A PRACTICAL APPROACH FOR PROBABILISTIC EVALUATION OF EARTHQUAKE SITE STABILITY APPLIED TO A CELLULAR WHARF BULKHEAD SYSTEM

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ABSTRACT

A generally applicable methodology for probabilistic evaluation of site liquefaction and submarine slope earthquake stability hazards, originally developed for application to a cellular wharf system, is formulated using available procedures based on (updatable) observations of site liquefaction and slope performance in previous earthquakes; the methodology is illustratively applied to the cellular wharf system. Pore pressure buildup, strength degradation, and associated slope stability effects are modeled. The probabilistic formulation accounts for uncertainty in site SPT characteristics and earthquake acceleration recurrence intervals. Numerical results (obtained without need of a computer) include probability estimates of stable site performance based on selected (1) minimum conventional factors of safety against liquefaction and (2) maximum calculated slope displacements, and using geotechnical data typically available on moderate, or larger, scale engineering projects. The methodology can be used to help clarify hazard risk levels and establish a sound basis for recommendation/ selection of project earthquake design details.

INTRODUCTION

The purpose of this paper is to present a general, practical approach for a probabilistic evaluation of earthquake site stability for shoreline sites located on cohesionless soils with nearby submarine slopes. The approach is documented and illustrated by its application to the State of Alaska's proposed Ferry Vessel Maintenance Facility at Ketchikan in southeastern Alaska (10); simplified geometric details of the project, including the cellular wharf bulkhead system, are shown in Fig 1. However, it is intended that the generality of the approach (for potential application to other projects) not be limited by the emphasis on Ketchikan project-specific data and assumptions, used here to demonstrate the approach.

Earthquake site stability as used here includes earthquake slope stability and earthquake-induced liquefaction in level ground. For important facilities adequate evaluation of the risk of stability loss by these hazards and determination of their acceptability in terms of economics and human safety is a necessary task; one which can be relatively uncertain and fuzzy.

The typical approach to stability is deterministic--using a minimum acceptable factor of safety. However, earthquake slope stability and liquefaction is often better formulated in a probabilistic manner. This provides a basis in the design process for quantifying risk to life and property, allowing evaluation of economic and safety tradeoffs between different potential design configurations and details (or retrofits) in support of optimum facility operation.

PROBABILISTIC FORMULATION

For any given site and facility the realization of stable site performance (S) over a given design life (of n years) requires that C, the actual "capacity" of the site to maintain site stability under earthquake loadings, be equal to or greater than D, the actual earthquake loadings or "demands" at the site occurring during the n year design life; i.e., S requires C>D. Because C, D and thus S are never known with certainty, they are random variables.

C, D and S can be modelled as random variables and evaluated probabilistically as follows. C can be equated to the average modified standard penetration test (SPT) blow counts N_1 (a random variable) for the site, $C(N_1)$, following Seed's observational correlations between site average N_1, and site liquefaction performance in previous earthquakes (5,6,7). A probability density function (PDF) of C in terms of N_1, $f[C(N_1)]$, can be estimated based on site SPT data. A site capacity relationship, $C(\underline{a})$, can be developed between site maximum effective earthquake ground acceleration, $\underline{a}$ (a random variable), and the minimum N_1 for the site estimated (e.g., using the procedure presented here) for (1) an acceptable no liquefaction condition and (2) an acceptable maximum slope displacement criterion. From this a functional relationship of the form $C(\underline{a})=g(N_1)$ can be developed. Based on site seismicity data the average yearly probability that any given D is exceeded can be estimated in terms of $\underline{a}$, $F[D(\underline{a})]$, and then in terms of N_1 by substituting $g(N_1)$ for $\underline{a}$ in $F[D(\underline{a})]$ to give $F[D(N_1)]$, the demand exceedance probability in terms of N_1. Then the yearly probability (reliability) of stable site performance, Ps, defined as $P(C>D)$ can be numerically estimated (e.g., using a programable calculator) by

$$Ps = 1-\int (f[C(N_1)]\cdot F[D(N_1)])\,dN_1 \qquad (1)$$

Finally, taking $F[D(\underline{a})]$ as time-independent (a common assumption), $\underline{Ps}$, the estimated probability of stable site performance for a design life of n years becomes $\underline{Ps}=Ps^n$.

