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Engineering Reliability of Navigation Structures, Supplement No. 1

by Thomas F. Wolff, Weijun Wang, Michigan State University

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Prepared for Headquarters, U.S. Army Corps of Engineers

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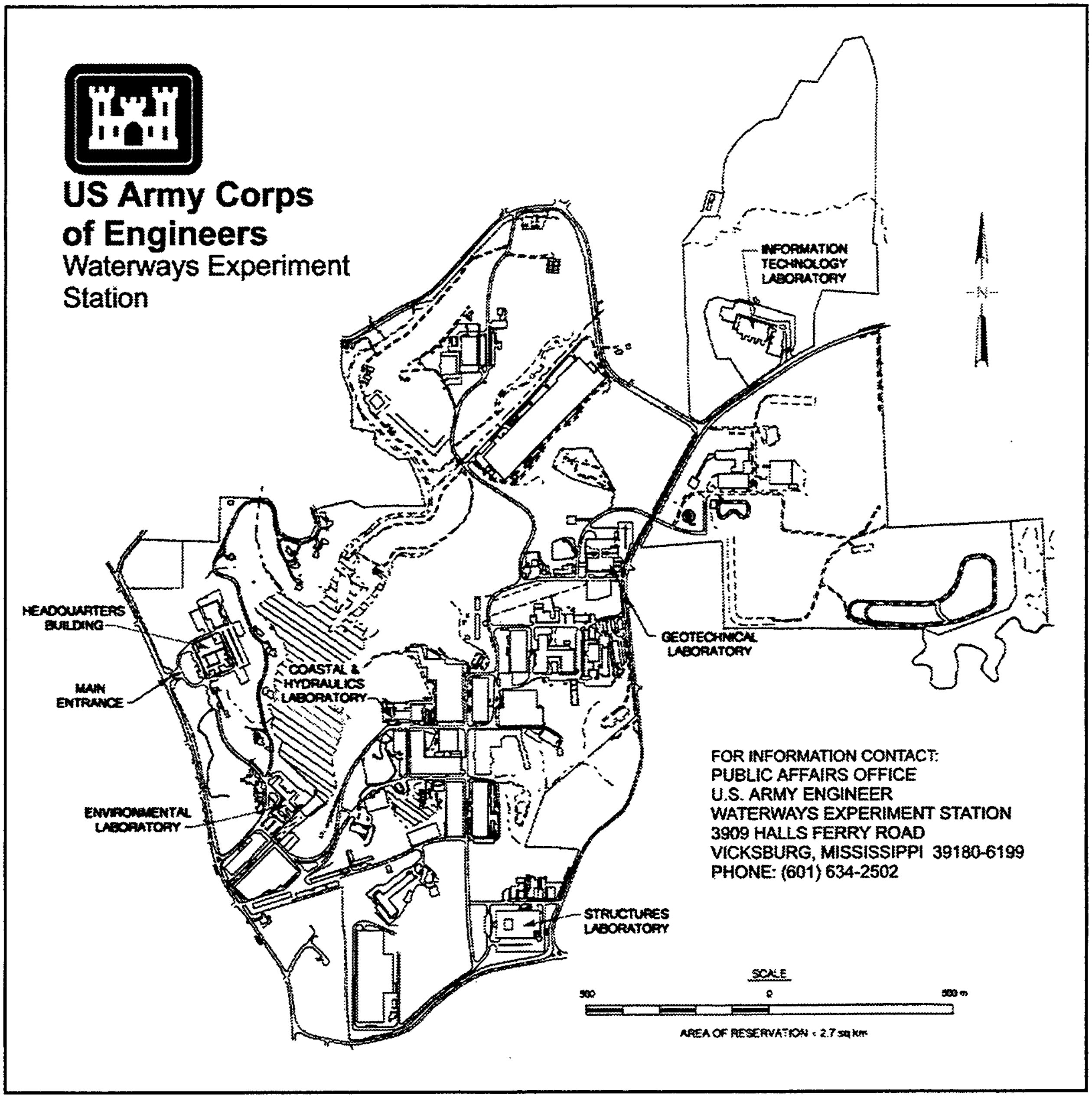

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Preface

The work reported herein was performed at Michigan State University under contract DACW39-92-M-0192 during the period December 1991 through March 1992. Funding was provided by the Civil Works Research and Development Program, Risk Analysis for Water Resource Investments of the U.S. Army Engineer Waterways Experiment Station (WES). The Principal Investigator for the project was Dr. Thomas F. Wolff, Associate Professor, Department of Civil and Environmental Engineering. Much of the methodology development and all of the analyses were performed by Mr. Weijun Wang, Graduate Research Assistant. The technical monitor for the Corps of Engineers was Dr. Mary Ann Leggett, Information Technology Laboratory (ITL), WES. Helpful guidance and comments were provided by Mr. Donald Dressler, Engineering Division, Directorate of Civil Works, Headquarters, U.S. Army Corps of Engineers, and Dr. Reed Mosher, ITL, WES. The work was performed under the general supervision of Mr. H. Wayne Jones, Chief, Scientific and Engineering Applications Center, and Dr. N. Radhakrishnan, Director, ITL.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander and Deputy Director was COL Leonard G. Hassel, EN.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurements used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
degrees (angle)	0.017455329	radians
feet	0.3048	meters
inches	2.54	centimeters
pounds	4.448222	newtons
kips	4.448222	kilonewtons
tons	2.224111	kilonewtons
kips per square foot	47.880260	kilopascals
ton per square foot	23.940130	kilopascals

PART I: INTRODUCTION

BACKGROUND

Since 1990, the Corps of Engineers has been involved in a research program to identify and develop methods to apply engineering reliability analysis to the assessment of the relative condition of navigation structures. In turn, such assessments are to be used in conjunction with traffic and economic evaluations in the decision-making process for allocating funds for major rehabilitation projects. In pursuing this effort, components of the research and development work have been performed by the writers at Michigan State University; by Jaycor, Inc., Vicksburg, Mississippi, and by Mr. Thurman Gaddie, consultant, Cinncinnati, Ohio. The research has been coordinated by the Information Technology Laboratory, Waterways Experiment Station with general supervision by the Structures Branch, Office, Chief of Engineers, U.S. Army. Based on the research component performed by the writers at Michigan State, which focused on the stability of gravity monoliths, a report was published (Wolff and Wang, 1992) entitled *Engineering Reliability of Navigation Structures.* That report summarized the development and testing of several reliability analysis procedures by their application to actual structures on the Monongahela River in Pennsylvania and the Tombigbee River in Alabama.

SCOPE OF THIS REPORT

As a result of the work performed for the previous report (Wolff and Wang, 1992), additional items of study were identified and these are the subject of this report. They include:

> Further investigation of the effects of the uplift pressure distribution and its associated uncertainty and refinement of its treatment in analysis.
>
> Investigation of the effects of uncertainty in structural geometry and anchor performance.

Further investigation of the appropriate manner to model the shear strength of foundation rock.

Further investigation of the effects of backfill level and wall friction (vertical shear) for Demopolis Locks and Dam monolith L-17.

Performance of "best estimate" analyses using revised parameters for random variables based on review of previous analyses.

PART II: LOCKS AND DAM NO. 2, MONOLITH M-16, RESULTANT LOCATION ANALYSES

PROBLEM STATEMENT

In the previous report (Wolff and Wang, 1992), monolith M-16 at Monongahela River Locks and Dam No. 2 was extensively analyzed as a model for the development of reliability analysis procedures. It was selected as a "first example" because of its simple geometry, and was used to assess the differences in reliability index obtained using different performance functions and different probabilistic methods. The relative contributions of various random variables to the overall uncertainty was also studied. As the previous research progressed, the relationship of uplift pressure and its associated uncertainty to the reliability index was identified as an area where further study was desired. This part of the present report summarizes studies to determine how various reliability indices vary as a function of the foundation uplift pressure and its associated uncertainty.

THE UPLIFT PARAMETER, E

To conveniently characterize the uplift force on a hydraulic structure for reliability analysis, the previous research defined a parameter, E, which may be considered a drainage efficiency or effectiveness. The relationship of E to the assumed uplift pressure assumption is illustrated in figure 2.1. The concept of drain efficiency has been used by the Corps in dam design where stilling basin drains are sometimes considered to be only partially effective in reducing uplift pressure and force. For a gravity monolith with no seepage cutoffs nor drains, uplift pressure is commonly assumed to vary linearly across the monolith base from headwater to tailwater (or upper pool to lower pool); the value of E is taken as zero for this condition and the uplift force per lineal foot of structure is calculated as:

$$U = (1/2)(H_U + H_L)B\gamma_w \tag{2.1}$$

where

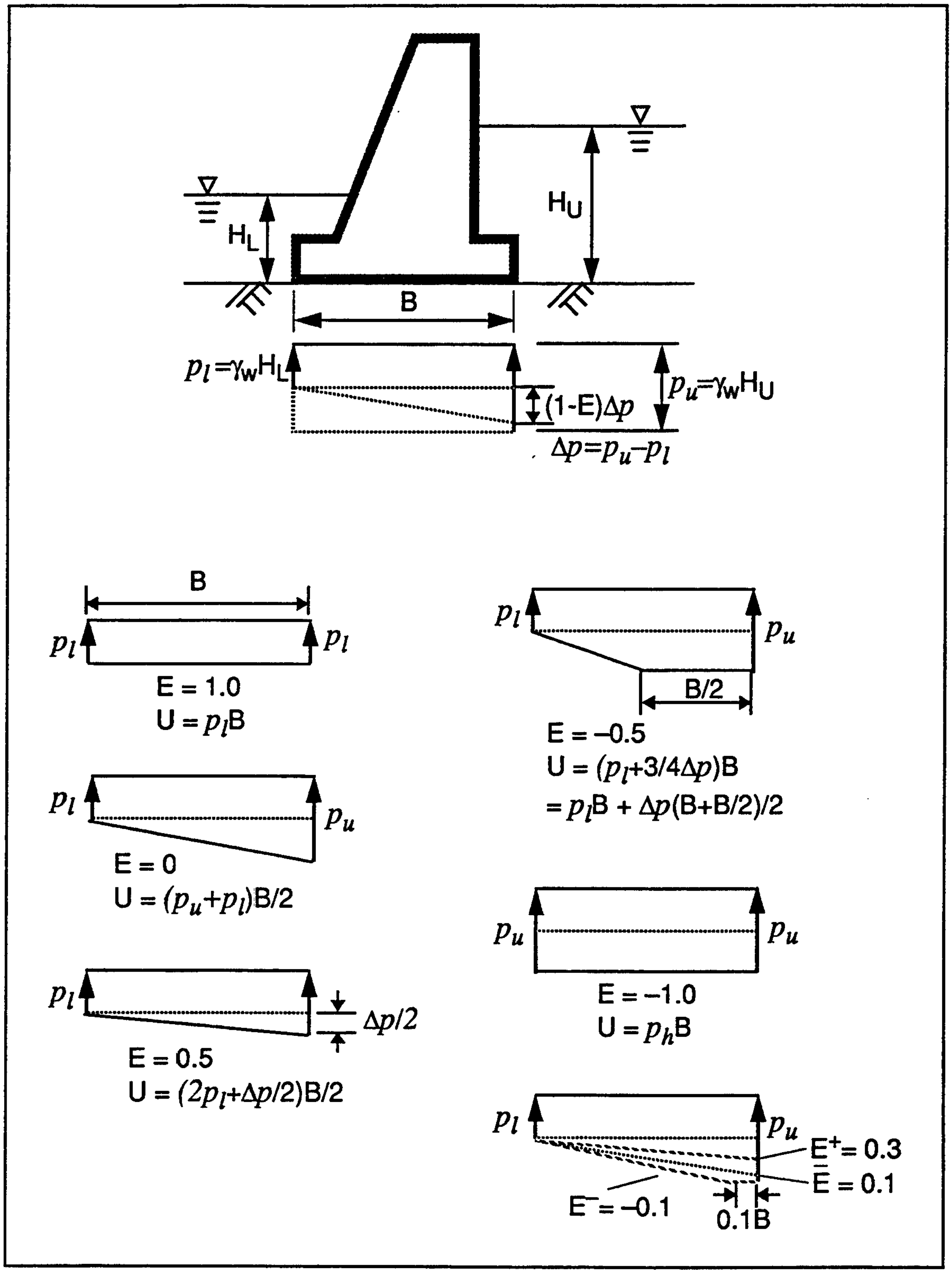

Figure 2.1 Definition of Uplift Parameter, E

H_U is the pressure head at the upper end of the monolith base,

H_L is the pressure head at the lower end of the monolith base,

B is the width of the monolith base, and

γ_w is the unit weight of water

For a perfectly drained foundation where the uplift pressure corresponds to tailwater across the entire monolith base, the uplift force is:

$$U = H_L B \gamma_w \tag{2.2}$$

Defining E as a drainage efficiency parameter that varies linearly from 0.0 for the case of no foundation drains to 1.0 for the case of perfect foundation drains, the above two equations can be generalized as:

$$U = (1/2)[2H_L + (1-E)(H_U - H_L)]B\gamma_w \tag{2.3}$$

Using this definition, the uplift force can be modeled as a random variable in reliability analysis by assigning an expected value and standard deviation to the uplift parameter, E.

The above definition can be extended to model uplift pressures greater than that corresponding to a linear variation from headwater to tailwater by using negative values of E. A commonly encountered case in which a greater uplift force is assumed is where the effective base resultant force lies outside the middle third of the base (or kern), implying tension over a portion of the base. In this case, it is common practice to assume full headwater pressure over the region of the base not in compression. It can be shown that equation 2.3 still holds for this assumption by defining E as minus one plus the fraction of the base in compression (PC):

$$E = -1 + PC \tag{2.4}$$

For example, if 75 percent of the base is in compression, substituting the value $E = -0.25$ in equation 2.4 will yield the correct value for the uplift force U.

Early in the previous research project, relatively large positive values of E were assumed for analysis of monolith M-16. These reflected observed uplift pressures beneath dam monoliths, which often correspond to tailwater. For certain of the analyses, the uplift pressure was modeled assuming $E[E] = 2/3$ and $\sigma_E = 0.1047$. As studies progressed, it was reasoned that such assumptions may be unconservative for M-16 as it is an intermediate lock wall monolith and not a dam monolith. For these later studies, the values $E[E] = 0.25$ and $\sigma_E = 0.1443$ were assumed.

ROTATIONAL STABILITY RELIABILITY INDICES

The studies in this part were performed to assess the effects of the expected value and standard deviation of E on the reliability index, β. For rotational stability (commonly called *overturning*), five different definitions of the reliability index had been defined and described in some detail in the previous report: β_{kern}, $\beta_{1/4}$, $\beta_{1/6}$, β_{toe}, and β_{FS}. The latter two are defined in terms of true limit states in that they measure a probabilistic "distance" to a condition of impending rotational instability. The first three definitions of β are measures of performance that quantify the likelihood that the effective base resultant force lies in various regions of the base. β_{FS} is a reliability index related to the probability that the factor of safety defined by the moment-ratio about the toe is greater than one. β_{toe} is a reliability index related to the probability that the effective base resultant force acts within the base, and β_{kern}, $\beta_{B/4}$, and $\beta_{B/6}$ relate to the likelihood the resultant force acts more than B/3, B/4, or B/6 centerward of the toe or edge of the base, respectively. These latter three definitions in turn are measures of the likelihood that 100, 75, or 50 percent of the base acts in compression.

RANDOM VARIABLES

To assess the effects of the parameter E on the reliability of monolith M-16, the cross-section shown in figure 2.2 was analyzed. The random variables were assigned the

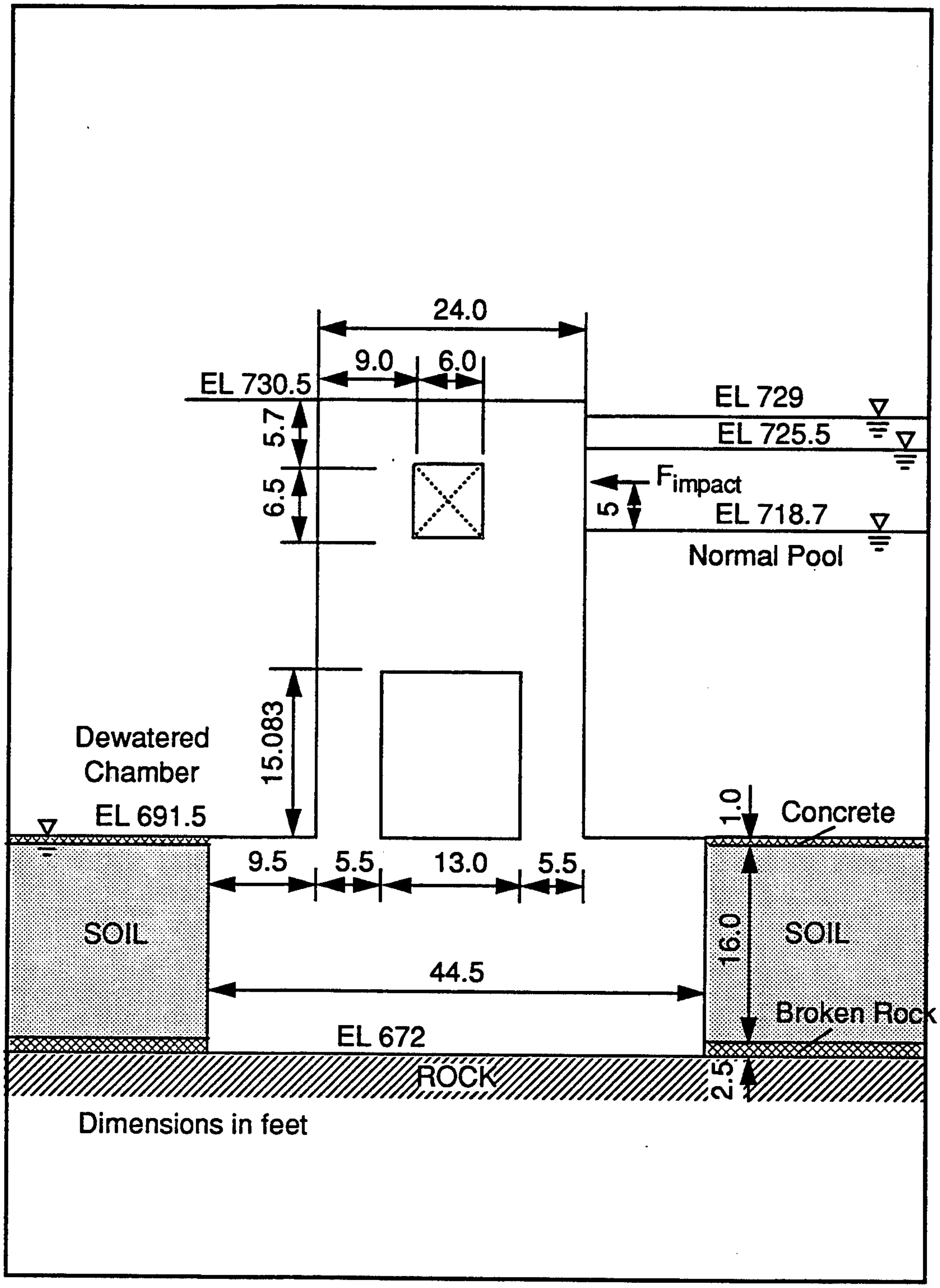

Figure 2.2 Locks & Dam No.2, Monolith M-16, Cross-Section

values shown in Table 2.1.

Table 2.1
Locks and Dam No. 2, Monolith M-16
Random Variables for Resultant Location Analyses

Variable	Expected Value	σ	V (%)
(1) γ_{soil}	0.0755	0.003775 kcf	5.0
(2) ϕ'_{soil}	33 deg	3.3 deg	10.0
(3) $\gamma_{concrete}$	0.15 kcf	0.0075 kcf	5.0
(4) Lateral Force, F	1.0 kips/ft	0.5 kips/ft	50.0
(5) Uplift parameter, E	Varies	0.2	--

ANALYSES AND RESULTS

Only the fully dewatered maintenance condition (maintenance condition (A) in the previous report) was analyzed. A free body diagram for this case taking all random variables at their expected values is shown in Figure 2.3. Probabilistic moments of the performance function for all analyses were determined using the Taylor's series method. The expected value of E was varied from -0.8 to 0.8 in increments of 0.2, and the standard deviation of E was assigned the values 0.1, 0.2, 0.4. These combinations equate to a total of 27 probabilistic analyses, each of which required 11 deterministic analyses.

