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## Journal
THIRD ITALIAN WORKSHOP ON LANDSLIDES: HYDROLOGICAL RESPONSE OF SLOPES THROUGH PHYSICAL EXPERIMENTS, FIELD MONITORING AND MATHEMATICAL MODELING, 9

## ISSN
1878-5220

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## Publication Date
2014

## DOI
10.1016/j.proeps.2014.06.009

Peer reviewed

Available online at www.sciencedirect.com

SciVerse ScienceDirect

Procedia Earth and Planetary Science 00 (2014) 000–000

Procedia
Earth and Planetary Science

www.elsevier.com/locate/procedia

The Third Italian Workshop on Landslides

# Role of Probabilistic Methods in Sustainable Geotechnical Slope stability Analysis

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**Abstract**

Sustainable stable slope for flood protection systems is important areas are of continuing interest, especially among developers of recommended practices and design standards for geotechnical engineers. The Factor-of-Safety used in conventional geotechnical engineering, is the ratio of the capacity (force, stress, deformation, displacement) of a structure to the demand imposed on or induced in the structure. The Factor-of-Safety is intended to be greater than unity. Currently, this important index of safety and reliability is evaluated without regard to the degree of uncertainty involved in its calculation. This is particularly true for levee slope stability design, where the uncertainties in the demand and capacity are very large. This paper advances reliability based slope stability approaches and strategies that address such uncertainties. Probabilistic methods have been applied successfully in practice detailed in this paper to two primary categories of uncertainties encountered in geotechnical engineering; natural variabilities and modeling uncertainties.


Selection and peer-review under responsibility of Dipartimento di Ingegneria Civile, Design, Edilizia e Ambiente, Seconda Universit di Napoli.

*Keywords*: Risk Assessment and Management; Slope Stabilit; Probabilistic Method; Geotechnical Engineering; uncertainties

## 1. Introduction

Current state-of-practice relies heavily in the deterministic characterization and assessment of performance of civil engineered infrastructure. Flood defense systems, especially levees, have been evaluated in terms of the Factor-of-Safety, where the capacity of the system is compared with the demand imposed on or induced in the system. The significant uncertainties associated with the capacity and demand render deterministic modeling potentially misleading. Two structures with the same Factor-of-Safety can have substantially different probabilities of failure.

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While efforts have been made to assess levee vulnerability, results from traditional engineering approaches are frequently questionable because they do not adequately account for uncertainties included in analytical modeling, natural variability, or human and organization factors.

This work builds on risk assessment approaches to develop a method for quantifying the contribution of uncertainty to engineering analyses of Factors-of-Safety and thereby produce a more accurate and informative method in geotechnical sustainability of an engineered system. The probability of failure is directly influenced by how well the system is understood, and how much uncertainty exists with the performance of the system. This paper presents a slope stability method that accounts for two types of uncertainty: Type I – inherent uncertainty, and Type II – modeling uncertainty. Two important types of additional uncertainty are not included in these analyses: Type 3 – human and organizational task performance, and Type 4 – knowledge development and utilization.

This paper focuses on the lateral stability behavior of the flood protection levee system protecting Sherman Island, one of the important western islands in the California Delta. Sherman island is of critical importance to the region and state because of the critical infrastructures that pass under, on and over it, including: natural gas pipelines; regional and interregional electricity transmission lines; two deepwater shipping channels that run alongside the island; and the presence of State Highway, a link between major expressways. The work evaluates current (year 2010) with incorporation of variations in levee capacity and hydrological demand arising from human activities and global climate change.

## 2. Lateral Slope Instability Mechanisms

The performance of slope stability is customarily assessed in terms of comparing two variables: Driving Force and/or Moment (Demand) and the Resisting Force and/or Moment (Capacity). Calculation of the variables allows a Factor-of-Safety to be computed for a given levee [e.g., Seed et al, 2008]. However, for these analyses to be useful, they must characterize a valid failure mechanism. In general, this requires: a) valid application of soil mechanics principles, b) knowledge of the geology and site conditions, c) knowledge of the properties of the soil at the site, d) assessment of uncertainties associated with important properties in capacity, demand, and, more importantly, the models used, and e) accurate identification of the potential failure mechanism which causes unsatisfactory levee lateral stability performance.