Available N_1 data from the Ketchikan site (10) (and others) suggests $f[C(N_1)]$ can be adequately described by a lognormal PDF where m is the median N_1 and σ is the standard deviation of $\ln N_1$:

$$f[C(N_1)] = \frac{1}{N_1\sqrt{2\pi}\sigma}\exp\left[-\left(\frac{1}{\sqrt{2}\sigma}\ln\frac{N_1}{m}1\right)^2\right] \qquad (2)$$

At Ketchikan, site N_1 data was characterized (10) using the lognormal distribution (based on results of the subsurface investigation and subsequent geostochastic analyses) in three ways, as shown on Fig 2: (1) for earthquake slope stability: average values in the critical potential failure zone Z, $N_1(Z)$ ($m=21$, $\sigma=1.2$); (2) for site liquefaction: site average, $N_1(S)$, based on all site data (m=24, σ=1.3); (3) for localized instability due to liquefaction: testhole averages, $N_1(TH)$ (m=25, σ=6.2).

$F[D(\underline{a})]$ can be adapted from a site seismicity study or, if not available, using the empirical relationship found and reported by Algermissen and Perkins (1)

$$F[D(\underline{a})] = (\underline{a}(R)/\underline{a})^{2.33}/T(R) \quad (3)$$

where $\underline{a}(R)$ and $T(R)$ represent a "reference" acceleration and average return period, obtained from available earthquake acceleration risk maps or other data. $F[D(N_1)]$ is obtained by substituting $g(N_1)$, from $C(\underline{a})=g(N_1)$, for $\underline{a}$ in Eq 3.

For Ketchikan, based on available information (10), an estimate for the reference event was judged to be $\underline{a}(R)=0.31g$ at $T(R)=10000$ years with a reasonable lower-bound $T(R)=1000$ years; two representative local magnitudes were used for design earthquake conditions: M=6.5 and M=7.5. At Ketchikan, as often the case elsewhere as well, actual (future) earthquake occurrence is the most uncertain single, major factor affecting site stability.

EARTHQUAKE SITE STABILITY EVALUATION

During earthquake loading cyclical shear strains are produced in the ground. In cohesionless soils these strains cause a densification of the soil which leads to a progressive increase in soil pore water pressures causing a corresponding decrease in the effective strength of the soil. Cohesionless soil can progressively loose strength, both during and immediately after an earthquake, and weaken until: (a) slopes can no longer maintain their shape against gravity or provide lateral support to adjacent land, and/or (b) level ground can no longer provide necessary strength to maintain adequate support of any structures founded on or in it. Both conditions "a" and "b" above are commonly referred to as liquefaction, although condition "a" can be more specifically described as earthquake slope instability.

The methodology developed for evaluation of earthquake slope stability at the Ketchikan site models increases in soil pore water pressure and their affect on soil shear strength and resulting slope behavior due to earthquake loadings (5,8,10). The methodology was executed as summarized in the following. First, r_u, the average pore pressure ratio within Z, the potential failure zone, due to design earthquake loading was estimated. Estimates were based on the relationships developed in Appendix I, using procedures and data presented by Seed and Idriss (6) for flat ground liquefaction in combination with procedures and data presented by Seed for earthquake-induced pore pressure development in dam embankments (5) supplemented by the data of Vaid and Finn (9). For the Ketchikan site, r_u was estimated as a function of $\underline{a}$ for the two design earthquake local magnitudes, M=7.5 and M=6.5. The reduction in soil strength in Z as a function of r_u was evaluated as an equivalent average angle of internal friction, ϕe, via:

