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Reliability Analysis of Anchored and Cantilevered Flexible Retaining Structures

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ABSTRACT: The primary objectives of this paper are to describe the details of a reliability analysis procedure for the geotechnical design of anchored and cantilevered flexible retaining structures using the 2002 AASHTO Bridge Design Specifications and to apply the procedure to evaluate the relationship among the major design parameters and the reliability index (β).

The load models employed in the analyses consist of the apparent earth pressure (AEP) concept for anchored walls and the Rankine earth pressure distribution for cantilevered walls. The load statistics are derived primarily from the bias observed in the AEP model, in addition to the statistical variation in the primary input parameters, namely the Rankine coefficient of active earth pressure (K_a) and the soil unit weight (γ_s). The resistance statistics for anchor pullout and the passive resistance of discrete embedded vertical elements are based solely on field test data. The statistical variation in the passive resistance of continuous embedded vertical elements is evaluated by considering the variation of the primary input parameters, namely the Rankine coefficient of passive earth pressure (K_p) and γ_s.

For anchored walls, the geotechnical limit states of anchor pullout and passive resistance are considered. Only overturning (passive resistance) is considered for cantilevered walls. The analyses demonstrate that the most important parameter in the reliability analysis for these types of walls is the statistical variability of the geotechnical resistance. Reliability indices are calculated using Monte Carlo simulations for anchored and cantilevered flexible retaining walls of typical dimensions. The results of the analyses also indicate that the reliability index of such structures designed according to existing geotechnical practice can vary significantly as functions of the degree to which engineering judgment is permitted and the type of soil strength data employed in resistance predictions.

1 INTRODUCTION

This paper provides a description of a reliability analysis procedure to evaluate the degree of reliability of the existing geotechnical design of anchored and cantilevered flexible retaining structures, as expressed by the reliability index (β), using the LRFD Bridge Design Specifications (AASHTO, 2002). A review of the applicable design models is presented, and the statistical variations of both earth load and resistance are provided. The load and resistance statistics are expressed in terms of bias (λ), which represents the measured data normalized by the corresponding predicted values, and coefficient of variation (COV), which is equal to the standard deviation (S.D.) normalized by the mean value. Limit state functions are formulated, and definitions of the probability of failure (P_F) and β are presented.

Trial geotechnical designs are subsequently performed for both anchored and cantilevered flexible retaining structures. For the sake of simplicity, it is assumed that the spacing of discrete vertical embedded anchor wall elements is such that each acts independently (i.e., interaction effects are neglected). In addition, the flexible cantilever retaining structures are assumed to consist of continuous wall elements. Reliability analyses are conducted for discrete anchor wall elements embedded in cohesionless and cohesive soils that retain cohesionless and stiff cohesive soil. Only cohesionless soil is considered as far as the geotechnical design of continuous flexible cantilever walls is concerned. Values of β are reported for the limit states of anchor pullout and passive resistance. Significant variations in β are observed in existing geotechnical design practice.

As far as the anchor pullout limit state is concerned, the relatively wide range of presumptive ultimate soil-grout bond stresses reported in the literature make the quantification of pullout resistance statistics a challenge. The reliability analyses for anchor pullout were performed assuming that lower bound (PTI, 1996) values of ultimate soil-grout bond stress, as a function of general soil type, relative density (cohesionless bond), and unconfined compressive strength (cohesive bond), are adopted. This scenario most likely corresponds to a "truly presumptive" ground anchor design basis (i.e., instances in which no prior load testing or experience is available before anchor installation, and no verification or proof testing is performed during anchor installation). Use of the lower bound ultimate bond stresses reported by PTI (1996) as the benchmark for the reliability analyses is consistent with the 2002 AASHTO LRFD Bridge Design Specifications in that these values are, at present, the only ones reported in the existing (2002) Commentary. While each ground anchor is proof-tested to a load equal to or exceeding 133% of the unfactored design load, the ultimate capacity (ultimate limit state) of such an anchor typically cannot be inferred from such a test, which represents a quasi-serviceability check, unless the maximum proof test load cannot be maintained at the anchor head.

For the passive resistance limit state, the resulting degree of reliability is highly dependent upon the degree of conservatism employed in the selection of the geotechnical strength parameters, in addition to the type of laboratory test used to characterize the soil strength properties. This is especially true for walls embedded in cohesive soils.

Preliminary recommendations are set forth to make the reliability of flexible retaining structures more consistent with that inherent in the design of bridge superstructures (AASHTO, 2002).

2 LOAD MODELS AND STATISTICS

2.1 Flexible Anchored Retaining Structures

Insofar as the structural and geotechnical design of flexible anchored retaining structures is concerned, estimates of the design earth load have traditionally been calculated from the Apparent Earth Pressure (AEP) envelopes popularized by Terzaghi and Peck (1967), which were originally developed to facilitate the design of struts for internally-braced excavations. While modifications to these AEP distributions have been recently recommended for anchored retaining structures (GeoSyntec, 1999), the total earth load applied using these new distributions yield values that are generally equivalent to the total earth load using the original AEP envelopes. On this basis, the statistics obtained from data used to develop the original AEP diagrams for internally-braced excavations are applicable to the design of flexible anchored retaining structures.

2.1.1 Walls Retaining Cohesionless Soils

For walls retaining cohesionless soils, Terzaghi and Peck (1967) recommended a uniform design AEP of $0.65K_a\gamma_sH$, where

K_a = Rankine coefficient of active earth pressure = $[1-\sin\bar{\phi}] / [1+\sin\bar{\phi}]$,
$\bar{\phi}$ = effective stress friction angle,
γ_s = soil unit weight, and
H = wall height.

This design recommendation was developed on the basis of strut data collected and synthesized by Flatte (1966) from internally-braced subway excavations in Berlin, Munich, and New York, as shown in Figure 1.

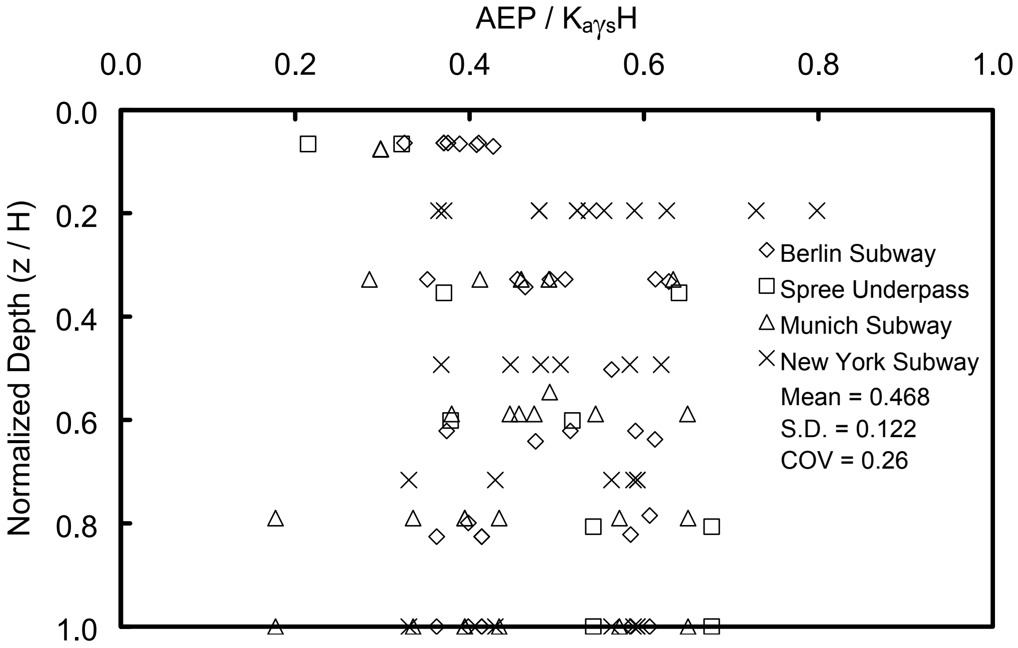

Figure 1. AEP/$K_a\gamma_s$H Versus z/H for Cohesionless Soils (Flaate, 1966)

The data presented in Figure 1 demonstrate that an AEP of $0.65K_a\gamma H$ for cohesionless soil is more representative of an *upper bound* to the earth load rather than a *mean* value. Hence, as far as reliability analysis is concerned, the load bias (λ_{AEP}) for anchored walls retaining cohesionless soils is less than 1.0. The data presented in Figure 1 indicate that λ_{AEP} = 0.468/0.65 = 0.72. However, to be conservative, a value of λ_{AEP} = 0.50/0.65 = 0.77 (corresponding to the ratio of earth thrust calculated from Rankine theory to that evaluated from the AEP diagram) is adopted in the reliability analyses. Rather than employing COV_{AEP} = 0.26 reported in Figure 1 for cohesionless soils, the AEP variability was incorporated by simulating random values of the key design parameters, namely γ_s and K_a (by simulating $\overline{\phi}$). The variability associated with measuring or estimating these parameters is the primary source of the variability in the data presented in Figure 1.

A summary of typical ranges and average values of COV for in-situ γ_s are provided in Table 1. Using the mean COVs reported by Phoon and Kulhawy (1999), a typical $COV_{\gamma s}$ of 0.08 appears to be appropriate for natural soil deposits. It should be recognized that in the calibration of the AASHTO LRFD Code for superstructures (Nowak, 1999), the statistical parameters adopted for dead load (DL) were λ_{DL} = 1.0 and COV_{DL} = 0.10. Therefore, a nominal $COV_{\gamma s}$ of 0.10 is conservatively adopted for γ_s. In addition, it is assumed that $\lambda_{\gamma s}$ = 1.0.

Table 1. Reported COVs of In-Situ Soil Unit Weight ($COV\gamma_s$)

Property	$COV\gamma_s$ Range	Average $COV\gamma_s$	Reference(s)
Submerged (buoyant) unit weight	0.00 - 0.10	-	Lacasse and Nadim (1996); Duncan (2000)
Dry unit weight for fine-grained soil	0.02 - 0.13	0.07	Phoon and Kulhawy (1999)
Total unit weight for fine-grained soil	0.03 - 0.20	0.09	Phoon and Kulhawy (1999)
Total unit weight	0.05 - 0.15	-	Meyerhof (1995)

Phoon, et al. (1995) reported the following general ranges of COV associated with the measurement or estimation of $\bar{\phi}$:

- Good Quality Laboratory Measurements: COV_ϕ = 0.05 to 0.10
- Indirect Correlations with Good Field Data (i.e., CPT q_c): COV_ϕ = 0.10 to 0.15
- Strictly Empirical Correlations (i.e., SPT N Value): COV_ϕ = 0.15 to 0.20

First-order estimates of COV_{Ka} can be calculated by considering the aforementioned ranges of COV_ϕ and by assuming that the relationship between K_a and $\bar{\phi}$ is linear at a particular reference (mean) $\bar{\phi}$. Such first order estimates of COV_{Ka} at reference values of $\bar{\phi}$ = 30°, 35°, and 40° are provided in Table 2. In reality, the relationship between K_a and $\bar{\phi}$ is non-linear. Therefore, the first order estimates of COV_{Ka} in Table 2 were not used directly in the reliability analyses. Rather, values of $\bar{\phi}$ were generated randomly, and the corresponding value of Rankine K_a for each randomly generated value of $\bar{\phi}$ was subsequently calculated.

Table 2. General Ranges of COV_ϕ and
COV_{Ka} (First Order Analysis) for Cohesionless Soils

Parameter	Reference $\bar{\phi}$ (deg)	SPT N Value	CPT q_c	Laboratory Measurements (TC, DS)
Degree of Variability:		"high"	"medium"	"low"
COV_ϕ :		0.15 - 0.20	0.10 - 0.15	0.05 - 0.10
COV_{Ka}:	30	0.19 - 0.27	0.13 - 0.19	0.06 - 0.13
	35	0.24 - 0.33	0.16 - 0.24	0.08 - 0.16
	40	0.30 - 0.41	0.19 - 0.30	0.09 - 0.19

TC = triaxial compression
DS = direct shear

2.1.2 Walls Retaining Stiff Cohesive Soils

For walls with a high factor of safety against basal instability, the expression for Rankine K_a for structures retaining cohesive soil is given as follows:

$$K_a = 1 - \frac{4\,s_u}{\gamma_s\,H} \tag{1}$$

where s_u = mean undrained shear strength within wall height H. If $s_u > \gamma_s H/4$, the resulting calculated value of K_a becomes negative. This seemingly problematic discrepancy is explained by the fact that the stresses and deformations in stiff cohesive soils correspond to a quasi-elastic state rather than a state of limiting equilibrium (as assumed in the calculation of K_a). Therefore, the AEP diagram recommended by Terzaghi and Peck (1967) for stiff cohesive soil does not explicitly consider K_a; rather, it consists of a trapezoidal pressure distribution with a maximum pressure ordinate ranging between $0.20\gamma_s H$ and $0.40\gamma_s H$.

The wide range of recommended design AEP for stiff cohesive soils makes it difficult to accurately define a load bias (λ_{AEP}). Typically, an AEP of $0.20\gamma_s H$ is used for "short term" conditions and $0.40\gamma_s H$ for "long term" conditions. However, Terzaghi, et al. (1996) state that an AEP of $0.20\gamma_s H$ be used "only when results of observations on similar cuts in the vicinity so indicate. Otherwise, a lower limit (on AEP) should be taken as $0.30\gamma_s H$." Considering this wide range of recommended design AEP, it is expected that the design engineer will typically adopt an AEP = $0.40\gamma_s H$ for the design of flexible anchored retaining walls in stiff cohesive soil. Again, this tendency toward conservatism will inherently yield a value of λ_{AEP} that is less than 1.0. From a practical standpoint, λ_{AEP} for stiff cohesive soils can range from 0.50 to 1.00, with a "typical" (average) value of 0.75, which closely corresponds to the value of λ_{AEP} = 0.77 adopted for cohesionless soils.

Unfortunately, only a limited amount of data exists for anchored walls in stiff cohesive soil. However, a significant amount of data does exist for strutted and anchored walls in soft to medium clays and interbedded sand and stiff clay. Therefore, the variability of these measurements will be presented and subsequently adopted for anchored walls in stiff cohesive soil.