Experience with the behavior of slope and often with their catastrophic failure has led to identification of different failure mechanisms caused by lateral instability. The mechanisms described below are the result of change in one or more parameters in the levee system associated with either capacity or demand leading to the levee failing and thus flooding.

### *2.1. Surface Sloughing*

A shear failure in which a surficial portion of the levee moves down slope is termed a surface slough. In this case, failure occurs when the slope/levee soil material had low strength or insufficient resistance to erosion. For example, after days of high water, the surface layers of the levee (by phreatic water movement) become saturated, rendering them heavier and potentially weaker due to lower effective strength relative to the underlying layers. The heavier and weaker surficial layers begin to yield away, resulting in the surface layer sliding down the slope of the levee. If such failures are not monitored and addressed as they occur and repaired, they can become progressively larger.

### *2.2. Shear Failure*

A shear failure involves sliding of a portion of a slope or the levee along with its foundation. Although failure surfaces are typically nonlinear, they are frequently approximated as circular in shape (in two-dimensional cross-sections) and they occur where weak strata exist within a soil deposits. In this case, localized failure (yielding) begins at some point of depth within the slope and progresses upslope and/or downslope, until the failure surface is expressed at the surface. When soft soils are present in the foundation, slope failure tends to manifest themselves as deeper rotational failures. For frictional materials like sand and silt, slope failures tend to be surficial failures [e.g., Terzaghi and Peck, 1967]. Since cohesive soils have a relatively low value of effective internal friction and high cohesion in terms of total stress, it makes the soil relatively strong at shallow depths and weak at deep depths. Peat is especially prone to rotational failure of failure by spreading, particularly under the action of horizontal hydraulic forces [e.g., Bell, 2000].

## 3. Quantification of Reliability

In this work, the quantification of uncertainty draws from statics and probability analysis. Static deals with analysis of data. In addition, statics involves geotechnical and hydrological results from the past or geotechnical, and hydrological data from experiment or trials. Probability deals with the analysis of likelihoods of outcomes from experiment or trials whose outcomes are not known or cannot be known in advance [Bea, 2006]. As deterministic modeling does not explicitly quantify uncertainty, stochastic modeling, makes it more possible to identify "extreme" values (often called "outliers") that are often the cause of failure.

### *3.1. Type I (Aleatory) Uncertainty Evaluation*

Many different types of probability density distributions can be used to characterize the uncertainties associated with an analytical model. For the sake of simplicity, the work presented here is based on Normal, and Lognormal probability distributions. Most soil parameters have uncertainties that can be well characterized with a Lognormal distribution, defined by the Mean, μ, and Standard Deviation, σ, of the natural logarithm of a set of the random variable, X. The relationship between the Coefficient of Variation, COV, and Standard Deviation with these parameters is given by (1):

$$\sigma_{lnx} = \sqrt{ln(\ 1 +\ V_x^2} \quad (1)$$

Alternatively, through validated "expert" judgment, testing, or typical values, a 90th percentile, X90, 10th percentile, X10, can be used to define the Standard Deviation, σ, of the distribution:

$$\sigma_{lnx} = 0.39(ln\, X_{90} - ln\, X_{10}) \quad (2)$$

To ascertain Type I uncertainties, soil properties were summarized and statistically analyzed in order to determine the Mean (μ), Standard Deviation (σ), and Coefficient of Variation (COV) (statistical characteristics). In order to evaluate the quality of the available data, the calculated COV was compared with accepted ranges based on previous studies performed by [Duncan, 2000]. For soil strength properties, laboratory data from previous field exploration programs were used to determine the necessary statistical characteristics.

Here introduce the paper, and put a nomenclature if necessary, in a box with the same font size as the rest of the paper. The paragraphs continue from here and are only separated by headings, subheadings, images and formulae. The section headings are arranged by numbers, bold and 10 pt. Here follows further instructions for authors.