$$\phi e = \tan^{-1}\left[(1-r_u)\ \tan(\phi')\right] \qquad (4)$$

ϕ', the average effective angle of internal friction in Z, was estimated as $(27+0.3N_1)°$ for $10 \leq N_1 \leq 40$ (4). An equivalent infinite slope having a slope angle Be giving the same static (and pseudostatic) factors of safety as the potential failure surfaces (for all ϕ') was evaluated as:

$$Be = \tan^{-1}\left[\frac{\tan(\phi')}{SFS(\phi')}\right] \qquad (5)$$

where $SFS(\phi')$ is the static (or pseudostatic) limit equilibrium factor of safety using a given value of ϕ' calculated (e.g., using a modified Bishop's method of slope stability) for any particular design geometry and loading. Next, Be and ϕe were input to a Newmark slope stability analysis (2,3) to calculate a dynamic resistance of the slope at the end of earthquake loading, N; where:

$$N = \tan(\phi e)\cos(Be)-\sin(Be) \qquad (6)$$

N, normalized by earthquake acceleration $\underline{a}$, $N/\underline{a}$, was calculated for various N_1 and $\underline{a}$ and plotted on the upper-bound envelope of permanent slope displacements determined by Franklin and Chang (2) to give an indication of potential slope displacement. These results were evaluated and are summarized in Fig 3 for "zero" slope displacement and "large" (>100 inches) displacement to provide plots of site N_1, $C(N_1)$, required to maintain site slope stability under potential earthquake loadings, measured by $\underline{a}$ for the two design local magnitudes, M=7.5 and M=6.5. From this plot a linear $C(\underline{a})$ envelope, shown in Fig 3, was chosen to provide a conservative interpretation of Ps for slope stability based on the criteria: (a) high factor of safety against liquefaction (FS>1.0) for lower values of N_1 (reflecting concern over potentially significant shear strain potential (7) and tendency for lateral movement should liquefaction begin to develop before slope displacement) decreasing with increasing N_1, and becoming tangent to FS=1.0 for liquefaction at higher values of N_1 (reflecting limited shear strain potential, and therefore limited tendency to flow laterally should liquefaction occur), with (b) as a margin of safety against potential flow sliding, keeping calculated maximum permanent slope displacements to zero at all values of N_1.

The yearly probability of stable site performance related to earthquake slope stability, Ps(S), was then evaluated numerically using Eq 1 with (the CDF of) $f[C(N_1)]$ for slope stability, $N_1(Z)$, shown graphically in Fig 2, and $F[D(N_1)]$ evaluated using Eq 3 and substituting $g(N_1)$ for $\underline{a}$ using $C(\underline{a})=g(N_1)$ from Fig 3:

$$C(\underline{a}) = 0.014(N_1-6.5) \qquad (7)$$

Ps(S) was taken as independent for any given succession of n years--giving the probability of stable site performance related to earthquake slope stability during a design life of n years, $\underline{P}s(S)$, equal to $Ps(S)^n$. Calculation results are displayed in Fig 4.

Potential liquefaction at the Ketchikan site was modeled using Seed's procedures (6,7). Results of the analysis, summarized in Fig 3, provided plots (similar to what was done for slope stability) of site N_1, $C(N_1)$, required to maintain site stability (FS=1.0) against potential liquefaction due to potential earthquake loadings, measured by $\underline{a}$, for M=7.5 and M=6.5. Scrutiny of these results suggested that the same linear $C(\underline{a})$ envelope chosen for earthquake slope stability would also be appropriate for liquefaction--and for the same reasons: the desirability of having a high factor of safety against liquefaction at lower values of N_1 because of the increased tendency for less dense soils (low N_1) to flow (significant shear strain potential) when liquefied, and allowing a FS=1.0 against liquefaction at higher values of N_1 because of the limited tendency of denser soils (high N_1) to flow (limited shear strain potential) if liquefied. This strategy considered possible (but unidentified) brittle failure effects and earthquake-induced soil densification/settlement effects by requiring higher factors of safety against liquefaction for lower density (low N_1) soils; this gave a lower probability of stable site performance related to liquefaction, Ps(L), than would be calculated using a $C(\underline{a})$ based on FS=1.0 against liquefaction for all potential N_1 values. Further conservatism against uncertainty in Ps(L) and Ps(S) was (prudently) maintained by assuming the conditional probability of liquefaction given FS=1.0 against liquefaction equal to 1.0 in all cases.