Flaate (1966) summarized and evaluated the measured strut loads for several excavations in soft to medium cohesive soils in Chicago, Japan, England, and Norway. A statistical summary of the $AEP/K_a\gamma_sH$ data for soft to medium clays with N (i.e., γ_sH/s_u) > 4 and base stability number N_b (i.e., γ_sH/s_{ub}) < 5.14 is provided in Table 3. Three data sets are considered in the table: (1) all struts, (2) only struts within the bottom three-quarters of wall height H, and (3) only the "critical" (i.e., most heavily loaded) struts.

Table 3. Summary of Statistical Data on $AEP/K_a\gamma_sH$ for Soft to Medium Clays with N > 4 and N_b < 5.14

Data Set	$AEP/K_a\gamma_sH$ Statistics				
	Mean	S.D.	Range	COV_{AEP}	n
All Struts	0.655	0.307	0.04-1.33	0.47	89
Struts Within Bottom 0.75H	0.764	0.254	0.30-1.33	0.33	67
"Critical" Struts	0.924	0.255	0.46-1.33	0.28	25

Measured values of K_a for cases in which N > 4 and N_b < 5.14 were determined by summing the strut loads in a particular strutted section (expressed as load per unit length along wall) and dividing by $0.5\gamma_sH^2$. This calculation presumes a triangular distribution of lateral earth pressure on the bracing. For cases in which N_b > 5.14, the values of computed K_a were modified by a correction factor ΔK as follows:

$$\Delta K = \frac{2\sqrt{2}\,d}{H}\left(1 - \frac{(2+\pi)\,s_{ub}}{\gamma H}\right) \tag{2}$$

where s_{ub} = mean undrained shear strength of the basal soil. Statistics on $(K_a)_{measured}/(K_a)_{computed}$ are shown in Table 4.

Table 4. Summary of Statistical Data on $(K_a)_{measured}/(K_a)_{computed}$ for Soft to Medium Clays with N > 4

Data Set	$(K_a)_{measured}/(K_a)_{computed}$ Statistics				
	Mean	S.D.	COV_{Ka}	Range	n
N_b < 5.14 (ΔK = 0)	0.975	0.240	0.25	0.66-1.68	35
All N_b (including ΔK correction)	1.009	0.248	0.25	0.66-1.68	42

In addition, O'Rourke (1975) reported AEP values obtained from two excavations made in interbedded sand and stiff clay in Washington, D.C., along with AEP data from other braced and tied-back cuts in similar soils. The data were divided into two general populations: cuts with H > 15 meters and H < 15 meters. Corresponding back-calculated values of AEP, normalized by γ_sH (i.e., AEP/γ_sH), from the measurements on struts and tiebacks for cuts with H > 15 meters are plotted in Figure 2 versus normalized depth. The data in Figure 2 include the maximum measurements from all strut and tieback levels for each cut, and indicate a mean AEP/γ_sH = 0.176 (S.D. = 0.051, COV_{AEP} = 0.29, n = 38). In general, the great majority of AEP data fall within an envelope of width $0.25\gamma_sH$. Figure 3 shows AEP data only for the "critical" (i.e., most highly loaded) strut or tieback level for each cut. The statistics corresponding to the data in Figure 3 indicate a mean $AEP/\gamma_s H = 0.248$ (S.D. = 0.032, $COV_{AEP} = 0.13$, n = 5). Back-calculated values of $AEP/\gamma_s H$ from the measurements on struts are plotted in Figure 4 versus normalized depth for cuts with H < 15 meters. The data in Figure 4 include the maximum measurements from all strut levels for each cut, and indicate a mean $AEP/\gamma_s H = 0.116$ (S.D. = 0.045, $COV_{AEP} = 0.39$, n = 31). In general, the great majority of AEP data fall within an envelope of width $0.20\gamma_s H$. In Figure 5, only data for the critical strut level of each cut are provided. The statistics corresponding to the data in Figure 5 indicate a mean $AEP/\gamma_s H = 0.165$ (S.D. = 0.043, $COV_{AEP} = 0.26$, n = 5).

On the basis of the data presented in Tables 3 and 4 and Figures 2 through 5, a $COV_{AEP} = 0.28$ for stiff cohesive soil was selected for the reliability analyses.

Incidentally, the "long term" condition for a cohesive soil essentially represents a drained situation. Therefore, from a theoretical standpoint, the use of the effective stress friction angle of a stiff clay to calculate a Rankine value of K_a, and subsequently adopting a design $AEP = 0.65K_a\gamma_s H$, may be a more appropriate approach in the evaluation of the operative long term earth pressures. If $\bar{\phi} = 30°$, for instance, $K_a = 0.33$. Hence, $0.65K_a = 0.22 < 0.40$. On this basis, it is highly unlikely that the upper bound AEP coefficient of $0.40\gamma_s H$ for a stiff cohesive soil for the "long term" condition will ever be achieved. From an effective stress point of view, a $\bar{\phi}$ as low as 15° ($K_a = 0.59$) would be required for $0.65K_a$ to equal 0.40 (i.e., $0.65K_a\gamma_s H = 0.40\gamma_s H$). However, such a low value of $\bar{\phi}$ would not be representative of a stiff clay. (It must be recognized that the additional horizontal load generated from the presence of free water must be considered in such a drained evaluation of a stiff cohesive soil).

2.2 Cantilevered Flexible Retaining Structures

The typical earth load model for cantilevered flexible retaining structures makes use of the Rankine theory of active earth pressure. In this study, only cohesionless soil will be considered as an earth load source for cantilevered flexible retaining structures. This earth load is a function of soil unit weight (γ_s) and the Rankine coefficient of active earth pressure (K_a), which is a function of the effective stress friction angle ($\bar{\phi}$). The variability of γ_s and $\bar{\phi}$ were addressed in Section 2.1. A further description of the limit state equations for cantilevered flexible retaining structures, considering both load and resistance, is provided in Section 4.2.2.

The total active earth pressure per unit length of wall can be calculated as follows:

$$H_a = \frac{K_a \gamma_s (H + D_o)^2}{2} \qquad (3)$$

where H = exposed wall height and D_o = depth of embedment used to calculate the nominal active earth load behind the wall.

Teng (1962) suggested that the actual depth of embedment D should be approximately 20% greater than D_o (i.e., $D = 1.2\ D_o$) to account for discrepancies in the actual pressure distributions acting on the embedded portion of a continuous flexible wall from the presumed triangular distributions.

In the reliability analyses, a 2-ft high soil surcharge is applied at the top of a continuous flexible cantilever wall to simulate a traffic load. This load is transmitted to the wall by using Rankine theory.

3 RESISTANCE MODELS AND STATISTICS

3.1 Pullout Resistance of Anchors Bonded in Cohesionless and Cohesive Soils

The unfactored nominal (ultimate) pullout resistance (Q_a) of a straight ground anchor in soil can be computed as follows:

$$Q_a = \pi\ d\ \tau_a\ L_b \qquad (4)$$

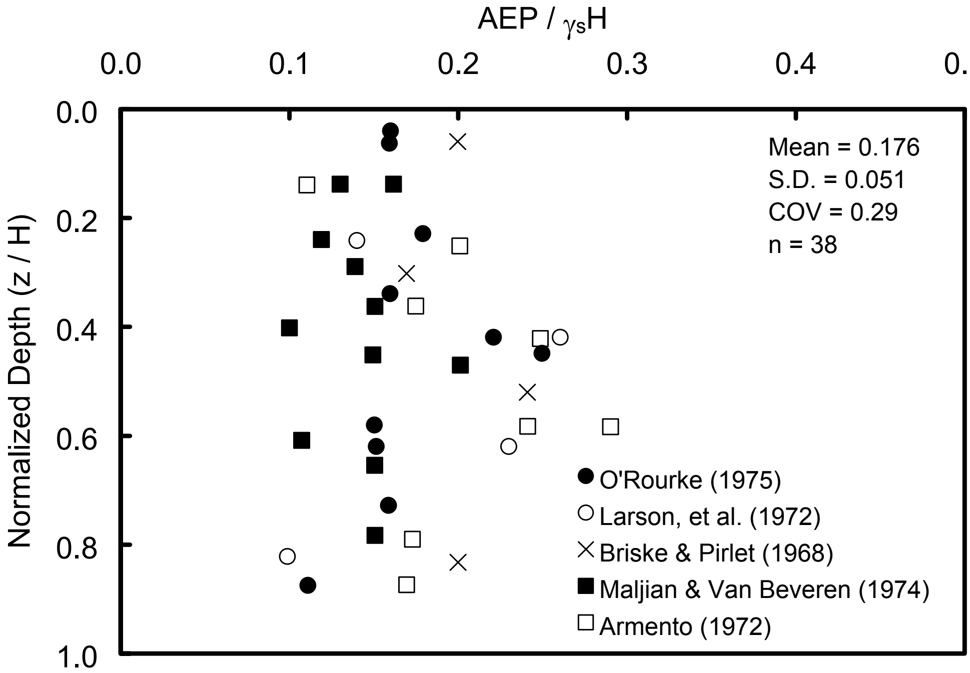

Figure 2. AEP/γ_sH Versus Normalized Depth for Cuts in Interbedded Sand and Stiff Clay, H > 15 meters, Including Data from All Strut and Tieback Levels (O'Rourke, 1975)

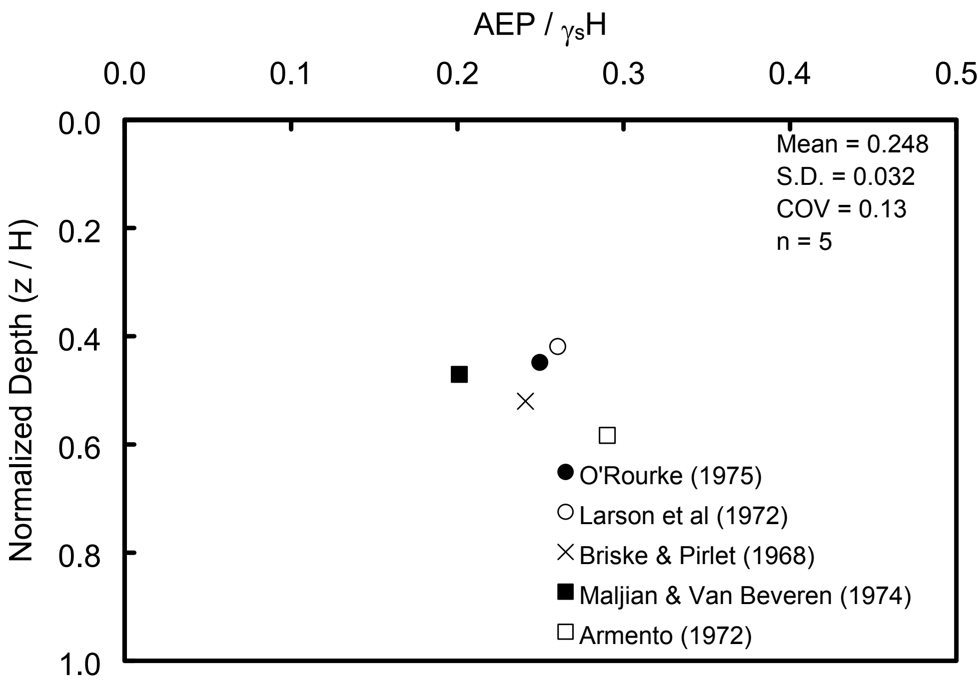

Figure 3. AEP/γ_sH Versus Normalized Depth for Cuts in Interbedded Sand and Stiff Clay, H > 15 meters, Considering Only the Critical Strut or Tieback Level (O'Rourke, 1975)

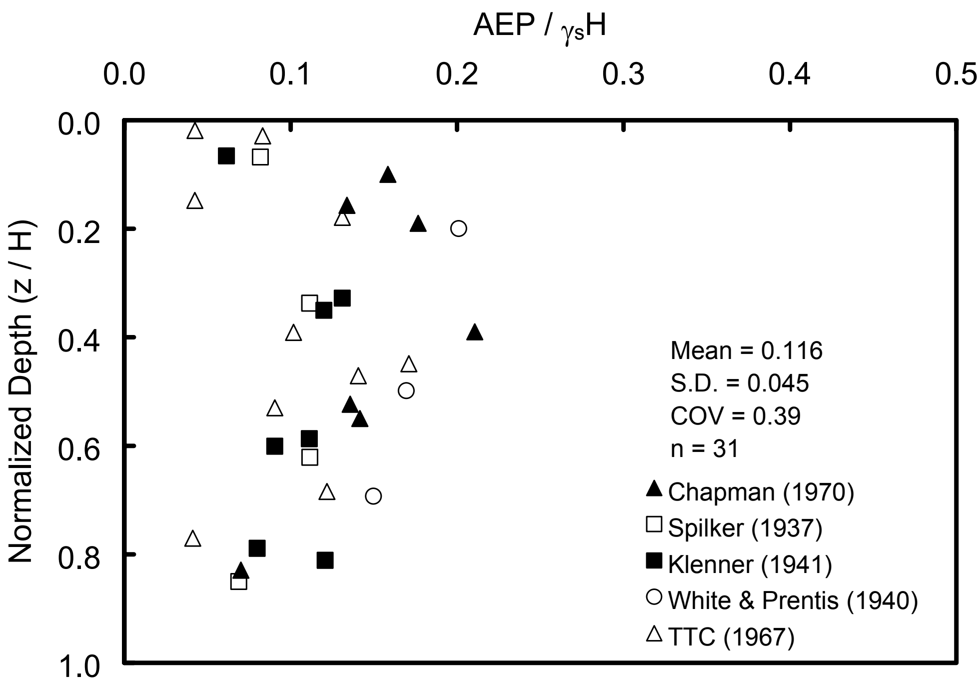

Figure 4. AEP/γ_sH Versus Normalized Depth for Cuts in Interbedded Sand and Stiff Clay, H < 15 meters, Including Data from All Strut and Tieback Levels (O'Rourke, 1975)