### *3.2. Type II (Epistemic) Uncertainty Evaluation*

Type II uncertainties are those due to modeling, observation errors, statistical, and measurement. This type of uncertainty is also known as epistemic, since this category of uncertainty is information sensitive in that data gathering, improved modeling, or measurement will reduce the uncertainties [Bea, 2006]. Type II (Epistemic) uncertainty is often ignored and be characterized with two parameters: 1) Bias, and 2) COV of Bias. Bias is defined as (3):

$$Bias(\psi) = True\ or\ Measured\ Value\ /\ Nominal\ or\ Predicted\ Value \quad (3)$$

Determination of central tendency characteristics of the Bias, ($B_{50}$, B Mean); and a dispersion measurement of the Bias (COV of Bias) provide the essential statistical characteristics needed in these analyses of Type II uncertainties. Several approaches should be utilized to determine these characteristics. These include field and laboratory measurements, model testing (prototype or scaled), and expert judgment [Bea, 2003; Baecher and Christian, 2003].

Determination of the Bias associated with analytical models is one of the most critical parts of determining valid Factors-of-Safety and probabilities of failure. Analytical results derived from mathematical models can and often do differ from those in the field. All analytical models have flaws, defects and limitations due to the assumptions and ‘boundary conditions’ included in these models. The best way to characterize the Bias is to compare the results from the analytical models that will be employed by the engineers with results from high quality prototype field tests.

### *3.3. Probability of Failure*

The probability (or likelihood) of failure ($P_f$) can be estimated in various ways. The most straightforward method is to numerically integrate the product of two distributions:

$$P_f = \sum F_c(s) f_d(s) \Delta s \quad (4)$$

Where Fc is the conditional probability that the capacity is equal to or less than a given demand, $f_d$ is the probability density distribution for the demand. This is the general expression and can be used for any form of the distributions and can incorporate the correlation between the capacity and demand. Assuming the distributions of demands and capacities can be reasonably characterized as Lognormal and independent, the Safety Index, β, can be computed by (5):

$$\beta = (ln(C50\ /\ D50\ ))/\surd(\sigma(ln\ C)^\wedge 2 + \sigma(ln\ D)^\wedge 2) \quad (5)$$

$C_{50}$ and $D_{50}$ are the median (i.e., 50th percentile) values of the capacity and demand, respectively. The ratio of $C_{50}/D_{50}$ is the equivalent of the traditional definition of the deterministic Factor of Safety. The values of $\sigma_{lnC}$ and $\sigma_{lnD}$ are the standard deviations of the lognormal distributions the capacity and demand. The probability of failure, $P_f$, can then be determined from the Safety Index:

$$P_f = 1 - \Phi\ (\beta) \quad (6)$$

Where Φ (β) is the standard cumulative normal probability function for the Safety Index. As the Factor-of-Safety increases, the Safety Index increases, and the likelihood of failure decreases. In contrast, as the uncertainty in the demand and capacity increases (which can be represented either as Type I or Type II uncertainties), the likelihood of failure increases. Probabilistic analyses thus give more information than a single value of the Factor-of-Safety. Determination of the likelihood of failure also allows assessment of the importance of uncertainties associated with each parameter in the reliability of the slope stability system.

## **4. Sherman Island Levee Slope Stability Vulnerability**

A variety of hazards including hydraulic loading (storms, flood), and earthquakes threaten the levee slopes surrounding Sherman Island. Sherman Island is located at the ‘gateway’ to the Sacramento – San Joaquin River Delta, California. This delta forms the hub of the State’s water distribution system. About two-thirds of all Californians and millions of acres of irrigated farmland rely on the Delta for water from the State Water Project and federal Central Valley Project. Delta water is vital to California’s economy, fifth largest in the world, and its growing population, expected to reach 53 million by 2030 (Department of Water Recourses, 2008).

While efforts have been made to assess slope stability vulnerability, the results are based on more traditional engineering estimation approaches that do not more fully account for uncertainties included in modeling, natural variability, and/or human and organization factors. Probabilistic approaches to analysis, like those described herein, are often overlooked or neglected frequently because geotechnical engineers are unfamiliar with the procedures for both identifying and quantifying uncertainty. As discussed below, not incorporating the full range of uncertainties into an analysis and decision making can actually lead to incorrect decisions and slope failure and ultimately, worsened consequences, if only because of a false appreciation of the likelihood of failure of the engineered system. In the absence of full consideration of uncertainties, slopes could appear safer than it is in reality. All such considerations are heightened in an era of increasing reliance on infrastructure systems protected by levees and climate-related hazards.