Ps(L) was evaluated numerically using Eq 1 with (the CDF of) $f[C(N_1)]$ for liquefaction, $N_1(S)$, shown graphically in Fig 2, and $F[D(N_1)]$ the same as for earthquake slope stability. Ps(L), as Ps(S), was taken as independent for any given succession of n years--giving the probability of stable site performance related to liquefaction during a design life of n years, $\underline{P}s(L)$, equal to $Ps(L)^n$. Calculation results are displayed in Fig 4.

In addition, because of the generally random density variation across the site--reflected in the distribution of $N_1(TH)$, test hole averages--the probability, $P\ell$, was explored of localized liquefaction occurring in limited (but unknown) zones of the site. This condition might be characterized by partial weakening of the site with the occurrence of initial liquefaction in limited pockets of loose soil (surrounded by denser soil). This potential phenomenon is not considered capable of compromising the functional integrity of structural components that are designed and constructed to act as a highly ductile, integral unit under earthquake loading.

$P\ell$ was calculated using Eq 1 with $f[C(N_1)]$ based on the distribution of $N_1(TH)$ shown graphically in Fig 2. A variety of possible situations were assumed and $P\ell$ calculated for sensitivity. Evaluation of results suggested a reasonable upper bound $P\ell$ of about 0.01 (per year), yielding a yearly probability of stability as 0.99 (1-.01) or 99%, and for an n year design life a probability of stability equal to 0.99^n, as shown on Fig 4.

SITE STABILITY CONCLUSIONS AND LIMITATIONS

Evaluation results are presented in Fig 4 in terms of Ps, the estimated probability of stable site performance, for liquefaction, Ps(L) and $P\ell$, and slope stability, Ps(S). Proper use of the evaluation in the design process rests on a clear understanding of the implications and limitations of Ps as a measure of earthquake site stability. First, all Ps values assume and require adequate static slope stability. This would include maintaining control of both tidal lag groundwater levels within and behind the bulkhead system and construction (dredging and pile driving) operations to prevent undermining, disturbing or compromising the integrity of submarine slopes. Increasing the static factor of safety will increase Ps under any given earthquake loading; therefore, within practical limits, a facility can be made more stable against earthquake loading, increasing Ps, by: deepening or widening bulkhead cells, moving the facility away from slopes, decreasing groundwater levels by drainage, densifying the site, buttressing or flattening submarine slopes. Second, the stability analyses assume the cellular bulkhead system is internally stable. Internal stability can be improved by densifying the soil and backfill both within and between sheet pile cells (10). Third, the "P" in Ps measures the <u>degree of belief</u> that the site is in fact "stable;" i.e., that it is capable of providing a facility--designed and constructed to act as a ductile, integral unit under earthquake loading--an adequate degree of stability "s" in terms of limited permanent ground deformations under future earthquake loadings. Ps estimates are themselves random variables; they have an inherent level of conservatism dependent on the conservatism of the formulations of C and D on which they are based. For this evaluation Ps has the expected or probable ranges shown in Fig 4 with absolute bounds of 0 and 1. The utility of Ps comes directly from its use as a design tool: examining quantitative and qualitative implications to facility reliability, cost and safety, including sensitivity comparisons of different evaluation assumptions and design factors; further utility comes indirectly from the tendency of the Ps evaluation process to require systematic consideration of uncertainty and explicit documentation of assumptions and methods. Interpreting an "exact" meaning of Ps (for a given site/facility) requires detailed understanding of the evaluation process (the data bases, computational methods, definitions, objectives, interpretations and assumptions--both explicit and implicit, objective and subjective) considered in context with relevant design, construction, and facility operation factors. Clearly, the need for judgement in evaluating, assessing and utilizing Ps in the design process is unavoidably fundamental. Finally, any requirements to reduce impacts on the facility due to unanticipated negative "surprises" must ultimately rely on resilient, earthquake resistant design.