The values obtained for β_{toe} and β_{FS} from these analyses are summarized in Table 2.2 and all β values are plotted in various fashions in Figures 2.4 through 2.7. It is of interest to note from Table 2.2 that the expected factors of safety against overturning

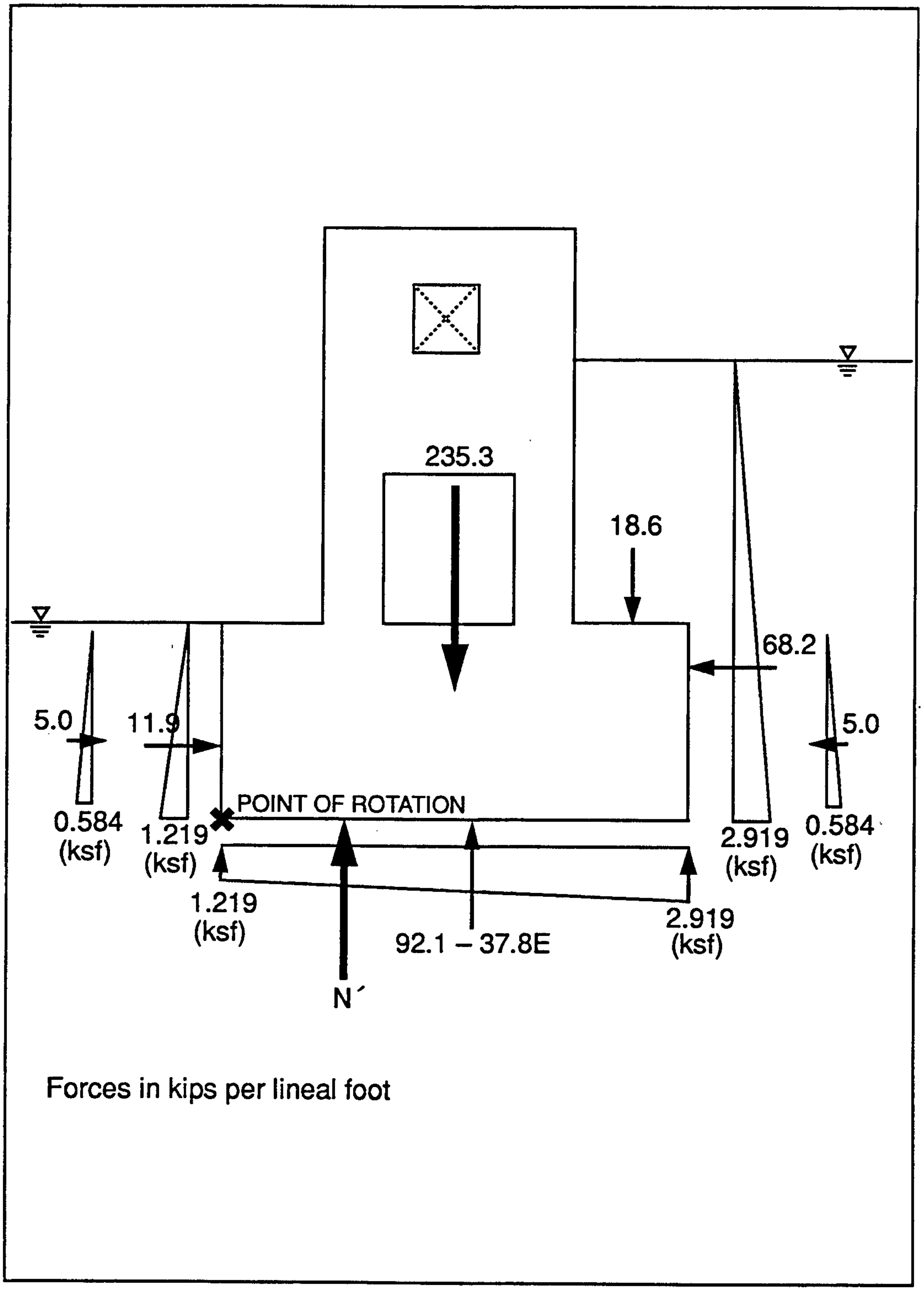

Figure 2.3 Locks and Dam No. 2, Monolith M-16,Free Body Diagram, Resultant Location Analysis—Maintenance Condition

Table 2.2
Locks and Dam No. 2, Monolith M-16
Resultant Location Analysis Results
Maintenance Condition

E[E]	σ_E	$E[X_R]$ (ft)	σ_{X_R}	β_{toe}	E[FS]	σ_{FS}	β_{FS}
-0.8	0.1	11.44	1.042	10.98	1.29	0.055	6.00
-0.6	0.1	11.25	0.964	11.66	1.32	0.058	6.24
-0.4	0.1	11.48	0.905	12.69	1.36	0.063	6.59
-0.2	0.1	12.05	0.859	14.02	1.42	0.070	7.04
0.0	0.1	12.83	0.838	15.30	1.50	0.081	7.44
0.2	0.1	13.76	0.724	19.01	1.61	0.089	8.49
0.4	0.1	14.59	0.629	23.21	1.73	0.100	9.52
0.6	0.1	15.34	0.549	27.96	1.88	0.112	10.51
0.8	0.1	16.02	0.481	33.29	2.05	0.128	11.44
-0.8	0.2	11.44	1.105	10.35	1.29	0.057	5.84
-0.6	0.2	11.25	0.965	11.66	1.32	0.064	5.65
-0.4	0.2	11.48	0.972	11.82	1.36	0.076	5.44
-0.2	0.2	12.05	1.059	11.38	1.42	0.093	5.25
0.0	0.2	12.83	1.197	10.72	1.50	0.119	5.04
0.2	0.2	13.76	1.051	13.09	1.61	0.135	5.62
0.4	0.2	14.59	0.929	15.70	1.73	0.153	6.16
0.6	0.2	15.34	0.826	18.57	1.88	0.177	6.63
0.8	0.2	16.02	0.739	21.69	2.05	0.208	7.04
-0.8	0.4	11.44	1.327	8.62	1.29	0.062	5.32
-0.6	0.4	11.25	0.967	11.63	1.32	0.083	4.32
-0.4	0.4	11.48	1.203	9.55	1.36	0.114	3.59
-0.2	0.4	12.05	1.629	7.40	1.42	0.155	3.13
0.0	0.4	12.83	2.085	6.15	1.50	0.212	2.80
0.2	0.4	13.76	1.851	7.43	1.61	0.242	3.09
0.4	0.4	14.59	1.654	8.82	1.73	0.279	3.34
0.6	0.4	15.34	1.486	10.32	1.88	0.327	3.56
0.8	0.4	16.02	1.343	11.93	2.05	0.388	3.73

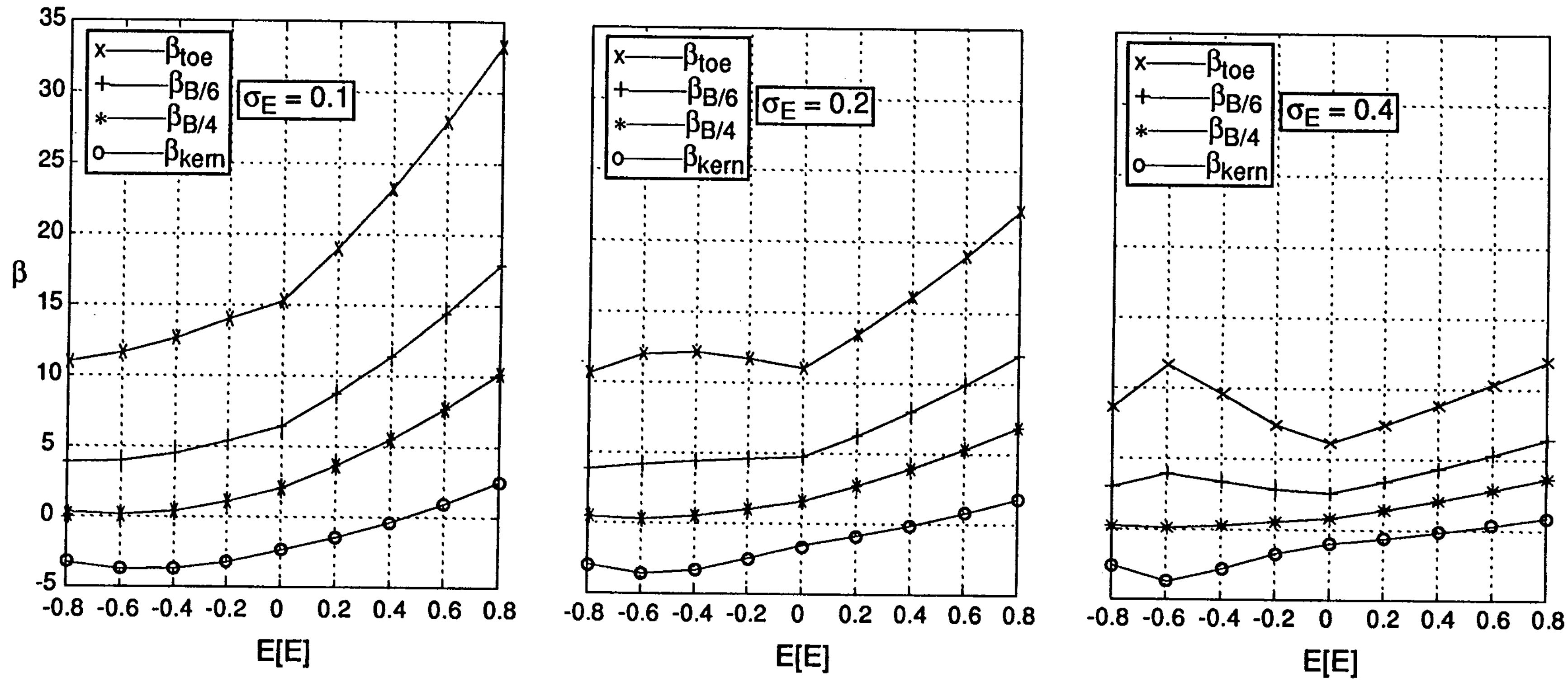

Figure 2.4 Locks and Dam No. 2, Monolith M-16, Resultant Location Analysis
β versus E[E] with $\sigma_E = 0.1, 0.2$ and 0.4

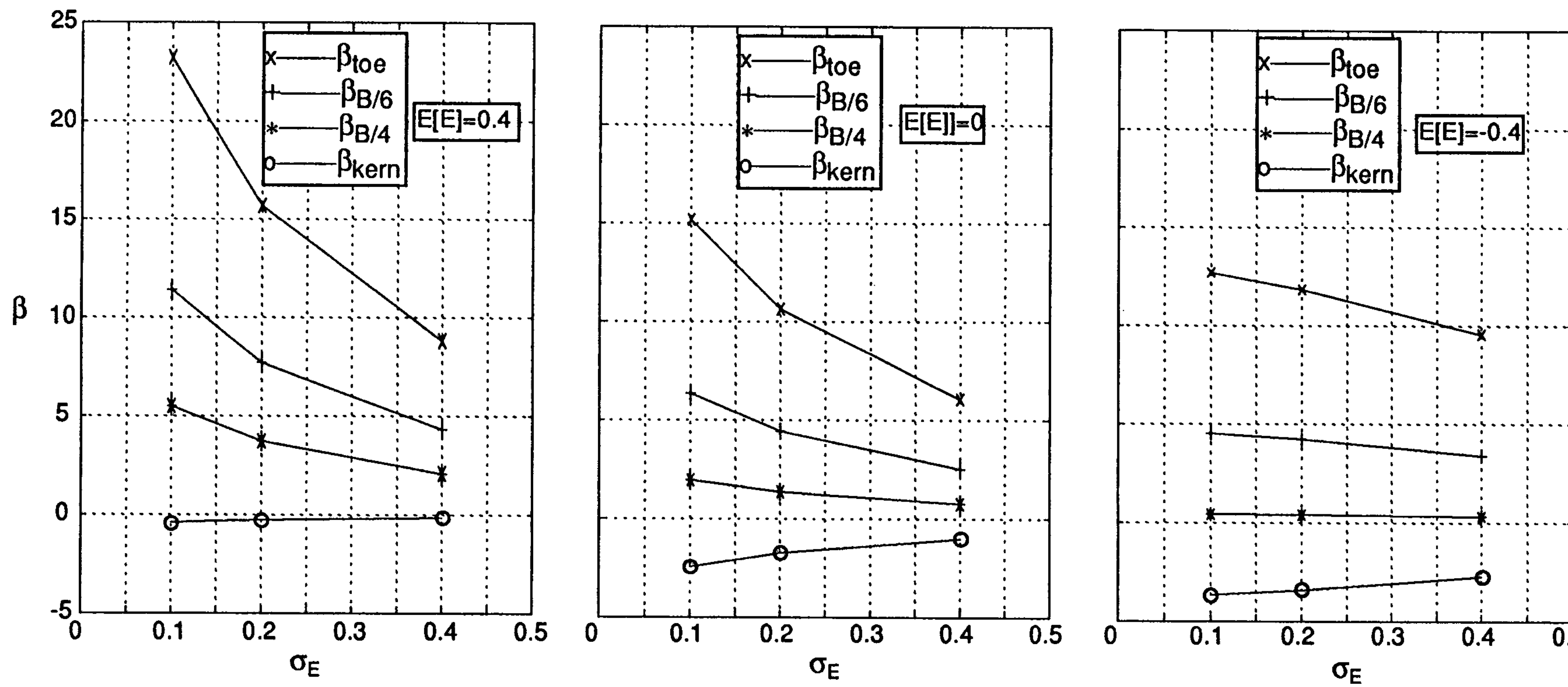

Figure 2.5 Locks and Dam No. 2, Monolith M-16, Resultant Location Analysis
β versus σ_E with E[E] = 0.4, 0 and –0.4

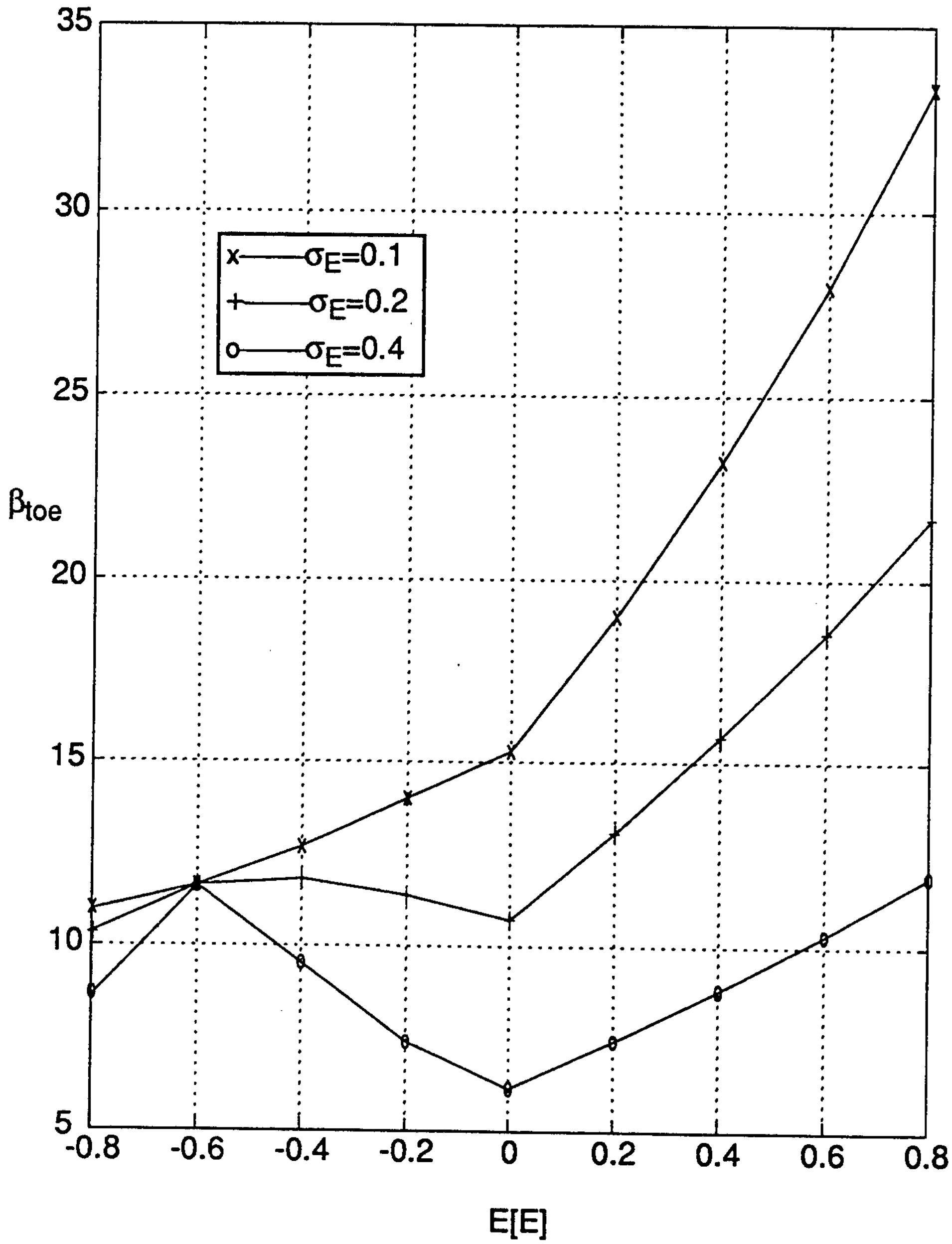

Figure 2.6 Locks and Dam No. 2, Monolith M-16, Resultant Location Analysis, β_{toe} versus E[E]

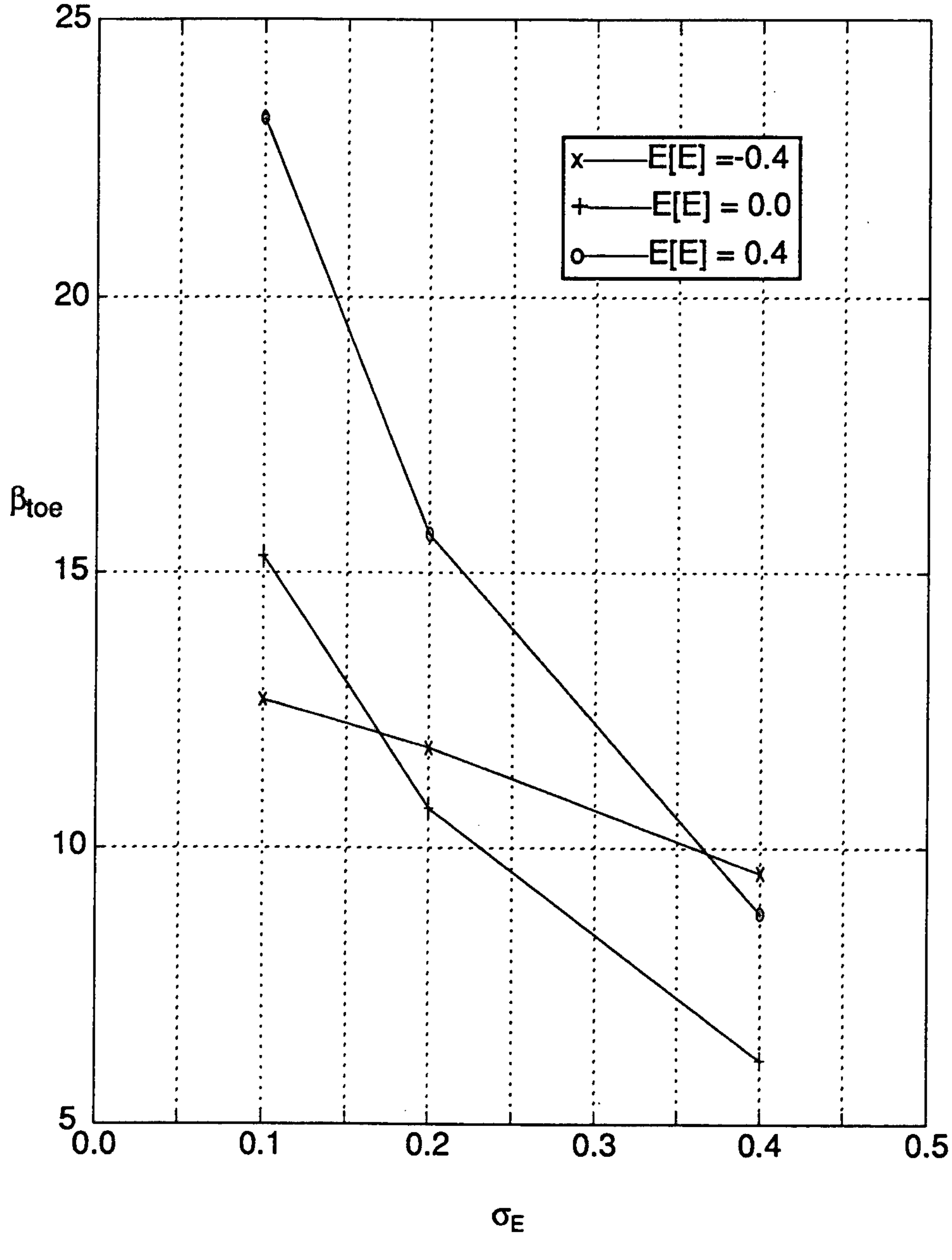

Figure 2.7 Locks and Dam No. 2, Monolith M-16, Resultant Location Analysis, β_{toe} versus σ_E

for all 27 analyses are generally below 2.0, which might commonly be considered unacceptable for deterministic results. However, the fact that the standard deviation of the factor of safety is generally below 0.10 leads to relatively large reliability indices, ranging from near 3.0 to more than 10.0. Alternatively stated, the factor of safety is not particularly high, but its range of probable values is fairly tight and there is a high degree of certainty that it is not near unity.

Figure 2.4 illustrates the effect of varying the expected value of [E] on β for different definitions of β and three values of σ_E. Where the uncertainty in E[E] is not great (σ_E = 0.1), the value of β is very sensitive to E[E]; for example, as E[E] increases from -0.8 to 0.8, β_{toe} increases from about 11 to 34 and $\beta_{B/4}$ increases from about 0 to about 10. As σ_E is increased to 0.2 or 0.4, the sensitivity of β to E[E] diminishes, but β itself decreases due to the increased uncertainty. As the uncertainty in the uplift pressure becomes relatively large (σ_E = 0.4) an interesting phenomenon is noted; the lowest values for β_{toe} and $\beta_{B/6}$ are for the condition E[E] = 0. Near this point, both the magnitude and the shape of the uplift diagram are uncertain as the entire base may be in compression or only a portion. As E[E] increases, the total uplift decreases, improving the reliability; as E[E] decreases below $-\sigma_E$, the uplift becomes more certain (it becomes certain that the base is not in compression), and β increases slightly.

Figure 2.5 plots the same information, but illustrates the effect of σ_E on β. As the standard deviation increases, the reliability index approaches zero. Alternatively stated, as the uncertainty becomes large, the reliability tends to a 50-50 situation. For the usual case where $\beta > 0$, the reliability index decreases as the standard deviation of E increases.

Figure 2.6 summarizes the variation of β_{toe} vs E[E] from the three parts of Figure 2.4 on one plot, and Figure 2.7 summarizes the variation of β_{toe} vs. σ_E from three parts of figure 2.5 on one plot. Again, it is noted that the lowest value of β occurs for E[E] = 0 and σ_E = 0.4; the uncertainty as to whether the entire base is in compression results in a lower reliability index than the case where it is known that the base is not in compression. This points out an interesting paradox in reliability theory: a condition that is known to be somewhat adverse may be more reliable than a condition that is probably not so adverse but more uncertain.

The reason for the greater sensitivity of the reliability index for positive values of E than for negative values of E is illustrated in figure 2.8. As E increases from 0.0 to 1.0, the uplift force U and net overturning moment M decrease linearly, as the point of application of U remains constant. As E decreases from 0.0 to -1.0, the uplift force increases linearly, but the moment increases at a decreasing rate as the point of application moves toward the center.

To summarize these findings, it was found that the reliability index can be relatively sensitive to both E[E] and σ_E when σ_E is reasonably small (< 0.2). A standard deviation for E greater than 0.4 implies such great uncertainty that it could be stated that uplift conditions are virtually unknown. For most studies where piezometric data are not available, assuming σ_E equal to 0.2 would appear justified. For the case of an intermediate wall monolith such as M-16 where the foundation could be either drained to or blocked from one side or the other depending on the foundation rock jointing, taking E[E] = 0 would appear to be a reasonable assumption, and uncertainty is maximized near this point. Using these two assumptions (E[E] = 0.0, σ_E = 0.2) for the maintenance condition modeled, the values β_{toe} = 10.72 and β_{FS} = 5.04 are obtained, which imply adequate reliability against overturning. These values are emphasized in Table 2.2. In the previous report, for the maintenance (A) condition where more favorable uplift conditions were assumed, β_{toe} was found to be 48 and β_{FS} was found to be 13.