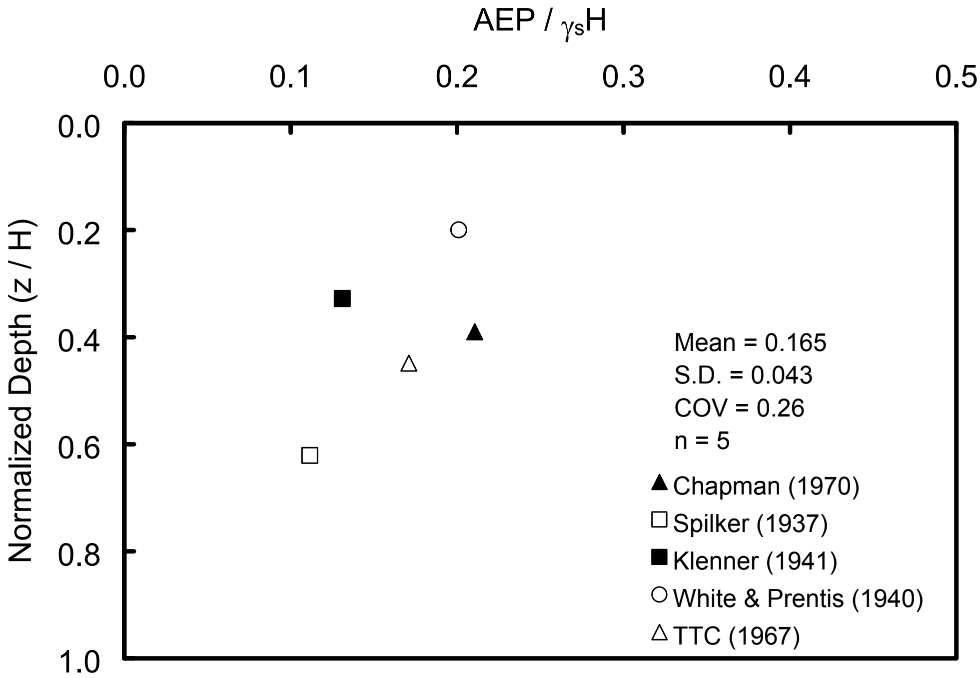

Figure 5. AEP/γ_sH Versus Normalized Depth for Cuts in Interbedded Sand and Stiff Clay, H < 15 meters, Considering Only the Critical Strut or Tieback Level (O'Rourke, 1975)

where d = diameter of anchor drill hole, τ_a = nominal (unfactored) presumptive ultimate anchor bond stress, and L_b = anchor bond length.

Guideline values of τ_a for the preliminary design of anchors bonded in cohesionless and cohesive soils, as reported by PTI (1996), are provided in Tables 5 and 6, respectively. It should be recognized that only the boldfaced (lower bound) values provided in these tables are included in the Commentary of the 2002 AASHTO LRFD Bridge Design Specifications. These values of τ_a are intended to estimate the geotechnical pullout resistance of anchors at the preliminary design stage, and are based on geologic and boring data, soil samples, laboratory testing, and previous experience. As presumptive values, τ_a should be verified as part of a pre-production test program or load testing during construction.

The development of resistance statistics for the ultimate anchor bond stress (τ_a) is therefore complicated by the fact that for each range of soil type and either SPT N value or unconfined compressive strength (q_u) in Tables 5 and 6, respectively, a large range of τ_a exists. The values of τ_a presented in Tables 5 and 6 represent the typical range of measured ultimate bond stresses. However, since the selection of τ_a in the preliminary design stage is also driven by local experience and prior observations of the contractor or design engineer, these design considerations are nearly impossible to quantify on a statistical basis. Therefore, the reliability analyses were conducted on the assumption that, for a particular soil type and range of SPT N or q_u, the lower bound (minimum) value of τ_a is selected in the preliminary design stage. This assumption is consistent with the 2002 AASHTO LRFD Bridge Design Specifications in that only the boldfaced (lower bound, or minimum) values of τ_a in Tables 5 and 6 are reported in the existing Commentary.

In-situ pullout tests in different soil types were collected from published sources and personal communications, as summarized by Hegazy (2003). Measured clay-grout bond strength data were obtained from Barley and McBarron (1997), Bruce (1998), Ostermayer (1975), and Woodland, et al. (1997). In addition, measured sand-grout bond strength data were obtained from Barley and McBarron (1997), Jones (1997), Liao, et al. (1997), and Ostermayer (1975). For each pullout test, a value of bias (λ) was defined as the measured ultimate bond stress divided by the presumptive bond stress. The mean and COV of λ (presumed normally-distributed), as calculated relative to the minimum, maximum, and average values of τ_a provided in Tables 5 and 6, are summarized in Table 7. On this basis, the data would seem to indicate that for ground anchors bonded in sand (cohesionless soil) and designed using lower bound values of τ_a: mean λ = 2.20 and COV_λ = 0.74. Likewise, for ground anchors bonded in clay (cohesive soil) and designed using lower bound values of τ_a: mean λ = 2.56 and COV_λ = 0.71.

Clearly, the presumed normally-distributed nature of the data results in very high values of both mean λ and COV_λ. The use of such high values of COV in reliability analyses, despite the high values of resistance bias λ, would produce unrealistically low values of β, considering the conservative nature of the presumed design procedure (i.e., use of lower bound τ_a). Therefore, the data were evaluated further to check the initial normal distribution assumption.

For each data set (anchors in cohesionless and cohesive soil), the result of each pullout test was assigned a standard normal variable (mean = 0, S.D. = 1). The value of standard normal variable was subsequently plotted against the resistance bias λ (measured bond/presumed bond) for each pullout test. A truly normal distribution of data would result in a linear plot.

The results of such an analysis for anchors in cohesionless soils are provided in Figure 6. It was discovered that one of the anchors in the database for cohesionless soils would have failed a proof test conducted to 133% of the unfactored design load. This data point was removed in the calculation of the standard normal variables in Figure 6. The non-linear nature of the data in Figure 6 for higher bias (λ) values demonstrates that the actual distribution of pullout resistance is not normal. For lower biases, however, the plot is somewhat linear. In such instances, an equivalent normal distribution of data can be assigned to the lower tail of the resistance curve, which more closely represents the operative condition at the design point, or the most likely condition at failure, by drawing a line tangent to these data. The inverse of the slope of this tangent line (i.e., 1/m) represents the equivalent standard deviation, while the bias corresponding

Table 5. Ultimate Unit Bond Stress for Anchors in Cohesionless Soils (PTI, 1996)

Anchor/Soil Type (Grout Pressure, MPa)	Soil Compactness or SPT Resistance[1] (Blows/0.3 m)	Presumptive[2] Ultimate Bond Stress, τ_a (MPa)
Gravity Grouted (<0.35)		
Sand or Sand-Gravel Mixtures	Medium Dense to Dense; 11-50	**0.07**-0.14
Pressure Grouted (0.35-2.8)		
Fine to Medium Sand	Medium Dense to Dense;11-50	**0.08**-0.38
Medium to Coarse	Medium Dense; 11-30	**0.11**-0.66
Sand w/Gravel	Dense to Very Dense; 30-50+	**0.25**-0.97
Silty Sands	---	**0.17**-0.41
	Medium Dense to Dense; 11-40	**0.21**-1.38
Sandy Gravel	Dense to Very Dense; 40-50+	**0.28**-1.38
Glacial Till	Dense; 31-50	**0.30**-0.52

(1) Corrected for overburden pressure.
(2) Only boldfaced values are included in the Commentary of the AASHTO (2002) LRFD Bridge Design Specifications

Table 6. Ultimate Unit Bond Stress for Anchors in Cohesive Soils (PTI, 1996)

Anchor/Soil Type (Grout Pressure, MPa)	Soil Stiffness or Unconfined Compressive Strength (MPa)	Presumptive[1] Ultimate Bond Stress, τ_a (MPa)
Gravity Grouted (<0.35)		
Silt-Clay Mixtures	Stiff to Very Stiff (0.10-0.38)	**0.03**-0.07
Pressure Grouted (0.35-2.8)		
High Plasticity Clay	Stiff (0.10-0.24)	**0.03**-0.07
	V. Stiff (0.24-0.38)	**0.07**-0.17
Medium Plasticity Clay	Stiff (0.10-0.24)	**0.10**-0.25
	V. Stiff (0.24-0.38)	**0.14**-0.35
Medium Plasticity Sandy Silt	V. Stiff (0.24-0.38)	**0.28**-0.38

(1) Only boldfaced values are included in the Commentary of the AASHTO (2002) LRFD Bridge Design Specifications

Table 7. Statistical Properties of Data Bias for Anchor Pullout Resistance (Hegazy, 2003), Presumed Normal Distribution

Soil Type	Value of τ_a used in Design	Mean Bias, $\lambda^{(1)}$	$COV_\lambda^{(1)}$	n
Sand	*Min*	*2.20*	*0.74*	84
	0.5*(Min + Max)	0.94	0.49	
	Max	0.63	0.47	
Clay	*Min*	*2.56*	*0.71*	59
	0.5*(Min + Max)	1.48	0.75	
	Max	1.05	0.75	

(1) Presumed Normal Distribution

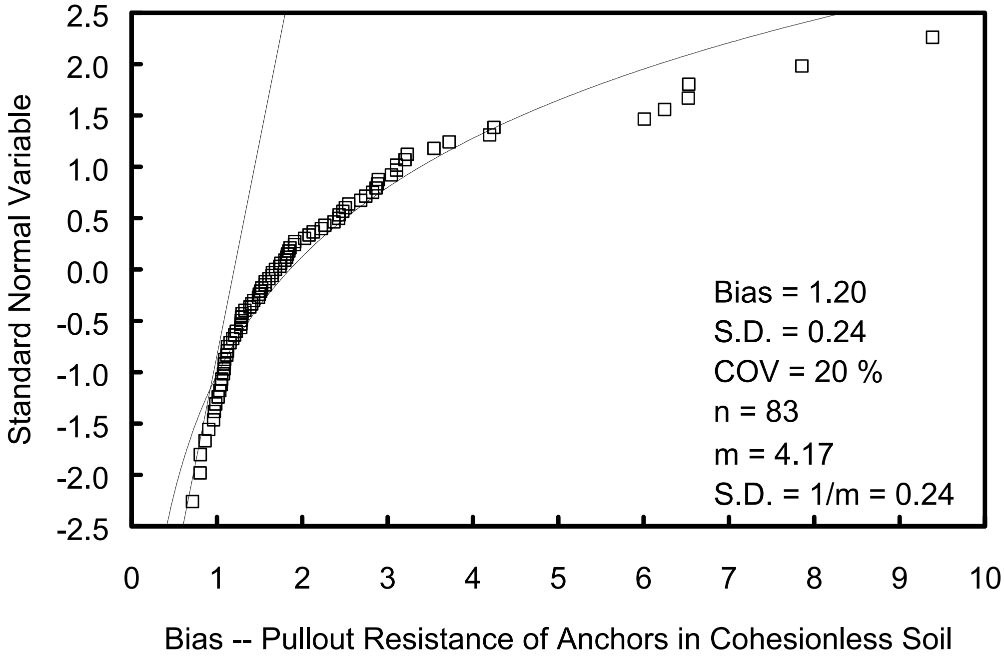

Figure 6. Development of Equivalent Normal Resistance Distribution Near The Design Point for Anchors in Cohesionless Soil, Using Lower Bound τ_a (Failed Proof Test Excluded)

to a standard normal variable of zero along the tangent line represents the equivalent bias. These data are plotted in Figure 6, and the resulting equivalent normal distribution near the design point has a mean λ = 1.20 and COV_λ = 0.20.

Similar analyses were performed for anchors in cohesive soil, the results of which are provided in Figure 7. It was discovered that all of the anchors in the database for cohesive soils would have passed a proof test conducted to 133% of the unfactored design load. The resulting equivalent normal distribution near the design point has a mean λ = 1.40 and COV_λ = 0.20.

The statistics presented in Figures 6 and 7 are more representative of the real design situation, provided that lower bound presumptive values of ultimate bond stress (τ_a) are employed in the preliminary stage of ground anchor design. Again, this assumption is consistent with the 2002 AASHTO LRFD Bridge Design Specifications in that only the boldfaced (lower bound, or minimum) values of τ_a in Tables 5 and 6 are reported in the existing Commentary. Therefore, the statistics reported in Figures 6 and 7 are employed in the subsequent reliability analyses for anchor pullout resistance.

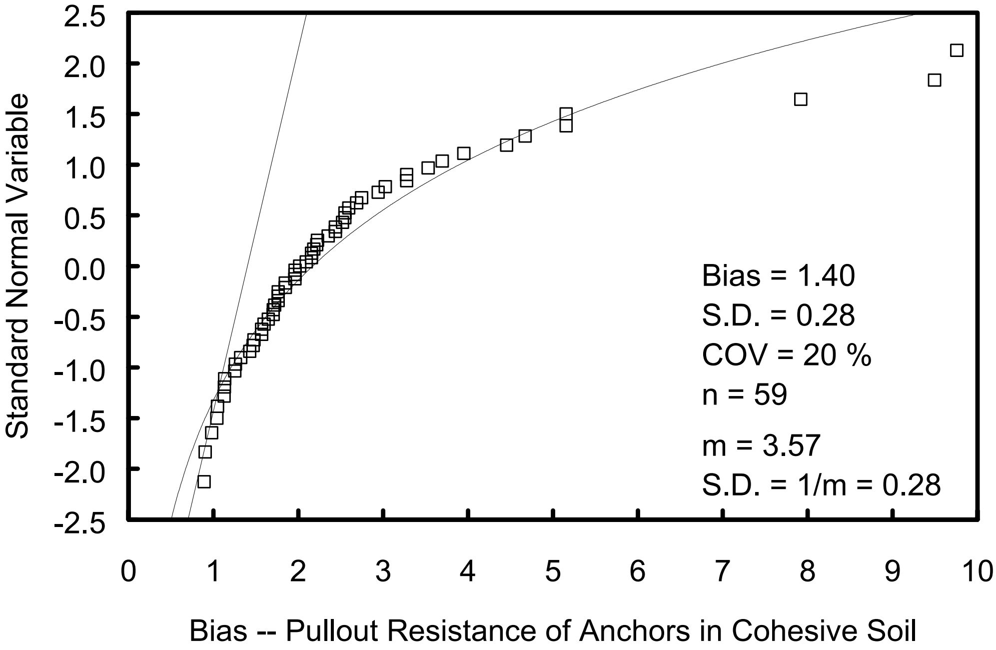

Figure 7. Development of Equivalent Normal Resistance Distribution Near The Design Point for Anchors in Cohesive Soil, Using Lower Bound τ_a

3.2 Passive Resistance (Embedment)

3.2.1 Discrete Vertical Elements

The passive resistance of retaining structures with discrete vertical embedded elements has typically been evaluated using the relationships developed by Broms (1964a, 1964b) for single laterally loaded piles, as shown in Figure 8. Details of the Broms method are provided in the subsequent subsections.