## **5. Type I (Aleatory) Uncertainty Evaluation**

### *5.1. Hydraulic Loading of Flood*

Sea level rise directly affects the probability of levee slope stability failure on Sherman Island through higher normal tide levels and therefore higher flood stages during winter storm and spring snowmelt. The indirect effect of sea level rise is that it can increase the chance of occurrence of current '100-year flood elevations.' Consequently, it is important to include potential sea level rise data into the levee reliability analysis. Since elevated water levels would result in increased hydrostatic loads acting on the waterside of levees, the sea level rise is a major factor influencing future levee stability. A variety of estimates exist for evaluating potential sea level rise between now and 2100. The Intergovernmental Panel on Climate Change has predicted an increase in sea level of 0.1m to 0.65m by 2150 (URS, 2005) over 1990 levels. A more recent analysis predicts a sea level rise of 0.5m to 1.4m by 2150 (Rahmstorf, 2007).

Storm events as the precipitating hazard were chosen for these analyses because levee failures and flooding have occurred relatively frequently during past storm events in the Sacramento San Joaquin Delta. A major storm scenario also allows the analysis to account for human interactions during the event, particularly with respect to flood fighting. Due to advances in meteorology, atmospheric models and global weather monitoring, and storm forecasting it is possible to have longer lead times to mobilize flood fighting before a storm event (sometimes 10 days in advance). This study selected to evaluate the 2, 50 and 100 year flood event for the current conditions (year 2010). To this end, previous studies were used (mainly the Delta Risk Management Strategy, DRMS, 2009), to determine the peak storm water levels at Sherman Island. Once this was determined, representative river stage hydrographs (time versus water elevations) were developed based on those associated with similar past events.

### *5.2. Site Geology and Soil Characterization*

The Sacramento-San Joaquin Delta region has been an area of ground subsidence and soil deposition for over 140 million years [S.E. Ingebritsen, U.S. Geological Survey, 2000]. During the period from 70,000 to 100,000 years ago (the last glacial period at the end of the Pleistocene era) sea level was as much as 111.25 m (365 feet) below present sea level. The delta area was then a fluvial and alluvial system, with fast flowing rivers typically depositing course grained sediments (predominantly sand) in alluvial fans and channels.

Approximately 38,000 meters (125,000 feet) of levees protect Sherman Island. The south levee on Sherman Island consists of dredged loose to medium sand and silt. Beneath the levee is a thick layer of peat/organic soil. This peat/organic soil layer is typically 10.6 m (35 feet) thick in the fields away from the levee but it has been consolidated under the weight of the levee. Underlying the peat/organic is an approximately 20-foot-thick layer of soft clay, under which is a dense sand stratum.

A total of 67 soil borings were analyzed to determine the soil properties under the Sherman Island levees. Of these borings, 42 had sufficient high quality data and were considered sufficiently reliable to characterize the soil properties. Other borings were rejected because of a lack of Standard Penetration Test (SPT) blow counts (N), limitations of testing equipment, unreliable or missing soil parameters, or limited depth. The remaining 42 borings all extended to layers of dense to very dense sand that underlie the surficial soils. Based on analyses of data from these borings, the levee section is comprised primarily of sand with some clay layers at a few locations. Low Standard Penetration Test (SPT) N values in the layers 0 to 9.14 m (30 feet) below the levee's crown show that soil strength is relatively low.

To ascertain Type I uncertainties, soil properties were summarized and statistically analyzed in order to generate a mean (μ), standard deviation (σ), and coefficient of variation (COV, the ratio of standard deviation to mean value of variable). A discussion of the uncertainty for each parameter is presented Table 1.