ACKNOWLEDGMENT

$K\alpha$ as a function of α and N_1 (Appendix I) was interpreted from Vaid and Finn's data (9) by N. Paruvakat. Thanks is given to the Alaska State DOTPF, particularly W. Slater, for the opportunity to develop the results presented here.

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APPENDIX I: Approximation of Average r_u Within A Failure Zone (Z)

The expression for r_u referenced by Seed (6) for liquefaction (average conditions)

$$r_u = \frac{2}{\pi} \sin^{-1}\left[\left(\frac{N}{N_\ell}\right)^{0.7}\right] \qquad (0<r_u<1.0) \qquad \text{(I.1)}$$

was used with substitution of an expression for N/N_ℓ based on an estimate of $FS[\alpha>0]$, the average factor of safety against r_u=1.0 within a failure zone Z having an average initial shear stress ratio $\alpha>0$. Seed's Fig 13 (6) liquefaction data was used to develop Eq I.2 (note, however, Eq I.3 may be an improved expression) for $1.0<N/N_\ell<0.0$

$$\frac{N}{N_\ell} = A-B\cdot FS[\alpha>0] \quad (I.2)$$

$$\frac{N}{N_\ell} = \frac{2}{\pi}\sin^{-1}\left[\left(2-FS[\alpha>0]\right)^x\right] \quad (I.3)$$

A,B and x are rough functions of earthquake magnitude, M, and can be (e.g., piecewise linear) functions of FS[α>0]; N/N_ℓ is limited to 1.0 (r_u=1.0) at FS[α>0]<1 and 0(r_u=0) at high FS[α>0]. FS[α>0] was modeled (5) as

$$FS[\alpha>0] = FS\cdot\bar{K}\alpha \quad (I.4)$$

where FS is the average factor of safety against flat ground liquefaction (α=0) and $\bar{K}\alpha$ is the average cyclic stress ratio factor within Z. FS was formulated using Seed's (6) empirical data and relationships for flat ground liquefaction, as

$$FS = \frac{a+bN_1}{c\cdot \underline{a}} \quad (I.5)$$

where: a and b are constants appropriately determined from linearized increments of Seed's Fig 11 (6); and c is the estimated average value of $(0.65\sigma_o r_d/g\sigma_o')$ within Z. $\bar{K}\alpha$ was estimated using an interpretation of data presented by Vaid & Finn (9) for Kα as a linear function of α and relative density (converted to N_1) between 0<α<0.1 with a lower limit of 1.0 (α=0, all N_1; all α, N_1<11) and upper limits of $(N_1+1)/12$ (α>0.1, 11<N_1<23) and 2.0 (α>0.1, N_1>23) and weighted over Z by u, v, w: the proportion of Z having, respectively, α=0(u), 0<α<.1(v), and α>0.1(w) where u+v+w=1.0. Note, in all cases, 1.0<$\bar{K}\alpha$<2.0. After substitution, the expression for r_u is obtained as Eq I.6; if Eq I.3 is used instead of Eq I.2, Eq I.7 is obtained