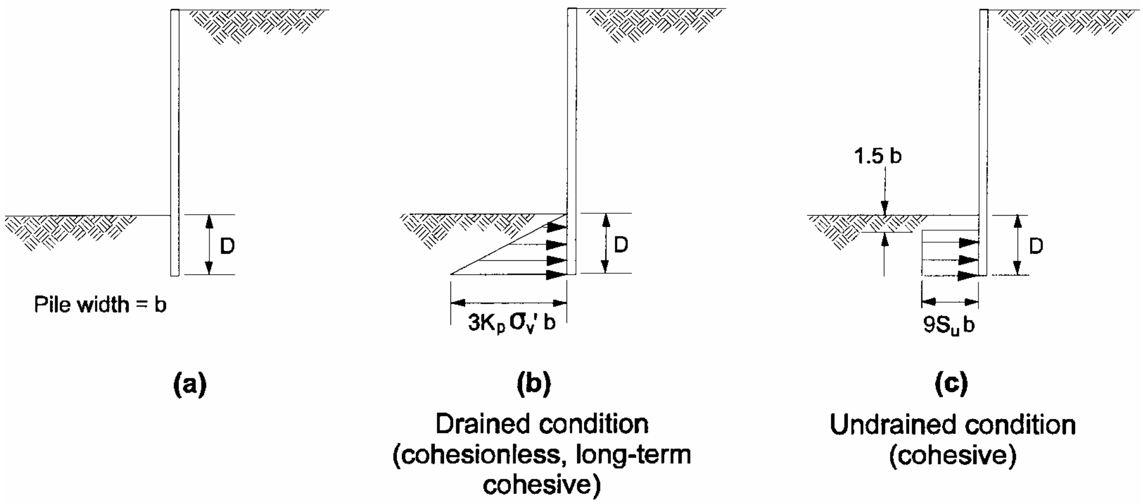

Figure 8. Broms Method for Evaluating the Nominal (Ultimate) Passive Resistance of Discrete Vertical Wall Elements (Broms, 1964a, 1964b)

Chen and Kulhawy (1994) reported the results of a case history evaluation of single, unrestrained (free-headed) laterally-loaded rigid drilled shafts embedded in both cohesionless and cohesive soils. The hyperbolic method, as described by Manoliu, et al. (1985), was used to interpret the ultimate lateral capacity from results of both laboratory and field lateral load tests. These interpreted ultimate lateral capacities (H_h) subsequently were compared to the predicted ultimate capacity (H_p) obtained from the Broms method. The resulting normalized statistics, which are summarized in the following subsections, are used to perform the reliability analyses for embedded discrete vertical wall elements.

Since the experimental data are based on load tests of single, free-headed, rigid drilled shafts, the resulting normalized statistics provide insight into the variability of the unit passive resistance prescribed by the Broms model. However, the fixity conditions of the embedded portion of a vertical wall element differ from that of a laterally-loaded, free-headed, rigid drilled shaft. Therefore, the equations used to calculate the value of H_p for these two scenarios are not the same. Nevertheless, the normalized statistics from the drilled shaft load tests can be applied to embedded discrete wall elements with circular cross-sections, such as concrete-encased H-piles in pre-drilled holes spaced far enough apart to act independently, by virtue of the consistent ultimate unit passive pressure prescribed for each fixity condition. The statistics can also be conservatively applied to vertical elements with square cross-sections. However, the statistics are not directly applicable to closely-spaced vertical elements or continuous embedded wall segments.

3.2.1.1 Elements Embedded in Cohesionless Soils

In the case of cohesionless soils (or drained, "long term" conditions), the passive pressure is assumed to act over three pile widths (3b); however, this effective width cannot exceed the horizontal spacing (S_H) between the discrete vertical elements. Therefore, the ultimate passive resistance (H_p) of a discrete vertical wall element embedded in cohesionless soil can be calculated as follows:

$$H_p = \frac{K_p\ \gamma_s\ D^2 (3b)}{2} \leq \frac{K_p \gamma_s\ D^2 (S_H)}{2} \tag{5}$$

where K_p = Rankine coefficient of passive earth pressure = $[1+\sin\bar{\phi}]$ / $[1-\sin\bar{\phi}]$. Equation 5 assumes that the embedded portion of the vertical element either translates laterally as a rigid body, or experiences a combination of lateral translation and rotation about the lowermost anchor, to mobilize the full triangular distribution of passive resistance over depth D. This assumption is reasonably valid, provided that the vertical element does not yield structurally before the ultimate passive resistance expressed by Equation 5 is mobilized.

On average, the triaxial extension (TE) effective stress friction angle is approximately 12% greater than the triaxial compression (TC) effective stress friction angle (i.e., $\bar{\phi}_{te} \approx 1.12\ \bar{\phi}_{tc}$). Chen and Kulhawy (1994) summarized the results of cone tip resistance (q_c) data to estimate values of $\bar{\phi}_{tc}$ using the correlation proposed by Kulhawy and Mayne (1990), and subsequently converted these data to equivalent values of $\bar{\phi}_{te}$. The average values of $\bar{\phi}_{te}$ and $\bar{\phi}_{tc}$ in the Chen and Kulhawy lateral load test database for rigid shafts in cohesionless soils, considering all laboratory and field experiments, were approximately 42° and 38°, respectively. The relative ratio of $K_p(42°)/K_p(38°)$ = 5.04/4.20 = 1.20; this ratio can be used to express the statistical data in terms of both TE and TC laboratory strength data, as inferred from the Kulhawy and Mayne (1990) correlation with the CPT.

For each load test, the interpreted (measured) hyperbolic capacity (H_h) was compared to the predicted ultimate capacity (H_p) obtained from the Broms method. In the case of single, free-headed, laterally-loaded rigid piles, H_p is calculated as follows:

$$H_p = \frac{K_p\ \gamma_s\ D^3 (b)}{2\,(D+e)} \tag{6}$$

where e = lateral load eccentricity. Equation 6 is based on the assumption that the pile rotates rigidly about a point of fixity located above the pile tip. Statistical data on the ratio of H_h/H_p for single rigid drilled shafts embedded in cohesionless soils, presuming a normal distribution to all available data, are provided in Table 8.

The data indicate that, in general, the Broms model provides conservative estimates of ultimate passive resistance for independent rigid elements embedded in cohesionless soils, regardless of whether TE or TC soil strength data are used in the prediction model. It should be recognized that the statistics provided in Table 8 represent the lumped effect of soil property evaluation and model (Broms, 1964b) uncertainties. Unfortunately, the model error associated with the Broms method cannot be deterministically isolated from the soil property error.

The procedure used to evaluate the anchor pullout data at the design point was also applied to the passive resistance statistics (field and lab experiments combined). For the sake of convenience, it is presumed that the prediction of passive resistance in cohesionless soils is typically made using triaxial compression (TC) strength data rather than triaxial extension (TE) data. The standard normal variable for each test is plotted versus the bias λ (measured capacity / predicted capacity using TC data) in Figure 9. The data indicate that the passive resistance is not normally distributed over the entire range of bias. However, a linearization of the lower tail of the resistance distribution results in an equivalent normal distribution near the design point represented by $\lambda = 1.05$ and COV = 0.16. These statistics are used in the subsequent reliability analyses to evaluate the passive limit state for discrete anchor wall elements embedded in cohesionless soil.

3.2.1.2 Elements Embedded in Cohesive Soils

For cohesive soils (or undrained conditions), H_P for discrete vertical wall elements can be calculated by:

$$H_p = 9\, s_u\, b\, (D - 1.5b) \qquad (7)$$

but shall not exceed:

$$H_p = (4s_u - \gamma_s H)\, S_H\, (D - 1.5b) \qquad (8)$$

where H = exposed wall height. Again, the difference in fixity conditions will alter the calculation of H_p for a free-headed, rigid, laterally-loaded pile, from the Equation 7. Further details are provided by Chen and Kulhawy (1994). Equation 8 is a hybrid expression incorporating the methods of Broms (1964a) and Teng (1962) to ensure that the calculated ultimate passive resistance of closely-spaced vertical wall elements does not exceed the total passive resistance of a continuous wall.

Chen and Kulhawy (1994) evaluated the lateral capacity of single rigid drilled shafts embedded in cohesive soils. As far as the evaluation of s_u is concerned, the laboratory strength test that is most applicable to this particular problem is the consolidated-anisotropically undrained triaxial extension (CK_oUE) test. Values of s_u obtained from CK_oUE tests can be linked to the CIUC test method through the a_{TEST} parameter. In obtaining equivalent CK_oUE s_u values from CIUC tests, a typical value of a_{TEST} = 0.406 (corresponding $\bar{\phi}_{tc} = 33°$) was applied to the data.

Both laboratory and field lateral load tests were evaluated by Chen and Kulhawy, and the statistical results, presuming a normal distribution to the data, are summarized in Table 9. These data represent the lumped effect of soil property evaluation and model (Broms, 1964a) uncertainties.

Table 8. Statistical Data on the H_h/H_p Ratio for Single Rigid Drilled Shafts Embedded in Cohesionless Soils – Presumed Normal Distribution (Chen and Kulhawy, 1994)

Statistic	Field Load Test Data		Lab Load Test Data		All Data	
	TE	TC	TE	TC	TE	TC
Bias, $\lambda^{(1)}$:	1.15	1.36	1.31	1.62	1.29	1.55
S.D.:	0.44	0.52	0.53	0.86	0.52	0.61
COV:	≈ 0.38		≈ 0.40		≈ 0.40	
n:	10		55		65	

(1)— $\lambda = H_h/H_p$

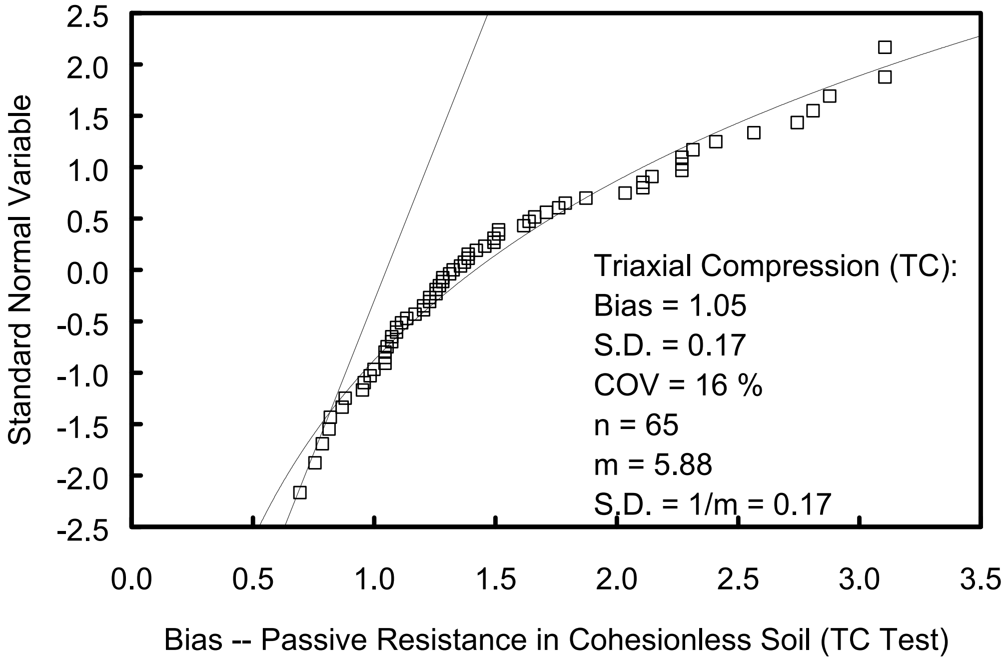

Figure 9. Development of Equivalent Normal Distribution Near the Design Point for Passive Resistance of Single Rigid Drilled Shafts Embedded in Cohesionless Soil (TC)

Table 9. Statistical Data on the H_h/H_p Ratio for Single Rigid Drilled Shafts Embedded in Cohesive Soils using Equivalent CK_oUE and CIUC Laboratory Strength Data -- Presumed Normal Distribution (Chen and Kulhawy, 1994)

Statistic	Field Load Test Data		Lab Load Test Data		All Data	
	$CK_oUE^{(2)}$ (TE)	$CIUC^{(2)}$ (TC)	$CK_oUE^{(3)}$ (TE)	CIUC (TC)	CK_oUE (TE)	CIUC (TC)
Bias, $\lambda^{(1)}$:	2.29	0.93	2.28	0.93	2.28	0.93
S.D.:	1.02	0.41	0.80	0.32	0.88	0.36
COV:	≈ 0.45		≈ 0.35		≈ 0.38	
n:	21		47		68	

(1) – $\lambda = H_h/H_p$
(2) – As inferred from UU & UC tests
(3) – As inferred from CIUC tests

Again, analyses were performed to check the normal distribution assumption for passive resistance. The results indicate that the resistance distribution is not normal over the entire range of bias. Plots of standard normal variable versus bias from each lab and field test are provided in Figures 10 and 11 for predictions using TE (CK_oUE) and TC (CIUC) strength data, respectively. The equivalent normal statistics associated with the lower tail of the resistance near the design point, as provided in these figures, are employed in the subsequent reliability analyses.

3.2.2 Continuous Wall Elements

The typical passive resistance model for the embedded portion of cantilevered flexible retaining structures makes use of the Rankine theory of passive earth pressure. In this study, only cohesionless soil will be considered as a source of passive resistance for continuous cantilevered flexible retaining structures. The total passive resistance per unit length of wall is a function of soil unit weight (γ_s) and the Rankine coefficient of passive earth pressure (K_p), which is a function of the effective stress friction angle ($\bar{\phi}$), is given as follows:

$$H_P = \frac{K_p \gamma_s (D_o)^2}{2} \tag{9}$$

where D_o is the depth of embedment used to calculate the nominal passive resistance. Teng (1962) suggested that the actual depth of embedment D should be approximately 20% greater than D_o (i.e., D = 1.2 D_o) to account for discrepancies in the actual pressure distributions acting on the embedded portion of a continuous flexible retaining wall from the presumed triangular active and passive distributions.