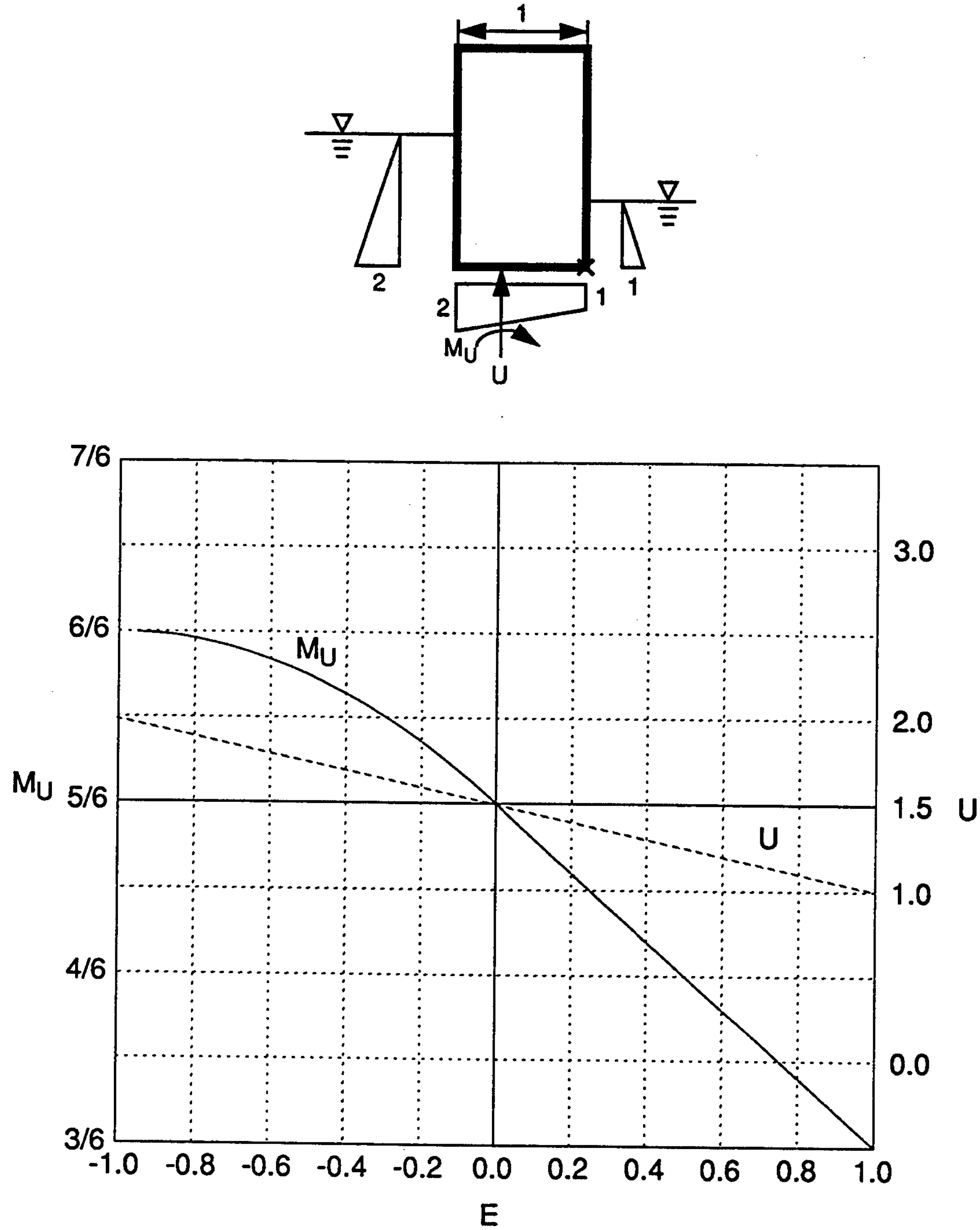

Figure 2.8 Uplift Force U and its Moment M_U versus E Factor

PART III: LOCKS AND DAM NO. 3, MONOLITH M-20, RESULTANT LOCATION ANALYSES

PROBLEM STATEMENT

In the previous report (Wolff and Wang, 1992), monolith M-20 at Monongahela River Locks and Dam No. 3 was selected for analysis because rock anchors had been installed in 1978 to improve stability against overturning, and the adequacy of the rock anchors was in question due to concerns for possible corrosion. For these follow-on studies, the following changes were made from the original analyses:

> The base elevation was taken to be a random variable. During the course of the previous study, drawings and reports for various projects made it apparent that the actual founding elevation of a particular monolith was often uncertain.
>
> The expected value of the uplift parameter E was taken as a random variable and its expected value was functionally related to the percent base in compression rather than merely assumed. This is described in more detail later in this Part.
>
> A judgmental probability that individual anchors were functional or not functional was incorporated into the analysis.
>
> Values were calculated for a new definition of the reliability index for rotational stability (overturning) suggested by the Corps of Engineers in a preliminary draft of an Engineer Technical Letter (U.S. Army, 1992).

The geometry of monolith M-20 is illustrated in the free-body diagram in Figure 3.1. The planned anchor installation called for 6 anchors, four on the tailwater side and two on the chamber side. However, only three were satisfactorily installed on the tailwater side, and only one was satisfactorily installed on the chamber side.

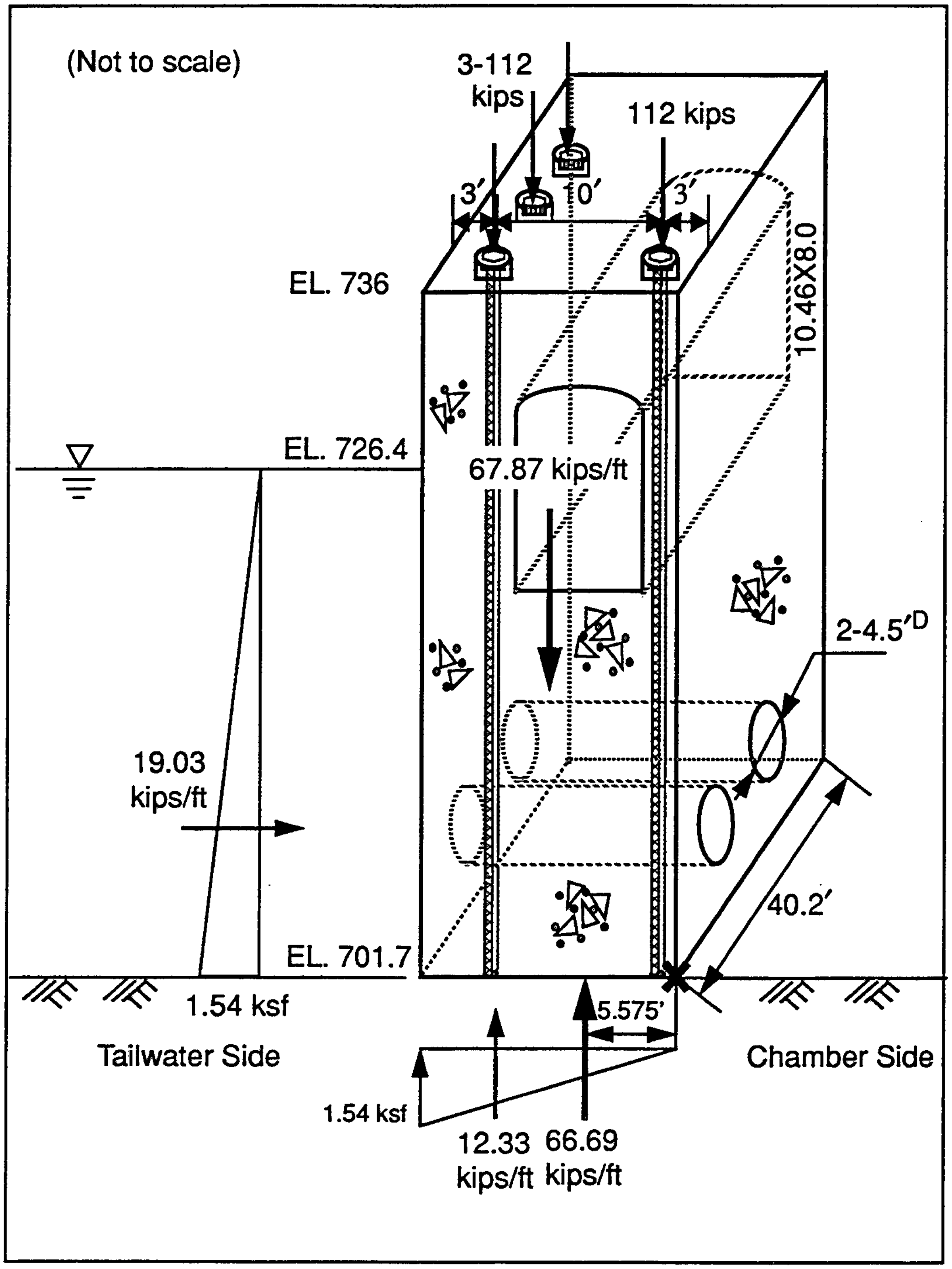

Figure 3.1 Locks and Dam No. 3, Monolith M-20, Resultant Location Analysis, Maintenance Condition (A), 3+1 Anchors

WATER LEVELS

Monolith M-20 was re-analyzed for three previously-defined water level conditions:

	Tailwater Elevation	Chamber Water Elevation
Normal Operating	726.9	718.7
Maintenance (A)	726.4	701.0
Maintenance (B)	732.0	701.0

"Tailwater" above refers to the water level in the river chamber extension, which is normally left open to tailwater. The "normal operating" case thus models pool in the extended river chamber and tailwater in the main chamber, a manner of lock operation understood to seldom occur in practice, but a combination of normal water levels in locations opposite to their usual ones that would most adversely affect stability considering the anchor layout. In addition to these previously-defined water levels, the relationship of the reliability index to the tailwater level during maintenance dewatering was assessed by performing analyses for two additional extreme tailwater levels, 715.0 and 736.0 (top of the lock wall and higher than can physically occur) and developing a set of curves.

BASE ELEVATION

Review of as-built drawings during the previous research indicated that the founding elevations of navigation structure monoliths were often uncertain due to ambiguous or incomplete records. To assess the effect of such uncertainty, the founding elevation of Monolith M-20 was taken as a random variable with an expected value of 701.7 ft and a standard deviation of 0.3 ft. As the practical range of a random variable can be taken as approximately 3 standard deviations, this assumption reflects a maximum uncertainty in the base elevation of about plus or minus 1 ft.

CHARACTERIZATION OF ANCHOR FORCES

The previous research effort included a review of published information for rock anchors in general and the rock anchor installation at Locks and Dam No. 3 in particular. For Locks and Dam No. 3, particulars of the installation were also learned from interviews with current and former Pittsburgh District personnel and from consulting reports (Colletti, 1990). From these reviews, the following information was considered in the analysis:

1. For properly installed cement-grouted rock anchors, the anchor force typically reduces a few percent in the first few days to weeks. If restressed after this initial relaxation, additional relaxation with time is usually relatively small. However, Colletti (1990), based on tests by TVA, expressed concern that creep effects may be much greater for resin-grouted anchors such as those at Locks and Dam No. 3.

2. A number of problems occurred during the installation of rock anchors at Locks and Dam No. 3. Due to hole size, anchor size, and poor mixing of epoxy resin, some anchorages never set up. Overcoring of one failed anchor showed no encapsulation by the resin over the entire bond length (Colletti, 1991). The anchor installation contract ended with fewer satisfactory anchors installed than had been specified.

3. Data on the anchor installation are incomplete and in some case ambiguous unless interpreted by witnesses to the installation. For example, it was learned that the notation "N.G." on an as-built drawing meant "no good," as expected; however, it was also learned the notation "OK" meant not that the anchor was satisfactory but rather that the anchor failed at installation but the monolith was calculated to be stable without it. (Although this fact was eventually clarified for this study, it would appear to have considerable implications to a comprehensive program of evaluation of aging navigation structures). Boring

logs and records of material quantities and anchor lengths for individual anchor installations were apparently not made or kept; the construction data consist primarily of the dates and locations of anchor installations and values of lockoff loads.

4. Below the founding elevation and within the anchor length, there is known to be a coal seam with the potential to corrode an anchor tendon not perfectly encased in grout; however, there are essentially no site-specific data available to develop a mathematical model relating anchor force to time, coal seam presence and chemical properties, grouting effectiveness, and tendon properties that would be more than sheer speculation.

Considering the above information, it was considered more reasonable to model uncertainty in anchor forces in the context of the probability of individual anchors presently carrying either the original load or no load rather than in the context of a probability-versus-time model. Alternatively stated, it was assumed that an anchor insufficiently protected by grout in the coal seam has lost tension by the present time, an anchor sufficiently protected is still carrying its initial load, and the sufficiency of protection is a random event with a judgmentally-assigned probability.

To estimate the probability of an individual random anchor being functional in 1992 (about 14 years after installation) the following question was posed to several knowledgeable personnel in various meetings and conversations:

If a random anchor were selected at Locks and Dam No. 3, and you were asked to wager as to whether it would still be functional before it is actually tested, what chance would you assign to the anchor being functional ?

The respondents expressed considerable doubt regarding anchor functionality. Based on their responses, a value of 0.50 was assigned for the probability that an individual random anchor was functional.

The probabilistic analyses were thus performed for three assumptions:

1. All (3 + 1) anchors are functional

2. No anchors are functional

3. The (3 + 1) anchors are statistically independent and each is functional with a 0.5 probability.

To model assumption (3), the binomial distribution is employed. From the binomial distribution, the probability of r out N anchors being functional if each anchor is functional with probability p is:

$$Pr(r|N,p) = \frac{N!}{r!\,(N-r)!}p^{r}(1-p)^{N-r} \tag{3.1}$$

From the above, the probability of 0, 1, 2, and 3 functional tailwater-side anchors is 1/8, 3/8, 3/8, and 1/8 respectively. As the expected total anchor force, T, can be expressed as (r)(E[T_i]) and E[T_i] = 112 for each anchor, the probability distribution on T is :

r	T	p(T)
0	0	1/8
1	112	3/8
2	224	3/8
3	336	1/8

The expected value of the total anchor force is :

$$E[T] = p_0T_0 + p_1T_1 + p_2T_2 + p_3T_3 \tag{3.2}$$
$$= (1/8)(0) + (3/8)(112) + (3/8)(224) + (1/8)(336)$$
$$= 168 \text{ kips}$$

The variance of the total anchor force is :

$$Var[T] = (1/8)(0-168)^2 + (3/8)(112-168)^2 + (3/8)(224-168)^2 + (1/8)(336-168)^2 \tag{3.3}$$
$$= 3528 + 1176 + 1176 + 3528$$
$$= 9408 \text{ kips}^2$$

and the standard deviation is the square root of the variance:

$$\sigma_T = 96.99 \text{ kips} \tag{3.4}$$

The expected value and standard deviation can also be obtained directly from the properties of the binomial distribution. For three tailwater-side anchors, each with probability 0.5 of functioning, the expected number of functional anchors is:

$$E[N_{fa}] = Np = (3)(0.5) = 1.5 \tag{3.5}$$

and the standard deviation is:

$$\sigma_{N_{fa}} = \sqrt{Np(1-p)} = 0.8666 \tag{3.6}$$

As T_i is a random variable with an expected value of 112 kips the expected value of the total tailwater-side anchor force is then:

$$E[T] = p_1E[T_1] + p_2E[T_2] + p_3[T_3] \tag{3.7}$$

$$= \quad (0.5)(112) + (0.5)(112) + (0.5)(112)$$
$$= \quad 168 \text{ kips}$$

or,

$$E[T] = E[N_{fa}]\,(112) \tag{3.8}$$
$$= (1.5)(112)$$
$$= 168 \text{ kips}$$

The variance of the total anchor force is :

$$Var[T] = E[T]^2 \sigma_{Nfa}^{\ 2} \tag{3.9}$$
$$= (112)^2(.866)^2$$
$$= 9407 \text{ kips}^2$$

and the standard deviation is the square root of the variance:

$$\sigma_T = (112)(.866) = 96.99 \text{ kips} \tag{3.10}$$

The total force from the three tailwater-side anchors can thus be modeled as a random variable with an expected value of 168 kips and a standard deviation of 97 kips. In a similar fashion, the force from the single chamber-side anchor can be modeled as a random variable with an expected value of 56 kips and a standard deviation of 56 kips.

UPLIFT PARAMETER, E, FOR BASE NOT ENTIRELY IN COMPRESSION

Where a portion of the base of a monolith is not in compression, full headwater uplift pressure is usually assumed over that part of the base not in compression (see, for example, the free body diagram in figure 3.2). In Part II of this report, it was shown that the value of the uplift parameter E could be made consistent with this assumption by taking E to be minus one plus the percent of the base in compression:

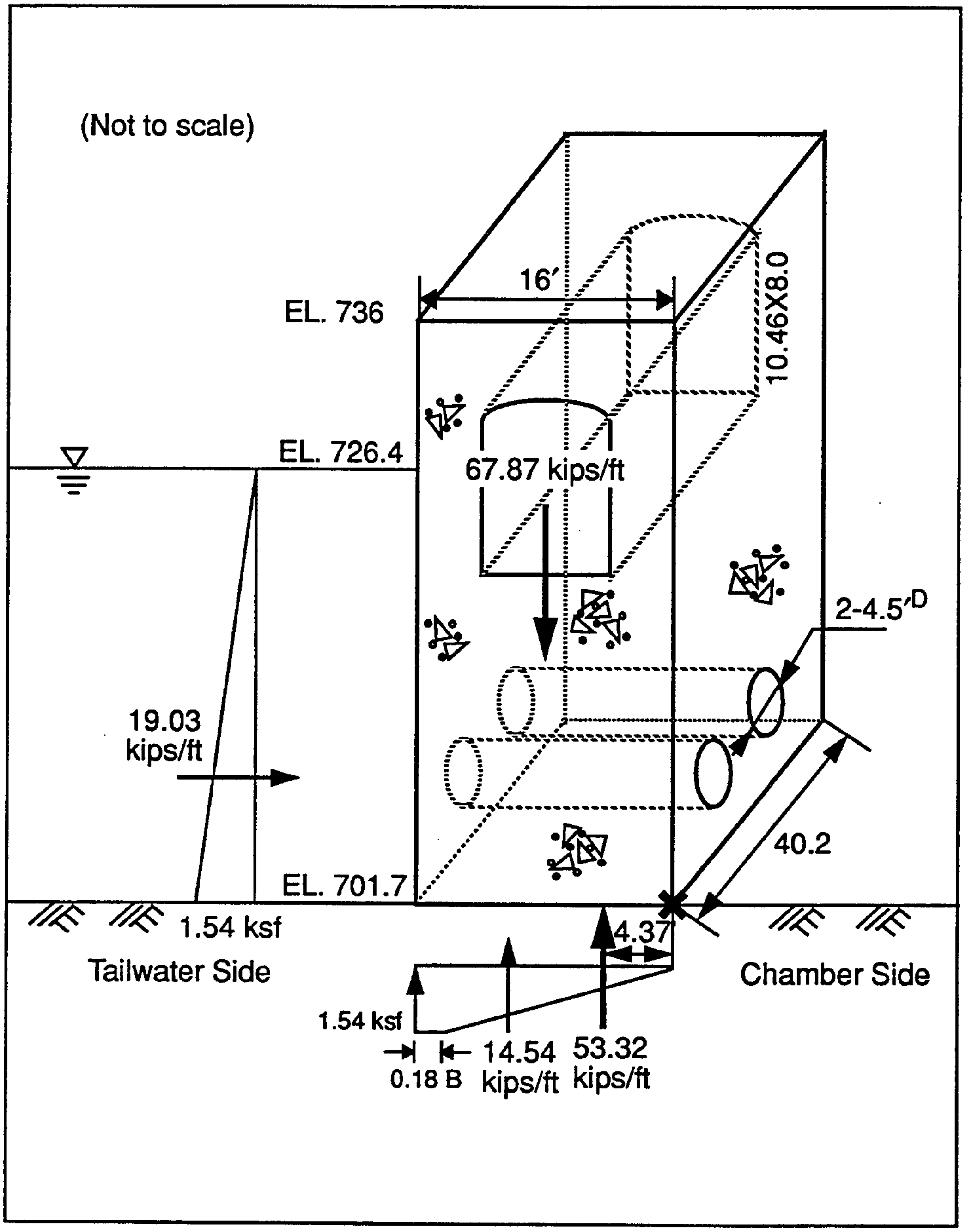

Figure 3.2 Locks and Dam No. 3, Monolith M-20, Resultant Location Analysis, Maintenance Condition (A), No Anchors

$$E = -1 + PC \tag{3.11}$$

where PC is the fraction of the base in compression. As the percent of the base in compression is a function of the random variable E, as well as other random variables, an iterative analysis is required to obtain consistent values of PC and E when the entire base is not in compression. When the entire base is in compression, E depends on the judgment of the engineer. These relationships are illustrated in figure 3.3.

For all analyses in the previous report, the expected value of E was assumed. For the analyses herein and in following parts of the report, the expected value of E was obtained by iteration. First a value for E[E] was assumed, and the percent base in compression was calculated for the condition of all random variables at their expected values. The expected value of E was then revised in accordance with equation 3.11 above and the analysis repeated until consistent values were obtained for the "expected value" case of the Taylor's series method.

RANDOM VARIABLES

Based on the above discussions and the previous report, the random variables considered for these analyses and the values assigned to them are summarized in Table 3.1.

AN ADDITIONAL RELIABILITY INDEX DEFINITION

In the previous report, several reliability index definitions were developed for rotational stability analysis, commonly called *overturning* analysis. β_{FS} is a probabilistic measure of the number of standard deviations by which the expected value of the natural log of the moment-ratio factor of safety exceeds zero. β_{toe}, $\beta_{B/6}$, $\beta_{B/4}$, and β_{kern} are probabilistic measures of the number of standard deviations of the distance by which the location of the effective base resultant force lies centerward of the toe, one-sixth point, quarter point, and one-third point of the structure base,

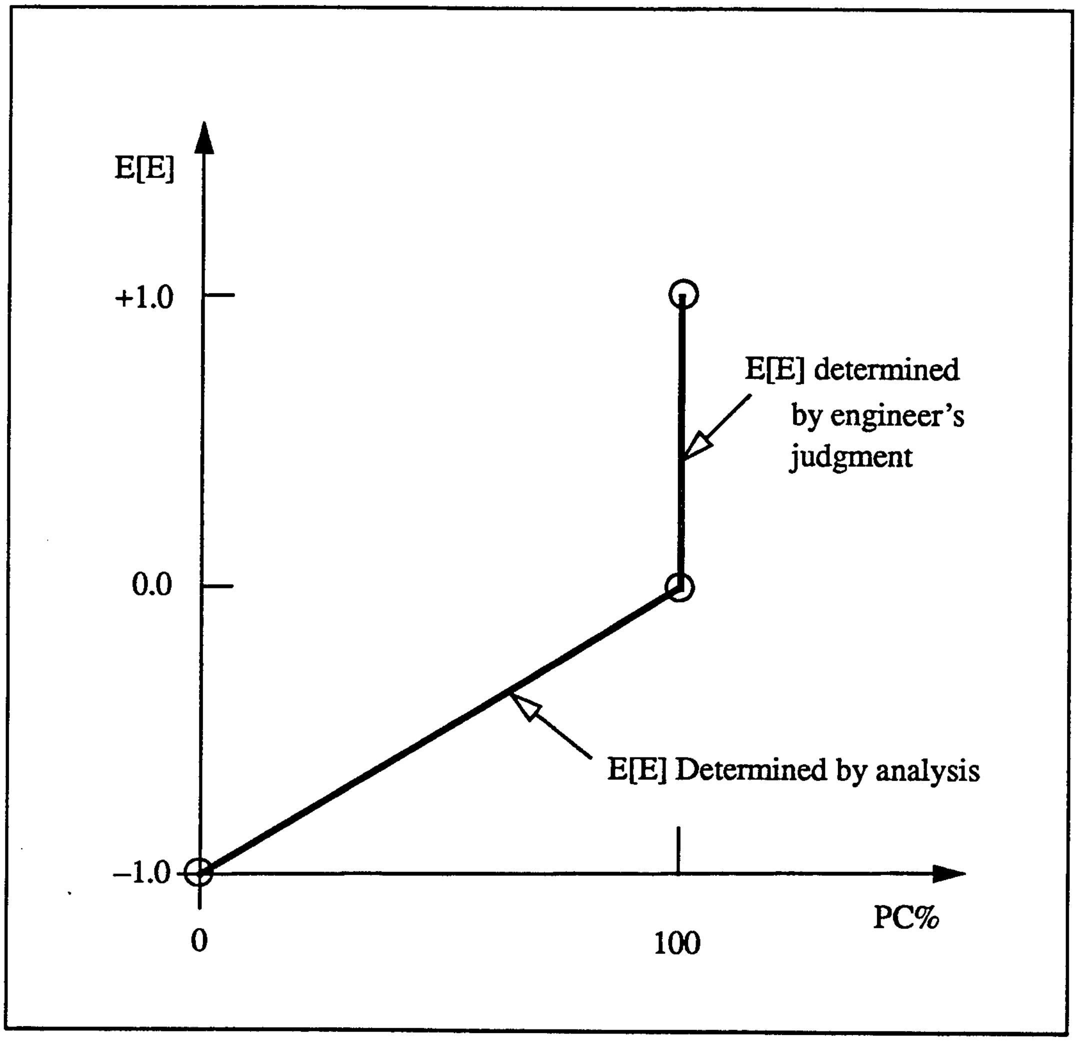

Figure 3.3 Relationship between E[E] and Percent Base in Compression

respectively. In the previous report, it was noted that only β_{FS} and β_{toe} are true limit states, consistent with common usage in structural reliability literature wherein β is a probabilistic "distance" to a limit state. The remaining definitions of β measure the "distance" between the resultant force location and arbitrary points on the structure base which can be considered "performance states."