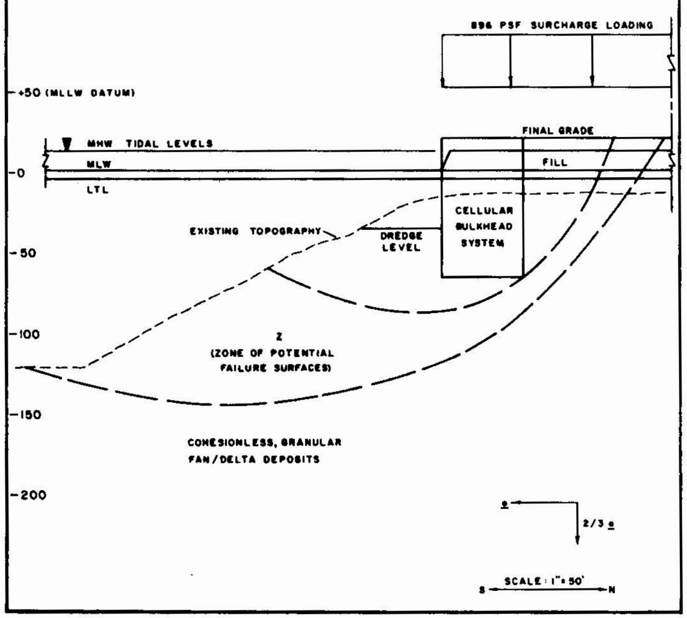
$$r_u = \frac{2}{\pi}\sin^{-1}\left[A-B\left(\frac{a+bN_1}{c\cdot \underline{a}}\right)\bar{K}\alpha\right]^{0.7} \quad (I.6)$$

$$r_u = \frac{2}{\pi}\sin^{-1}\left[\frac{2}{\pi}\sin^{-1}\left(2-\left(\frac{a+bN_1}{c\cdot \underline{a}}\right)\bar{K}\alpha\right)^x\right]^{0.7} \quad (I.7)$$

In this study (a) for M=7.5: A=3.7, B=2.7 with N/N_ℓ=0 for FS[α>0]>1.4, a=0, b=0.0105, c=1.1, $\bar{K}\alpha$=0.43+.052N_1 for 11<N_1<23 and a=-.066, b=.0134, c=1.1, $\bar{K}\alpha$=1.6 for N_1>23, (b) for M=6.5: A=2.0, B=1.3 for 1.1<FS[α>0]<1.6 and N/N_ℓ=0 for FS[α>0]>1.6; all other values were the same as for M=7.5 except a and b were increased by 16%.