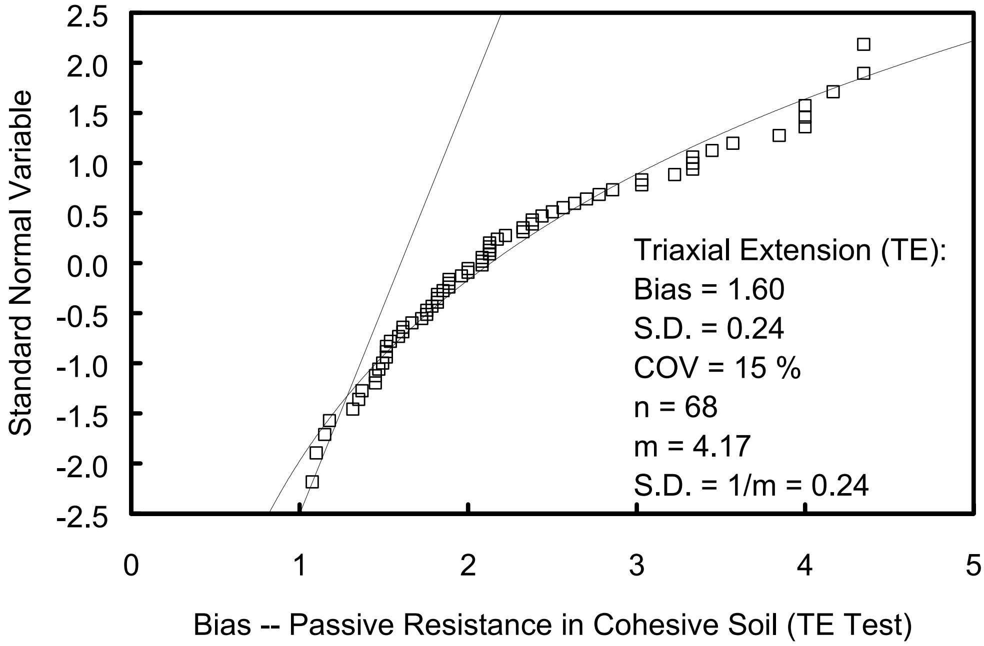

Figure 10. Development of Equivalent Normal Distribution Near Design Point for Passive Resistance of Rigid Drilled Shaft Embedded in Cohesive Soil (TE)

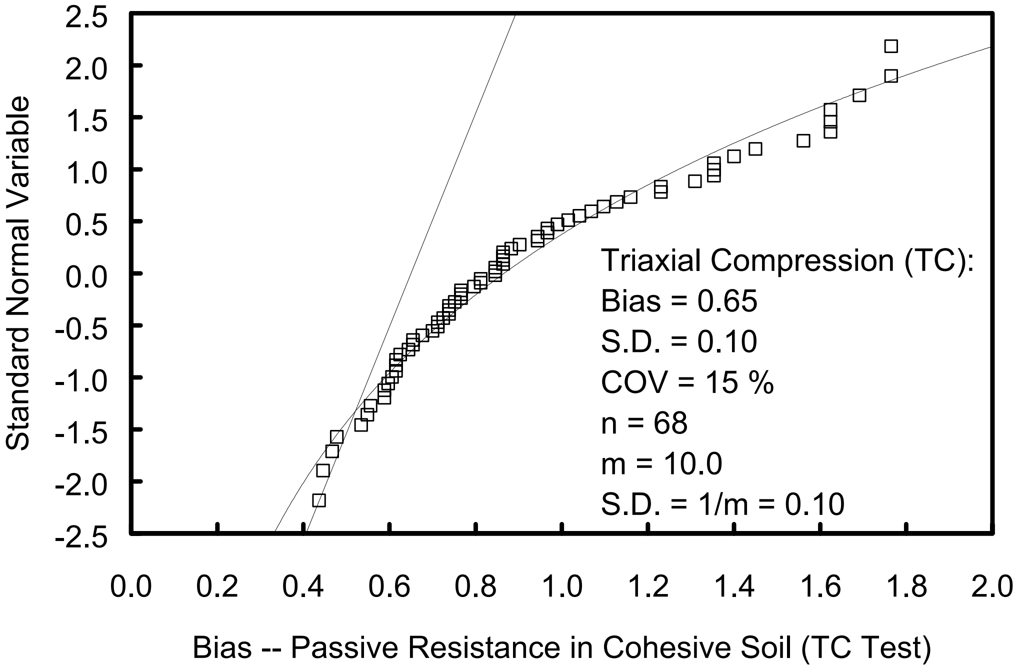

Figure 11. Development of Equivalent Normal Distribution Near Design Point for Passive Resistance of Rigid Drilled Shaft Embedded in Cohesive Soil (TC)

The variability of γ_s and $\bar{\phi}$ were addressed in Section 2.1. First-order estimates of COV_{Kp} can be calculated by considering the ranges of COV_{ϕ} reported in Section 2.1 and by assuming that the relationship between K_p and $\bar{\phi}$ is linear at a particular reference (mean) $\bar{\phi}$. Such first order estimates of COV_{Kp} at reference values of $\bar{\phi}$ = 30°, 35°, and 40° are provided in Table 10. In reality, the relationship between K_p and $\bar{\phi}$ is non-linear. Therefore, the first order estimates of COV_{Kp} in Table 10 were not used directly in the reliability analyses. Rather, values of $\bar{\phi}$ were generated randomly, and the corresponding value of Rankine K_p for each randomly generated value of $\bar{\phi}$ was subsequently calculated.

Table 10. General Ranges of COV_{ϕ} and COV_{Kp} (First Order Analysis) for Cohesionless Soils

Parameter	Reference $\bar{\phi}$ (deg)	SPT N Value	CPT q_c	Laboratory Measurements (TC, DS)
Degree of Variability:		"high"	"medium"	"low"
COV_{ϕ} :		0.15 - 0.20	0.10 - 0.15	0.05 - 0.10
COV_{Kp}:	30	0.16 - 0.21	0.11 - 0.16	0.06 - 0.11
	35	0.20 - 0.25	0.14 - 0.20	0.07 - 0.14
	40	0.23 - 0.29	0.16 - 0.23	0.09 - 0.16

TC = triaxial compression
DS = direct shear

4 LIMIT STATE FUNCTIONS

The basic format of the limit state functions considered in this study is expressed as:

$$g = R - Q \tag{10}$$

where R is the resistance (load carrying capacity), and Q is load effect. However, both R and Q are expressed in terms of parameters such as load components, soil properties, and dimensions.

In the reliability analyses and calculation of reliability indices, all wall components (e.g., embedment depth D and anchor bond length L_b) are dimensioned according to particular load and resistance factors, γ and ϕ, respectively. In addition, the values of R and Q represent the nominal load and resistance, and are based upon the dimensions of the elements, which are sized according to these values of γ and ϕ.

Expressions for the limit state functions of anchor pullout resistance and passive resistance (embedment) are provided in the subsequent subsections.

4.1 Anchor Pullout Resistance

For anchor failure by pullout, the limit state function (g_{PO}) may be specified in terms of forces as follows:

$$g_{PO} = Q_a - T \tag{11}$$

where Q_a = pullout resisting force of straight ground anchor (Section 3.1) and T = anchor force along the longitudinal axis of the anchor.

4.4.1. Cohesionless Soils

In case of a single level anchored wall retaining cohesionless soil (GeoSyntec, 1999), the longitudinal anchor force (T) is defined as:

$$T = \frac{T_h}{\cos i_a} \tag{12}$$

where

$$T_h = \frac{(23h - 10h_1)h}{54(h - h_1)} K_a \gamma_s h \, S_H \tag{13}$$

i_a = anchor inclination angle (with respect to horizontal)

h, h_1 = dimensions shown on Figure 12.

4.4.2. Cohesive Soils

In case of a single level anchored wall retaining cohesive soil (GeoSyntec, 1999), the horizontal component of anchor force (T_h) is defined as:

$$T_h = \frac{(23h - 10h_1)h}{54(h - h_1)} 0.4 \gamma_s h \, S_H \tag{14}$$

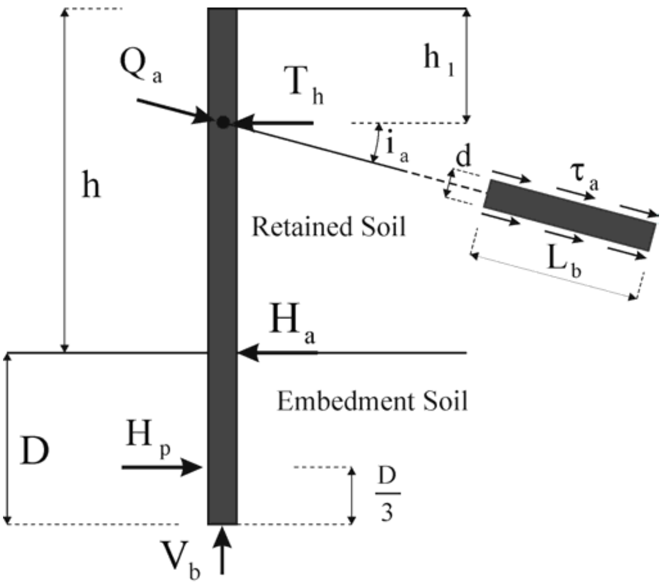

Figure 12. Geometry, and Forces Acting on Single Level Anchor Walls
Retaining Cohesionless or Cohesive Soils

4.2 Passive Resistance (Embedment)

4.2.1 Discrete Wall Elements (Anchored Walls)

In the case of passive resistance of the embedded portion of a discrete vertical anchored wall element, the limit state function (g_{PR}) may be specified in terms of resisting force (passive resistance) and the retained soil base reaction force (load) as follows:

$$g_{PR} = H_p - H_a \tag{15}$$

where H_p = passive pressure resisting force (Section 3.2 and Figure 12) and H_a = component of active earth pressure applied at the base of the exposed wall (Figure 12). H_a is defined for single level anchor walls retaining cohesionless and stiff cohesive soils (GeoSyntec, 1999) in the following subsections.

4.2.1.1 Walls Retaining Cohesionless Soils

For single level anchor walls retaining cohesionless soils, the component of active earth pressure applied at the base of the wall (H_a) can be expressed as:

$$H_a = \frac{13\left(h - 2h_1\right)h}{54\left(h - h_1\right)} K_a \gamma_s h\, S_H \tag{16}$$

4.2.1.2 Walls Retaining Stiff Cohesive Soils

For single level anchor walls retaining stiff cohesive soils, the component of active earth pressure applied at the base of the wall (H_a) can be expressed as:

$$H_a = \frac{13\left(h - 2h_1\right)h}{54\left(h - h_1\right)} 0.4 \gamma_s h\, S_H \tag{17}$$

4.2.2 Continuous Wall Elements (Cantilever Walls)

In the case of passive resistance of the embedded portion of a continuous cantilevered wall element, the limit state function (g_{PR}) depends upon the driving and resisting moments calculated about a depth D_o below grade at the front face of the wall:

$$g_{PR} = M_R - M_D \tag{18}$$

where M_R = sum of resisting moments (from passive earth pressure) and M_D = sum of driving moments (from active earth pressure and surcharge).

To account for discrepancies between actual and observed earth pressure distributions, the actual embedment depth D should be 20% greater than the depth D_o used to calculate the nominal active and passive earth pressures acting on a continuous flexible cantilevered retaining wall (Teng, 1962).

5 RELIABILITY ANALYSIS PROCEDURE

Reliability analysis procedures are presented in existing texts, such as Nowak and Collins (2000), and will not be described in detail here. In general, for each of the limit state functions presented in the previous section, the structure is safe if $g \geq 0$; otherwise, it fails. The probability of failure (P_F) is equal to:

$$P_F = \mathrm{Prob}(R - Q < 0) = \mathrm{Prob}(g < 0) \tag{19}$$

and the reliability index (β) is defined as a function of P_F as follows:

$$\beta = \Phi^{-1}(P_F) \tag{20}$$

where Φ^{-1} is the inverse standard normal distribution function.

In the current study, values of the safety margin (g) were generated using the Monte Carlo simulation technique. The resulting cumulative distribution function of g was plotted on the normal probability scale to determine β (i.e., the value of Φ^{-1} corresponding to g = 0 for the as-designed structure). The relationship between β and P_F is summarized in Table 11.

Table 11. Relationship Between Reliability Index (β) and Probability of Failure (P_F)

β	P_F
2.50	0.99×10^{-2}
3.00	1.15×10^{-3}
3.50	1.34×10^{-4}
4.00	1.56×10^{-5}
4.50	1.82×10^{-6}
5.00	2.12×10^{-7}
5.50	2.46×10^{-8}
1.96	10^{-1}
2.50	10^{-2}
3.03	10^{-3}
3.57	10^{-4}
4.10	10^{-5}
4.64	10^{-6}
5.17	10^{-7}

6 EXISTING RELIABILITY INDICES

This section summarizes the results of reliability analyses that have been performed to evaluate the degree of reliability of the existing geotechnical design of anchored and cantilevered flexible retaining structures, as expressed by the reliability index (β), using the LRFD Bridge Design Specifications (AASHTO, 2002). The applicable load and resistance factors (AASHTO, 2002) are provided in Table 12.

A significant change in the passive resistance factor was made in the Interim LRFD Bridge Design Specifications (AASHTO, 2002). The passive resistance factor is now equal to 1.0 (corresponding to a factor of safety of 1.5, presuming the existing γ_{EH} = 1.5 is used). The passive resistance factor prior to this change had been equal to 0.6. Therefore, values of ϕ = 1.0 and 0.6 for passive resistance are considered in the reliability analyses.

