lund*, 2009]. The currently revised Dutch law will update the safety standards using flood probability instead of exceedance frequency in the Water Act by 2017 [*Eijgenraam et al*., 2014]. The *USACE* [1996] provided procedures with risk and uncertainty analysis to estimate expected benefits of proposed flood damage reduction plans, quantitatively and qualitatively representing the likelihood and consequences of capacity exceedance, as an extension and expansion of traditional formulation and regulations in other guidance materials [e.g., *USACE*, 2000, 2006].

Most traditional risk analysis of levees only accounts for levee failure from overtopping, but levees often fail before overtopping due to intermediate geotechnical failures [*Wolff*, 1997; *Gui et al*., 1998; *Cenderelli*, 2000; *Foster et al*., 2000]. *Wolff* [1997] modeled multiple failure modes and created a combined failure probability, assuming individual modes are independent, finding that underseepage and through-seepage are the most common intermediate failure modes (before overtopping), which also may trigger failure by erosion and slope instability. Although some studies have analyzed the probability of intermediate levee failure as a function of water level [*USACE*, 1996; *Meehan and Benjasupattananan*, 2012; *Kind*, 2014; *Jongejan and Maaskant*, 2015], they generally do not identify specific levee geotechnical characteristics. The current study extends this approach by explicitly including through-seepage as a representative of overall intermediate geotechnical failure modes in levee risk analysis based on synthetic levee performance curves, parameterizing levee fragility with levee height and crown width.

As lower levees are more likely to fail by overtopping and narrower levees are more likely to fail geotechnically [*Bogárdi and Máthé*, 1968; *Wood*, 1977; *Tung and Mays*, 1981a; *Tung and Mays*, 1981b], levee height and crown width are two major parameters in levee planning. Other significant planning parameters include waterside slope angle and landside slope angle for a general levee with a trapezoidal cross section, as well as levee materials and compaction. With a complete planning study, later design considers practical levee construction (from a geotechnical perspective) to provide protection and benefits consistent with accepted standards. Levee design in the U.S. usually follows federal and local criteria and guidance, such as the 100 year urban flood protection developed by National Flood Insurance Program and FEMA [*NRC*, 2013], Bulletin 192-82 by California *Department of Water Resources* [1982], and Public Law 84-99 by the federal government [*USACE*, 2009], and the 200 year flood protection required by the Urban Levee Design Criteria to meet California's requirements [*Department of Water Resources*, 2012].

Section 2 of this paper describes a risk-based optimization model for single levee planning, including model description, intermediate geotechnical levee failure, and risk-based analysis incorporating overtopping and overall intermediate geotechnical failures. Section 3 presents and discusses illustrative applications of this model for a small rural levee with new levee construction and upgrading a large existing urban levee. Section 4 presents a sensitivity analysis of levee fragility curves that represent the intermediate failure probabilities, analyzes impacts from major economic parameters and from varying flood frequency driven by climate change. Section 5 concludes with key findings.

2. Risk-Based Optimization for a Single Levee

Typical optimization for levee risk analysis is to minimize all flood-related costs, including costs of expected (residual) flood damages and costs of flood protection (here levee construction or upgrade) [*Kind*, 2014]. For this study, a model combining simple representations of hydraulic levee failure and economic cost is used to examine levee planning parameters (height and width) by minimizing annual expected total costs, including expected annual damage and annualized construction costs.

To develop the framework for risk-based optimal levee planning, several assumptions are applied: levee planning is based on minimizing total annual expected net cost; levee material is homogeneous and isotropic, with constant soil conductivity; through-seepage can develop during a flood event with sufficiently pervious levee material and/or sufficiently long flood duration; defects (e.g., cracks or animal burrows) that cause through-seepage are not considered; levee slope instability and surface erosion failures are neglected; spatial correlation of levee and foundation material is ignored; a stationary lognormal flood flow frequency distribution is assumed; Manning's coefficient is assumed constant and independent of flood level, seasonal variation, or other factors; and no impacts exist from climate change, urbanization, or other uncertainties. Analyses below are all based on these assumptions, which would limit the utility of the optimization model and can be addressed in future work.

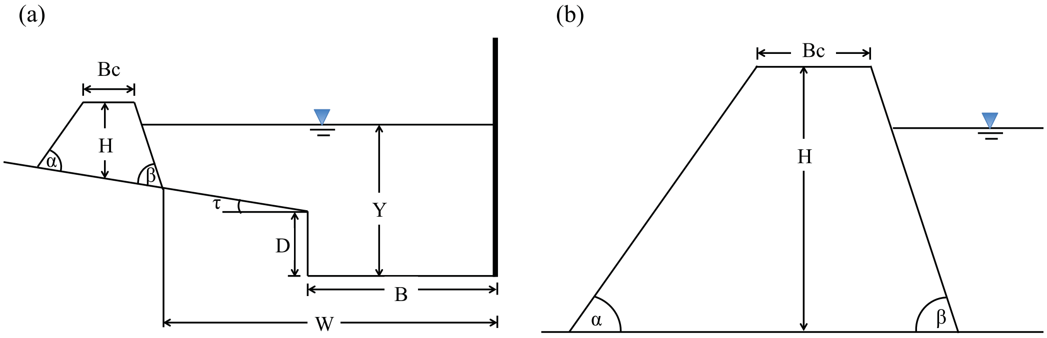

Figure 1. (a) Idealized cross section of a channel with a single levee; (b) the basic schematization of a levee cross section.

2.1. Model Description

This study uses a simple idealized channel with a single levee on one side of a river reach and a high bank on the other side (that never fails) (Figure 1a). B is channel width, W is total channel and floodplain width to the toe of the levee, D is channel depth, Y is water level, τ is slope of the floodplain, α is the levee landside slope, β is the levee waterside slope, H is levee height, and Bc is levee crown width. To simplify the levee optimization considering both overtopping and intermediate geotechnical failures, levee height and crown width are assumed to be the two dominant variables affecting reliability, as surrogates for overtopping and intermediate failures (Figure 1b).

2.2. Intermediate Geotechnical Levee Failure From Through-Seepage

Overtopping failure is assumed to occur when the water level exceeds the top of a levee, with a probability estimated using an annual flood flow frequency distribution. Owing to geometric uncertainty, erosion and flood fighting, overtopping may occur slightly before or above the "overtopping" heights assumed here. Figure 2a shows a general annual flood flow frequency distribution, here assumed to be lognormal. Figure 2b is a sample rating curve, here using Manning's Equation, to relate flow and water level/stage [*Wolff*, 1997]. Climate change could affect the flood flow frequency distribution and optimal levee planning, but is only briefly discussed in this paper [*Zhu et al*., 2007; *Hansen et al*., 2012]. Uncertainty in rating curves can arise from Manning's n, floodplain development, channel geometry, and streamflow measurement [*Pappenberger et al*., 2006; *Di Baldassarre and Claps*, 2010]. *Domeneghetti et al*. [2012] assess rating-curve uncertainty and its effects on hydraulic model calibration. Here we begin with a static optimization model without uncertainties often considered in real cases.