Table 3.1

Locks and Dam No. 3, Monolith M-20

Random Variables for Resultant Location Analyses

Variable	Expected Value	σ	V (%)
(1) $\gamma_{concrete}$	0.15 kcf	0.0075 kcf	5.0
(2) Anchor Force	112 kips/ea	2/24 kips/ea	2.0
(3) Functional Anchors	0 3 1.5	0 0 0.8666	0 0 57.7
(4) Base Elevation	701.7	0.3	--
(5) Lateral Force, F	0.80 kips/ft	0.4 kips/ft	50.0
(6) Uplift Parameter, E	Varies, set by iteration	0.2	--

In February 1992, in a draft Engineer Technical Letter (ETL), the Corps of Engineers (U.S. Army, 1992), proposed another criteria for evaluating rotational stability. A shortcoming of the moment-ratio approach is that ambiguity may arise as to whether a particular moment term should be in the numerator or denominator of the ratio. For example, resisting-side water might be considered to cause a positive resisting moment or a reduction to the driving moment. To circumvent this dilemma, the Corps proposed a definition in terms of the base resultant force location, which is unique regardless of the sign of any moments. Under the Corps' definition, the ratio of capacity to demand for overturning stability would be taken as:

$$\frac{C}{D} = \frac{B}{B-2X_R} \tag{3.12}$$

where B is the base width and X_R is the distance from the toe of the base to the location of the effective base resultant force. The above definition can be shown to be equivalent to the moment-ratio factor of safety for the special case where the monolith weight acts through the midpoint of the monolith base (U.S. Army, 1992). For the above definition, it might be reasoned that the entire base width, B, represents a "capacity" to support foundation stresses, and the distance B-2X_r is a measure of how well the load on the foundation is apportioned along the base. For a resultant force at the center of the base, the denominator goes to infinity, and there is considered to be infinite safety against overturning. For a resultant force at the toe of the monolith, the denominator equals the numerator, and rotational instability is impending. The resulting reliability index, β, herein referred to as $\beta_{C/D}$, is calculated similar to β_{FS} (Wolff and Wang, 1992) with C/D substituted for FS:

$$\beta = \frac{E[\ln(C/D)]}{\sigma_{\ln(C/D)}} = \frac{\ln\left[\dfrac{E(C/D)}{\sqrt{1+V^2{}_{C/D}}}\right]}{\sqrt{\ln[1+V^2{}_{C/D}]}} \tag{3.13}$$

ANALYSES AND RESULTS

The normal operating case and the maintenance (A) case were each analyzed for three anchor assumptions, the maintenance (B) case was analyzed for five anchor assumptions, and two other maintenance conditions were analyzed for three anchor assumptions, for a total of seventeen probabilistic analyses. For each of these analyses, the reliability index was calculated for β_{toe}, β_{FS}, and $\beta_{C/D}$. Free body diagrams for the maintenance (A) condition with and without anchors are shown on Figures 3.1 and 3.2, respectively, and example calculations for the mean value analyses of these conditions are shown in Tables 3.2 and 3.3. Results of the analyses are shown in Tables 3.4 and 3.5. Comparison of the results for β_{FS} and $\beta_{C/D}$ indicate similar trends. As would be expected, the reliability increases as the tailwater level during dewatering decreases and the reliability increases as the number of functional anchors increases. For similar pool levels, values are typically lower than found in the previous report, apparently due to consideration of the parameter E.

Figure 3.4 provides some insight into the relationships among reliability index, number of functional anchors, and pool level during dewatering. Assuming that it is desired to maintain the reliability index β_{FS} above 4.0 during maintenance dewatering, the tailwater must be below about elevation 728 if no anchors are functional, and below about 731 if all anchors are functional. It appears from this plot that reliability is more sensitive to tailwater elevation than to the number of functional anchors.

CONTRIBUTIONS TO UNCERTAINTY

One advantage of using the Taylor's series method for calculating probabilistic moments is that the variance of the performance function is obtained as a summation of terms which represent the contribution to the total uncertainty associated with each random variable. These terms consist of the variance of each random variable multiplied by the square of the partial derivative of the performance function with respect to that random variable. The ratio of each of these terms to the total variance represents the contribution of each random variable to the total uncertainty. To illustrate these contributions, Table 3.6 summarizes the "partial variances" for the moment ratio factor of safety for the maintenance (A) condition with 3 + 1 fully functional anchors and 3 + 1 anchors functional with p = 0.5.

From Table 3.6, it can be seen that the uplift parameter E contributes the greatest

Table 3.2
Locks and Dam No. 3, Monolith M-20
Resultant Location Analysis Using Expected Values
Maintenance Condition (A), No Anchors

	V (kips/ft)	H (kips/ft)	Arm (ft)	M_R (kips-ft/ft)	M_O (kips-ft/ft)
Concrete	(.15)[(16)(736-701.7)–(10.46)(8)–(2)π(4.5/2)^2(16)/40.2] = (.15)(452.46)=67.87		8.0	542.95	
Pool		(.0624)(726.4-701.7)^2/2= (.0624)(305.05) =19.03	24.7/3 =8.267		156.72
Uplift force (1)	–(1.541)(16)(.18) = –4.438		(.82+.18/2) (16)=14.56		64.62
Uplift force (2)	–(1.541)(16)(1–.18)/2 =–10.109		(.82)(16)(2/3) =8.747		88.42
	ΣV= 53.32	ΣH= 19.03		ΣM_R=542.95	ΣM_O= 309.76

$SM=\Sigma M_R-\Sigma M_O= 542.95-309.76=233.19$

$X_R=233.19/53.32=4.373$ ft

PC = (3)(4.373)/(16) = 82%

FS = (542.95)/(309.76)=1.75

Table 3.3
Locks and Dam No. 3, Monolith M-20
Resultant Location Analysis Using Expected Values
Maintenance Condition (A), 3+1 Anchors

	V (kips/ft)	H (kips/ft)	Arm (ft)	M_R (kips-ft/ft)	M_O (kips-ft/ft)
Concrete	67.87		8.0	542.95	
Anchors: Outboard Inboard	 (3)(112)/40.2=8.358 (1)(112)/40.2=2.786		 13.0 3.0	 108.66 8.36	
Pool		19.03	8.267		156.72
Uplift force	−(1.541)(16)/2 = −12.328		(16)(2/3) =10.667		131.50
	ΣV= 66.685	ΣH= 19.03		ΣM_R=659.97	ΣM_O= 288.22

$SM=\Sigma M_R-\Sigma M_O= 659.97-285.12=371.75$

$X_R=371.75/66.685=5.575$ ft

PC = (3)(5.575)/(16) = 100%

FS = (659.97)/(288.22)=2.29

Table 3.4
Locks and Dam No. 3, Monolith M-20
Resultant Location Analysis Results

Water levels	Anchor	PC (%)	$E[X_R]$ (ft)	σ_{X_R} (ft)	β_{toe}	E[FS]	σ_{FS}	β_{FS}	E[C/D]	$\sigma_{C/D}$	$\beta_{C/D}$
Normal	No Anchors	89.87	4.793	.358	13.39	1.70	.114	7.85	2.49	.279	8.14
	3+1	100	5.914	.247	23.91	2.09	.130	11.84	3.83	.457	11.25
	3+1 R=0.5	100	5.433	.456	11.92	1.91	.158	7.73	3.12	.556	6.33
Maint. (A)	No Anchors	82.0	4.373	.379	11.55	1.75	.167	5.87	2.21	.343	5.04
	3+1	100	5.574	.276	20.19	2.29	.195	9.71	3.30	.348	11.29
	3+1 R=0.5	95.11	5.073	.445	11.39	2.04	.211	6.88	2.73	.453	6.03
Maint. (B)	No Anchors	5.80	0.309	.735	.42	1.02	.089	0.21	1.04	.298	0.01
	3+1	49.47	2.638	.426	6.19	1.29	.099	3.27	1.49	.296	1.94
	3+1 R=0.5	30.45	1.624	.781	2.08	1.15	.110	1.39	1.25	.314	0.80
	4+2	59.78	3.188	.380	8.40	1.40	.106	4.43	1.66	.297	2.78
	4+2 R=0.5	37.69	2.010	.763	2.63	1.20	.118	1.81	1.34	.322	1.10

Table 3.5
Locks and Dam No. 3, Monolith M-20
Resultant Location Analysis Results
Effect of Tailwater during Dewatering

Anchor	Tail-Water	PC (%)	$E[X_R]$ (ft)	σ_{X_R} (ft)	β_{toe}	E[FS]	σ_{FS}	β_{FS}	E[C/D]	$\sigma_{C/D}$	$\beta_{C/D}$
No anchors	715 *	100	7.311	.231	31.64	5.70	.684	14.48	11.62	.754	37.80
	726.4	81.99	4.373	.379	11.55	1.75	.167	5.87	2.21	.343	5.04
	732	5.80	.309	.735	0.42	1.02	.089	0.21	1.04	.297	.001
	736 *	0	-4.482	1.395	-3.21	0.78	.083	-2.38	0.64	.287	-1.25
3+1 anchors	715 *	100	7.802	.228	34.28	6.93	.806	16.62	40.46	2.616	52.25
	726.4	100	5.574	.276	20.19	2.29	.195	9.71	3.30	.348	11.29
	732	49.47	2.638	.426	6.19	1.29	.099	3.28	1.49	.296	1.94
	736 *	0	-0.752	.721	-1.04	0.95	.083	-0.62	0.91	.287	-0.45
3+1 anchors R=0.5	715 *	100	7.577	.316	23.99	6.31	.815	14.26	18.92	11.70	4.88
	726.4	95.1	5.073	.445	11.39	2.04	.211	6.88	2.73	.453	6.03
	732	30.45	1.624	.781	2.08	1.15	.110	1.39	1.25	.314	0.80
	736 *	0	-2.352	1.371	-1.72	0.87	.094	-1.37	0.77	.296	-0.88

* Cannot physically occur; for curve developoment only.

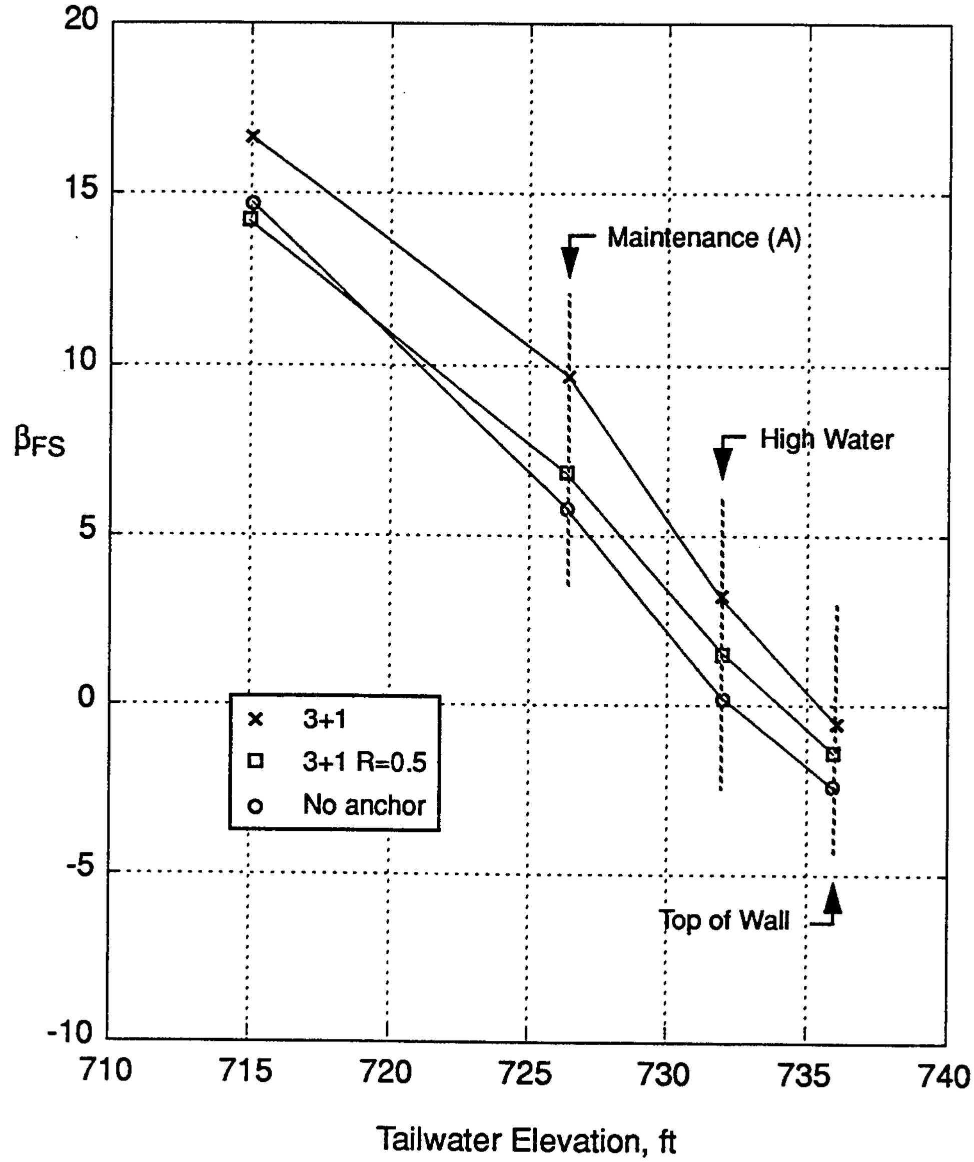

Figure 3.4 Locks and Dam No. 3, Monolith M-20, β_{FS} versus Tailwater, Resultant Location Analysis, Maintenance Condition (A)

uncertainty. If anchors are not functional, the majority of the uncertainty (61.8%) is attributable to uncertainty in E. For anchors functional with 50 percent probability, uncertainty due to E drops to 42 percent of the total and the anchors account for 26 percent. On-site investigations to better define uplift conditions and anchor forces may justify a reduced uncertainty in the random variables E and T and could lead to an increased β.

Table 3.6

Locks and Dam No. 3, Monolith M-20

Resultant Location Analyses

Contributions to Uncertainty

Variable	Maintenance (A) 3 + 1 Anchors		Maintenance (A) 3 + 1 Anchors (p=0.5)	
	Variance Component	Percent of Total Variance	Variance Component	Percent of Total Variance
$\gamma_{concrete}$	0.008 870 431	23.4	0.008 497 232	19.1
Base elevation	0.000 399 294	1.1	0.000 382 495	0.9
Tailwater-side anchor force	0.000 018 947	<0.1	0.011 343 530	25.5
Chamber-side anchor force	0.000 000 336	<0.1	0.000 213 651	0.5
Uplift parameter, E	0.023 471 670	61.8	0.018 899 900	42.4
Impact force	0.005 224 259	13.8	0.005 224 259	11.7
Total	0.037 984 940	100.0	0.044 548 780	100.0

PART IV: LOCKS AND DAM NO. 3, MONOLITH M-20, SLIDING ANALYSES

PROBLEM STATEMENT

The purpose of the studies summarized in this part was to provide a revised sliding analysis of monolith M-20 at Locks and Dam No. 3, consistent with the assumptions and revisions previously described in Part III of this report and certain other revisions. These other revisions include:

Reconsideration of the rock strength parameters

Calculation of the cohesion component of the base shear strength along only that portion of the base that is in compression.

CHARACTERIZATION OF STRENGTH PARAMETERS

During review of the previous report by the Corps of Engineers, concern was expressed regarding the appropriate probabilistic characterization of the shear strength parameters for the foundation rock at Lock and Dam No. 3. Analyses had been performed using the following peak strength parameters :

$E[c] = 11$ ksf, $\quad V_c = 70\ \%$

$E[\phi] = 56.3$ deg $\quad V_\phi = 45\ \%$

$\rho_{\phi,c} = -0.7$

Analyses had also been performed using the following residual strength parameters:

$E[\phi] = 32.6$ deg $\quad V_\phi = 50\ \%$

These parameters were based on linear regression analyses of direct shear tests on intact rock cores and engineering judgment based on consideration of the variability of test results for samples from Locks and Dams No. 2, 3, and 4.

However, the monolith was constructed in a cofferdam that could not be unwatered and is considered to be "poorly founded". In preparing the draft Engineer Technical Letter (ETL) on reliability analysis (U.S. Army, 1992), the Corps of Engineers reasoned that the appropriate strength might be modeled somewhere between peak and residual values. Assuming that each strength is representative with probability 0.5, a third set of strength parameters was obtained; these are referred to as the "ETL" strengths in Table 4.1. The "ETL" assumption that the strength may be peak or residual with probability 0.5 leads to extremely large values for the coefficients of variation, 43 percent for the ϕ parameter and 140 percent for the c parameter. Furthermore, the correlation coefficient $\rho_{\phi,c}$ goes from strongly negative to slightly positive, further increasing the uncertainty. The resulting uncertainty is so great that it is tantamount to saying the strength is virtually unknown.

It was suggested by Waterways Experiment Station personnel that the writers likewise consider other strength characterizations. As the principal investigator had no personal engineering experience with the materials in question, a survey was sent to potentially knowledgeable personnel. The plot of the previously-used strength characterizations shown in Figure 4.1 was furnished and respondents were asked to mark on this figure their own judgment as to the likely (or expected value) "operative" strength and its plus or minus one standard deviation range. The principal response came from the Pittsburgh District, who recommended that the "operative" strength be taken as a residual strength with $E[\phi] = 24$ degrees and $V_\phi = 25$ percent, a strength lower than that obtained from the regression analysis of residual strengths in the previous report. The Pittsburgh response was based on direct residual shear tests from Boring MW-7, drilled in monolith M-20 in June 1977, after the publication of the *Engineering Condition Survey* (USAEWES, 1976) that had been used as a data source for the previous report. Nevertheless, the significance of the strength value suggested by Pittsburgh District lies not so much in its lower numerical values but in the opinion that the residual strength is the appropriate characterization. Test results reported with

Table 4.1
Locks and Dam No. 3, Monolith M-20
Random Variables for Sliding Analysis

Variable	Mean	σ	V (%)
(1A) c_{rock} (Peak. MSU)	11.0 (ksf)	7.7 (ksf)	70.0
(2A) ϕ_{rock} (Peak. MSU)	52.4 °	12.9 °	24.62
$\rho_{c,\phi\,(peak)} = -0.70$			
(1B) c_{rock} (Residual, MSU)	0.0	0.0	
(2B) ϕ_{rock} (Residual, MSU)	30.5 °	13.0 °	42.6
(1C) c_{rock} (ETL)	5.5 (ksf)	7.7 (ksf)	140.0
(2C) ϕ_{rock} (ETL)	41.5 °	17.0 °	40.1
$\rho_{c,\phi\,(ETL)} = 0.19$			
(1D) c_{rock} (Residual, Pittsburgh)	0.0	0.0	
(2D) ϕ_{rock} (Residual, Pittsburgh)	24.0 °	6.0 °	25.0
(3) $\gamma_{concretel}$	0.15 (kcf)	0.0075 (kcf)	5.0
(4) Anchor force	112 (kips/anchor)	2.24 (kips/anchor)	2.0
(5) Base elevation	701.7 (ft)	0.3 (ft)	
(6) Lateral force, F	0.80 (kips/ft)	0.4 (kips/ft)	50.0
(7) Uplift parameter, E	Varying	0.2	

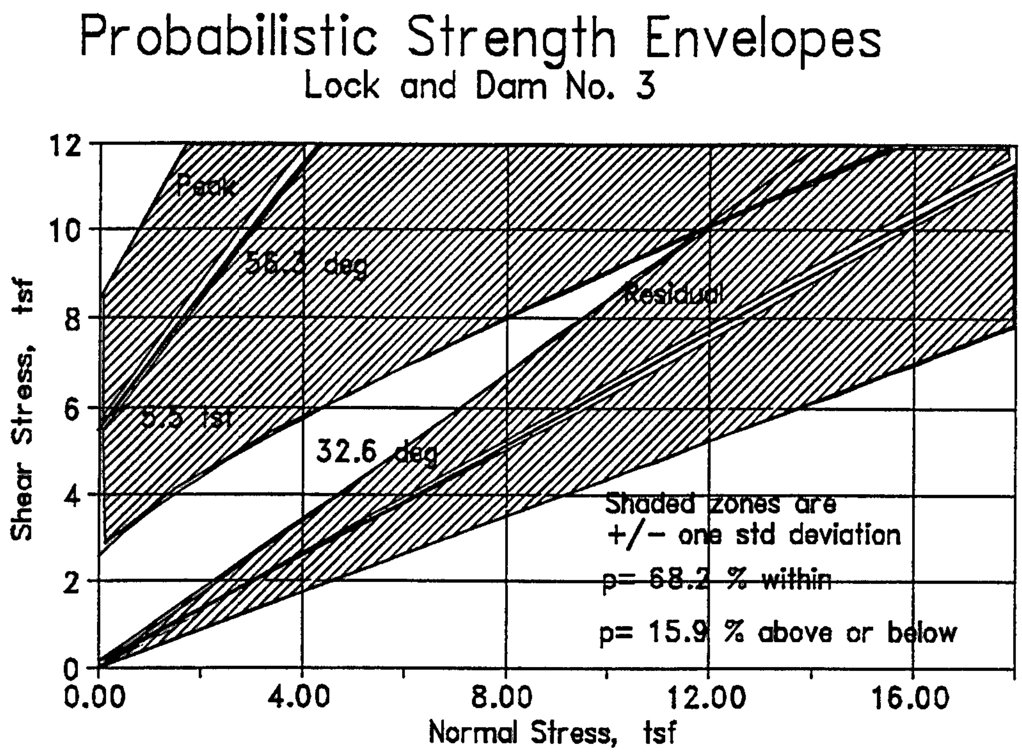

Figure 4.1 Probabilistic Strength Envelopes, Locks and Dam No. 3

the 1977 borings showed peak strengths in the range of ϕ = 52 to 63 degrees and cohesion intercepts of 0 to 26.9 ksf, and residual strengths in the range ϕ = 20 to 34 degrees, values generally similar to data previously analyzed.