Table 12. Load and Resistance Factors for the Geotechnical Design of Flexible Anchored and Cantilevered Retaining Structures (AASHTO, 2002)

Load or Resistance	Symbol	Value
Horizontal Active Earth Pressure (EH)	γ_{EH}	1.50
Live Load Surcharge (LL)	γ_{LL}	1.75
Passive Resistance	ϕ	1.00 (changed from 0.60)
Anchor Pullout – Bonded in Cohesionless Soil	ϕ	0.65
Anchor Pullout – Bonded in Cohesive Soil	ϕ	0.70

6.1 Anchored Walls

These reliability analyses for flexible anchored retaining walls (with one row of anchors) consider earth pressure as the only load source. The nominal geotechnical and geometric parameters employed in the analyses are as follows:

γ_s = 18.9 kN/m^3 (120 pcf), $\bar{\phi}$ = 35°, τ_a = 100 kPa (14.5 psi)
h = 5.0 meters (16.4 feet)
S_H = 3.0 meters (9.8 feet)
h_1 = h/3 = 1.67 meters (5.5 feet)
d = anchor diameter = 0.15 meter (6 inches)
b = width of embedded element = 0.457 meter (18 inches)
i_a = 15°

6.1.1 Anchor Pullout

A summary of the baseline statistical and design parameters associated with the pullout resistance of ground anchors are provided in Table 13. The associated values of reliability index (β) are given in Tables 14 through 16 and Figures 13 through 15.

For anchor walls retaining cohesionless soil, the italicized values of β in the shaded cells of Tables 14 and 15 correspond to the typical λ_{AEP} = 0.77 (as described in Section 2.1.1). Values of β assuming λ_{AEP} = 1.00 are shown in these tables for comparison. For anchor walls retaining stiff cohesive soil, the italicized values of β in the shaded cells of Table 16 correspond to the typical λ_{AEP} = 0.75 (as described in Section 2.1.2). Values of β assuming λ_{AEP} = 0.50 and 1.00 are also shown in this table for comparison.

Table 13. Summary of Baseline Statistical and Design Parameters for Ground Anchor Pullout Resistance

Earth Load Source	Anchor Embedment Material	Anchor Resistance Statistics	Anchor Resistance Factor, ϕ	Earth Load Design Parameters & Statistics:	β Reported in Table No. (Figure No.)
Cohesionless Soil	Cohesive Soil	λ = 1.4 COV = 0.2 (Figure 7)	0.70	Model Bias: λ_{AEP} = 0.77 Parameters: γ_s = 120 pcf $COV_{\gamma s}$ = 0.1 $\lambda_{\gamma s}$ = 1.0 $\bar{\phi}$ = 35°, λ_ϕ = 1.0 COV_ϕ = 0.05 to 0.20	15 (14)
	Cohesionless Soil	λ = 1.2 COV = 0.2 (Figure 6)	0.65		14 (13)
Stiff Cohesive Soil	Cohesive Soil	λ = 1.4 COV = 0.2 (Figure 7)	0.70	Load Statistics: λ_{AEP} = 0.75 COV_{AEP} = 0.28 (Includes Lumped Model & Property Error)	16 (15)
	Cohesionless Soil	λ = 1.2 COV = 0.2 (Figure 6)	0.65		16 (15)

Table 14. Existing β for the Pullout Resistance of Ground Anchors Bonded in Cohesionless Soil for Walls Retaining Cohesionless Soil

Variation in $\bar{\phi}$ for Retained Soil	Resistance Statistics (Figure 6): λ = 1.20, COV = 0.20	
	Earth Load Bias $\lambda_{AEP} = 0.77$	Earth Load Bias $\lambda_{AEP} = 1.00$
	Earth Load Factor: $\gamma_{EH} = 1.50$ Anchor Pullout Resistance Factor: $\phi = 0.65$	
	Reliability Index, β	
$COV_{\phi} = 0.05$	*3.57*	3.15
$COV_{\phi} = 0.10$	*3.50*	3.02
$COV_{\phi} = 0.15$	*3.37*	2.84
$COV_{\phi} = 0.20$	*3.16*	2.59

Note: Use of lower bound values of τ_a (i.e., minimum boldfaced values shown in Table 5) for ground anchor design is presumed

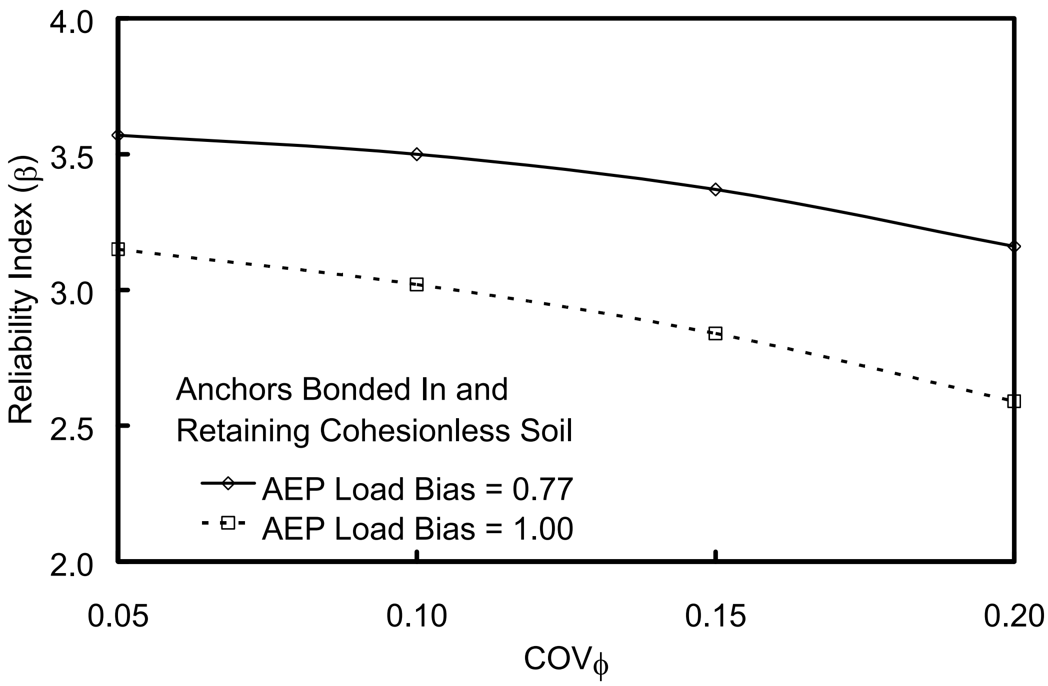

Figure 13. Sensitivity of β to COV_{ϕ} and λ_{AEP} for the Pullout Resistance Limit State of Anchors Bonded In and Retaining Cohesionless Soil

Table 15. Existing β for the Pullout Resistance of Ground Anchors Bonded in Cohesive Soil for Walls Retaining Cohesionless Soil

Variation in $\bar{\phi}$ for Retained Soil	Resistance Statistics (Figure 7): λ = 1.40, COV = 0.20	
	Earth Load Bias λ_{AEP} *= 0.77*	Earth Load Bias λ_{AEP} = 1.00
	Earth Load Factor: γ_{EH} = 1.50 Anchor Pullout Resistance Factor: ϕ = 0.70	
	Reliability Index, β	
COV_ϕ = 0.05	*3.70*	3.28
COV_ϕ = 0.10	*3.64*	3.18
COV_ϕ = 0.15	*3.52*	3.00
COV_ϕ = 0.20	*3.30*	2.78

Note: Use of lower bound values of τ_a (i.e., minimum boldfaced values shown in Table 6) for ground anchor design is presumed

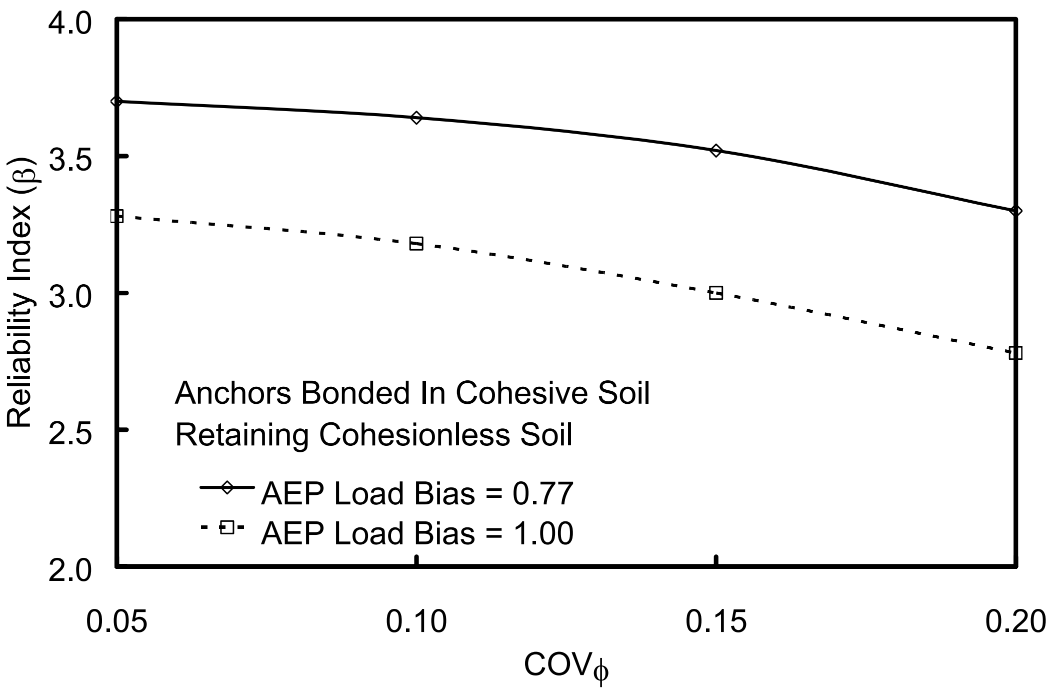

Figure 14. Sensitivity of β to COV_ϕ and λ_{AEP} for the Pullout Resistance Limit State of Anchors Bonded In Cohesive Soil and Retaining Cohesionless Soil

Table 16. Existing β for Pullout Resistance of Ground Anchors for Walls Retaining Stiff Cohesive Soil

Anchor Embedment Soil and Resistance Factor	Anchor Pullout Resistance Statistics (Figures 6 & 7)	Earth Load Bias, λ_{AEP}		
		1.00	*0.75*	0.50
		Reliability Index, β		
Cohesionless (ϕ = 0.65)	λ = 1.20 COV = 0.20	2.85	*3.40*	3.90
Cohesive (ϕ = 0.70)	λ = 1.40 COV = 0.20	3.03	*3.55*	4.00

Notes: Earth load COV_{AEP} = 0.28; Active Earth Load Factor γ_{EH} = 1.50; Use of lower bound values of τ_a (i.e., minimum boldfaced values shown in Tables 5 and 6) for ground anchor design is presumed

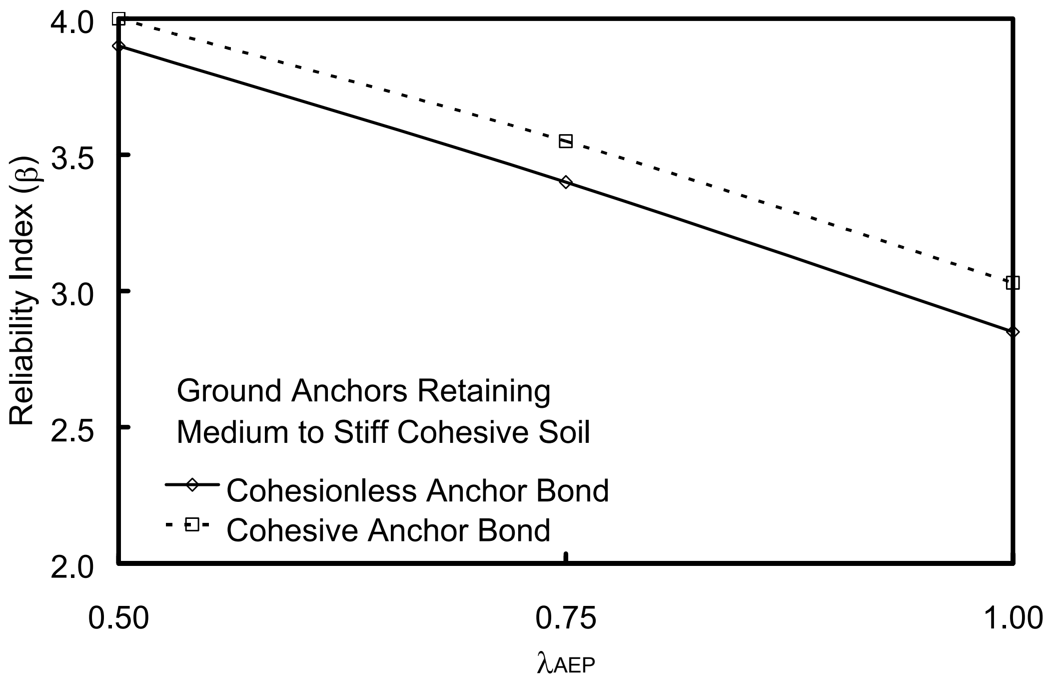

Figure 15. Sensitivity of β to λ_{AEP} for the Pullout Resistance Limit State of Ground Anchors Bonded In Cohesionless and Cohesive Soils and Retaining Stiff Cohesive Soil

The reliability indices using baseline statistics, as reported in the shaded cells in Tables 14 through 16, range from approximately 3.2 to 3.7 with an average of approximately 3.5, which corresponds to the target reliability index for superstructure design in the existing LRFD Bridge Design Specifications (AASHTO, 2002). These values of β for the anchor pullout limit state are valid for the existing load and resistance factors (AASHTO, 2002) *only if the minimum values of ultimate soil-grout bond stress (τ_a) reported in Tables 5 and 6 are employed in preliminary design*. This assumption is consistent with the LRFD Bridge Design Specifications (AASHTO, 2002) in that the Commentary now cites only the minimum (boldfaced) values of τ_a reported in Tables 5 and 6. However, higher values are permitted by the code. In addition, the Commentary recognizes that the use of these minimum values of τ_a can be conservative.