Geotechnical failure modes are often represented by levee fragility curves that graphically summarize how levee failure probability varies with water levels. Here we assume that the levee fragility curves summarize all relevant physical, geological, and hydraulic factors that affect intermediate levee failure likelihood. Figure 2c illustrates three levee fragility curves commonly estimated by professional judgment [e.g., *Wolff*, 1997; *Vorogushyn et al*., 2009], and later used for a minimum crown width and adjusted for wider widths. Levee failure probability for water levels below the toe of levee is zero and above the levee's top is one. Failure probability for a levee in "good" condition remains low when water level is low and increases dramatically when water level approaches the levee height. In contrast, the levee in "poor" condition has a high failure probability even at low water levels. Levees in "fair" condition tend to be in good condition at low water levels, but come to resemble poor-quality levees at higher water levels. The "good" condition is more applicable for planning new levees, while "fair" and "poor" conditions are more applicable for planning of evaluating and upgrading existing levees. The failure probability between the toe and the crest of the levee is uncertain given that these curves are typically based on professional judgment [*Perlea and Ketchum*, 2011]. Many physical and geological parameters can affect levee intermediate failure, for example, levee slopes, soil properties, and water fluctuation. Other ways to provide more precise fragility curves would be through geotechnical experiments or extensive numerical analysis. Intermediate failure probabilities can be estimated for varying water levels to approximate the fragility curve.

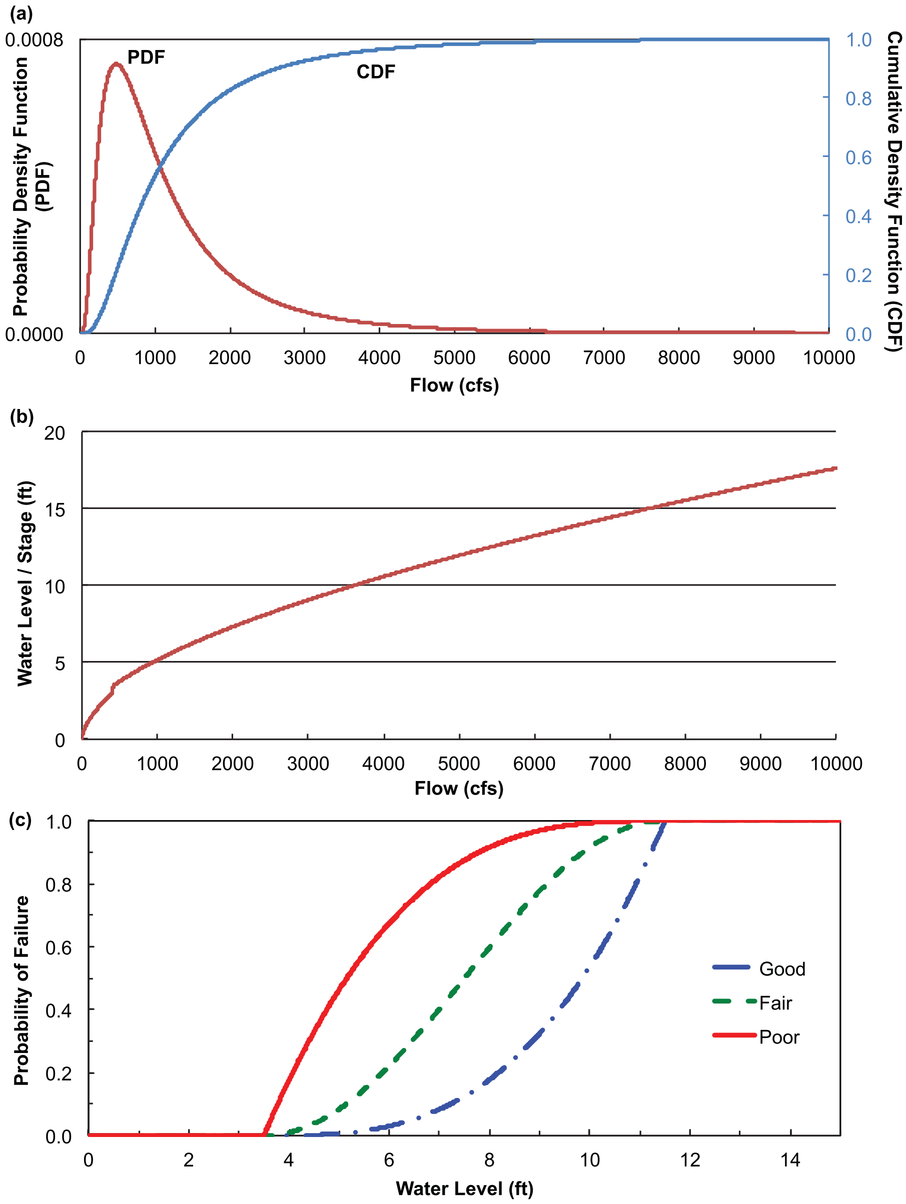

Figure 2. (a) A lognormal annual flood flow frequency distribution; (b) a sample rating curve converting flow to water level or stage using Manning's Equation; (c) sample levee fragility curves for levees in good, fair, and poor conditions.

Combined with the flow frequency curve where lower flows are more frequent, Figure 2c implies a high likelihood of levee failure before overtopping.

Levee fragility depends on levee geometry such as levee height, crown width, side slopes, and properties such as soil conductivity and compaction [*Kashef*, 1965; *USACE*, 2000]. Here in addition to levee height H, levee crown width Bc is a second decision variable because of its influence on intermediate failure performance curves and the wide range of acceptable values [*USACE*, 2006]. Through-seepage is chosen to represent general intermediate failure where theory and solution are comparatively well developed and levee crown width can be a representative plan variable. Underseepage is a common levee intermediate failure [e.g., *Mansur and Kaufman*, 1956; *Turnbull and Mansur*, 1961] when a levee is less permeable than its foundation. Here overall intermediate risk is calculated from through-seepage, assuming sufficiently pervious levee materials that through-seepage exceeds underseepage. For a levee with more underseepage, the fragility curve can be degraded with higher failure probabilities or the problem and model can be reformulated to explicitly represent failure from underseepage.

Crown width can be used to calculate seepage through a levee using, for example, geotechnical relationships given in Schaffernak's solution for through-seepage [*Das*, 2010]. Alternative solutions for through-seepage also could be applied, for example, Casagrande's solution (1932) and Pavlovsky's solution (1931). Independent variables in Schaffernak's solution include water level, levee height, crown width, landside angle, and waterside angle. This model's main assumptions are: (1) the levee's base is assumed to be impervious, implying through-seepage as the primary cause of intermediate failure and disregarding underseepage failure; (2) the waterside slope angle is less than 30° (the selected 2:1 horizontal to vertical ratio satisfies); and (3) the hydraulic gradient is constant and equals the free-surface slope as water flows through the levee according to the Dupuit assumption [*Das*, 2010]. Steady-state through-seepage assumed in Schaffernak's solution would represent longer floods or where protected lands are below sea level, but would overpredict seepage for short-duration floods. Here the flood is assumed to have enough duration for through-seepage to fully develop.