To further clarify the issue of use of peak or residual strength, the literature on rock strength was reviewed, specifically the report by Nicholson (1983) for WES, a paper by Underwood (1976), former chief geologist of the Corps of Engineers, and a paper by Deere (1976). The shear behavior for discontinuous rock described by these references is illustrated in Figure 4.2. For discontinuous rock, the failure envelope is

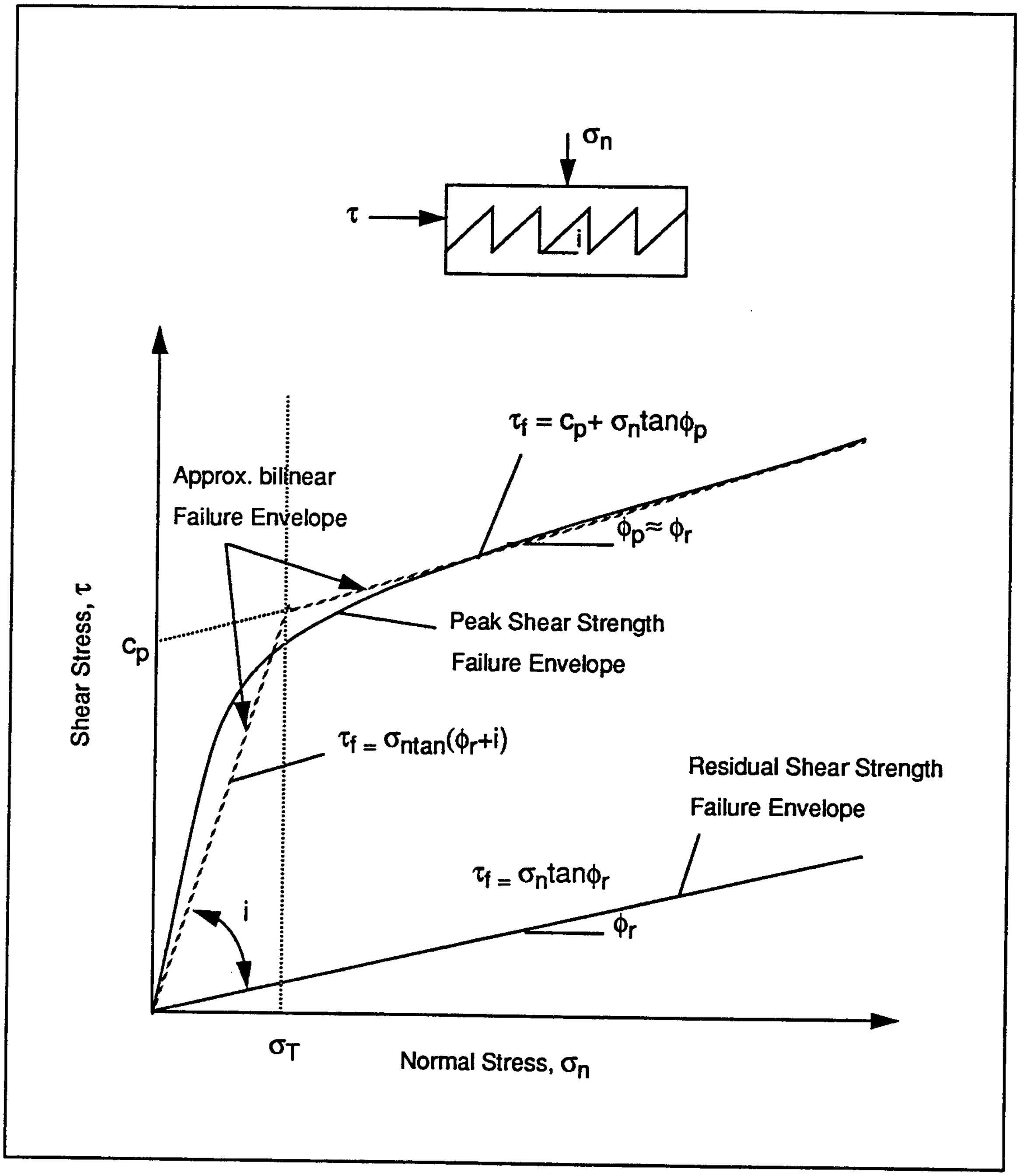

Figure 4.2 Typical Curvilinear Failure Envelope and Bilinear Approximation for Modeled Discontinuous Rock

typically curvilinear, but can be approximated as bilinear. The irregularities along rock joints (or the foundation) can be characterized by an interface angle *i* which produces an additional frictional shear resistance over and above the residual strength at low normal stresses as the strongly cohesive rock components must slide up and over each other to fail in shear. At normal stresses greater than σ_T, failure occurs through intact rock pieces and the cohesion of the intact rock governs the failure conditions. According to Deere, the *i* angle is often in the range of 5 to 15 degrees but may be as high as 30 to 40 degrees for very irregular jointing.

This curvilinear strength envelope concept is superimposed on data from the Monongahela River materials in figure 4.3. The circles marked "peak strength" and the x's marked "residual strength" represent results of the recently-furnished 1977 direct shear testing under monolith M-20; the solid lines represent the results of regression analyses on that data; and the dotted lines represent the expected values used in the previous report. The actual effective stresses at the base of the monolith are in the range 2.0 to 5.0 ksf (14 to 34 psi). In reviewing this figure, it is apparent that, if the foundation consists of broken rock components, foundation stresses may be low enough that the irregularity angle *i* may still be contributing some frictional resistance. In this case, the previous "MSU" peak strengths are somewhat conservative and not unreasonable. If, in fact, the foundation is smooth or contains a horizontal, previously sheared surface, the "Pittsburgh" or "MSU" residual strengths are appropriate. It should be recalled that, in all cases, it is implied there is a 0.5 probability that the strength is below any expected value selected.

The strength parameters used for analysis are summarized in Table 4.1, and include (A) a slightly reduced MSU peak strength that includes the combined effects of residual friction angle, interface angle, and cohesiveness of broken rock pieces at low normal stresses; (B) a slightly reduced MSU residual strength; (C) the ETL intermediate strength assumption, and (D) the Pittsburgh residual strength assumption.

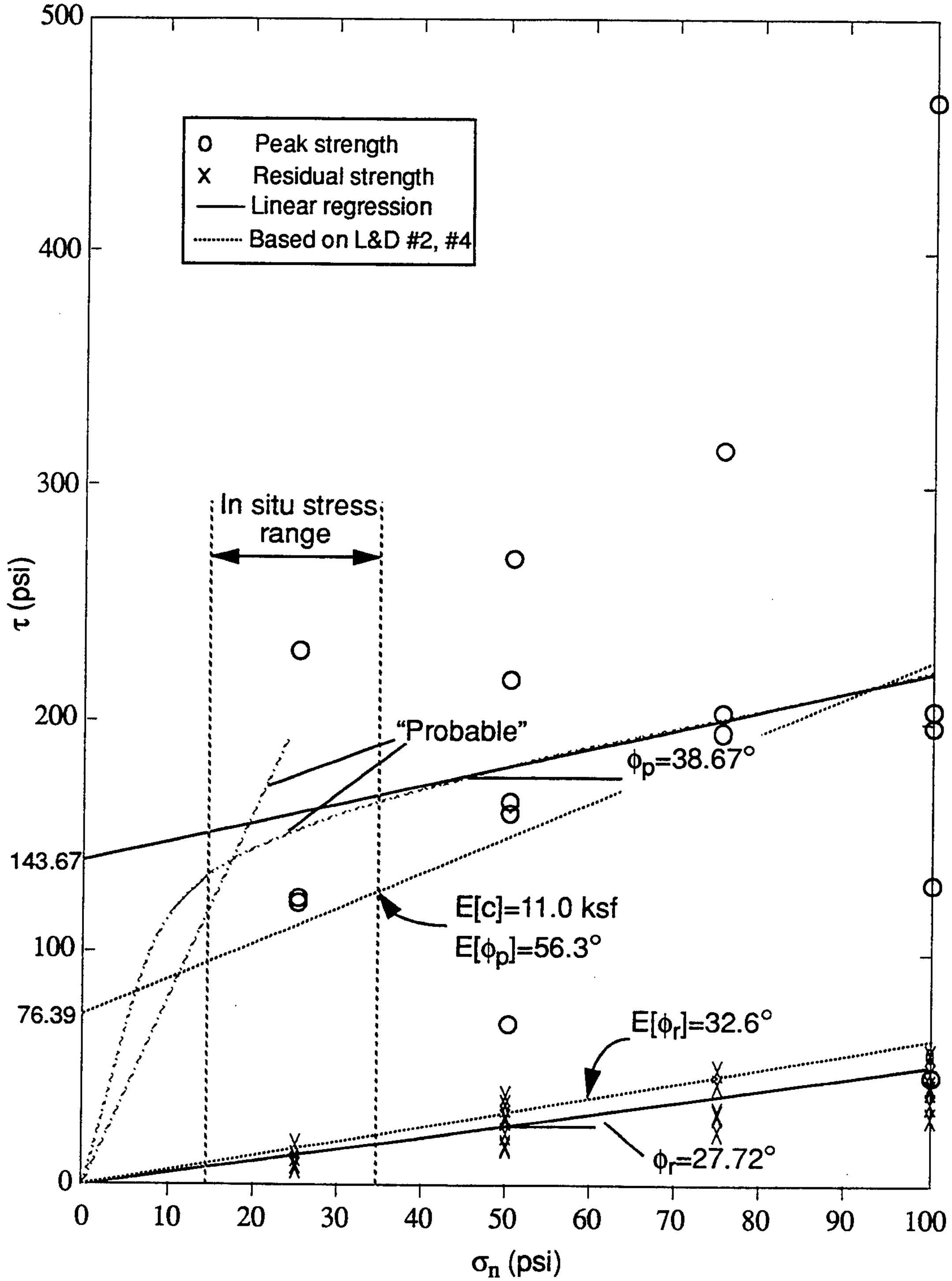

Figure 4.3 Locks and Dam No. 3, Shear Strength from Direct Shear Test Results

RANDOM VARIABLES

All assumptions regarding random variables are summarized in Table 4.1. These include the strength assumptions summarized above and the other values discussed in Part III.

ANALYSES AND RESULTS

All sliding analyses were performed using the "simple method" for sliding (Wolff and Wang, 1992) combined with the Taylor's series method for obtaining the mean and standard deviation of the factor of safety. Cohesion forces were calculated only along that part of the base in compression as obtained from the deterministic rotational analyses using expected values. The expected values of the uplift parameter, E, for each case were determined as part of the resultant location analysis described in Part III.

The results of the analyses are summarized in Tables 4.2 through 4.4 for the normal operating, maintenance (A), and maintenance (B) cases, respectively, and are discussed below.

DISCUSSION

Effect of Number of Anchors. A review of the tabulated results shows that, for the normal operating and maintenance (A) conditions, the number of functional anchors has some effect on the reliability index, β, but the effect is relatively small compared to the resultant location analyses where the moment arm of the anchors comes into play. For the more severe maintenance (B) condition, the monolith stability is greatly dependent on the additional normal force supplied by the anchors, and the number of functional anchors has a considerable effect on the reliability index.

Effect of Water Level. A review of the tabulated results indicates that the reliability index decreases somewhat going from the normal operating condition to the maintenance (A) condition, but significantly decreases for the maintenance (B)

Table 4.2
Locks and Dam No. 3, Monolith M-20
Sliding Analysis Results
Normal Operating Condition

Anchors	Strength	$E[\phi]$ (°)	σ_ϕ	$E[c]$ (ksf)	σ_c	$\rho_{c,\phi}$	$E[FS]$	σ_{FS}	β
No anchors	Peak	52.4	12.9	11.0	7.7	−0.7	19.31	8.295	6.99
	Residual	30.5	13.0	0.0	0.0	0.0	2.40	1.304	1.46
	ETL	41.5	17.0	5.5	7.7	0.19	10.61	10.576	2.43
	Pittsburgh	24.0	6.0	0.0	0.0	0.0	1.81	0.535	1.91
Anchors 3+1	Peak	52.4	12.9	11.0	7.7	−0.7	23.67	10.0	7.61
	Residual	30.5	13.0	0.0	0.0	0.0	2.98	1.615	1.89
	ETL	41.5	17.0	5.5	7.7	0.19	13.03	12.900	2.69
	Pittsburgh	24.0	6.0	0.0	0.0	0.0	2.25	0.657	2.69
Anchors 3+1 R=0.5	Peak	52.4	12.9	11.0	7.7	−0.7	21.71	9.273	7.31
	Residual	30.5	13.0	0.0	0.0	0.0	2.69	1.465	1.69
	ETL	41.5	17.0	5.5	7.7	0.19	11.93	11.879	2.57
	Pittsburgh	24.0	6.0	0.0	0.0	0.0	2.04	0.600	2.32

Table 4.3
Locks and Dam No. 3, Monolith M-20
Sliding Analysis Result
Maintenance Condition (A)

Anchors	Strength	$E[\phi]$ (°)	σ_ϕ	$E[c]$ (ksf)	σ_c	$\rho_{c,\phi}$	E[FS]	σ_{FS}	β
No anchors	Peak	52.4	12.9	11.0	7.7	−0.7	11.18	4.268	6.36
	Residual	30.5	13.0	0.0	0.0	0.0	1.65	0.895	0.72
	ETL	41.5	17.0	5.5	7.7	0.19	6.25	5.845	1.91
	Pittsburgh	24.0	6.0	0.0	0.0	0.0	1.24	0.666	0.61
Anchors 3+1	Peak	52.4	12.9	11.0	7.7	−0.7	14.18	5.397	7.03
	Residual	30.5	13.0	0.0	0.0	0.0	2.06	1.116	1.17
	ETL	41.5	17.0	5.5	7.7	0.19	7.91	7.426	2.21
	Pittsburgh	24.0	6.0	0.0	0.0	0.0	1.56	0.452	1.41
Anchors 3+1 R=0.5	Peak	52.4	12.9	11.0	7.7	−0.7	12.89	4.928	6.73
	Residual	30.5	13.0	0.0	0.0	0.0	1.87	1.014	0.98
	ETL	41.5	17.0	5.5	7.7	0.19	7.19	6.756	2.08
	Pittsburgh	24.0	6.0	0.0	0.0	0.0	1.41	0.414	1.06

Table 4.4
Locks and Dam No. 3, Monolith M-20
Sliding Analysis Results
Maintenance Condition (B)

Anchors	Strength	$E[\phi]$ (°)	σ_ϕ	$E[c]$ (ksf)	σ_c	$\rho_{c,\phi}$	$E[FS]$	σ_{FS}	β
No anchors	Peak	52.4	12.9	11.0	7.7	–0.7	2.07	1.237	1.04
	Residual	30.5	13.0	0.0	0.0	0.0	0.79	0.434	-0.72
	ETL	41.5	17.0	5.5	7.7	0.19	1.35	1.006	0.12
	Pittsburgh	24.0	6.0	0.0	0.0	0.0	0.60	0.183	-1.87
Anchors 3+1	Peak	52.4	12.9	11.0	7.7	–0.7	5.56	1.637	5.81
	Residual	30.5	13.0	0.0	0.0	0.0	1.15	0.627	0.03
	ETL	41.5	17.0	5.5	7.7	0.19	3.24	2.613	1.31
	Pittsburgh	24.0	6.0	0.0	0.0	0.0	0.87	0.257	-0.62
Anchors 3+1 R=0.5	Peak	52.4	12.9	11.0	7.7	–0.7	4.01	1.206	4.56
	Residual	30.5	13.0	0.0	0.0	0.0	0.98	0.536	-0.30
	ETL	41.5	17.0	5.5	7.7	0.19	2.39	1.810	0.96
	Pittsburgh	24.0	6.0	0.0	0.0	0.0	0.74	0.223	-1.17

condition. As was noted for the resultant location analyses, the reliability index is very sensitive to the tailwater level during dewatering.

Effect of Strength Parameters. In general the reliability index values decrease as the strength assumptions are considered in the following order:

MSU peak, ETL, MSU residual, Pittsburgh residual

For the normal operating case, the reliability index values are 7 and above for the peak strength assumption, regardless of anchor assumptions. The ETL strengths yield β values on the order of 2.5, and the MSU and Pittsburgh residual strengths yield values between 1.5 and 2.7. The force contributed by the anchors adds little to the reliability at low differential heads. For the maintenance (A) condition, peak strength assumptions yield β values between 6 and 7, ETL assumptions yield β values on the order of 2.0, and the MSU and Pittsburgh residual strengths yield β values between 0.6 and 1.4. If residual characterizations are appropriate, there is a high probability that the monolith would slide if dewatered. If it has already been dewatered without sliding, there is a high probability that the strength is greater than residual.
For the maintenance (A) case, the results are similar, except that reliability index values are moderately lower.

For the maintenance (B) case, the reliability index values are much lower, and more dependent on the anchor assumptions than for the previous three cases. For the MSU and Pittsburgh residual strength assumptions, the monolith is more likely to slide than be stable.

CONTRIBUTIONS TO UNCERTAINTY

As discussed in Part III, the Taylor's series method can be used to determine the contribution to total uncertainty from each random variable. Table 4.5 summarizes the partial variances due to each random variable for the maintenance (A) condition, using peak strengths, with 3 + 1 fully functional anchors and 3 + 1 anchors functional with

$p = 0.5$.

From Table 4.5, it can be seen that the dominant source of uncertainty in sliding is the shear strength, which accounts for about 97 percent of the total uncertainty. Uncertainty in the total anchor force, even with a fifty percent probability of anchors being functional, contributes little to the uncertainty in sliding factor of safety.

Table 4.5

Locks and Dam No. 3, Monolith M-20,

Sliding Analyses

Contributions to Uncertainty

Variable	Maintenance (A) 3 + 1 Anchors		Maintenance (A) 3 + 1 Anchors (p=0.5)	
	Variance Component	Percent of Total Variance	Variance Component	Percent of Total Variance
(1A) c_{rock}	45.532 688	97.74	37.666 404	97.29
(2A) ϕ_{rock}	5.568 977		4.582 461	
$\rho_{c,\phi}$	-22.293 445		-18.393 081	
(3) $\gamma_{concrete}$	0.400 863	1.36	0.400 863	1.64
(4) Anchor force	<0.000 001	nil	0.000 152	0.06
(5) Base elevation	0.017 956	0.06	0.017 956	0.07
(6) Lateral force, F	NA	NA	NA	NA
(7) Uplift parameter, E	0.245 123	0.8	0.245 123	1.00
Total	29.472 162	100.0	24.519 958	100.0

PART V, LOCKS AND DAM NO. 3, MONOLITH L-8, RESULTANT LOCATION ANALYSES

PROBLEM STATEMENT

Monolith L-8 is an approach wall monolith at Locks and Dam No. 3. In the previous report, it was selected for analysis because its relatively small width-to-height ratio causes its effective base resultant force to lie outside the kern. Furthermore, the earth backfill contributes uncertainty in the horizontal normal force and vertical shear force transmitted to the structural wedge. A cross-section and free-body diagram for the monolith is shown in Figure 5.1

Subsequent to the previous report, additional analyses for monolith L-8 were performed by the Corps and included in the draft ETL (U.S. Army, 1992). The analyses reported herein differ from those in the previous report as follows:

> The uplift parameter, E, was determined iteratively as described in Part III of this report.
>
> The expected value of the backfill friction angle was reduced slightly from 32 degrees to 30.
>
> An additional water level, 732.9, was analyzed to compare to the example in the draft ETL.
>
> The differential head on the wall due to water in the backfill was increased from 1 ft to 2 ft and taken as a random variable to be consistent with the draft ETL.

The analyses reported herein differ from those in the draft ETL as follows:

> At-rest earth pressure conditions were assumed and the backfill strength was

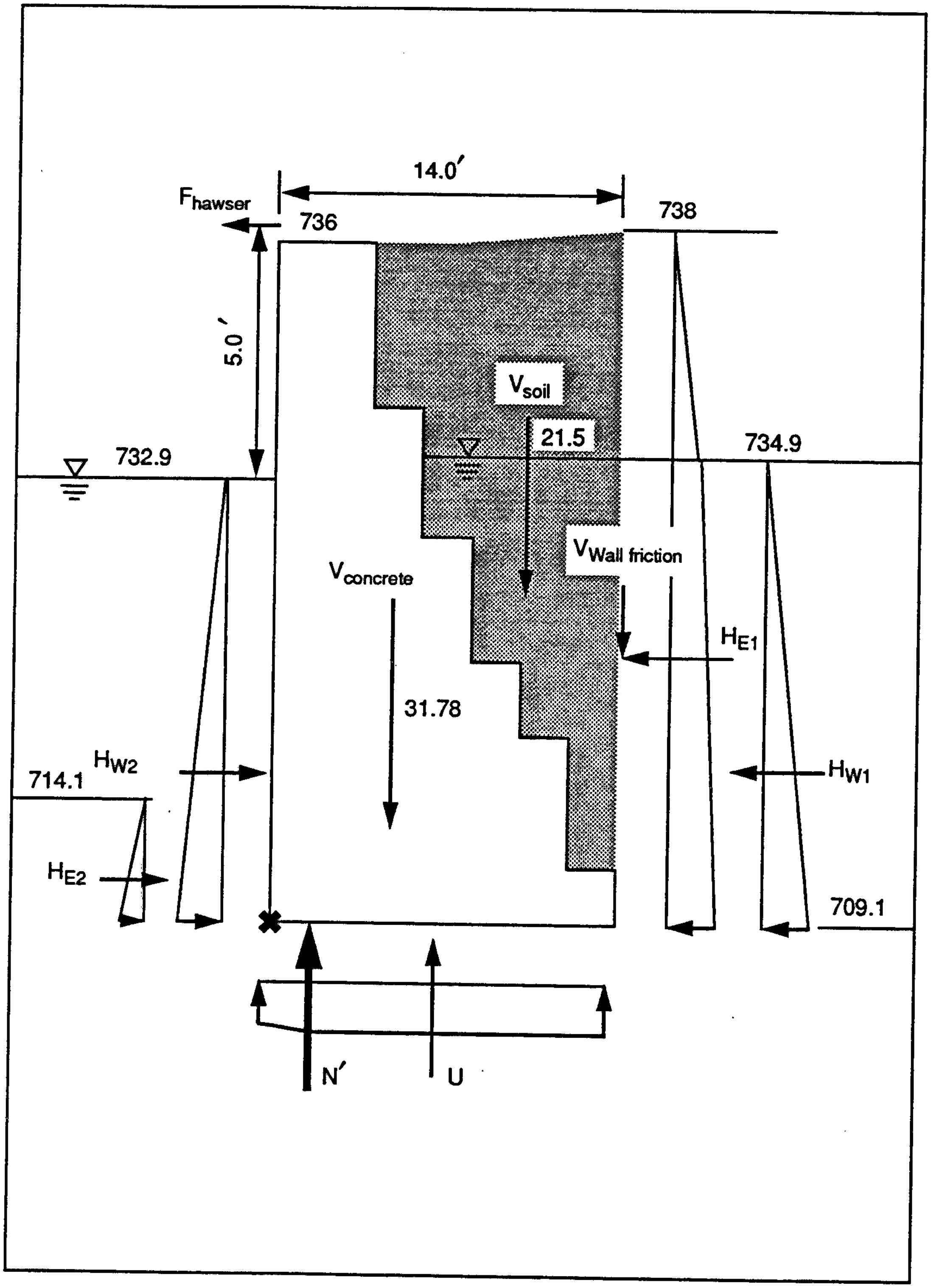

Figure 5.1 Locks and Dam No. 3, Monolith L-8, Free Body Diagram

taken as a random variable, rather than taking the earth pressure coefficient as a random variable between at-rest and active values.

The density of concrete is retained as a random variable.

Uncertainty in the uplift pressure is considered.

The Taylor's series method is used to calculate probabilistic moments rather than the point estimate method.

WATER LEVELS

Monolith L-8 is in the upper pool of Locks and Dam No. 3. It was analyzed for two pool levels, a normal pool level of 726.9, and a high water level of 732.9.

RANDOM VARIABLES

Random variables assumed for the analysis are summarized in Table 5.1. These are consistent with values used for the previous report with the exception of (4) the water elevation in the backfill, and (7) the uplift parameter, E. The water elevation in the backfill is taken as a random variable with an expected value of elevation 734.9 and standard deviation of 1.0 ft. The expected value of the uplift parameter, E, is obtained from an iterative analysis as described in Part III.

ANALYSIS AND RESULTS

An example calculation is summarized in Table 5.2. Results of the analyses are summarized in Table 5.3. Reliability index values for the two water levels and three definitions of reliability index vary between 0.14 and 0.71, with lower values for the higher water case. These are lower than the "ETL" values (1.26 to 1.28) due to the assumption of at-rest pressure conditions in the backfill. These are notably lower than values in the previous report (typically 3 to 5), apparently due to greater uplift force and considered uncertainty in the water level in the backfill.