FIGURE 1

CRITICAL CROSS SECTION FOR SLOPE STABILITY
KETCHIKAN VESSEL MAINTENANCE FACILITY



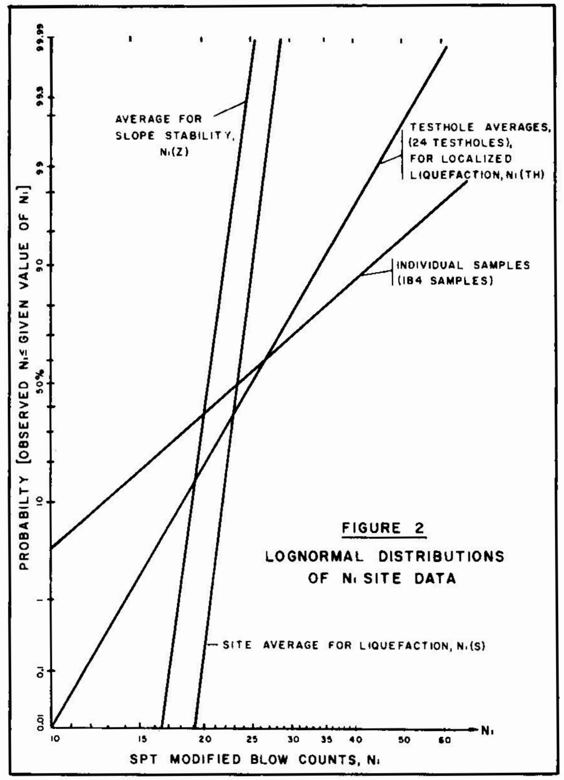


FIGURE 2

LOGNORMAL DISTRIBUTIONS OF N_1 SITE DATA

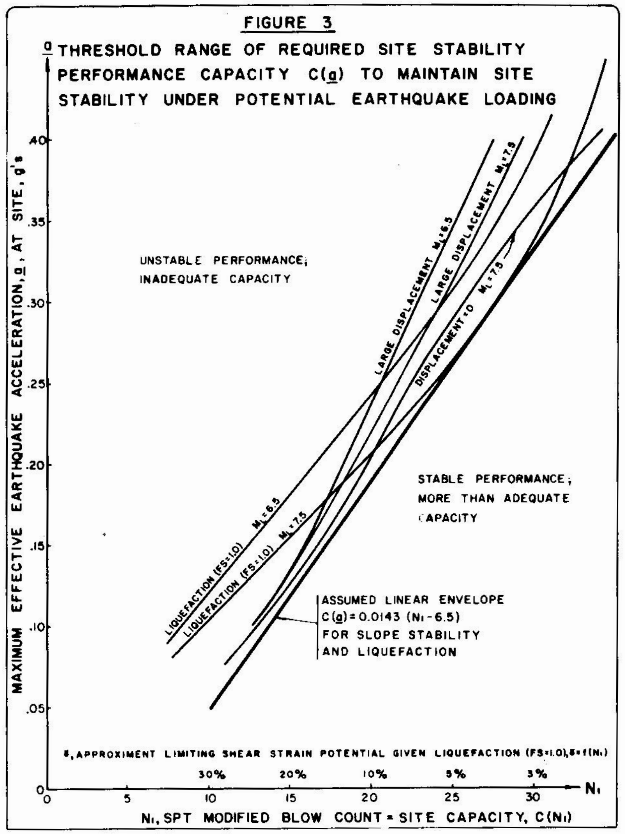


FIGURE 3

THRESHOLD RANGE OF REQUIRED SITE STABILITY PERFORMANCE CAPACITY C(a) TO MAINTAIN SITE STABILITY UNDER POTENTIAL EARTHQUAKE LOADING

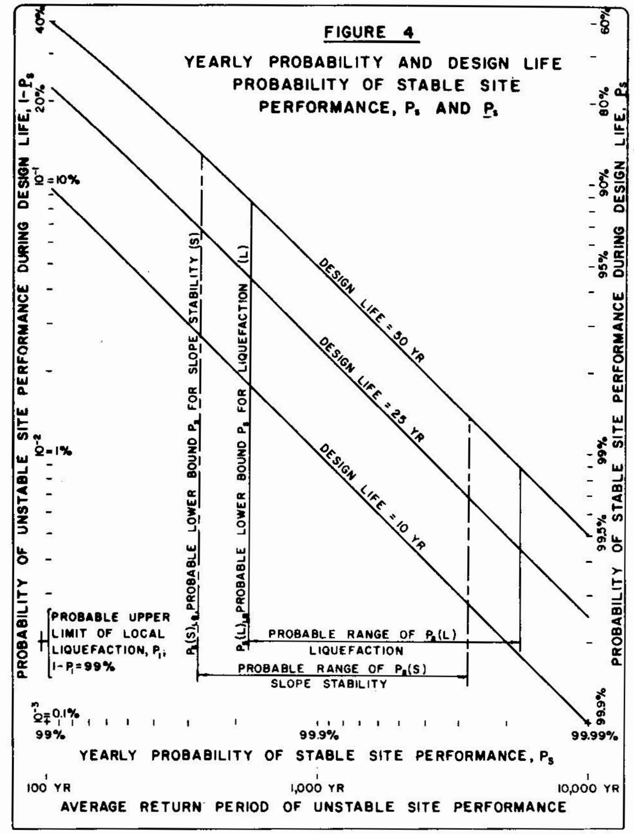


FIGURE 4

YEARLY PROBABILITY AND DESIGN LIFE PROBABILITY OF STABLE SITE PERFORMANCE, P_s AND $\underline{P}_s$