Schaffernak's solution uses L_s, the sloped elevation of the discharging water, and the soil hydraulic conductivity to calculate the rate of seepage per unit length of the levee. The hydraulic conductivity is assumed to be constant for all levee heights and crown widths. Given this assumption, relative seepage rates can be compared using the ratio of the sloped discharge elevations for two crown widths; so the rate of seepage can be calculated as:

$$q = k * L_s * \tan\alpha * \sin\alpha \tag{1}$$

where q is the rate of seepage per unit length of the levee, k is the soil conductivity which is assumed constant in this study, α is the angle of levee landside slope, and L_s is the sloped elevation of the discharging water defined in equation (2).

$$L_s = \frac{d}{cos\alpha} - \sqrt{\left(\frac{d}{cos\alpha}\right)^2 - \left(\frac{Y}{sin\alpha}\right)^2} \tag{2}$$

where Y is the water level, and d is the horizontal distance between the landside toe of the levee and the effective seepage entrance as defined in equation (3).

$$d = 0.3 * \frac{Y}{tan\beta} + \frac{H-Y}{tan\beta} + Bc + \frac{H}{tan\alpha} \tag{3}$$

where β is the angle of levee waterside slope, H is levee height, and Bc is crown width.

The relative seepage rates can be viewed as changes in the likelihood of levee through-seepage failure. At any given levee height, a wider levee would have a smaller sloped elevation L_s and a smaller seepage rate q, and therefore smaller exit velocities and lower through-seepage failure probability. So widening levee crown width decreases the likelihood of intermediate levee failure, which provides a basis for estimating the levee's intermediate geotechnical failure.

For numerical computation rather than theoretical analysis, a base probability distribution of levee failure with the minimum standard crown width is approximated by the following mathematical expressions (equation (4)) to explicitly represent levee fragility curves under good, fair, and poor conditions. Levee intermediate failure probabilities for other crown widths are normalized using a coefficient (COP_{int}) of relative sloped elevation of the discharging water that depends mostly on crown width (equation (5)).

$$P_L(Q) = \begin{cases} \left[\frac{(Y-H_{toe})}{H}\right]^3, & good\ \ levees \\ \frac{1+sin\left\{\pi * \left[\frac{(Y-H_{toe})}{H}\right] - \frac{\pi}{2}\right\}}{2}, & fair\ \ levees \\ 1+\left[\frac{(Y-H_{toe})}{H} - 1\right]^3, & poor\ \ levees \end{cases} \tag{4}$$

$$COP_{int} = \frac{L_s(Bc, Y)}{L_s(Bc\min, Y)} \tag{5}$$

where $P_L(Q)$ = probability of levee intermediate geotechnical failure from through-seepage and H_{toe} = height of the toe of levee. Mathematical formulas to represent levee fragility curves can take other

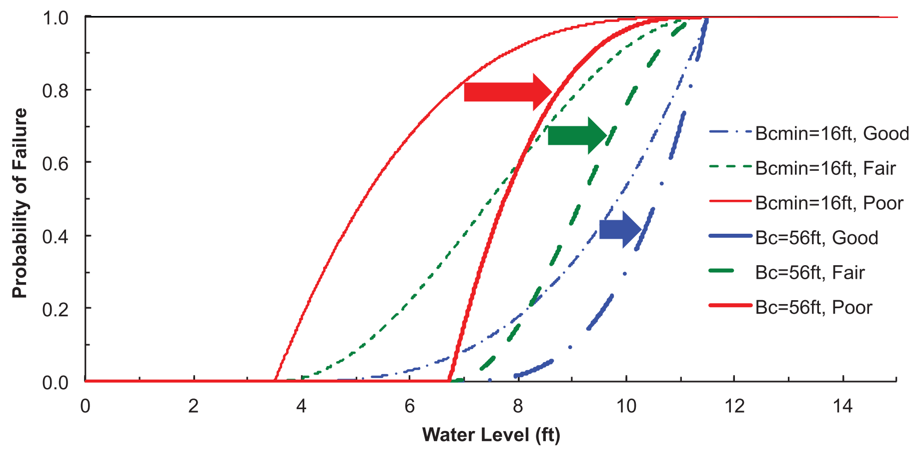

Figure 3. Levee failure probability curves for different crown widths in various conditions.

forms. For example, a linear function can be implemented for fair levees. This study illustrates one way to explicitly incorporate intermediate failure into levee risk-based analysis.

A wider crown width with a smaller sloped elevation L_s will have a smaller COP_{int} and corresponding lower failure probability. For example, given the base levee failure probability for a minimum standard crown width of $Bcmin=16$ ft (1 ft/feet = 0.3048 m), the decrease in levee failure probability for a crown width of $B cmax=56$ ft can be calculated at different water levels. In Figure 3, levee failure probability curves have the same pattern for the same levee condition with either crown width. For the same water level and levee condition, failure probability for $Bc=56$ ft is less than that for $Bcmin=16$ ft. The levee failure probability curves for a larger crown width shift down and to the right from the original case.

The new levee fragility curves depend on both crown width and levee height, but continue to represent the professional judgment in the original levee fragility curves. Sensitivity analysis on this representation of levee fragility is presented later. The above method develops a conceptual mathematical and geometrical representation of the additional effectiveness of larger crown widths on intermediate through-seepage levee failure, which is largely limited by the assumptions in Schaffernak's solution and ignoring material characteristics (e.g., hydraulic conductivity and soil properties). More detailed methods for calculating through-seepage and overall intermediate geotechnical failure probability could be substituted.

2.3. Risk-Based Optimization Model

This model assumes independent overtopping and intermediate geotechnical failures for a given water level. It also assumes no hydraulic uncertainty affecting the relationship between water level and flow. Ignoring hydraulic uncertainty reduces the information available for decision making and can reduce the accuracy of estimated expected damages, which should be avoided when adequate knowledge of the channel is available [*Tung and Mays*, 1981b; *Briant*, 2001]. Considering hydrologic uncertainties only, equation (6) calculates the expected (residual) annual damage cost of the system for combined intermediate and overtopping failures, for constructing new levees or upgrading existing levees. The first term represents the expected damage from intermediate failure when flow is below channel capacity Q_c, while the second term is the expected damage from overtopping failure when flow exceeds the channel capacity.

$$EAD=\int_0^{Q_c}\left[D(Q)*P_q(Q)*P_L(Q)\right]dQ+\int_{Q_c}^{\infty}\left[D(Q)*P_q(Q)\right]dQ \tag{6}$$

where $D(Q)$ = damage cost as a function of flow; Q_c = critical overtopping flow of the leveed channel; $P_q(Q)$ = probability density function of river flow; $P_L(Q)$ = probability of levee intermediate through-seepage failure as a function of flow.