Table 1: Southern Site, Material Properties and Uncertainty Associated with Each Layer

| Material | Property | Mean | Std. Dev. | Lower | Higher |
|---|---|---|---|---|---|
| **Peat 1** | $S_u$ (psf) | 700 | 120 | 600 | 810 |
| **Peat 1** | Unit Weight (pcf) | 85 | 20 | 78 | 92 |
| **Silt 1** | $S_u$ (psf) | 710 | 140 | 610 | 815 |
| **Peat 2** | $S_u$ (psf) | 350 | 120 | 250 | 400 |
| **Peat 2** | Unit Weight (pcf) | 72 | 10.5 | 70 | 78 |
| **Silt 2** | $S_u$ (psf) | 435 | 150 | 300 | 600 |

*5.3. Site Selection and Subsurface Condition*

Two sites were selected for evaluation: a) Southern Site and b) Northern Site. The procedure developed and applied to evaluate these sites can be applied to any site location on Sherman Island.

The levees of interest are constructed along the southern side of Sherman Island (the San Joaquin River side) and the Island's northern side (the Sacramento River bank). For purposes of this study, the profile considered as starting at the depth with a sand stratum below approximate elevation -21 m (-70 feet) (cite the elevation datum), above the sand is a layer of silty clay. The clay stratum is on the order of 6.1 m (20 feet) thick and overlain by peats that, in their natural state, are up to about 12.2 m (40 feet) thick and extend to the island surface. The levee fills are typically composed of peat, dredge materials and sandy fill, with the crown of the levees usually consisting of relatively clean sand. In some locations the levee also appear to be located directly over natural levees of the San Joaquin River, which are indicated by layers of silty, material within the peat stratum.

Soil stratigraphy reflects the depth and thickness of each soil type in the cross-section (Fig. 1). The stratigraphy is characterized by plotting the elevation of the surface, and on that placing the soil boring logs. The geometry of the different soil layers were characterized based on the soil boring logs and on regional site geological – geotechnical interpretations. The available information was used to create two plausible cross-sections identified as 'A' (less plausible), and 'B' (most plausible).

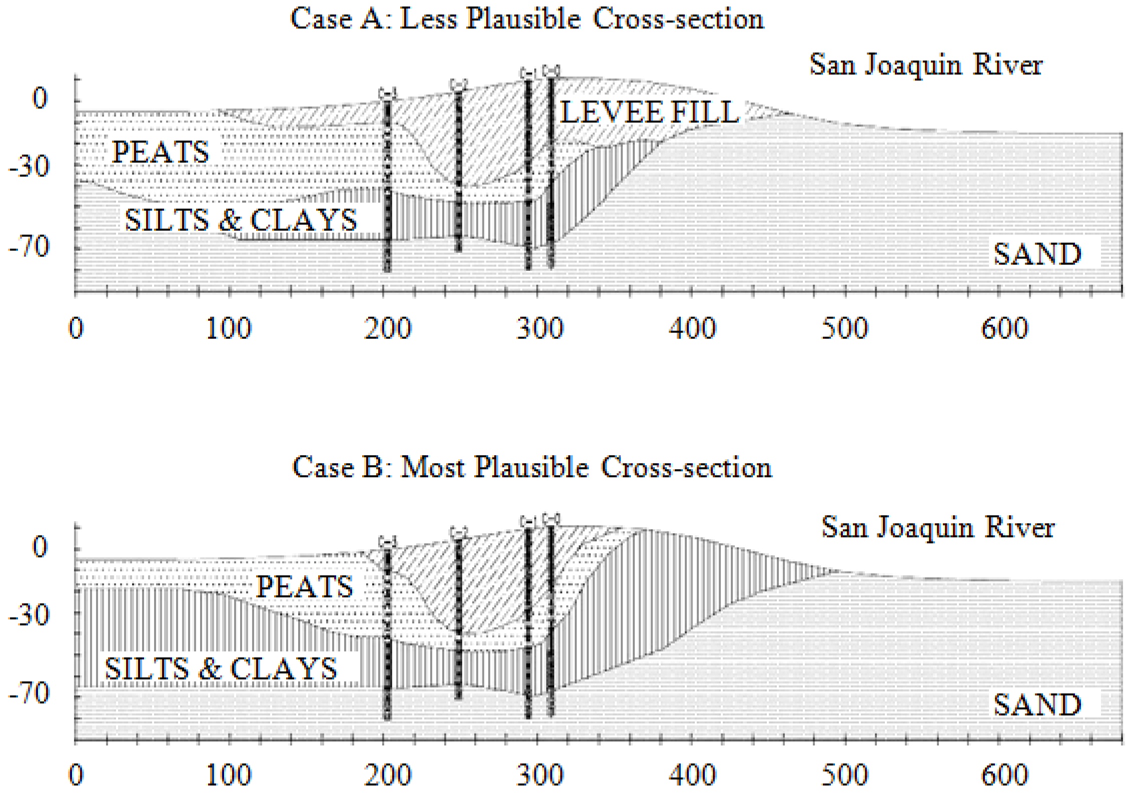


Figure 1: South Side Subsurface (a)Less Plausible Condition; (b) Most Plausible Condition