Table 5.1

Locks and Dam No. 3, Monolith L-8

Random Variables for Resultant Location Analysis

Variable	Mean	σ	V(%)
(1) γ_{soil}	0.13 kcf	0.0065 kcf	5.0
(2) ϕ'_{soil}	30 deg	3.0 deg	10.0
(3) $\gamma_{concrete}$	0.145 kcf	0.00725 kcf	5.0
(4) Water elevation in backfill	734.9	1.0	--
(5) Wall friction angle, δ	12.0	3.0	25.0
(6) Lateral Force, F	1.0 kips/ft	0.5 kips/ft	50.0
(7) Uplift parameter, E	Varies	0.2	--

Table 5.2
Locks and Dams No. 3, Monolith L8
Stability Analysis Using Expected Values
High Water Condition (Pool: 732.9)

ITEM	VALUE	VERT. (kips/ft)	HORI. (K_0) (kips/ft)	HORI. (kips/ft)	ARM (ft)	M_R (ft-k/ft)	M_O (ft-k/ft)
Concrete	0.145	31.784			4.7536	151.088	
Earth, V_{soil}	0.130	21.502			10.1866	219.032	
Earth, H_{E1}	$k = k_0$		−16.744		10.2752		172.045
Earth, H_{E1}	$k = k_a$			−11.162			
Earth, H^0_{E2}	$k = k_0$		0.4219		10.2752	0.703	
Earth, H^p_{E2}	$k = k_p$			2.5313			
Hawser Pull			−1.0	−1.0	27.90		27.90
Water, H_{W1}	0.0625		-20.80	-20.80	8.60		178.89
Water, H_{W2}	0.0625		17.70	17.70	7.93		−140.43
Wall Friction, δ	12°, $k = k_0$	3.559			14.0	49.826	
Wall Friction, δ	12°, $k = k_a$	(2.3727)					
Uplift, U	0.0625	−22.4474			7.036		157.938
TOTAL		34.398 (33.211)	20.322	12.731	0.680	420.65	397.24

$X_R = (\Sigma M_R - \Sigma M_O)/\Sigma V = (420.65-397.24)/34.398 = 23.41/34.398 = 0.68$
$C/D = B/(B-2X_R) = (14)/[14-2(0.68)] = 1.11$
$FS = \Sigma M_R/\Sigma M_O = 420.65/397.24 = 1.06$

Note: $K=K_0$ values used for resultant location analyses;
$K=K_a$, K_p values used for sliding analyses in chapter 6.

Table 5.3

Locks and Dam No. 3, Monolith L-8

Resultant Location Analysis Results

Pool Level	PC (%)	$E[X_r]$ ft	σ_{Xr}	β_{toe}	E[FS]	σ_{FS}	β_{FS}	E[C/D]	$\sigma_{c/d}$	$\beta_{C/D}$
726.9	24.45	1.141	1.597	0.71	1.12	0.212	0.51	1.19	0.465	0.29
732.9	14.58	0.680	1.563	0.44	1.06	0.163	0.30	1.11	0.376	0.14

Contributions to Uncertainty

Partial variances for the 732.9 pool condition due to the seven random variables are summarized in Table 5.4. It is seen that the uplift pressure and the water level in the backfill are the most significant sources of uncertainty, each contributing about 30 percent of the total. However, uncertainty in the wall friction angle and the horizontal impact force are not insignificant, each contributing between 17 and 19 percent of the total uncertainty.

Table 5.4

Locks and Dam No. 3, Monolith L-8

Resultant Location Analysis (732.9 Pool)

Contributions to Uncertainty

Variable	Variance Component	Percent of Total
(1) γ_{soil}	<0.000 001	nil
(2) ϕ'_{soil}	0.000 903	3.4
(3) $\gamma_{concrete}$	0.000 362	1.4
(4) Water elevation in backfill	0.008 075	30.2
(5) Wall friction angle, δ	0.005 080	19.0
(6) Lateral Force, F	0.004 583	17.2
(7) Uplift parameter, E	0.007 695	28.8
Total	0.026 699	100.0

PART VI: LOCKS AND DAM NO. 3, MONOLITH L-8, SLIDING ANALYSES

PROBLEM STATEMENT

Monolith L-8 was illustrated in Part V of this report. The assumptions for sliding analyses summarized in this part differ from those in the previous report as discussed for the resultant location analysis in Part V. Additionally, the four strength characterizations previously described in Part IV are considered.

These analyses differ from those for monolith L-8 presented in the draft ETL in that the earth pressure conditions are taken as active and passive rather than at-rest, and uncertainty in the uplift pressure is considered, and the Taylor's series mehtod is used.

WATER LEVELS

Water levels are the same as used for the resultant location analyses in Part V, 726.9 and 732.9.

RANDOM VARIABLES

Random variables used in the analyses are summarized in Table 6.1

RESULTS

The results of the sliding analyses are summarized in Table 6.2. For the reliability index denoted as β^1, the passive force was treated as a resistance; for the reliability index denoted as β^2, the passive force was treated as a reduction to the driving force. The difference in these definitions made little difference in β. For peak strengths, the reliability index was found to be between 1.60 and 2.40, depending on definition and water level. The ETL strength gives 1.21 and 0.96 (vs. 1.43 in the draft ETL). For the two residual strength assumptions, the reliability index was found to be between 0.29 and 0.34, a very low reliability. Similar to the findings for monolith M-20 in Part IV, the results suggest that the operative strength is greater than the residual strength, as there is a high probability that the monolith should have already slid given

Table 6.1
Locks and Dam No. 3, Monolith L-8
Random Variables for Sliding Analysis

Variable	Mean	σ	V (%)
(1A) c_{rock} (Peak. MSU)	11.0 (ksf)	7.70 (ksf)	70.0
(2A) ϕ_{rock} (Peak. MSU)	52.4 °	12.9 °	24.62
$\rho_{c,\phi\ (peak)} = -0.70$			
(1B) c_{rock} (Residual, MSU)	0.0	0.0	
(2B) ϕ_{rock} (Residual, MSU)	30.5 °	13.0 °	42.6
(1C) c_{rock} (ETL)	5.5 (ksf)	7.7 (ksf)	140.0
(2C) ϕ_{rock} (ETL)	41.5 °	17.0 °	40.1
$\rho_{c,\phi\ (ETL)} = 0.19$			
(3) γ_{soil}	0.13 (kcf)	0.0065 (kcf)	5.0
(4) ϕ'_{soil}	30 °	3.0 °	10.0
(5) $\gamma_{concretel}$	0.145 (kcf)	0.00725 (kcf)	5.0
(6) Saturation elevation	734.9 (ft)	1.0 (ft)	
(7) Wall friction angle	12.0 °	3.0 °	25.0
(8) Lateral force, F	1.0 (kips/ft)	0.5 (kips/ft)	50.0
(9) Uplift parameter, E	Varying	0.2	

Table 6.2
Locks and Dam No. 3, Monolith L8
Sliding Analysis Results

Pool level	Strength	$E[\phi]$ (°)	σ_ϕ	E[c] (ksf)	σ_c	$\rho_{c,}$	E[FS] (1)	σ_{FS}	β^1	E[FS] (2)	σ_{FS}	β^2
726.9	Peak	52.4	12.9	11.0	7.7	–0.7	5.24	3.677	2.30	5.97	4.373	2.40
	Residual	30.5	13.0	0.0	0.0	0.0	1.47	0.771	0.54	1.56	0.907	0.55
	ETL	41.5	17.0	5.5	7.7	0.19	3.23	2.878	1.15	3.60	3.394	1.21
	Pittsburgh	24.0	6.0	0.0	0.0	0.0	1.15	0.338	0.34	1.18	0.399	0.32
732.9	Peak	52.4	12.9	11.0	7.7	–0.7	4.46	3.912	1.60	5.15	4.747	1.70
	Residual	30.5	13.0	0.0	0.0	0.0	1.45	0.741	0.53	1.54	0.893	0.53
	ETL	41.5	17.0	5.5	7.7	0.19	2.83	2.670	0.90	3.19	3.225	0.96
	Pittsburgh	24.0	6.0	0.0	0.0	0.0	1.13	0.324	0.31	1.16	0.392	0.29

(1)—$FS = (T + H^{P}_{E2})/(\Sigma H - H^{P}_{E2}) = [\Sigma V\tan\phi + 3X_{R}c + H^{P}_{E2}]/(\Sigma H - H^{P}_{E2})$
(2)—$FS = T/\Sigma H = [\Sigma V\tan\phi + 3X_{R}c]/\Sigma H$

the residual characterization.

CONTRIBUTIONS TO UNCERTAINTY

Partial variances contributed by each of the random variables are summarized in Table 6.3. It is noted that the uncertainty is shared among a number of random variables, with no single variable predominating. The most significant random variables are, in descending order, the water level in the backfill, the uplift parameter E, the friction angle of the backfill soil, and the rock strength.

Table 6.3

Locks and Dam no. 3, Monolith L-8

Sliding Analysis (732.9 Pool, Peak Strengths)

Contributions to Uncertainty

Variable	Variance Component	Percent of Total
(1) c_{rock}	1.060 565	7.72
(2) ϕ_{rock}	2.313 651	
$\rho_{c,\phi}$	-2.193 035	
(3) γ_{soil}	0.005 535	0.04
(4) ϕ'_{soil}	1.208 779	7.90
(5) $\gamma_{concrete}$	0.294 811	1.93
(6) Water in backfill	4.328 104	28.28
(7) Wall friction angle, δ	2.418 910	15.80
(8) Lateral Force, F	2.005 551	13.10
(9) Uplift parameter, E	3.864 007	25.24
Total	15.306 878	100.0

PART VII: DEMOPOLIS LOCKS AND DAM, MONOLITH L-17, RESULTANT LOCATION ANALYSES

PROBLEM STATEMENT.

Demopolis Lock and Dam is located on the Tombigbee Waterway near Demopolis, Alabama. Monolith L-17 was described and analyzed in the previous report as a case history of a structure where remedial action was taken to correct a perceived design deficiency. Specifically, water in the backfill behind the lock wall was found to be higher than had been assumed for design, in turn causing the percent of the base in compression to be less than permitted by Corps' criteria at the time of the fifth periodic inspection in 1987. To improve rotational stability, 20 feet of backfill was removed and a drainage system was installed. The studies in the previous report compared the reliability index values before and after backfill removal, and showed that reliability had in fact been substantially increased by removal of the fill.

A free-body diagram of monolith L-17 before backfill removal is shown in Figure 7.1. Two separate studies are reported in this chapter:

Parametric studies using random variables from the original report.

A "best estimate" analysis using adjusted random variables.

The parametric studies investigated the effects of the uplift parameter E, the wall friction angle, δ, and the height of the backfill level on the reliability indices. These studies were performed early in the Phase II work and used the same standard deviation for the backfill water level (6.8 ft) as the first report; this value that was later realized to be unrealistically large. The "best estimate" analyses were based on revised piezometric levels reflecting piezometric data adjusted to the location of monolith L-17.

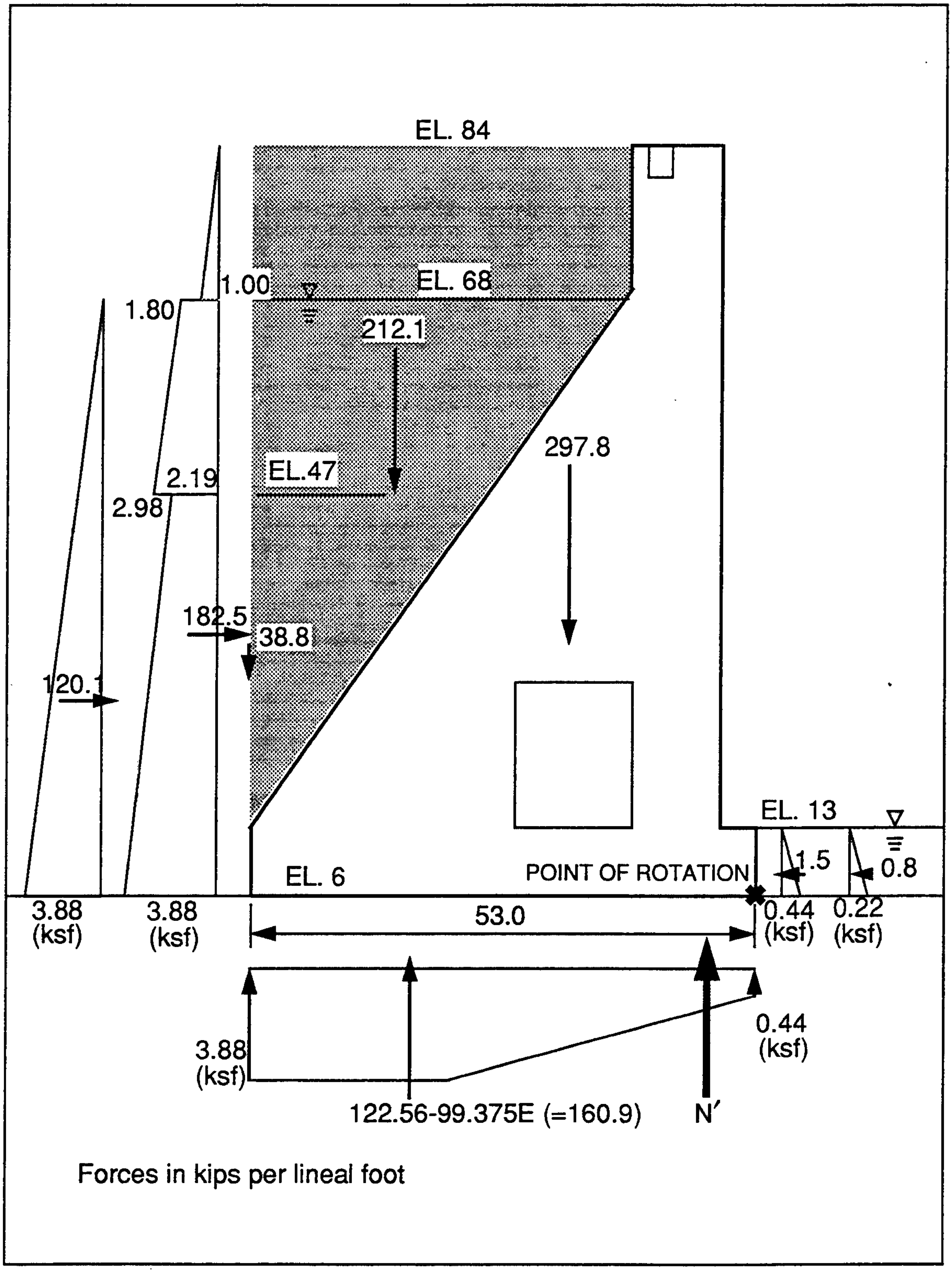

Figure 7.1 Demopolis Locks and Dam, Monolith L-17, Resultant Location Analysis, Free Body Diagram

WATER LEVELS

For all but four points of the parametric studies, analyses herein were performed for the maintenance dewatering condition, with water in the chamber drawn down to elevation 13.0. Water in the backfill was taken as a random variable; for all parametric studies, it was assigned an expected value of 68 ft and a standard deviation of 6.8 ft. For the "best estimate" analyses of the "before backfill removal" condition, it was assigned an expected value of 67 feet and a standard deviation of 1.7 ft based on measured piezometric elevations. For the "best estimate - after backfill removal" condition, it is assumed that soil has been removed to elevation 64 and a drainage system has been provided; the backfill water elevation was taken as a random variable with an expected value of 61 ft and a standard deviation of 1.5 ft.

RANDOM VARIABLES

Random variables considered for the various analyses and their probabilistic moments are summarized in Table 7.1. Variable (2), the friction angle of the embedment rock, was used to calculate at-rest pressure on the resisting side of the monolith. Variable (4) represents the set of earth pressure coefficients used on the driving side of the wall from top to bottom. For computational simplicity, these were taken to vary together (implying perfect correlation among the three layers); the earth pressure diagram is randomly higher or lower than the expected value. For the "after backfill removal' condition, the upper material is not present. Variable (5), δ, has been referred to as the "wall friction" angle in this and the previous report; in fact, it represents the angle of developed shear on a vertical plane from the heel of the monolith through the soil backfill. Variable (7), the lateral force, was only used for the four points in the parametric studies corresponding to normal pool or high water; it was deleted for the maintenance condition studies including the "best estimate analyses" as such a force should not be present during a dewatered condition.

Table 7.1

Demopolis Locks and Dam, Monolith L-17

Random Variables for Resultant Location Analysis

Variable	Expected Value	σ	V (%)
(1) γ_{conc}	0.15 kcf	0.0075 kcf	5.0
(2) $\phi_{rock\ embed}$	30 deg	11.46 deg	38.2
(3) γ_{soil}	0.125 kcf	0.00625 kcf	5.0
(4)$K_{backfill}$	0.5 (upper) 0.9 (CH,CL) 0.66 (SC,SM)	0.05 0.09 0.066	10.0
(5) δ	12 deg	3 deg	25.0
(6)Water Level in Backfill	68.0^{p} 68.0^{bb} 61.0^{ba}	6.8^{p} 1.7^{bb} 1.5^{ba}	-- -- --
(7) Lateral Force F	1.0^{nh} 0.0	0.5^{nh} 0.0	50.0 0.0
(8) Uplift parameter, E	Varies[p] Varies[bb,ba]	Varies[b] $0.2^{bb,ba}$	--

Notes

Notes	p	parametric studies
	bb	best estimate - before backfill removal
	ba	best estimate - after backfill removal
	nh	normal pool and high water cases

RESULTS OF PARAMETRIC STUDIES

Parametric studies investigated the effects of three different random variables: the uplift parameter E, the wall friction (vertical shear) angle δ, and the height of backfill.

Effects of Uplift Parameter E on Reliability Index. Figures 7.2 through 7.5 summarize the results of parametric studies to assess the effect of the expected value and standard deviation of the uplift parameter E on reliability indices. Figure 7.2 shows trends very similar to those shown in Figure 2.4 for monolith M-16 at Lock and Dam No. 2; the reliability increases with increasing E[E] and decreases with increasing σ_E. Likewise, Figure 7.3 shows trends very similar to those in Figure 2.5. The major difference from Demopolis to Lock and Dam No. 2 is that the reliability indices for the Demopolis monolith are much lower and are somewhat less sensitive to σ_E. It should be recalled that these values may be lower than "actual" due to the large variance assumed for the backfill water level, however it is clear that both E[E] and σ_E are significant random variables which must be considered in analysis. Figures 7.4 and 7.5 combine the information for β_{toe} in the previous two figures on a single graph for comparison; again, it is noted that β may tend to a minimum near E[E] = 0 when σ_E is large.

Effects of Backfill Level and Wall Friction on Reliability Index. For these parametric studies, iterative analyses were first made using expected values to relate the percent base in compression to the wall friction angle and the height of backfill. Results are shown in Figure 7.6. These in turn set the expected value of E to be used in the various analyses. Table 7.2 summarizes the relative effect of the backfill removal for three water level conditions in the lock chamber taking the expected value of the wall friction angle at 12 degrees. It is seen that the increase in reliability due to backfill removal is greater when expressed in terms of β_{toe} than in terms of β_{FS}, but in either case is significant. Table 7.3 summarizes the detailed effects of backfill level and wall friction angle for the maintenance dewatering case, and some of the results are plotted in Figures 7.7 through 7.10. Noting the different scales in Figures 7.7 and 7.9, which plot β_{toe} versus backfill removal and wall friction angle, it is noted that the