Greater flow usually leads to greater flood damage, but for large floods of deeply leveed floodplains, a fixed damage from levee failure should closely approximate. Since the magnitude of flood flow is accounted in the failure probability, with a constant damage cost equation (6) simplifies to:

$$EAD = D * \left\{ \int_0^{Q_c} \left[P_q(Q) * P_L(Q) \right] dQ + \left[1 - F_Q(Q_C) \right] \right\} \tag{7}$$

where D = constant flood damage cost; $F_Q(Q_C)$ = cumulative density function of flow Q_c. The assumed constant damage cost would involve large uncertainty from both the estimation of the lost economic welfare due to flooding, and the evolving damage potential in a long term. More accurate or elaborate damage cost functions could be used, if available. This static study can also be extended to a dynamic process that accounts for the long-term impacts from floodplain urbanization or climate change [*Zhu et al*., 2007].

The single levee planning can be optimized by minimizing the annual expected total cost (TC) under static condition, which is the sum of the expected (residual) annual damage cost (EAD) and annualized construction cost (ACC).

$$\text{Min } TC = EAD + ACC \tag{8}$$

Annualized construction cost for building a new levee or upgrading an existing levee (with existing height H_0 and crown width Bc_0) is based on levee volume and land area.

$$ACC = \left[\frac{r * (1+r)^n}{(1+r)^n - 1} \right] * (s * c * V + LC) \tag{9}$$

where r = real (inflation adjusted) discount or interest rate; n = number of useful years the levee will be repaid over; s = a cost multiplier to cover engineering and construction administrative costs; c = unit construction cost per volume; $V = L * \left[Bc * H + \frac{1}{2} * \left(\frac{1}{\tan\alpha} + \frac{1}{\tan\beta} \right) * H^2 \right] - L * \left[Bc_0 * H_0 + \frac{1}{2} * \left(\frac{1}{\tan\alpha} + \frac{1}{\tan\beta} \right) * H_0{}^2 \right]$ is the total volume of building a new levee (the second term disappears with $H_0 = 0, Bc_0 = 0$) or upgrading the existing levee along the entire length (L) of the reach; $LC = UC * A$ is the cost for purchasing land to build or upgrade the levee, with a unit land cost, UC, and the additional land area occupied by levee base, $A = L * \left[Bc + \left(\frac{1}{\tan\alpha} + \frac{1}{\tan\beta} \right) * H \right] - L * \left[Bc_0 + \left(\frac{1}{\tan\alpha} + \frac{1}{\tan\beta} \right) * H_0 \right]$ (the second term disappears for building a new levee). Land cost is an additional cost to represent the site-specific expense of purchasing land for levee construction. The long-term operation and maintenance costs of the levees are assumed constant for any planning and not included.

Physical constraints on this optimization include upper and lower limits of crown width and levee height, as well as nonnegativity of all variables. To meet the "level of protection" requirement, for example protecting from a 200 year flood, additional constraints can be added to such levee planning optimization that the overall failure probability or overflow failure probability is no less than or equal to a required exceedance frequency.

3. Model Applications

To show the feasibility and applicability of the model and to derive general implications from the model results, the developed risk-based optimization model is applied illustratively to a small rural levee with new levee construction and a large urban levee with existing levee evaluation and upgrade, roughly based on the Cosumnes River and Natomas in California. Hydraulic parameters and levee dimensions are formulated from previous studies [*Tung and Mays*, 1981b], following standards developed by DWR and the federal government, Bulletin 192-82 and PL 84-99, respectively. All economic values and costs are annualized.

3.1. Model Applications to a Small Rural Levee on Cosumnes River

The Cosumnes River has a median peak annual flow of 930 cfs (1 cfs = 0.0283 m^3/s), a mean annual peak flow of 1300 cfs, a land cost of \$3000 per acre (1 acre = 4046.86 m^2), and an assumed constant \$1.5 million damage cost if the area is flooded [*Maniery*, 1991]. This case examines planning for a new levee.

Adjusted channel geometry and levee-related parameters (Figure 1) include: $B=200$ ft, $W=250$ ft, $D=3$ ft, longitudinal slope of the channel and the floodplain section is 0.0005, Manning's roughness for the channel and floodplain are 0.05, floodplain slope is $tan\tau=0.01$, levee landside slope and waterside slope are set as $tan\alpha=1/4$ and $tan\beta=1/2$, respectively, and total levee length is $L=2640$ ft (0.5 mi). Construction cost parameters are: $c_{soil}=\$1.5/ft^3$, $r=0.05$, $s=1.3$, and useful life of the levee is 100 years. The minimum and maximum crown width standards are $Bcmin=16$ ft and $Bcmax=56$ ft, and the maximum levee height is set as $Hmax=15$ ft.

Because the annual expected total cost of the levee is a function of both levee height and crown width, enumeration of the annual expected total cost over all heights and widths can find the overall optimal levee planning. Figure 4 shows the annual expected total costs and least-cost levee height and crown width, with cost contours for all height and width combinations for good, fair, and poor levee conditions, respectively, given the above site-specific values. A levee is likely to be built to good condition; the fair and poor conditions examined here are for comparison. Levee height and levee crown width both vary in 0.1 ft increments. Contour intervals are \$0.02 million/year. The red dots show the optimal levee height and crown width.

From Figure 4, the annual expected total cost increases when levees become too low since the expected annual damage cost is relatively high, or when levees become too high since the annualized construction cost is relatively high. For similar reasons, levees with too narrow or too wide crown widths greatly increase the annual expected total cost. Minimum annual expected total cost occurs at an optimal combination of levee height and crown width. The contour plots show that the optimal levee height decreases with increasing crown width, for a comparable total cost. As levee crown width increases, intermediate failure probability decreases and therefore decreases the first term of the expected annual damage equation. As levee height increases, the channel capacity increases and decreases the probability of overtopping. Increasing levee height and crown width also increase construction cost. So the optimization should balance costs from flood damage and construction, while balancing the two planning decisions as they have similar impacts on costs. Fortunately, a fairly wide range of near-optimal solutions seems common around the optimal solution. Near the optimum, the height difference for the smallest contour interval (about 2 ft) is much smaller than the width difference (about 20 ft), because of the more likely intermediate failure that depends largely on levee crown width, and affected by the assumed levee geometry and other assumptions.