## 6. Type II (Epistemic) Uncertainty Evaluation

A good way to determine the central tendency and uncertainty characteristics of the analytical model Bias is to compare the results from the analytical models used to determine the Demands and Capacities associated with levee stability with the actual observed and measured performance characteristics of levees subjected to prototype conditions. For purpose of this study, prototype field test data from four different full-scale test sites were used. Data for these four different test sections are presented in the following technical papers:

- "Performance of Test Fill Constructed on Soft Peat", by Tillis et al. (1992)
- "Stability of Atchafalaya Levees", by Kaufman and Weaver (1967)
- "Design of Single- or Multi-stage Construction of Embankment Dams for the James Bay Project", Ladd et al. (1983)
- "Monitoring of the Test on the Dike at Bergambacht", by Koelewijn and Van (2003)

Measured data from these four cases are used to define the true or actual levee performance characteristics. Failure (Factor-of-Safety of unity) was defined as the imposed lateral loading that resulted in large lateral deformations of the test levee (fill) with no increases in the lateral loading. The goal was to replicate the field test condition in the computer analytical models with their associating stability methodologies and determine the resulting Factors-of-Safety for the prescribed test loading conditions. By comparing the results from the analytical models with observed field test data the corresponding Bias values were determined.

### *6.1. Bias Mean Value and Standard Deviation*

The four levee test cases that were selected for this study were used to develop sixteen Bias points (four for each limit equilibrium model) by comparing the ratio between measured field result and prediction of stability analytical model result. Table 2 summarizes the final result of Bias point calculation for each base case and its corresponding limit equilibrium methods.

Once the Bias associated with the slope stability analytical procedure were determined, "graphical statistics" methods were used to determine how different statistical distribution characteristics could be determined (Wilcoxon, F. 1947). The four different Bias values (n=1 to 4) for each analytical procedure were rank ordered and the plotting position (PP) determined from (7)

$$PP = {}^{n}/_{(N+1)} \tag{7}$$

Where n is rank (1, 2, 3, 4) of each Bias point and N is total number of Bias values. For the purpose of plotting, the author plotted these Bias points on different types of graphical statistical plotting papers (e.g. Normal, Lognormal, Weibull, Extreme Value, etc) and determined which distribution provided the 'best fit' to the data. The Lognormal distribution developed a 'best fit' to the Bias points. Based on the distributions, the Mean and Standard Deviation of the Bias could be determined (Table 2). As expected, the analytical methods produce 'conservative' Mean Factors-of-Safety (Mean Bias values less than unity) and have Bias COVs in the range of 17 to 20 percent.

Table 2: Bias Mean values and Standard Deviations for Four Analytical Slope Stability Methods

| Bias | Mean | Standard Deviation σ |
|---|---|---|
| **OMS** | 0.967 | 0.192 |
| **Bishop** | 0.930 | 0.158 |
| **Janbu** | 0.928 | 0.162 |
| **Spencer** | 0.932 | 0.157 |

## 7. Analysis

The methodology for modeling the lateral Capacity and Demand forces acting on the levee systems is based on the widely used definition of the Factor-of-Safety for slope lateral stability analysis. The Factor-of-Safety against sliding is computed based on the horizontal forces acting on each block. Components for each of the active, passive and neutral blocks are D as the driving force and R as the resisting force. For each block, D and R can be obtained by constructing the force polygon which consists of the weight of the block W, the normal force N on the slide plane, and the shear strength of soil being mobilized along the sliding plane. The uplift force U can also be considered in the polygon of forces when the effective strength parameters are used for freely draining material. The factor of safety with respect to the shear strength of soil can then be expressed as (8)

$$FS = {}^{\text{Total Horizontal Slope Material Resistance}}/_{\text{Driving Force Required for Equilibrium}} \tag{8}$$

Since the probability of failure $P_f$ is function of the Safety Index, $\beta$, and given that the distributions of demands and capacities can be reasonably characterized as Lognormal, then $\beta$ can be computed directly from (5). In (5) the ratio of $C_{50}/D_{50}$ is the equivalent of the median Factor-of-Safety. As the Factor-of-Safety increases, the Safety Index increases, and the likelihood of failure decreases.