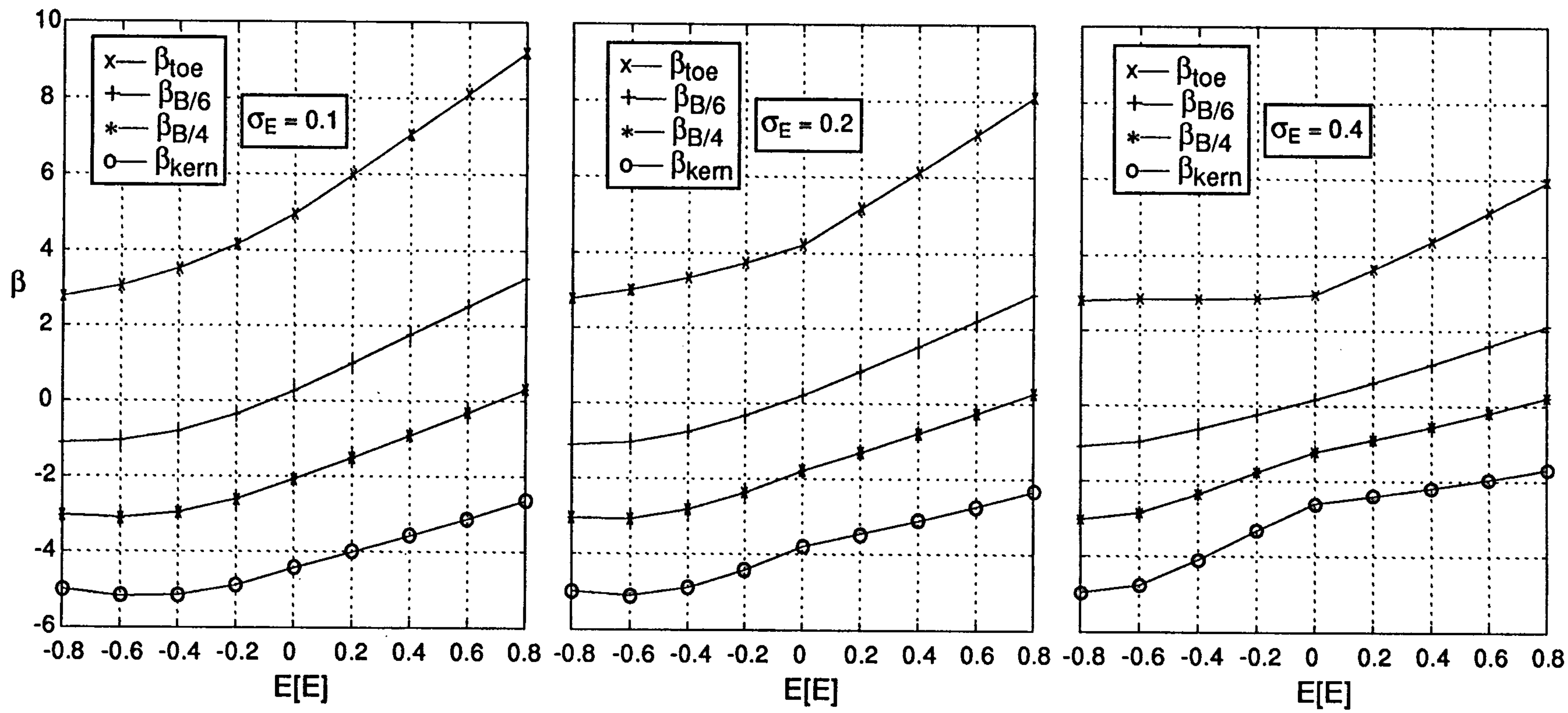

Figure 7.2 Demopolis Locks and Dam, Monolith L-17, Resultant Location Analysis

β versus E[E] with σ_E = 0.1, 0.2 and 0.4

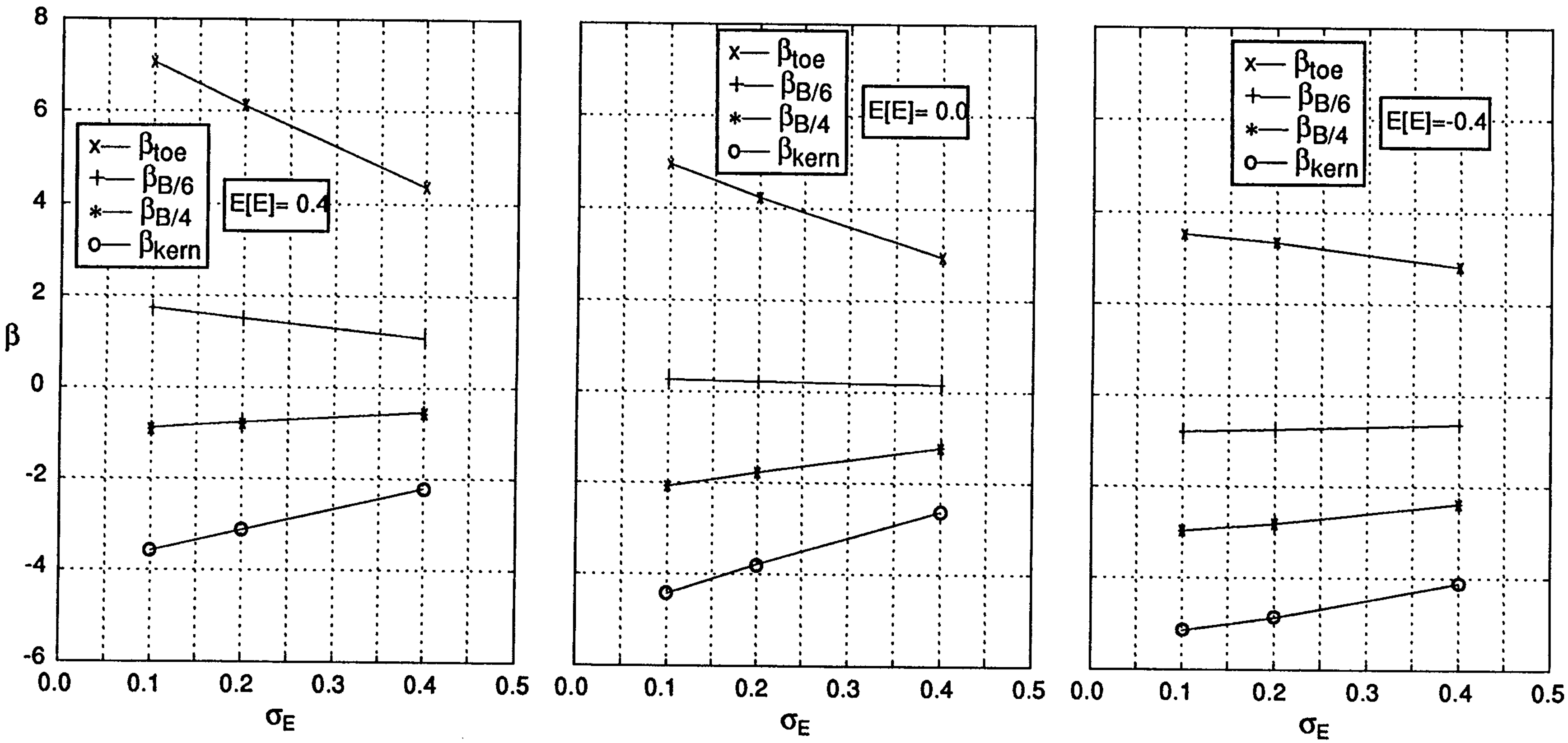

Figure 7.3 Demopolis Locks and Dam, Monolith L-17, Resultant Location Analysis

β versus σ_E with E[E] = 0.4, 0.0 and –0.4

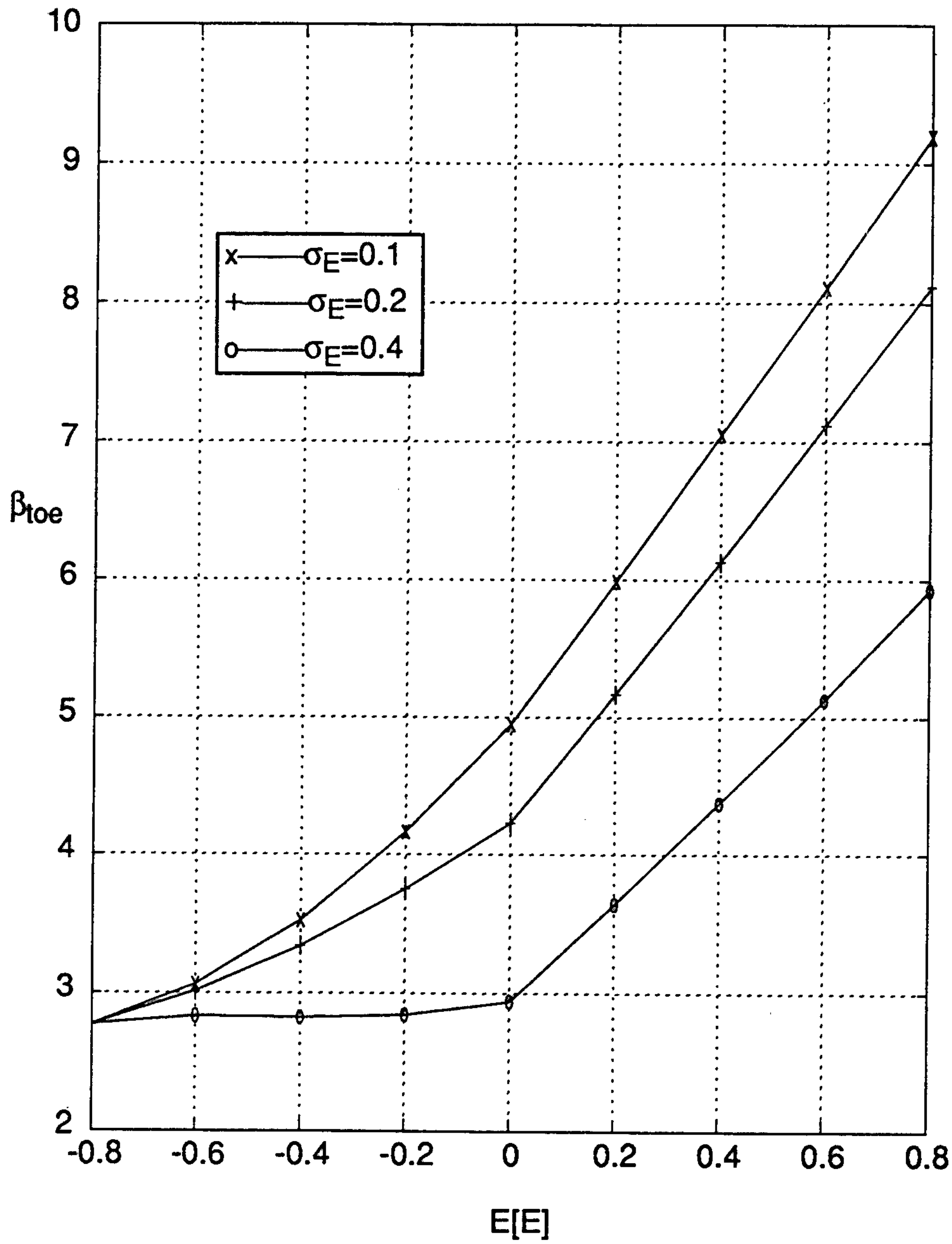

Figure 7.4 Demopolis Locks and Dam, Monolith L-17, Resultant Location Analysis, β_{toe} versus E[E]

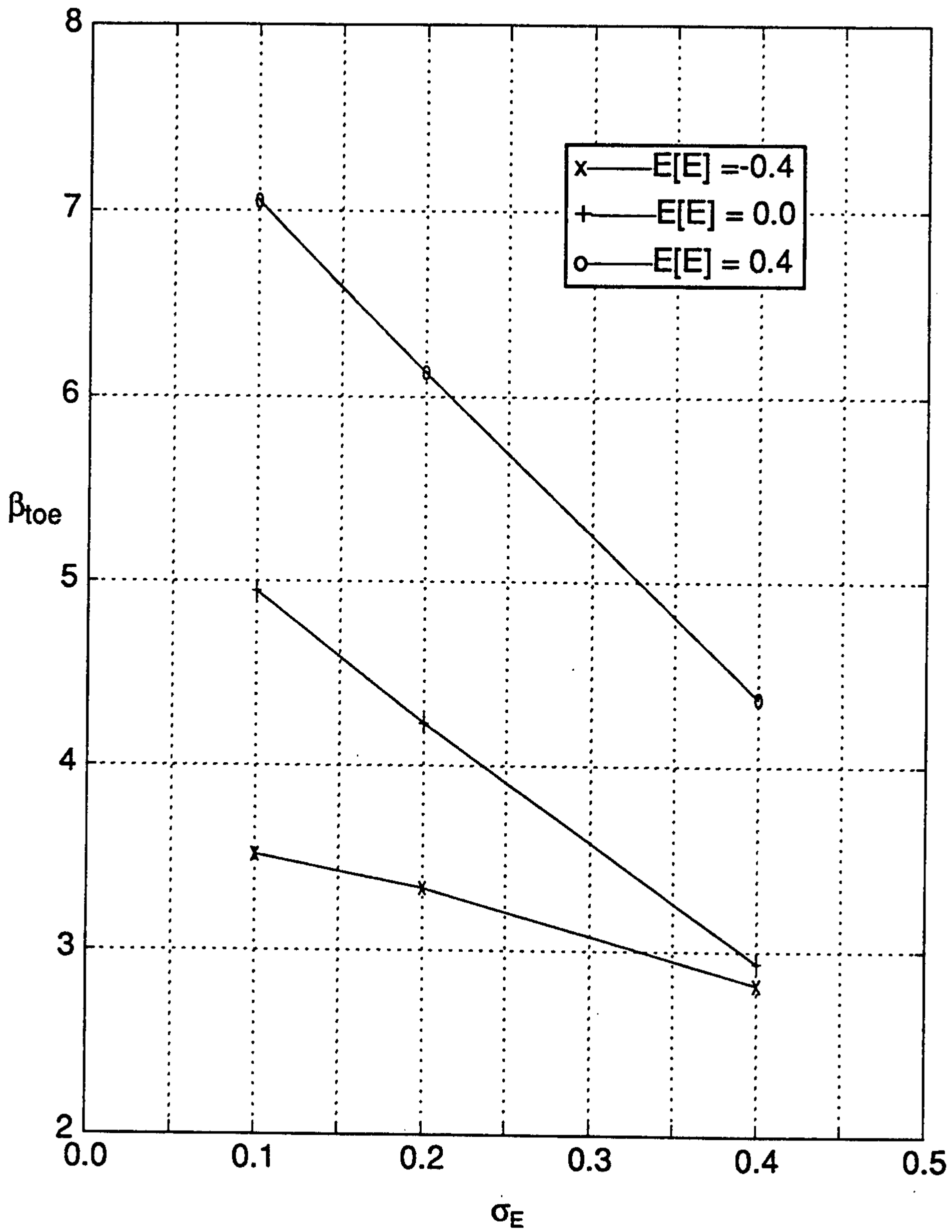

Figure 7.5 Demopolis Locks and Dam, Monolith L-17, Resultant Location Analysis, β_{toe} versus σ_E

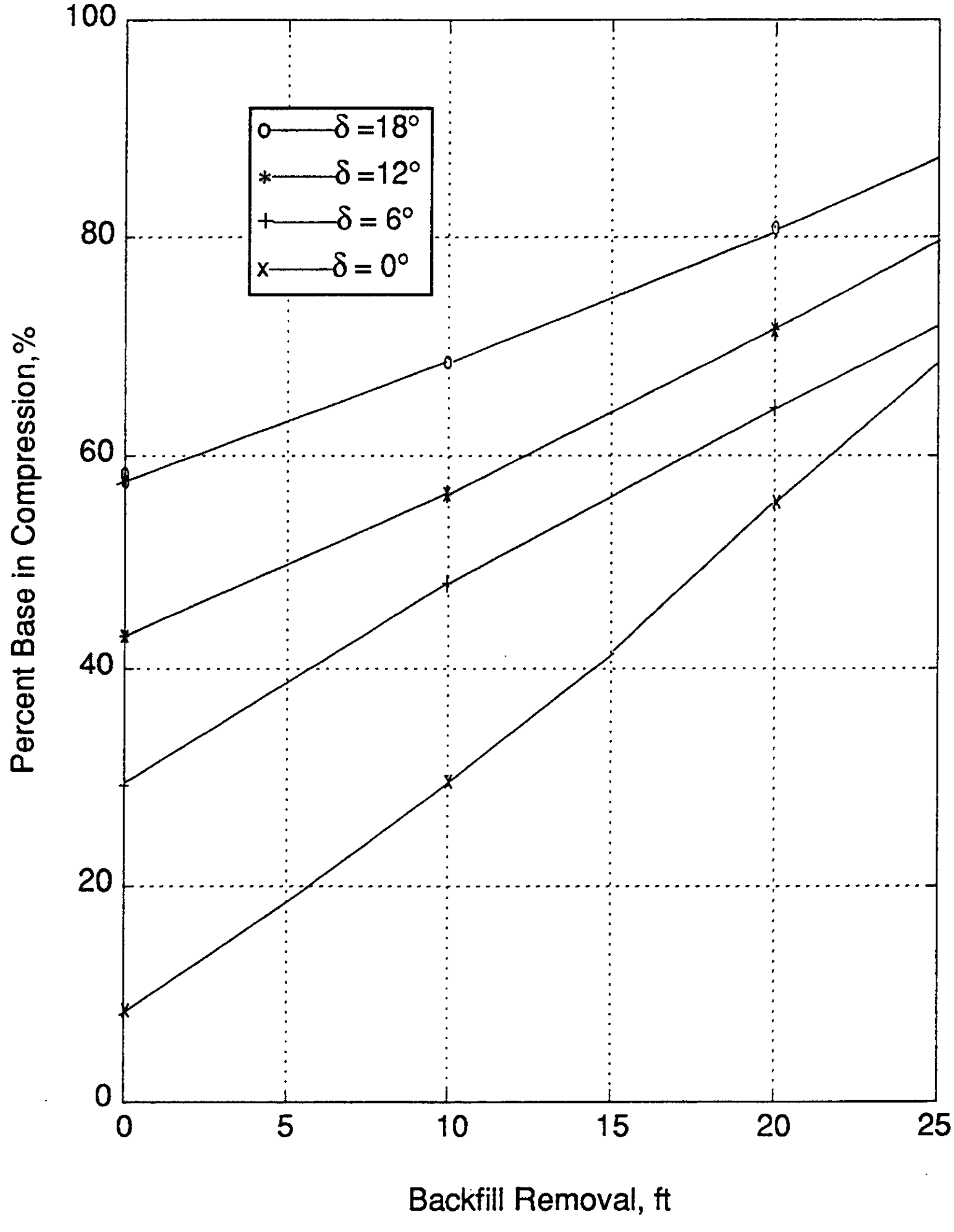

Figure 7.6 Demopolis Locks and Dam, Monolith L-17, Resultant Location Analysis, Percent Base in Compression versus Backfill Removal

Table 7.2
Demopolis Locks and Dam, Monolith L-17
Resultant Location Analysis Results, $\delta = 12°$

Backfill Level	Case	$E[X_R]$ (ft)	σ_{X_R}	β_{toe}	E[FS]	σ_{FS}	β_{FS}
No backfill removed	Normal	8.621	3.246	2.66	1.25	0.129	2.08
	Maint.	7.651	3.092	2.47	1.22	0.121	1.93
	High water	13.567	2.177	6.23	1.33	0.068	5.54
20 ft backfill removed	Normal	13.905	2.299	6.05	1.49	0.161	3.63
	Maint.	12.753	2.332	5.47	1.47	0.172	3.21
	High water	19.566	2.223	8.80	1.420	0.082	6.07

Table 7.3
Demopolis Locks and Dam, Monolith L-17
Resultant Location Analysis Results
Changing Wall Friction Angle and Backfill Level
Maintenance Condition

Backfill Level	Wall Friction (°)	$E[X_R]$ (ft)	σ_{X_R}	β_{toe}	E[FS]	σ_{FS}	β_{FS}
No backfill removed	0.0	1.636	4.058	0.40	1.04	0.097	0.34
	6.0	4.798	3.499	1.37	1.12	0.108	1.15
	12.0	7.651	3.092	2.47	1.22	0.121	1.93
	18.0	10.294	2.781	3.70	1.32	0.136	2.68
10 ft backfill removed	0.0	5.189	3.178	1.63	1.13	0.101	1.32
	6.0	8.661	2.943	2.55	1.21	0.116	1.91
	12.0	9.657	2.769	3.49	1.29	0.133	2.45
	18.0	11.698	2.634	4.44	1.39	0.153	2.94
20 ft backfill removed	0.0	9.818	2.312	4.25	1.31	0.130	2.68
	6.0	11.321	2.317	4.89	1.38	0.150	2.96
	12.0	12.753	2.332	5.47	1.47	0.172	3.21
	18.0	14.149	2.351	6.02	1.56	0.197	3.44

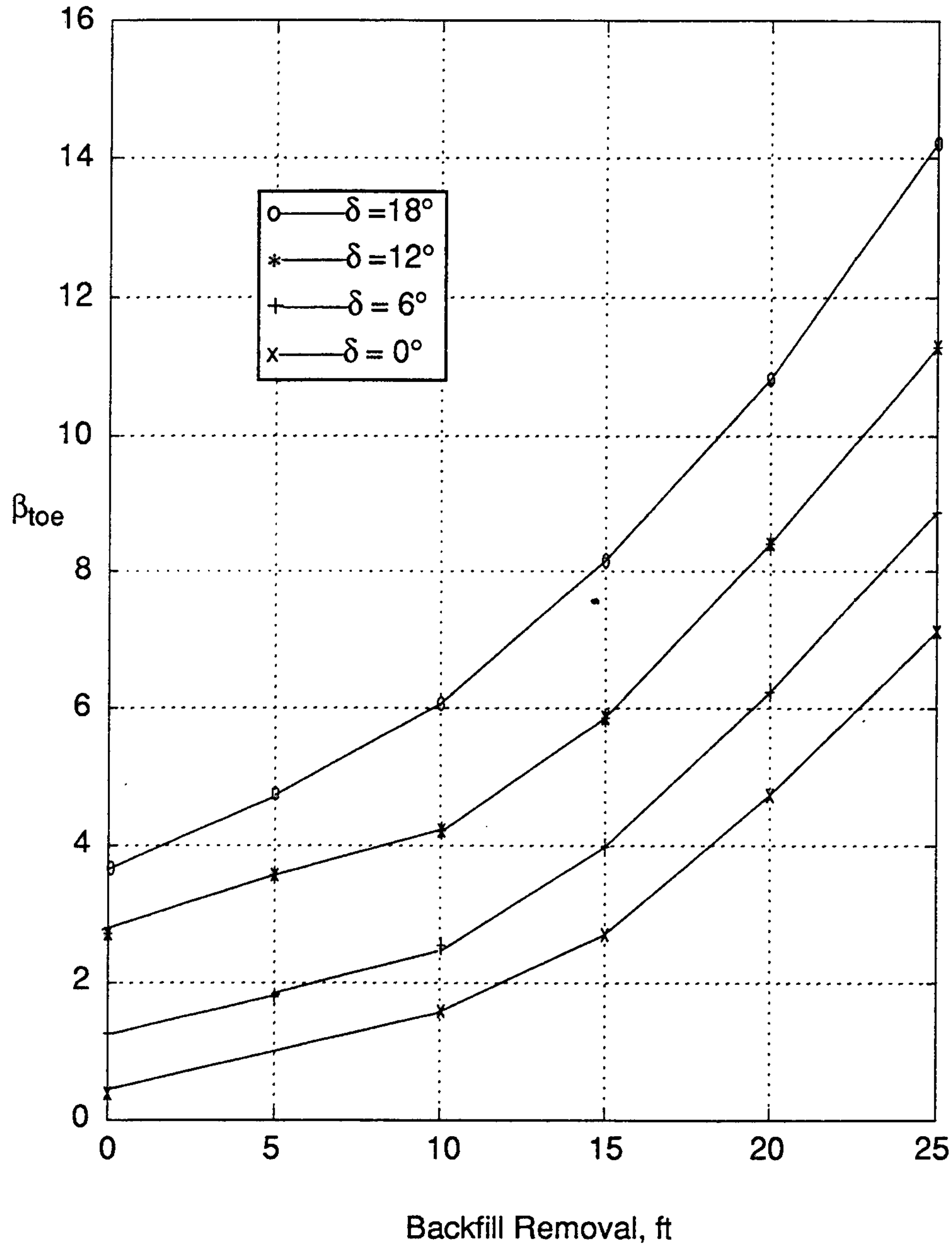

Figure 7.7 Demopolis Locks and Dam, Monolith L-17, Resultant Location Analysis, β_{toe} versus Backfill Removal

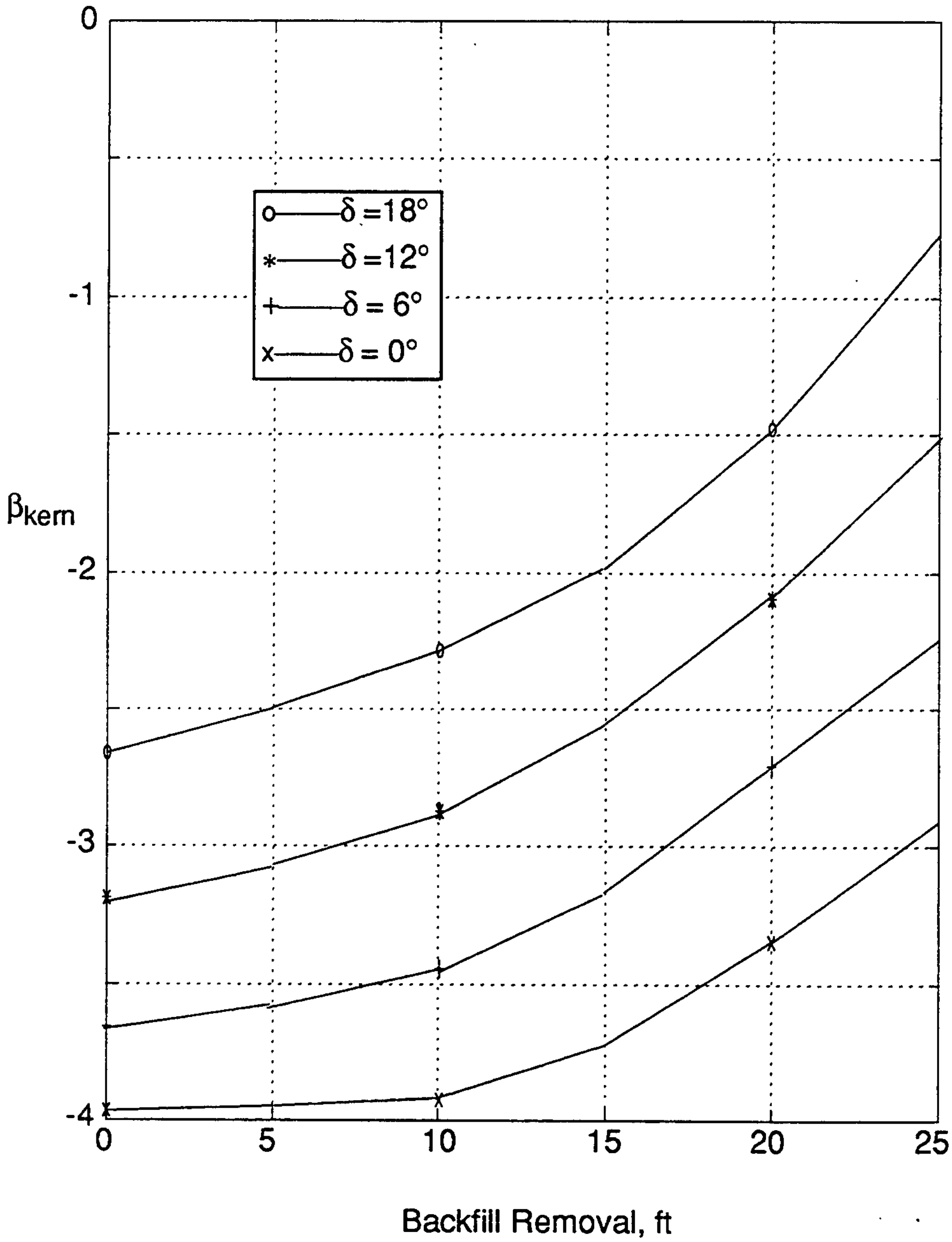

Figure 7.8 Demopolis Locks and Dam, Monolith L-17, Resultant Location Analysis, β_{kern} versus Backfill Removal