To show clearly how costs change with levee height and crown width, Figure 4a compares the annualized construction cost, expected annual damage cost, and annual expected total cost for a minimum levee crown width of $Bcmin=16$ ft and a maximum of $Bcmax=56$ ft, for a good Cosumnes levee, for instance. Generally, annual expected total cost is dominated by the expected annual damage cost for lower levees, and by the annualized construction cost for higher levees. The minimum of each total cost curve defines an optimum levee height for that crown width. The optimal levee height is $H^*=6.8$ ft for $Bcmin=16$ ft (also the global optimality in this case), while is $H^*=5.1$ ft for $Bcmax=56$ ft. The annual expected total cost for the maximum crown width exceeds that of the minimum crown width, as a result of a big increase in annualized construction cost and a small increase in expected annual damage cost. Similarly, Figure 5b compares the costs for a levee height of $H=4$ ft and a levee height of $H=6$ ft, for a good Cosumnes levee. General trends of costs curves and conclusions are similar. The optimal levee crown width is $Bc^*=33.4$ ft for $H=4$ ft, while is $Bc^*=19.9$ ft for $H=6$ ft. The annual expected total cost for a levee height of $H=4$ ft exceeds that of $H=6$ ft, due to a small decrease in annualized construction cost and a large increase in expected annual damage cost. Comparing the results in Figures 5a and 5b, changes in levee height lead to greater changes in costs than changes in levee crown width.

Table 1 summarizes the optimal results for the three different levee conditions, as well as for the good levee planning including only overtopping failure to show the importance of considering intermediate failure.

Balancing different failure mechanisms that depend on levee height and crown width is significant in planning for reasonable levee dimensions, although constraints can prevent extreme and unrealistic results. From Table 1, the optimal good and poor levees are constrained by the minimum and maximum crown widths, respectively. The good levee would have a higher optimal height if optimal crown width decreased beyond the lower bound to balance damage and construction costs, while the poor levee would have a lower optimal height if optimal crown width increased beyond the upper bound. The optimal costs, crown width, and intermediate failure probability increase from good to poor levee conditions, differing from the

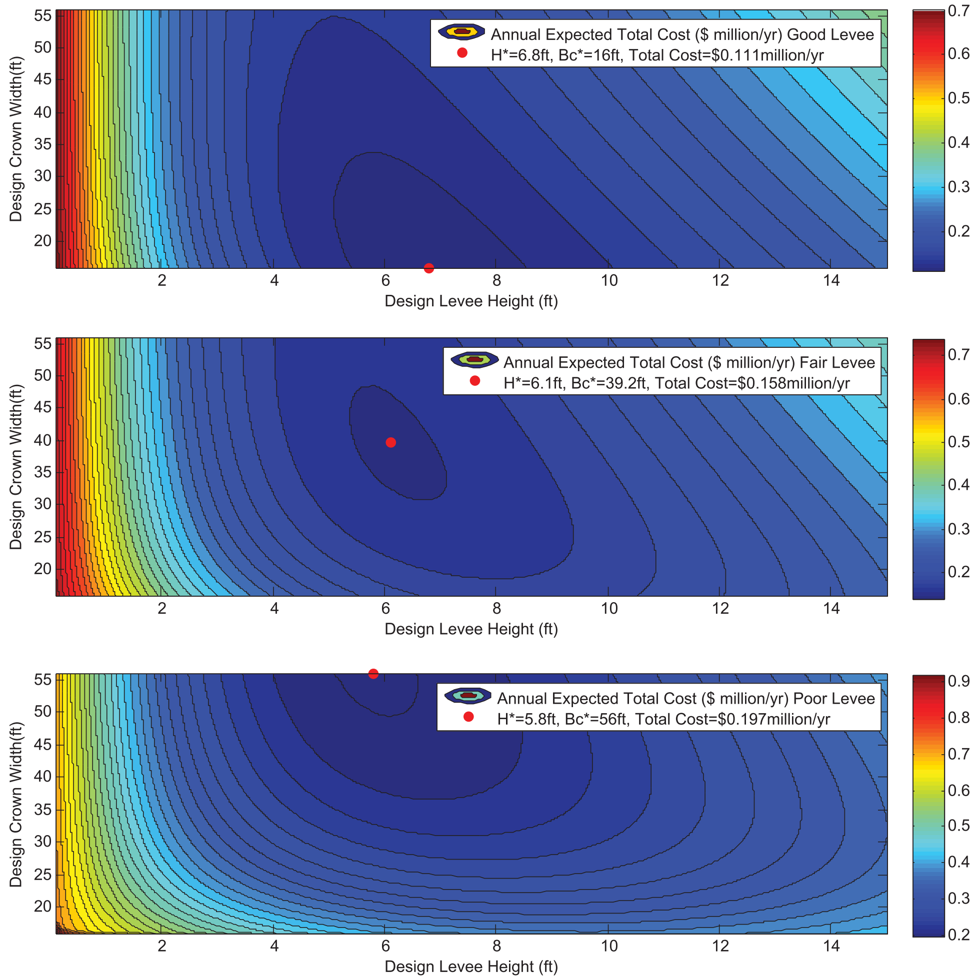

Figure 4. Contour plots of annual expected total costs with $0.02 million/year interval for various levee geometries with 0.1 ft levee height and crown width increments under different levee conditions (rural levee).

trends of others. Compared to big differences in optimum levee crown widths of good, fair, and poor conditions, the optimum levee height remains fairly constant. For the optimal combination, a levee with much narrower crown width has a slightly higher levee height, or a slightly higher levee has a much narrower crown width. This results from the geometry of the levee side slopes. The annualized construction cost as a function of levee volume is more sensitive to levee height than crown width; when a levee height increases by 1 ft, the base width increases by 6 ft ($1/tan\beta+1/tan\alpha=6$), which increases the horizontal distance of the seepage path and decreases seepage-related failures. A bigger change in levee crown width often can substitute for a smaller change in levee height.

From the above analysis, intermediate geotechnical failure is about 75% of overall failure probability in all cases, and overtopping failure is about 25%. Geotechnical failures are more likely here than failure by overtopping. A comparison between the risk-based analysis for overtopping failure only and for a combination of overtopping and intermediate failure also shows the significance of intermediate failure modes. Given the above optimized 16 ft crown width for the rural levee in good condition, we also optimize the planning considering overtopping failure only. The optimal levee height, failure probability, return year of the peak flow, and costs for the overtopping failure only condition (Table 1, last column) all are smaller than those for the combined condition, respectively. However, overtopping failure and the expected annual damage cost due to overtopping in the combined condition are less than those for overtopping failure only. Ignoring intermediate failure probability that actually exists also reduces the protection from overtopping.