### *7.1. NORTH AND SOUTH SITE CAPACITY AND DEMAND ANALYSES*

Many different methods relying on limit equilibrium principles use discretization of slices (e.g., Morgenstern and Price, 1965 and Spencer, 1967. Fundamentally, while all similar in nature, their differences lie in the fact that different equilibrium equations are enforced (moment, force), and different assumptions for the inclination of the inter-slices forces are considered. For example, Janbu's Generalized Procedure of Slices (1968) method only considers force equilibrium, which is similar to the equilibrium equation used in the General Method of Slices, whereas, Morgenstern and Price (1965) and Spencer (1967) methods satisfy both force and moment equilibriums. These procedures have been extensively documented and are available in many commercial computer programs. For the purpose of this study we used the 4 different stability methodologies: 1) Ordinary Method of Slices (OMS) (Fellenius, 1936), Simplified Bishop's Method (1955), Spencer (1967), and Janbu's Generalized Procedure of Slices (1968)

Traditionally, these analytical procedures tend to be ‘conservative’. Type II uncertainties are frequently implicit in design codes and guidelines. Problems develop due to the compounding of these ‘implicit conservatisms’ and lack of knowledge of how conservative the results are. In addition, what is conservative for one set of conditions may or not be conservative for another set of conditions.

Two-dimensional static lateral stability analyses were performed. The Mean Demand and Capacities were computed using the procedures described earlier for the South and North site locations. The South site plausible alternative cross-sections were analysed for the various flood case scenarios (hydrographs). Based on the levee cross-sections, Mean Demands and Capacities were calculated based on four lateral stability analysis methods and plausible ‘Lower’ and ‘Higher’ soil properties.

The Mean Factor-of-Safety determination is based on when the water remains at or near full flood stage long enough so the embankment becomes fully saturated and a condition of steady flow seepage occurs. This condition may be critical for deep levee slope stability. Previous experience and slope stability analysis indicate that deep failure may occur in levee slopes after the embankment becomes fully saturated in the southern portion of Sherman Island for both the ‘A’ and ‘B’ cross-sections. Failure generally occurs in these very plastic peat slopes. The failures appear to be the result of shrinkage during dry weather and moisture gain during wet weather or fluctuations in the water table with a resulting loss in shear strength due to a net increase in water content, plus additional driving force from water in cracks. Mean Factors-of-Safety were determined for both shallow and deep failure surfaces.

### *7.2. Probability of Levee Failure Due to Sliding With Consideration of Type I & II Uncertainties*

Probabilities of failure due to slope instability were successfully computed using the method for the 2, 50 and 100 year storm cases. These probabilities of failure not just increased with intensity of the storm (demands on the system) and age of system but also with consideration of Type II uncertainties.

This specification of the Type II uncertainties has two important effects on the estimation of the probabilities of levee failure (in this case, breaching leading to flooding of Sherman Island). The first is that they add to the total uncertainties that are addressed as part of the intrinsic uncertainties that include Type I—natural variability—uncertainty. The second effect of Type II modeling uncertainties on the estimation of Pf is that they affect the central tendency and distribution characteristics of the probabilistic descriptions used to define the demands and capacities for the infrastructure concerned. The Fig. 2 is showing the annual Pf with consideration of the type I and type II uncertainty the probability of failure for the levees of Sherman Island both on south and north side in the case of cross-sections ‘A’ (less plausible), and ‘B’ (most plausible).