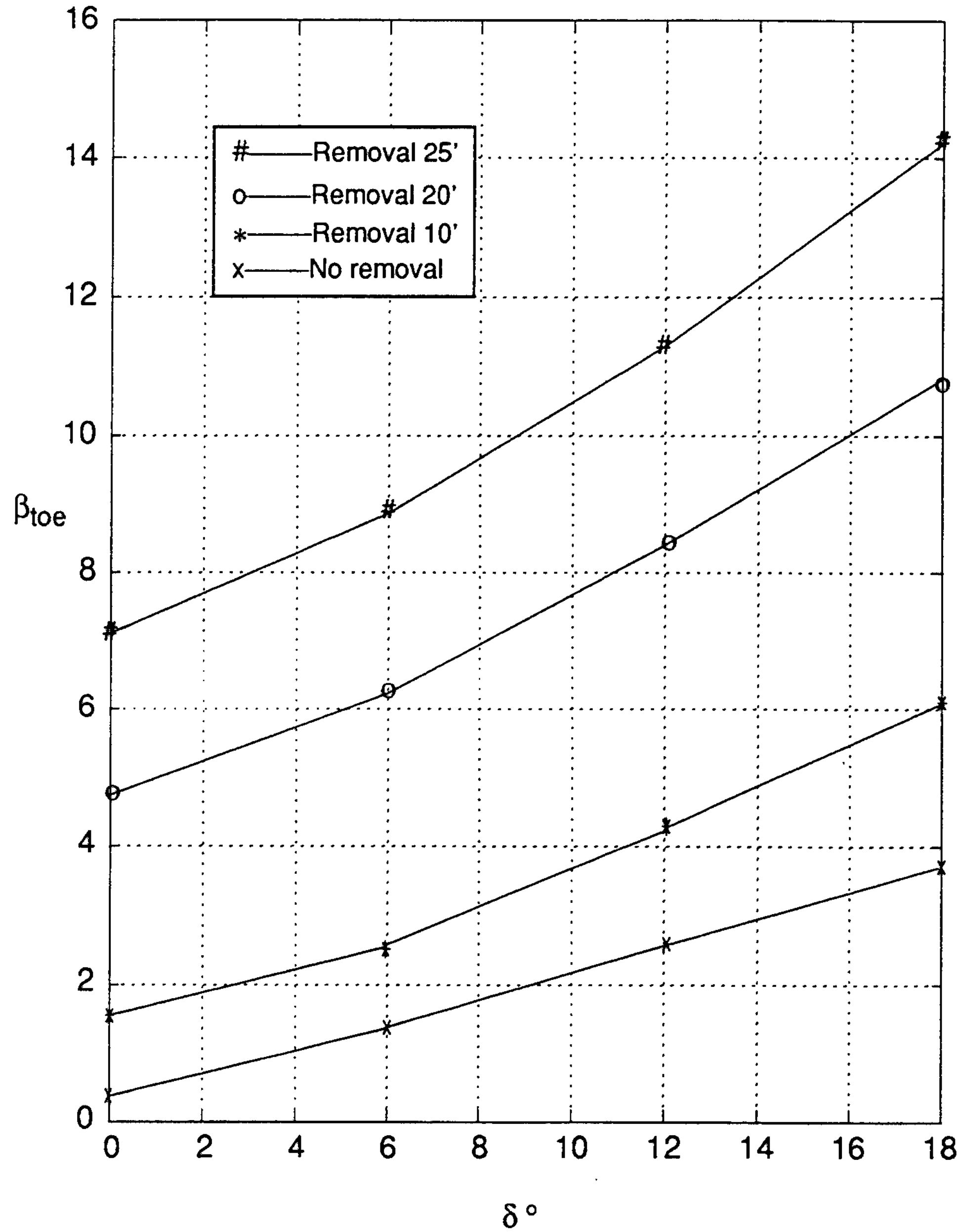

Figure 7.9 Demopolis Locks and Dam, Monolith L-17, Resultant Location Analysis, β_{toe} versus Wall Friction Angle δ

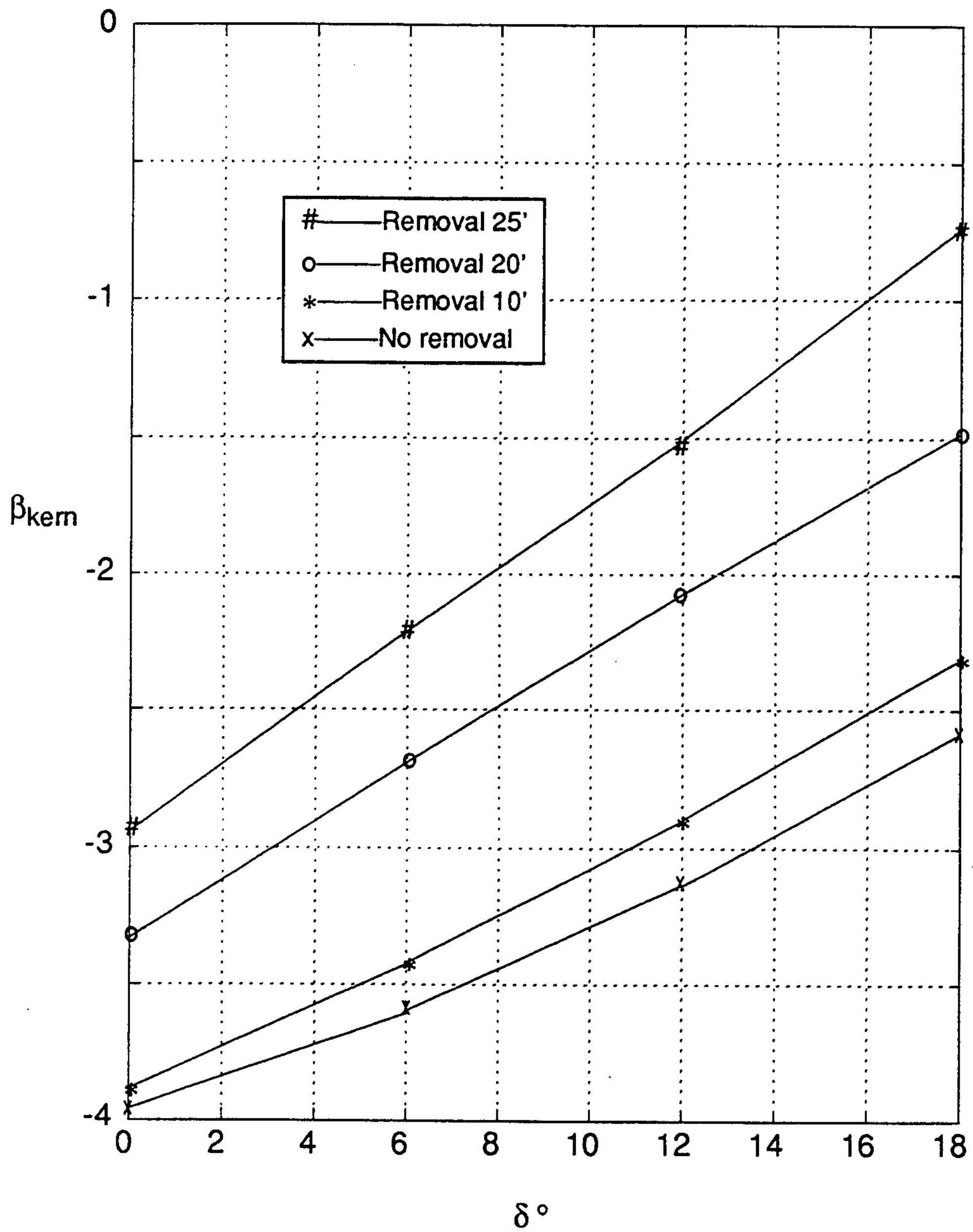

Figure 7.10 Demopolis Locks and Dam, Monolith L-17, Resultant Location Analysis, β_{kern} versus Wall Friction Angle δ

backfill level has more influence, although both variables are significant. Without any backfill removal and with δ taken as zero, the monolith would have an unacceptably low reliability index. With full backfill removal *or* a large wall friction angle, the reliability index would be considered acceptable. Given the assumptions of the parametric studies, it appears that if a moderate wall friction angle of, say, 9 degrees were assumed, about 15 ft of backfill removal may have been satisfactory.

BEST ESTIMATE ANALYSIS

Figure 7.1 previously shown illustrates the free body diagram for the expected value "best estimate - before backfill removal" analyses. Figure 7.11 shows the cross section and free-body diagram for the "best estimate - after backfill removal analysis. Random variables have been previously described in Table 7.1. An example calculation for the expected value case before backfill removal is provided in Table 7.4. Results of the before and after "best estimate" analyses are summarized in Table 7.5. It is seen that, for the "best estimate" assumption, which includes wall friction and reduced uncertainty in the backfill water level, β_{FS} and $\beta_{C/D}$ were on the order of 4.0, suggesting marginally adequate reliability if no action had been taken. However, removal of the backfill increased these values to the range 6 to 10, providing a high degree of reliability.

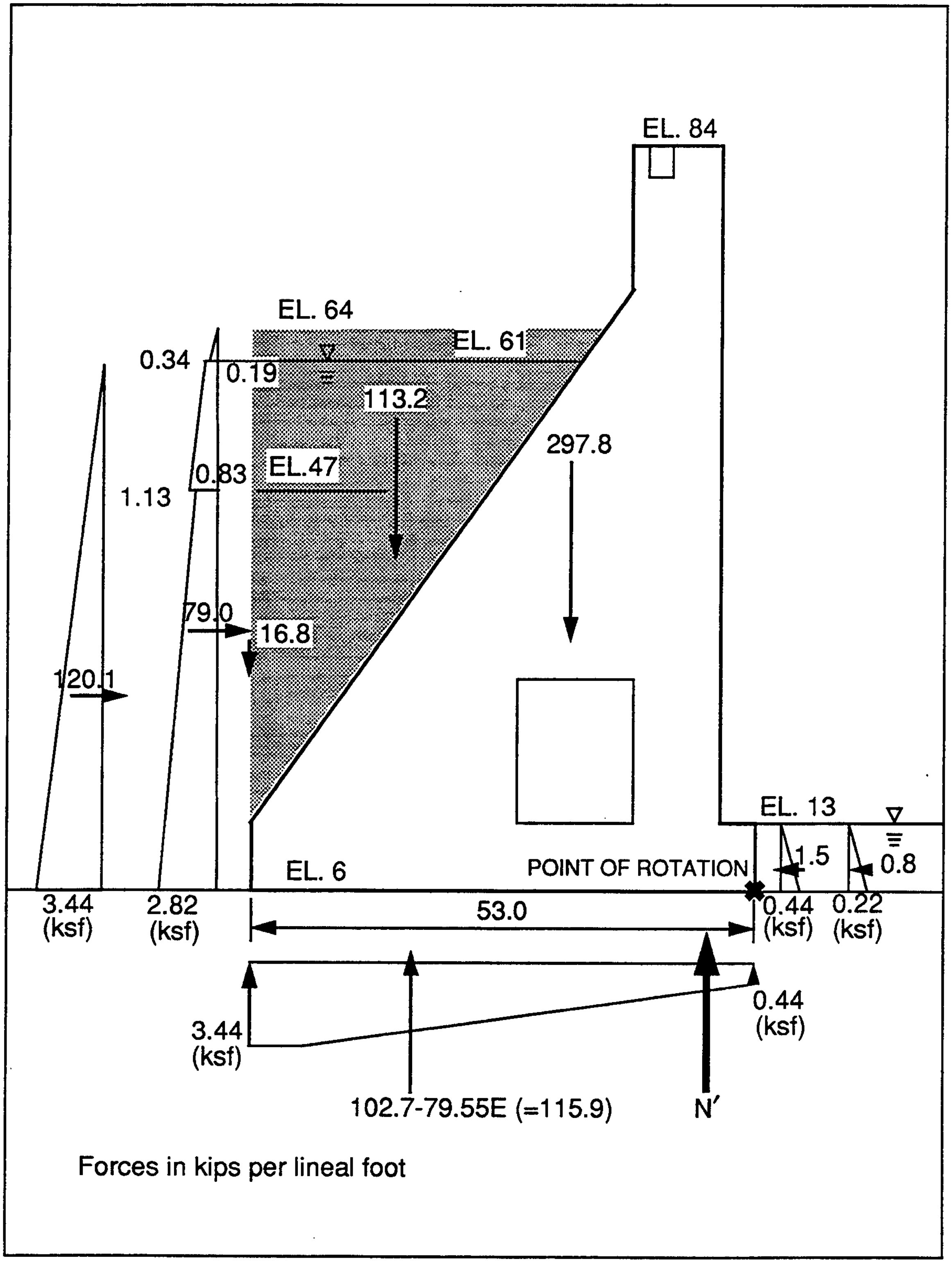

Figure 7.11 Demopolis Locks and Dam, Monolith L-17, Resultant Location Analysis, Free Body Diagram, 20 feet of Backfill Removed

Table 7.4
Demopolis Locks and Dam, Monolith L-17
Resultant Location Analysis Using Expected Values
Maintenance Condition, Original Backfill

	V (kips/ft)	H (kips/ft)	Arm (ft)	M_R (kips-ft/ft)	M_O (kips-ft/ft)
Concrete	297.825		21.107	6286.19	
Soil	209.625		37.733	7909.78	
Water (River)		-1.531	7/3		-3.57
Water (Land)		120.125	62/3		2482.58
Backfill (1)		$(84-68)^2/2$ (.125) (.5) = 8.0	16/3+62		538.67
Backfill (2)		[(84-68)(68-47)(.125) $+(68-47)^2/2$(.0625)] (.9) = 50.203	21/2+41 21/3+41		1946.70 595.35
Backfill (3)		{[(16)(.125)+(21)(.0625)] (41) $+(41)^2/2$(.0625)} (.66) =124.307	41/2 41/3		1837.54 473.83
Wall Friction	(182.51)tan(12°) =38.794		53	2056.08	
Overburden		-(49/2)(.0625)[1-sin(30°)] = -0.766	7/3	1.79	
Uplift force (1)	-(.4375)(53) =-23.188		53/2		614.48
Uplift force (2)	-(3.4375((.5117) (53) = -93.225		(.5117/2 +.4883)(53)		3676.80
Uplift force (3)	-(3.4375)(.4883) (53)/2 = -44.481		(.4883)(53) (2/3)		767.44
	$\Sigma V = 385.35$	$\Sigma H = 300.338$		$\Sigma M_R =$ 16253.84	$\Sigma M_O =$ 12929.82

$SM=\Sigma M_R-\Sigma M_O= 16253.84 - 12929.82 = 3324.02$ $X_R=3324.02/385.35= 8.626$ ft

PC = (3)(8.626)/(53) = 48.8% FS = (16253.84)/(12929.82)=1.26

C/D = 53/(53-2Xr)= 1.48

Table 7.5

Demopolis Locks and Dam, Monolith L-17

Maintenance Condition

Results of "Best Estimate" Resultant Location Analysis

Case	PC (%)	$E[X_R]$	σ_{XR}	β_{toe}	E[FS]	σ_{FS}	β_{FS}	E[C/D]	$\sigma_{C/D}$	$\beta_{C/D}$
Original Backfill Normal Pool	53.72	9.49	1.72	5.53	1.28	0.071	4.46	1.56	0.158	4.34
Original Backfill Maint.	48.81	8.62	1.76	4.91	1.26	0.074	3.88	1.48	0.147	3.94
Removed Backfill Normal Pool	88.52	15.64	0.95	16.4	1.63	0.094	8.42	2.44	0.219	9.93
Removed Backfill Maint	83.37	14.73	1.11	13.3	1.63	0.128	6.26	2.25	0.216	8.44

Part VIII: DEMOPOLIS LOCKS AND DAM, MONOLITH L-17, SLIDING ANALYSES

Problem Statement.

Demopolis Lock and Dam monolith L-17 was previous described in Part VII. This part summarizes the results of "best estimate" sliding analyses for the before and after backfill removal cases. The cross-sections used for the analyses were previously shown in Figures 7.1 and 7.11; however, the lateral earth pressures are different as active and passive conditions are used with the simple sliding model rather than at-rest pressures. Both the normal operating and maintenance conditions were analyzed, and both peak and residual strengths were considered for the foundation rock. For all sliding analyses, cohesion was used only along that portion of the foundation taken calculated to be in compression from the resultant location analyses.

Random Variables

Random variables used for the analyses are shown in Table 8.1, and are consistent with the "best estimate" assumptions from Part VII. Soil strengths are consistent with those in used in the previous report and are used to calculate active earth pressures. Foundation rock strengths are consistent with the previous report.

Table 8.1

Demopolis Locks and Dam, Monolith L-17

Random Variables for Sliding Analysis

Variable	Expected Value	σ	V (%)
(1) γ_{soil}	0.125 kcf	0.00625 kcf	5.0
(2) ϕ'_{soil}	30.0 deg	3.0 deg	10.0
(3) δ	12.0 deg	3.0 deg	25.0
(4A) c_{rock} (peak)	30.0 ksf	21.0 ksf	70.0
(5A) ϕ_{rock} (peak)	30.0 deg	11.46 deg	38.2
$\rho_{c,\phi} = -.70$			
(4B) c_{rock} (residual)	0.0	0.0	--
(5B)ϕ_{rock} (residual)	25.2	8.02	31.84
(6) $\gamma_{concrete}$	0.15 kcf	0.0075 kcf	5.0
(7) Water in Backfill	68.0 (before) 61.0 (after)	1.7 1.5	-- --
(8) Uplift parameter, E	-1 + E[PC]	0.2	--

RESULTS

Sliding analyses were performed using the "simple" method based on active and passive earth pressures described in the previous report. Results of the analyses are shown in Table 8.2. For the peak strength assumption, backfill removal resulted in large increases in the sliding factor of safety, but only moderate increases in the reliability index. As the backfill is removed, the component of uncertainty due to driving soil force is reduced, but the base shear strength then accounts for a greater proportion of the total uncertainty. As the shear strength is only calculated along the portion of the base in compression, it appears that removing the backfill and providing more base area in compression causes some compensating increase in uncertainty which somewhat offsets the improvement in reliability due to removing the fill. For the residual strength assumption, little improvement is noted in either the deterministic factor of safety or the reliability index; indeed values are sufficiently low that it can be concluded that the residual strength assumption does not match real conditions.

Table 8.2

Demopolis Locks and Dam, Monolith L-17

Results of "Best Estimate" Sliding Analysis

Strength	Water	Backfill	E[FS]	σ_{FS}	β_{FS}
Peak	Normal	Before	5.77	3.01	3.33
		After	14.19	8.46	4.53
	Maintenance	Before	4.83	2.46	3.04
		After	11.44	6.71	4.21
Residual	Normal	Before	0.94	0.35	-0.34
		After	1.28	0.47	0.51
	Maintenance	Before	0.86	0.32	-0.61
		After	1.11	0.42	0.12

PART IX
SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

SUMMARY

Additional probabilistic stability analyses have performed for several prototype navigation structures previously studied (Wolff and Wang, 1992). These analyses included parametric studies to assess the relative effects of various random variables and "best estimate" analyses using refined values.

CONCLUSIONS

Effect of Uplift parameter, E. For both cases studied, the effect of uncertainty in the uplift distribution parameter, E, was found to significantly affect the reliability index, even where differential heads are relatively low. As the parameter E is typically significant, when the base resultant force lies outside the kern, the value of E in resultant location analysis should be matched to the percent base in compression through an iterative procedure. Where the resultant base force lies within the kern, the value of E must be judgmentally assigned. Examination of results and their sensitivity to E indicate that using an expected value of 0.0 and a standard deviation of 0.2 seem appropriate in the absence of data that indicate otherwise.

Anchors Analyses for Locks and Dam No. 3, monolith M-20 suggest that the tailwater level during dewatering is of much more significance than the certainty or uncertainty in the anchor force provided. As water levels increase, the expected value of the overturning moment increases with little or no change in uncertainty, moving the capacity and demand distributions closer together and reducing β. As anchor forces change from certainly functional to fifty-fifty to certainly non-functional, the capacity distribution first spreads and then tightens, and its expected value decreases, decreasing β, but with less significance than changes in water level.

Shear Strength The strength characterization of foundation rocks is found to greatly affect the reliability index for sliding. Analyses using residual strengths typically indicate such low values of β that it is likely that the "operative" strength is greater than the residual condition or the structures would have already experienced sliding. Although peak strengths from residual shear tests interpreted by the Mohr-Coulomb failure criteria typically suggest high cohesion values which designers do not wish to rely on, and the founding conditions of the Monongahela monoliths are known to be poor, a review of the literature suggests that shear dilatancy of strong but broken or discontinuous foundation rock can account for the good performance of the monoliths with respect to translational stability. It is conjectured that the non-linear strength envelope illustrated in Figures 4.2 and 4.3 may be representative of the foundation strength at Locks and Dam No. 3, even given its poor construction methods.

Base Elevation Analyses indicated that uncertainty in the base elevation of monoliths, although real in practice, contributes little to uncertainty and can typically be neglected in analysis.

Wall Friction Analyses indicated that consideration of shear on vertical planes in the backfill, often referred to as wall friction whether along a wall face or not, significantly increases the reliability index. Taking wall friction as zero appears unduly conservative for analysis of existing structures that have performed well. Taking a modest expected value on the order of 12 degrees and a coefficient of variation of 30 percent probably retains some conservatism but should provide more realistic β values.

Backfill level at Demopolis As summarized further in Parts VII and VIII, removing the 20 feet of backfill at Demopolis substantially increased reliability. Reviewing the "best estimate" analyses, the reliability before backfill removal was marginally adequate (near 4.0) and a lesser amount of removal may have proven satisfactory.

RECOMMENDATIONS

An unusual aspect of the research project described in the previous and present reports is the time frame within which the results have been put into practice. Even at this writing, reliability analyses are being performed in various Corps' field offices on a number of structures. Although the methodology has been demonstrated as being capable of application to navigation structures, and appears to provide consistent and logical results, it will, like any new methodology, evolve and become better understood as more analyses are performed by more engineers for more structures. It is therefore recommended that Corps of Engineers recent and developing guidance for reliability analyses be reviewed and refined after a year or more of field experience has been obtained.

As pointed out in the recommendations of the previous report, additional research and development effort appears warranted in a number of areas; important among these are the appropriate characterization of shear strength and its spatial variability, more review of data for anchor performance and pore pressure distribution, adaptation of Corps' computer programs to probabilistic analyses, and training of engineers in probabilistic methods.

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13.ABSTRACT *(Maximum 200 words)*

This report summarizes the results of follow-on studies to an earlier project to develop probability-based methods for assessing the structural reliability of gravity monoliths at navigation locks and dams. In this report supplement, further investigations are made of the effects of several random variables on the reliability index, namely uplift pressure, water levels, wall friction angle, anchor performance, and structural geometry. Analyses are performed for four actual structures, three on the Monongahela River in Pennsylvania and one on the Tombigbee River in Alabama.

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