In this example, land cost (*LC*) has little effect on the optimal results, but the availability of land may constrain a levee's base area (*A*). The bottom width of the levee cross section increases 6 ft per foot of

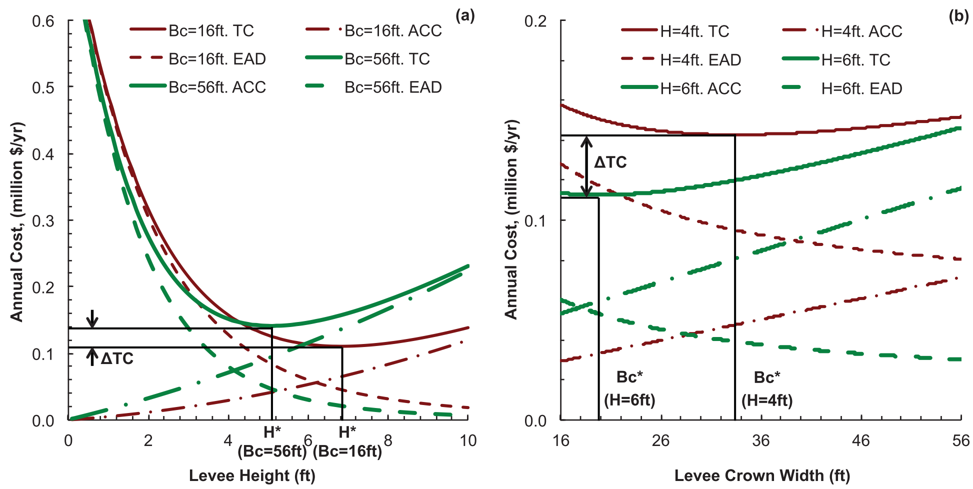

Figure 5. Annual expected total costs, annualized construction costs, and expected annual damage costs for (a) different levee crown widths and (b) different levee heights, for good levee condition (rural levee).

additional height and increases 1 ft per foot of additional crown width. As the unit land cost (*UC*) increases, the optimal plan will have a wider but slightly lower levee to balance different failure mechanisms for an optimal level of flood protection. Where the optimum crown width is too large for a given land area, alternative solutions are steeper landside slopes (following some slope steepness standards) or smaller crown widths. Structural solutions in levees (e.g., sheet piling) also can protect against seepage. In urban areas with high land prices, levees may be replaced with more expensive, but thinner flood walls. In contrast to building a new levee along the Cosumnes River protecting primarily agricultural land, next section looks at evaluating and upgrading an existing levee that protects an urban area from a major river.

3.2. Model Applications to a Large Urban Natomas Levee on Sacramento River

The Natomas Levee examined in this section protects a denser urban area along the Sacramento River, starting from the confluence with Natomas Cross Canal to the confluence with the American River (Figure 6). For this illustrative analysis, river flow frequency for the Sacramento River has an estimated mean peak annual flow of roughly 30,000 cfs [*Domagalski et al.*, 2000]. The coefficient of variation of the assumed lognormal-distributed peak annual flow is 1.0. The cost of land adjacent to the river is valued at $200,000 per acre. An assumed constant damage cost of roughly $1 billion occurs if the protected urban area is flooded. The channel depth, channel width, and levee length are roughly 10 ft, 1000 ft, and 19 mi (100,320 ft), respectively. Channel roughness and longitudinal slope of the stage are assumed to be 0.05 and 0.0005.

Most levees on the north side of the American River have crown widths of 30–60 ft, with roads on the crest. Waterside slope is $3H:1V$ and landside slope is $4H:1V$ at the steepest. For the examined urban Natomas Levee between the two confluences, we assume the existing levee has a height of $H_0=30$ ft, a crown width of $Bc_0=20$ ft, a waterside slope of $\tan\alpha=1/4$, and a landside slope of $\tan\beta=1/3$. The minimum and maximum crown width standards are $Bcmin=20$ ft and $Bcmax=90$ ft, and the maximum levee height is set as $Hmax=60$ ft.

Table 2 shows optimal results for the three different qualitative levee conditions of the levee, found by enumeration.

The optimized results in Table 2 for upgrading the urban levee show similar conclusions as the rural levee, except that the existing levee is already high enough. The optimized overtopping failure remains at the original height. If the existing levee is in good condition, it needs no upgrading or widening; otherwise the levee needs to be widened to reduce intermediate failure likelihood. The optimal intermediate failure probability increases as levee condition becomes worse, but all have more geotechnical failure probability than the constant overtopping failure probability. With relaxation of the maximum crown width constraint, optimal crown widths for poor levees increase further, decreasing optimal levee heights and annual expected total costs (TC) accordingly. However, given the extremely large optimal crown width for "poor" levees,

Table 1. Optimal Results and Comparison for Different Levee Conditions (Rural Levee)

Optimal Results	Good	Fair[a]	Poor[a]	Good (Overtopping Failure Only)
Annual expected total cost ($million/year)	0.111	0.158	0.197	0.071
Expected annual damage cost ($million/year)	0.046	0.066	0.086	0.023
Annualized construction cost ($million/year)	0.065	0.092	0.111	0.048
Levee height, *H* (ft)	6.8	6.1	5.8	5.6
Levee crown width, *Bc* (ft)	16	39.2	56	16 (same as GOOD)
Prob. of overtopping failure (1/year)	0.0080	0.0115	0.0135	0.0151
(contribution to overall failure)	(26%)	(26%)	(24%)	
Prob. of intermediate failure (1/year)	0.0226	0.0325	0.0438	0
(contribution to overall failure)	(74%)	(74%)	(76%)	
Prob. of overall failure (1/year)	0.0306	0.044	0.0573	0.0151
Return period (years)	125	87	74	66
Return period (years) (2ft freeboard[b])	291	224	198	182
Return period (years) (3ft freeboard[b])	394	321	291	271

[a]Presented for comparison only.
[b]Some federal and local standards (e.g., Bulletin 192-82 and PL 84-99) require an additional 2–3 ft freeboard on top of a levee, which increase the ability of a levee to resist flow with smaller failure frequency and larger return period.

building or reinforcing levees that are already classified as poor (particularly for urban areas) is not feasible in practice, where other solutions should be implemented.

Similar to the contour plots in Figure 4, the annual expected total costs of the urban levee show the same trends regarding levee height and crown width. As crown width increases, the intermediate failure probability decreases; as levee height increases, the capacity of the levee system increases and overtopping failure probability decreases. The optimum height and crown width balance failure mechanisms and the trade-off between expected annual damage costs and annualized construction costs.

Land cost also has little impact in this example, but land for levee construction may not be available or too expensive to purchase in densely populated urban areas. The current Natomas Levee is under improvement to increase flood protection and ensure it achieves an updated 200 year protection standards developed by FEMA, USACE, and the State of California. A wide and high levee to protect from a 200 year flood needs a large footprint. So structures requiring less land (e.g., slurry or flood walls) might be needed to reduce seepage and related failures. In California, most levees built originally in the early 1900s to protect agricultural land are more likely to fail. Improvements for risky levees can include slurry walls to mitigate seepage and channel capacity expansion to decrease loads on levees [*USACE*, 2000]. Acquiring understanding and acceptance from the general risk-averse public are also significant in levee planning from social and political perspectives [*Slovic et al*., 1977; *Harrington et al*., 1999; *Browne and Hoyt*, 2000].

4. Impacts From Relevant Uncertainties

Many factors could affect such optimization results, particularly estimation of the conceptual levee fragility curves, various economic parameters, and climate change alterations to hydrologic conditions [*Hansen et al*., 2012; *Abraham et al*., 2015].

4.1. Sensitivity Analysis on Levee Fragility Curves

In this study, levee fragility curves provide a framework to include through-seepage as a representative of intermediate failure in optimal levee planning. The proposed hybrid method of addressing intermediate failure probability combines professional judgment in the original levee fragility curves with a more mathematical and geometric representation of the effectiveness of greater crown widths. Sensitivity analysis of the levee fragility curves and their mathematical expressions is discussed with examples for the rural levee.