The reliability analyses presented in this section correspond to a "truly presumptive" ground anchor design basis (i.e., instances in which no prior load testing or experience is available before anchor installation, and no verification or proof testing is performed during anchor installation). Clearly, the degree of reliability at the preliminary design stage can be significantly degraded if higher values of τ_a are adopted without pre-production load tests or previous experience with similar soils and construction techniques. However, such experience or engineering judgment resulting in the use of higher values of τ_a at the preliminary design stage is rather difficult, if not impossible, to quantify in a statistical manner. In reality, existing geotechnical practice requires that each production ground anchor be proof-tested to a load typically equaling or exceeding 133% of the unfactored design load. Unfortunately, the ultimate capacity (ultimate limit state) of a ground anchor typically cannot be inferred from a proof test. If the proof test load is adopted as the ultimate capacity, reliability analyses can be performed assuming that the resistance (soil-grout bond) is deterministic (λ = 1.0, COV = 0.00). However, the resulting reliability indices would only represent a lower bound to β.

6.1.2 Passive Resistance

A summary of the baseline statistical and design parameters associated with the passive resistance of discrete vertical anchor wall elements are provided in Table 17. The associated values of reliability index (β) are given in Tables 18 through 21 and Figures 16 through 18.

Table 17. Summary of Baseline Design Parameters and Statistics for the Passive Resistance of Discrete Vertical Anchor Wall Elements

<table>
<tr><th>Earth Load Source</th><th>Passive Embedment Material</th><th>Passive Resistance Statistics</th><th>Passive Resistance Factors Considered, ϕ</th><th>Earth Load Design Parameters & Statistics:</th><th>β Reported in Table No. (Figure No.)</th></tr>
<tr><td rowspan="3">Cohesionless Soil</td><td>Cohesionless Soil ($\bar{\phi}_{tc}$ Prediction)</td><td>λ = 1.05
COV = 0.16
(Figure 9)</td><td rowspan="3">0.6 & 1.0</td><td rowspan="3"><u>Model Bias:</u>
λ_{AEP} = 0.77
<u>Parameters:</u>
γ_s = 120 pcf
$COV_{\gamma s}$ = 0.1
$\lambda_{\gamma s}$ = 1.0
$\bar{\phi}$ = 35°,
λ_{ϕ} = 1.0
COV_{ϕ} = 0.05 to 0.20</td><td>18 (16)</td></tr>
<tr><td>Cohesive Soil (TE Prediction)</td><td>λ = 1.60
COV = 0.15
(Figure 10)</td><td>19 (17)</td></tr>
<tr><td>Cohesive Soil (TC Prediction)</td><td>λ = 0.65
COV = 0.15
(Figure 11)</td><td>20 (18)</td></tr>
<tr><td rowspan="3">Stiff Cohesive Soil</td><td>Cohesive Soil (TE Prediction)</td><td>λ = 1.60
COV = 0.15
(Figure 10)</td><td rowspan="3">0.6 & 1.0</td><td rowspan="3"><u>Load Statistics:</u>
λ_{AEP} = 0.75
COV_{AEP} = 0.28
(Includes Lumped Model & Property Error)</td><td rowspan="3">21</td></tr>
<tr><td>Cohesive Soil (TC Prediction)</td><td>λ = 0.65
COV = 0.15
(Figure 11)</td></tr>
<tr><td>Cohesionless Soil ($\bar{\phi}_{tc}$ Prediction)</td><td>λ = 1.05
COV = 0.16
(Figure 9)</td></tr>
</table>

Table 18. Existing β for Passive Resistance of Discrete Vertical Anchor Wall Elements Embedded in Cohesionless Soil (TC Prediction) for Walls Retaining Cohesionless Soil

Variation in $\bar{\phi}$ for Retained Soil	Resistance Statistics, TC Prediction (Figure 9): $\lambda = 1.05$, COV = 0.16	
	Earth Load Bias: $\lambda_{AEP} = 0.77$	
	Earth Load Factor: $\gamma_{EH} = 1.50$	
	Passive Resistance Factor ϕ	
	1.0	0.6
	Reliability Index, β	
$COV_{\phi} = 0.05$	3.00	4.35
$COV_{\phi} = 0.10$	2.75	4.25
$COV_{\phi} = 0.15$	2.40	3.95
$COV_{\phi} = 0.20$	2.10	3.55

Note: For earth load evaluation, $COV_{\gamma s} = 0.10$, $\lambda_{\gamma s} = 1.0$.

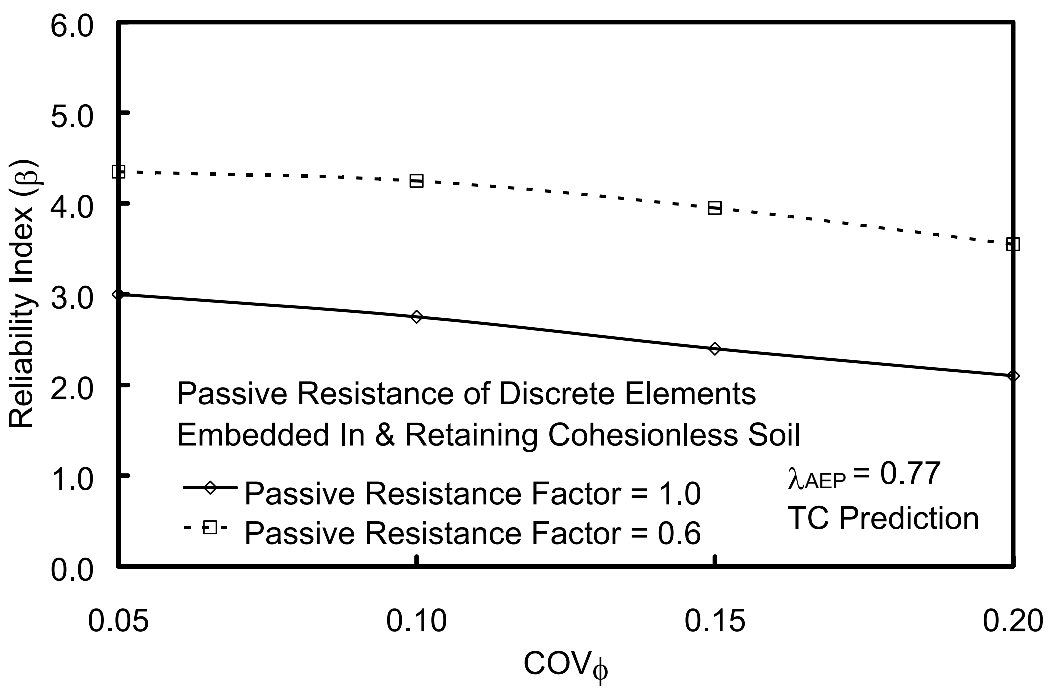

Figure 16. Sensitivity of β to Resistance Factor and COV_{ϕ} for the Passive Resistance Limit State of Discrete Elements Embedded In and Retaining Cohesionless Soil ($\lambda_{AEP} = 0.77$, TC Data used in Passive Resistance Prediction)

Table 19. Existing β for Passive Resistance of Discrete Vertical Anchor Wall Elements Embedded in Cohesive Soil (TE Prediction) for Walls Retaining Cohesionless Soil

Variation in $\bar{\phi}$ for Retained Soil	Resistance Statistics, TE Prediction (Figure 10): $\lambda = 1.60$, COV = 0.15	
	Earth Load Bias: $\lambda_{AEP} = 0.77$	
	Earth Load Factor: $\gamma_{EH} = 1.50$	
	Passive Resistance Factor ϕ	
	1.0	0.6
	Reliability Index, β	
$COV_{\phi} = 0.05$	4.50	5.35
$COV_{\phi} = 0.10$	4.30	5.25
$COV_{\phi} = 0.15$	3.95	5.10
$COV_{\phi} = 0.20$	3.45	4.90

Note: For earth load evaluation, $COV_{\gamma s} = 0.10$, $\lambda_{\gamma s} = 1.0$.

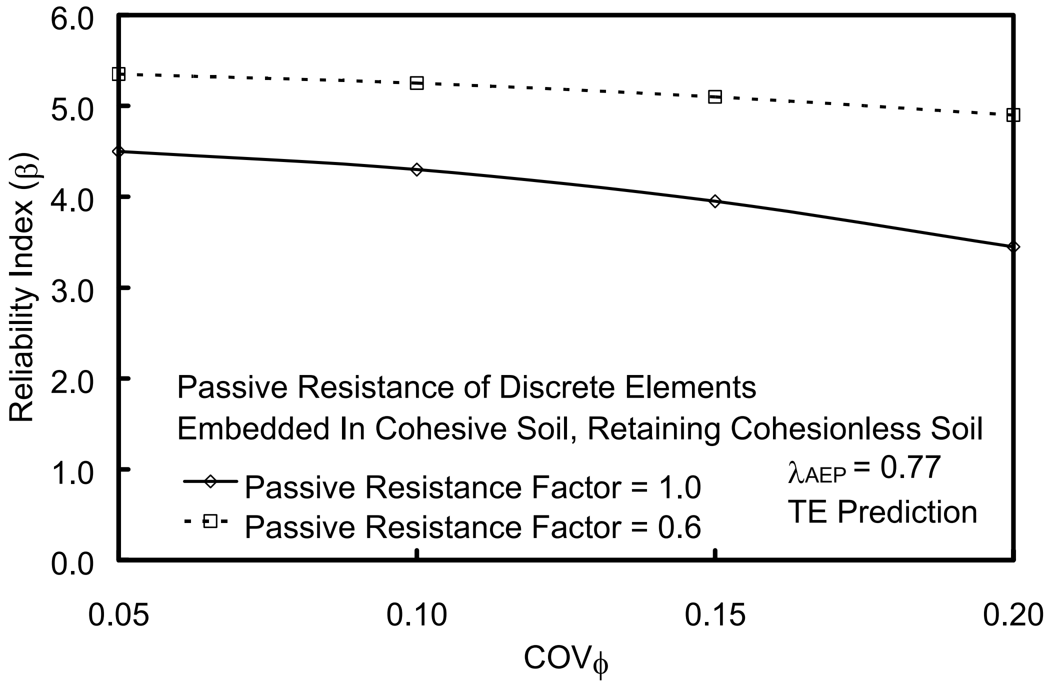

Figure 17. Sensitivity of β to Resistance Factor and COV_{ϕ} for the Passive Resistance Limit State of Discrete Elements Embedded In Cohesive Soil, Retaining Cohesionless Soil ($\lambda_{AEP} = 0.77$, TE Data used in Passive Resistance Prediction)

Table 20. Existing β for Passive Resistance of Discrete Vertical Anchor Wall Elements Embedded in Cohesive Soil (TC Prediction) for Walls Retaining Cohesionless Soil

Variation in $\bar{\phi}$ for Retained Soil	Resistance Statistics, TC Prediction (Figure 11): λ = 0.65, COV = 0.15	
	Load Bias: λ_{AEP} = 0.77	
	Earth Load Factor: γ_{EH} = 1.50	
	Resistance Factor ϕ	
	1.0	0.6
	Reliability Index, β	
COV_{ϕ} = 0.05	1.35	3.30
COV_{ϕ} = 0.10	1.15	3.00
COV_{ϕ} = 0.15	0.95	2.65
COV_{ϕ} = 0.20	0.80	2.25

Note: For earth load evaluation, $COV_{\gamma s}$ = 0.10, $\lambda_{\gamma s}$ = 1.0.

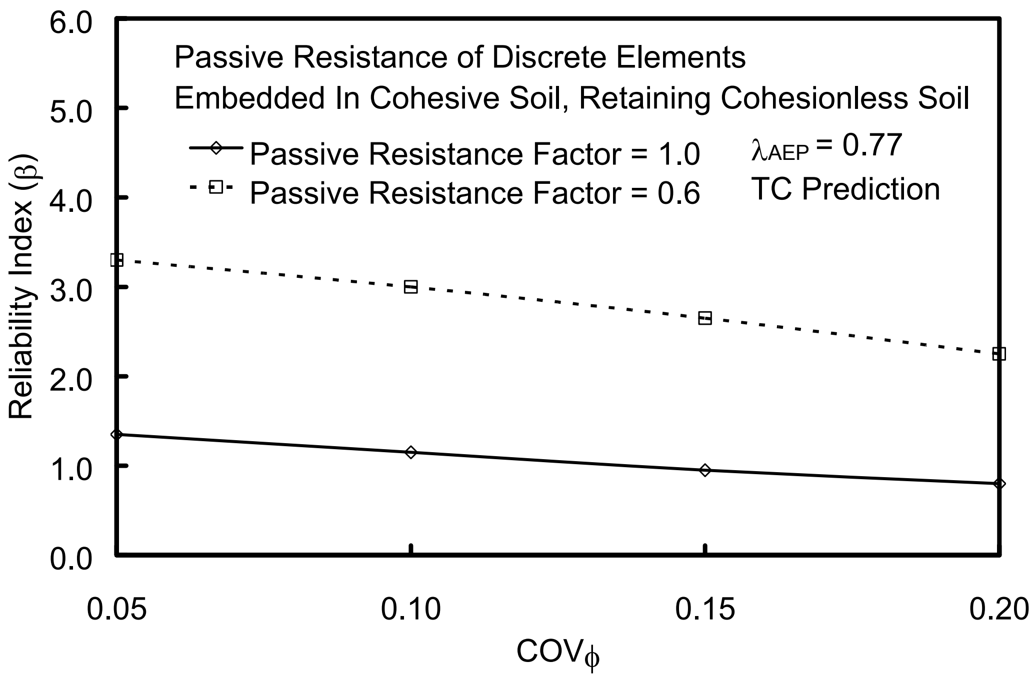

Figure 18. Sensitivity of β to Resistance Factor and COV_{ϕ} for the Passive Resistance Limit State of Discrete Elements Embedded In Cohesive Soil, Retaining Cohesionless Soil (λ_{AEP} = 0.77, TC Data used in Passive Resistance Prediction)

Table 21. Existing β for Passive Resistance of Discrete Vertical Anchor Wall Elements Embedded in Cohesive and Cohesionless Soils for Walls Retaining
Stiff Cohesive Soil

Passive Embedment Material	Passive Resistance Statistics	Passive Resistance Factor, ϕ	Load Bias, λ_{AEP}		
			1.00	*0.75*	0.50
			Reliability Index, β		
Cohesive	s_u(TE) Prediction: $\lambda = 1.60$ COV = 0.15 (Figure 10)	1.0	3.16	*4.00*	5.10
		0.6	4.60	*5.20*	5.80
	s_u(TC) Prediction: $\lambda = 0.65$ COV = 0.15 (Figure 11)	1.0	0.00	*0.97*	2.44
		0.6	1.70	*2.73*	4.00
Cohesionless	$\bar{\phi}_{tc}$ Prediction: $\lambda = 1.05$ COV = 0.16 (Figure 9)	1.0	1.57	*2.53*	3.75
		0.6	3.23	*4.05*	4.90

Note: Earth Load Factor $\gamma_{EH} = 1.50$; $COV_{AEP} = 0.28$

For anchor walls retaining cohesionless soil, the values of β in Tables 18 through 20 correspond to the typical $\lambda_{AEP} = 0.77$ (as described in Section 2.1.1). For anchor walls retaining stiff cohesive soil, the italicized values of β in the shaded cells of Table 21 correspond to the typical $\lambda_{AEP} = 0.75$ (as described in Section 2.1.2). Values of β assuming $\lambda_{AEP} = 0.50$ and 1.00 are also shown in this table for comparison.