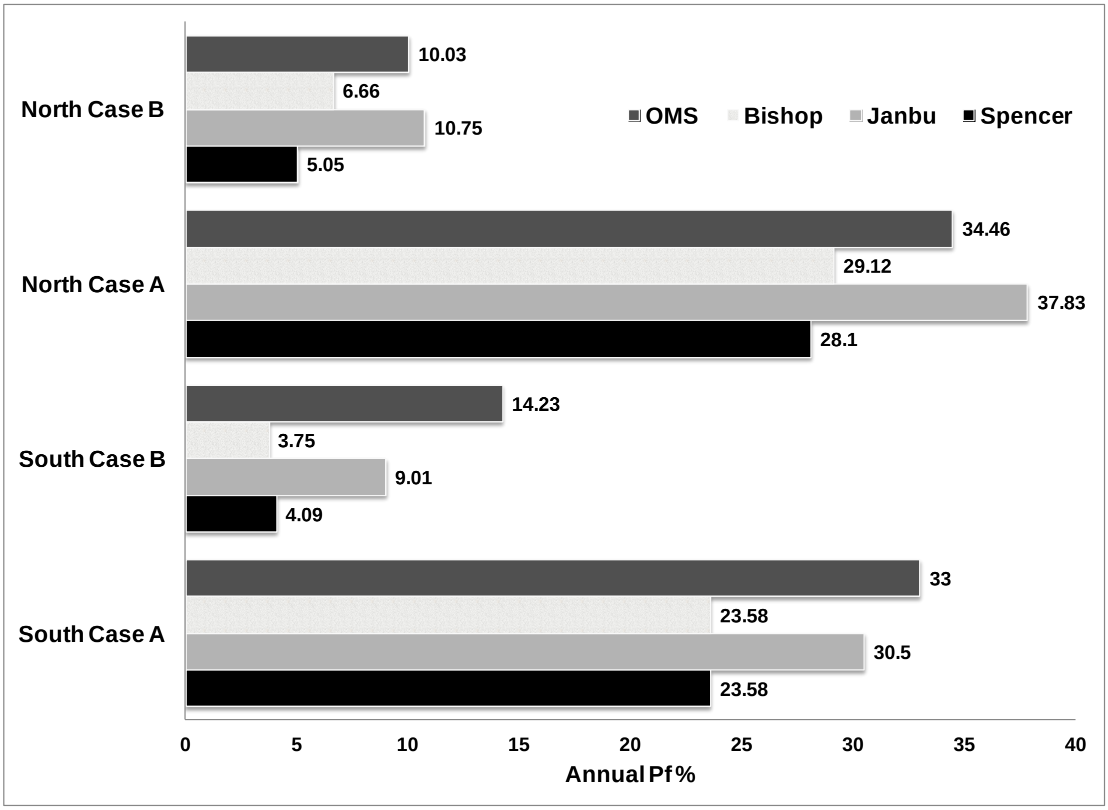


Figure 2: Annual Pf with consideration of the type I and type II uncertainty the probability of failure for the levees of Sherman Island both on South and North side in the case of cross-sections 'A' (less plausible), and 'B' (most plausible).

## 8. Conclusions

The goal of this paper was not to provide accurate results; rather it was to develop a method by which the likelihood of slope failure due to sliding could be determined using available information for sustainable geotechnical engineering system. Probabilities of failure due to slope instability were successfully computed using the method. These probabilities of failure not just increased with intensity of the storm (demands on the system) and age of system but also with consideration of type II uncertainties.

In this paper, the Type II (modeling) uncertainties were evaluated by making multiple comparisons between the results from prototype field tests and experiments and the results from analytical models that attempted to replicate or reproduce the results from these field analyses. This specification of the Type II uncertainties has two important effects on the estimation of the probabilities of levee failure (in this case, breaching leading to flooding of Sherman Island). The first is that they add to the total uncertainties that are addressed as part of the intrinsic uncertainties that include Type 1—natural variability—uncertainty. The second effect of Type II modeling uncertainties on the estimation of Pf is that they affect the central tendency and distribution characteristics of the probabilistic descriptions used to define the demands and capacities for the infrastructure concerned.

## 9. Acknowledgments

This material is based upon work supported by the National Science Foundation under Grant No. (EFRI-0836047). Any opinions, findings, and conclusions or recommendations expressed in this material are those of the author(s) and do not necessarily reflect the views of the National Science Foundation."

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