Increasing levee failure probability for any given levee height and crown width raises the levee fragility curve, while decreasing levee failure probability lowers the levee fragility curve. Figure 7 shows examples of changing levee fragility curves upward or downward for levees in good and poor condition with a maximum crown width of $Bcmax=56$ ft. Solid lines are the levee fragility curves used earlier, dashed fragility curves have a relative 20% decrease in failure probability, and dash-dot fragility curves have a relative 20% increase in failure probability.

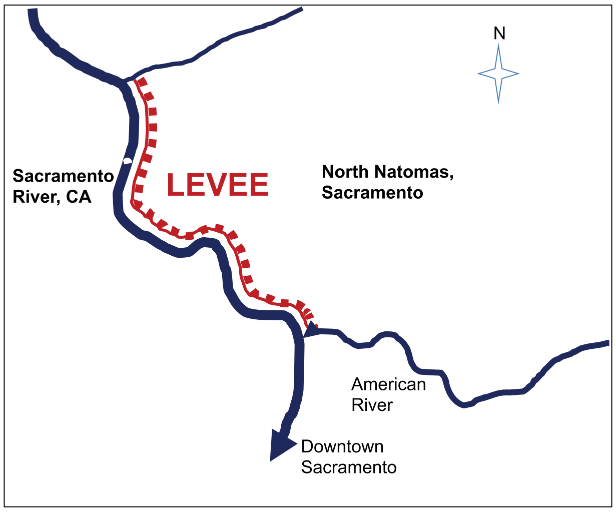

Figure 6. Nineteen miles of levee on the Sacramento River protecting the Natomas Basin to the east.

We can optimize the planning for the rural levee with these changes in levee fragility curves to show their impacts. *Hui* [2014] presents detailed results for a similar study with a relative 1%, 5%, and 20% increase and decrease in intermediate failure probability, for levees under good and poor conditions. Levee fragility curves would affect the optimal levee plan, but changes in optimal levee plans are relatively small if the fragility curves deviate within a modest range (e.g., ±5%). The optimal levee crown width is more sensitive to changes in levee fragility curves, indicating the effectiveness of raising, rather than widening a levee.

4.2. Analytical View of Trade-Offs in Planning Parameters

The first-order condition for minimizing the annual expected total cost of flood control requires the first partial derivatives of $TC(H, Bc)$ with respect to levee height H and levee crown width Bc equal zero. Assuming negligible land cost,

$$\frac{\partial TC}{\partial H}=D*\frac{\partial\left\{\int_0^{Q_c}\left[P_q(Q)*P_L(Q)\right]dQ-F_Q(Q_C)\right\}}{\partial H}+\frac{\partial\left[\frac{r*(1+r)^n}{(1+r)^n-1}*s*c*V\right]}{\partial H}=0 \quad (10)$$

$$\frac{\partial TC}{\partial Bc}=D*\frac{\partial\left\{\int_0^{Q_c}\left[P_q(Q)*P_L(Q)\right]dQ\right\}}{\partial Bc}+\frac{\partial\left[\frac{r*(1+r)^n}{(1+r)^n-1}*s*c*V\right]}{\partial Bc}=0 \quad (11)$$

Assuming uniform flow in the river channel, the overtopping capacity Q_c is determined solely by river cross-section geometry, which is levee height H in this case. Energy slope and channel roughness should not be greatly affected by levee modification. Therefore, from equations (10) and (11),

$$\frac{\partial\left\{\int_0^{Q_c}\left[P_q(Q)*P_L(Q)\right]dQ-F_Q(Q_C)\right\}}{\partial H}\Bigg/\frac{\partial\left\{\int_0^{Q_c}\left[P_q(Q)*P_L(Q)\right]dQ\right\}}{\partial Bc}=\frac{\partial V}{\partial H}\Bigg/\frac{\partial V}{\partial Bc} \quad (12)$$

The above equation holds for the optimal levee height and optimal levee crown width. Other than the enumeration method discussed previously for solving this optimization model, the optimal levee height and crown width also can be found by numerically solving the two first-order conditions simultaneously and verifying that a global minimum is attained.

Table 2. Optimal Results and Comparison for Different Levee Conditions (Urban Levee)

Optimal Results	Good	Fair[a]	Poor[a]
Annual expected total cost ($billion/year)	0.028	0.047	0.068
Expected annual damage cost ($billion/year)	0.007	0.018	0.025
Annualized construction cost ($billion/year)	0.021	0.030	0.043
Levee height, H (ft)	30	30	30
Levee crown width, Bc (ft)	20	47.5	90
Prob. of overtopping failure (1/year)	0.0012	0.0012	0.0012
(contribution to overall failure)	(16%)	(7%)	(5%)
Prob. of intermediate failure (1/year)	0.0062	0.0164	0.0242
(contribution to overall failure)	(84%)	(93%)	(95%)
Prob. of overall failure (1/year)	0.0074	0.0176	0.0254
Return period (years)	842	842	842
Return period (years) (2 ft freeboard[b])	1024	1024	1024
Return period (years) (3 ft freeboard[b])	1115	1115	1115

[a]Presented for comparison only.
[b]Some federal and local standards (e.g., Bulletin 192-82 and PL 84-99) require an additional 2–3 ft freeboard.

In the above equation, values of flood damage cost *D*, unit construction cost *c*, and economic discount rate *r* do not affect the optimal trade-off between levee height and crown width for this static optimization. Changes in these economic values do not affect the optimal substitution between levee height and crown width, though this may affect the optimal plan values. The trade-off between height and width primarily depends on flood flow frequency, levee failure probability, channel geometry, and levee side slopes. These results are similar to analytical results for levee height versus setback [*Zhu and Lund*, 2009].

4.3. Climate Change on Flood Risk Analysis

Climate change has received considerable attention in recent decades, and may worsen flooding problems [*Schreider et al*., 2000; *Milly et al*., 2002], particularly the frequency of floods, represented here by the mean annual peak flood flow and its variance [*Gleick*, 1989; *Zhu et al*., 2007; *Hansen et al*., 2012]. Urbanization could have similar effects as climate change where increased runoff coefficients will increase peak flows, but is not considered here.

To explore how hydrologic variation in a changed climate would affect the risk-based optimal levee plan, we can perform the optimization with varying hydrologic parameter values. For illustration, we calculate the optimal results for the rural Cosumnes levee with increasing mean annual peak flood flow, for a levee in good condition (Table 3).

From Table 3, increases in mean annual peak flood flow from climate change would increase the optimal annual expected total costs (to roughly the same percentages). Optimal crown width remains constant at the minimum value due to minimum standard, without which it should have greater changes than the fairly constant optimal levee height. Failure probabilities, return periods, contributions to overall failure from overtopping, and intermediate failures also change slightly. So a moderate increase in mean annual peak flood flow does not significantly affect the optimal levee plans. For this case, the small damage potential limits additional levee investments with much larger annual floods. Planning for urban levees would be more sensitive.

We also performed the optimization with increasing peak flow variation and increasing combined mean and variance, for various levee quality conditions. The optimal results are similar. Overall, the resulting minor changes in optimal levee plans suggest that a moderate change in hydrologic condition due to climate change would not affect the basic principles and guidelines for levee plans. Dynamic programming can further examine levee sizing over time with climate change [*Zhu et al*., 2007].

5. Conclusion

This study presents a quantitative risk-based analysis for optimal single levee plans considering overtopping failure and intermediate geotechnical failure through parameterizing levee fragility with levee height and crown width. Through-seepage is chosen to represent general intermediate failure since it is more likely when a levee is more permeable than its foundation, and through-seepage theory and solutions are well developed. By using geotechnical relationships given in Schaffernak's solution for through-seepage, levee crown width is added as an independent decision variable to mathematically and geometrically estimate intermediate geotechnical failure probability. In this way, conceptual levee fragility curves, which largely represent professional judgment, are quantitatively adjusted to include both levee height and crown width and represent both overtopping and intermediate failure modes.

In the developed risk-based optimal levee plan, levee height determines overtopping failure probability, while levee height and crown width together affect the likelihood of intermediate geotechnical failure. The optimal levee height and crown width are found by minimizing the annual expected total cost, which is the sum of expected (residual) annual damage and annualized construction costs. This approach could help optimize plans for new levees, and evaluate the current condition and improved plans to upgrade existing levees.

This risk-based optimization model is demonstrated for a rural levee on a small river and an urban levee on a major river in California, for building a new levee and upgrading an existing levee respectively. Increasing levee height primarily reduces overtopping failure, while increasing crown width decreases intermediate geotechnical failure in both large and more frequent smaller floods. As the probability of intermediate

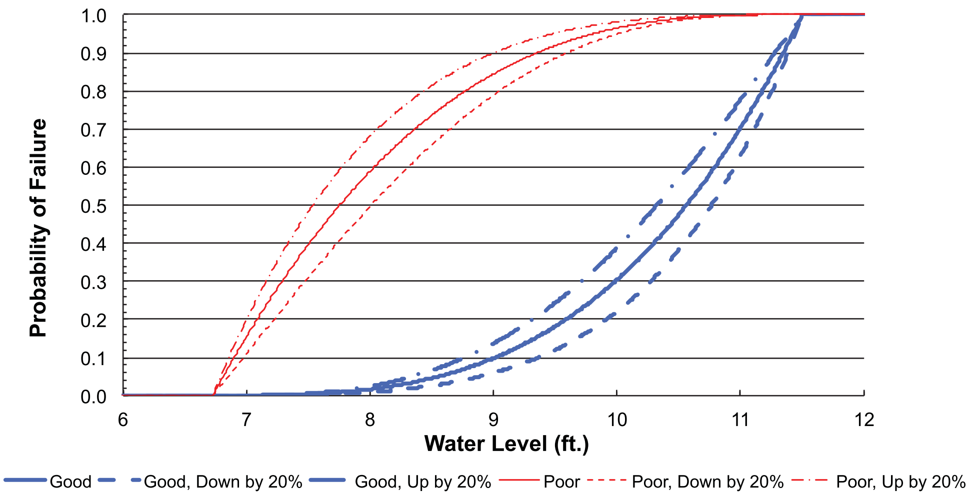

Figure 7. Levee fragility curves with different curvatures.

failure can be much larger than overtopping failure, intermediate failure should be included in such analyses. The optimal crown width of levees in poorer condition can be significantly larger than that in better condition, while the optimal levee height remains fairly constant across levee conditions. Balancing different levee failure mechanisms through changing levee height and crown width should prevent extreme sizes. More profound levee upgrades may be needed for poorer-quality levees. Optimal levees are higher and wider for the urban floodplain with greater damage potential than for the rural floodplain. The contribution of overtopping to overall failure is less for the urban levee, primarily because the increase in intermediate failure from the space-limited crown width.

In practice, levee height for overtopping protection will be the first consideration, and intermediate failure risk from other factors would then be considered. Building a lower levee to provide additional width would not normally be considered. Other methods are also likely to be considered where crown width needs to be increased, such as blanketing the water side with a more impervious material and providing a pervious toe drain on the land side. But the general guidance and implications derived here should be useful.

Many factors could affect the developed optimization model. The optimal levee crown width is more sensitive than the optimal levee height in response to relevant changes. These indicate the effectiveness of levee height in determining the optimal levee plans and in dampening changes in other parameters. Changes to other levee plan parameters have less impact on optimal plans and costs than changing the overall levee fragility curves representing intermediate failure mechanisms. Economic values do not affect the optimal trade-off between levee height and crown width in this static optimization, though they may affect changes in the optimal flow capacity over time. And the derived basic principles and guidelines for levee planning are not affected by moderate changes in hydrology due to climate change.

Table 3. Impacts From Climate Change by Varying Mean Annual Peak Flood Flow

	Increases in Mean Annual Peak Flood Flow					
Optimal Results	0	0.5%	1%	2%	5%	10%
Annual expected total cost ($million/year)	0.111	0.112	0.112	0.113	0.116	0.121
Expected annual damage cost ($million/year)	0.046	0.047	0.046	0.047	0.048	0.051
Annualized construction cost ($million/year)	0.065	0.065	0.066	0.066	0.068	0.071
Levee height, *H* (ft)	6.8	6.8	6.9	6.9	7	7.2
Levee crown width, *Bc* (ft)	16	16	16	16	16	16
Prob. of overtopping failure (1/year)	0.0080	0.0081	0.0078	0.0080	0.0083	0.0086
Prob. of intermediate failure (1/year)	0.0226	0.0229	0.0225	0.0230	0.0239	0.0252
Prob. of overall failure (1/year)	0.0306	0.0310	0.0303	0.0310	0.0322	0.0338
Return period (years)	125	123	128	124	120	116
Return period (years) (2 ft freeboard)	291	287	294	288	278	267
Return period (years) (3 ft freeboard)	394	390	397	390	379	366

With the assumptions and simplifications used in this risk-based analysis, further study should address limitations, such as by including more detailed descriptions of channel geometry, damage cost function, levee fragility curves, and failure modes, particularly with more physical, hydraulic, and geological factors (e.g., soil properties). The effect of levee length also should be analyzed, as longer levees should be more likely to fail. Soil and geometric properties are highly correlated over some distance, and levee sections can be considered probabilistically independent at greater distances [*Vanmarcke*, 2011]. A 2-D analysis can be considered representative of some "correlation length" of levee. A further extended dynamic optimization is promising to better represent time-dependent variations, for example, the nonstationarity in a changing climate [*Cheng and AghaKouchak*, 2014]. Particularly for infrastructure with a long lifetime, a dynamic risk tolerance by starting at a higher safety standard but allowing higher failure risk toward the end might be better than fixed risk levels.

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