For discrete wall elements embedded in cohesionless soil, the reliability indices for passive resistance (with resistance factor $\phi = 1.0$), as reported in Tables 18 through 20, range from approximately 2.1 to 3.0 with an average β of approximately 2.5. However, for $\phi = 0.6$, the reliability indices range from approximately 3.6 to 4.4 with an average β of approximately 4.0. Therefore, it is expected that the use of an intermediate $\phi = 0.8$, in conjunction with $\gamma_{EH} = 1.50$, would most likely yield an average β of approximately 3.25, which is rather close to the target reliability index of 3.5 for superstructure design in the existing LRFD Bridge Design Specifications (AASHTO, 2002). These findings are based on the assumption that the triaxial compression effective stress friction angle ($\bar{\phi}_{tc}$) is used to estimate the passive resistance from the Broms procedure.

An evaluation of the reliability associated with the passive resistance of discrete wall elements embedded in cohesive soil is complicated by the fact that different types of laboratory tests conducted on the same soil specimen can yield significantly different values of undrained shear strength (s_u). The data in Tables 19 through 21 indicate that if triaxial extension (TE) values of s_u (i.e., CK_oUE triaxial) are employed in the passive resistance prediction, the use of a resistance factor $\phi = 1.0$ would most likely yield values of β ranging from approximately 3.4 to 4.5, with an average value of approximately 4.0. However, if the same resistance factor of 1.0 is used in conjunction with triaxial compression (TC) values of s_u (i.e., CIUC triaxial), the resulting values of β are in the range of approximately 0.8 to 1.4, with an average value of approximately 1.0. Clearly, these lower values of β using TC strength data should be deemed unacceptable for conventional design practice. It should be noted that if a resistance factor of 0.6 is used in conjunction with triaxial compression (TC) values of s_u, the resulting values of β for passive resistance are in the range of approximately 2.2 to 3.3, with an average value of approximately 2.8. These values of β are still rather low compared to the target β of 3.5 for superstructure design.

To make the design of discrete wall elements embedded in cohesive soil somewhat consistent with superstructure design, it is recommended that a passive resistance factor $\phi = 1.0$ be used in conjunction with values of s_u obtained from a triaxial extension (TE) laboratory test. If such testing is not available, equivalent s_u(TE) data can be developed from the relationships among different laboratory test types proposed by Kulhawy and Mayne (1990) and Chen and Kulhawy (1994). Since the SPT is highly unreliable in cohesive soils, its use in developing values of s_u for the purpose of estimating passive resistance should be discouraged.

6.2 Flexible Cantilever Walls

These reliability analyses for flexible cantilever retaining walls consider both earth pressure and live load surcharge (2-ft of earth) as load sources. While a load factor γ_{LL} = 1.75 was applied to the live load earth surcharge, this load was assumed to be deterministic in nature. (Data have shown that the Boussinesq distribution for flexible walls is rather conservative).

The nominal geotechnical and geometric parameters employed in the analyses are as follows:

$\gamma_s = 18.9$ kN/m^3 (120 pcf)
H = 3.0 meters (9.8 feet)
$\bar{\phi}$, λ_ϕ → *Variable*

The degree of redundancy for flexible cantilever retaining walls is less than that of flexible anchored walls. While anchor walls obtain geotechnical resistance from two general sources (prestressed ground anchors and passive resistance), flexible cantilever walls rely solely on passive resistance. On this basis, it is assumed that the design engineer will be more conservative in the selection of the nominal design parameters, most notably the value of $\bar{\phi}$, in the design of cantilever walls compared to anchor walls. Recognizing this, the following three scenarios are considered for reliability analysis, in order of increased conservatism:

- A <u>mean value</u> of $\bar{\phi}_{tc}$ = 35° is obtained either from lab tests or empirical correlation with SPT or CPT data, and is used directly in design (λ_ϕ = 35°/35° = 1.0)
- <u>"Slightly Conservative"</u>: A mean value of $\bar{\phi}_{tc}$ = 37° is obtained either from lab tests or empirical correlation with SPT or CPT data, but a value of $\bar{\phi}_{tc}$ = 35° is used in design (λ_ϕ = 37°/35° = 1.06)
- <u>"More Conservative"</u>: A mean value of $\bar{\phi}_{tc}$ = 35° is obtained either from lab tests or empirical correlation with SPT or CPT data, but a value of $\bar{\phi}_{tc}$ = 31° is used in design (λ_ϕ = 35°/31° = 1.13).

As addressed in Sections 2.2, 3.2.2, and 4.4.4, the reliability analyses for continuous flexible cantilever retaining walls are based on the Rankine theory of active and passive earth pressure. In neglecting wall friction, the calculated earth load is on the high side, and the passive earth pressure is on the low side. In addition, triaxial compression (TC) friction angle values were assumed in the calculation of both load and resistance (i.e., K_a and K_p). While K_a is best calculated using $\bar{\phi}_{tc}$, $\bar{\phi}_{te}$ is more representative of the passive earth pressure condition. Since $\bar{\phi}_{te} \approx 1.12\ \bar{\phi}_{tc}$, on average, the use of $\bar{\phi}_{tc}$ to calculate K_p, as assumed in these analyses, is conservative. Therefore, the application of these assumptions in the reliability analyses result in calculated reliability indices which are most likely lower than what would be expected in the field.

A summary of the baseline statistical and design parameters associated with the passive resistance of continuous flexible cantilever retaining walls are provided in Table 22. The associated values of reliability index (β) are given in Tables 23 through 25 and Figures 19 through 21. An active earth load factor (γ_{EH}) of 1.50 is used in all of the analyses. Passive resistance factors of 0.6 and 1.0 are also considered.

Table 22. Summary of Baseline Design and Statistical Parameters for the Passive Resistance of Continuous Flexible Cantilever Walls in Cohesionless Soil

Strength Parameter Selection	Passive Resistance Statistics	Passive Resistance Factors Considered, ϕ	Earth Load & Resistance Parameters & Statistics:	β Reported in Table No (Figure No.)
<u>"Mean" Evaluation:</u> λ_{ϕ} = 35°/35° = 1.0	Lumped effect of Strength Parameter Selection, $COV_{\gamma s}$, and COV_{ϕ}	0.6 & 1.0	<u>Load and Resistance Model Bias:</u> λ = 1.00 <u>Parameters:</u> γ_s = 120 pcf $COV_{\gamma s}$ = 0.1 $\lambda_{\gamma s}$ = 1.0 $\bar{\phi}$ &, λ_{ϕ} → *Variable (See Strength Parameter Evaluation Column)* COV_{ϕ} = 0.05 to 0.20	23 (19)
<u>"Slightly Conservative":</u> λ_{ϕ} = 37°/35° = 1.06				24 (20)
<u>"More Conservative":</u> λ_{ϕ} = 35°/31° = 1.13				25 (21)

A visual evaluation of Figures 19 through 21 indicates that β is highly sensitive to COV_{ϕ}. An evaluation of the mean strength parameter condition (Table 23, Figure 19) indicates that if COV_{ϕ} is between 0.05 and 0.10 (consistent with laboratory evaluations of $\bar{\phi}$), the use of a resistance factor ϕ = 1.0 results in β ranging from 1.85 to 3.25, while the use of ϕ = 0.6 results in β ranging from 3.9 to 6.0. On this basis, to achieve a mean target β of 3.5 consistent with superstructure design (AASHTO, 2002) an intermediate passive resistance factor of ϕ = 0.8 is recommended if *mean laboratory TC strength data* are employed in the prediction.

An analysis of the results presented in Table 23 (Figure 19) also indicates that for values of COV_{ϕ} in the range of 0.10 to 0.15 (consistent with mean estimates of $\bar{\phi}$ obtained from CPT data), the resulting values of β range from 2.65 to 3.90 if a passive resistance factor of 0.6 is used. Therefore, the use of ϕ = 0.6 is recommended to achieve a mean β of approximately 3.3 (≈3.5) if mean CPT strength data are used to estimate $\bar{\phi}$.

An evaluation of the "more conservative" strength evaluation given in Table 25 (Figure 21) demonstrates that for values of COV_{ϕ} ranging from 0.15 to 0.20 (consistent with estimates of $\bar{\phi}$ obtained from SPT data), the resulting values of β range from 2.6 to 3.5 if a passive resistance factor of 0.6 is used. Therefore, the use of ϕ = 0.6 is recommended to achieve a mean β of approximately 3.0 if SPT strength data are used to estimate a conservative value of $\bar{\phi}$. From a practical standpoint, the mean value of $\bar{\phi}$ estimated from SPT N data should be multiplied by ≈ 0.9 before being used to estimate a nominal (unfactored) passive resistance.

Table 23. Existing β for Passive Resistance of Continuous Flexible Cantilever Walls Embedded in Cohesionless Soil and Retaining Cohesionless Soil, Using Mean Strength Parameters

Variation in $\bar{\phi}$ for Retained and Embedment Soil	$\lambda_{\phi} = 35°/35° = 1.00$	
	$\gamma_s = 120$ pcf, $\lambda_{\gamma s} = 1.0$, $COV_{\gamma s} = 0.10$	
	Load & Resistance Model Bias: 1.00	
	Earth Load Factor: $\gamma_{EH} = 1.50$	
	Passive Resistance Factor ϕ	
	1.00	0.60
	Reliability Index, β	
$COV_{\phi} = 0.05$	3.25	6.00
$COV_{\phi} = 0.10$	1.85	3.90
$COV_{\phi} = 0.15$	1.25	2.65
$COV_{\phi} = 0.20$	0.95	2.00

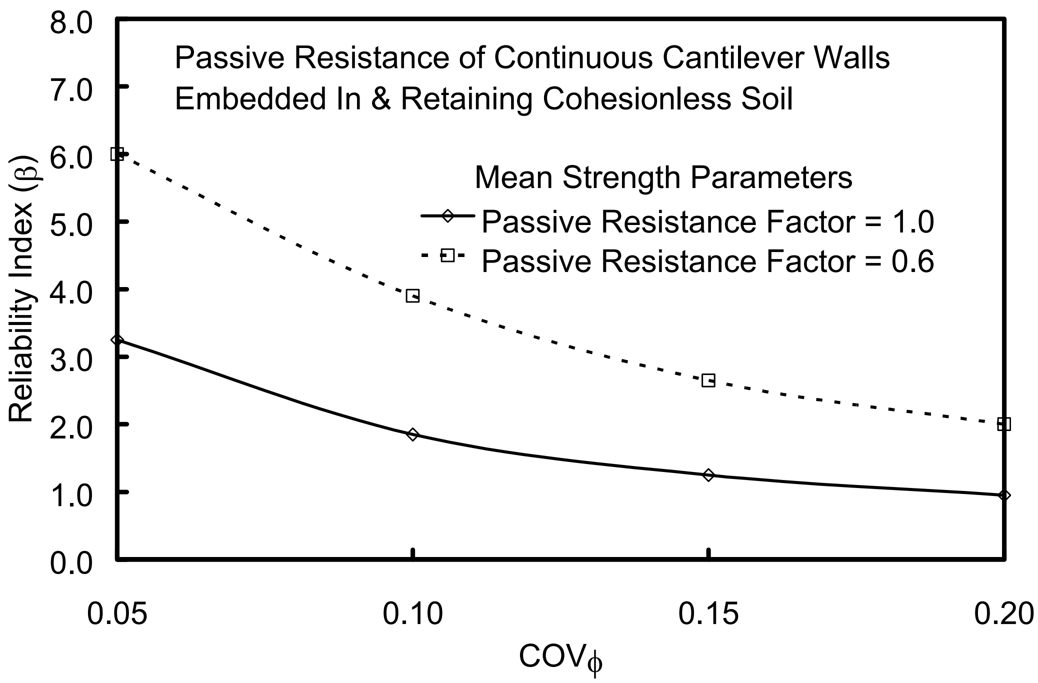

Figure 19. Sensitivity of β to Resistance Factor and COV_{ϕ} for the Passive Resistance Limit State of Continuous Cantilever Walls Embedded In and Retaining Cohesionless Soil (Using Mean Strength Parameters)

Table 24. Existing β for Passive Resistance of Continuous Flexible Cantilever Walls Embedded in Cohesionless Soil and Retaining Cohesionless Soil,
Using “Slightly Conservative” Strength Parameters

Variation in $\bar{\phi}$ for Retained and Embedment Soil	$\lambda_{\phi} = 37°/35° = 1.06$	
	$\gamma_s = 120$ pcf, $\lambda_{\gamma s} = 1.0$, $COV_{\gamma s} = 0.10$	
	Load & Resistance Model Bias: 1.00	
	Earth Load Factor: $\gamma_{EH} = 1.50$	
	Passive Resistance Factor ϕ	
	1.00	0.60
	Reliability Index, β	
$COV_{\phi} = 0.05$	4.30	7.00
$COV_{\phi} = 0.10$	2.40	4.40
$COV_{\phi} = 0.15$	1.65	2.90
$COV_{\phi} = 0.20$	1.20	2.20

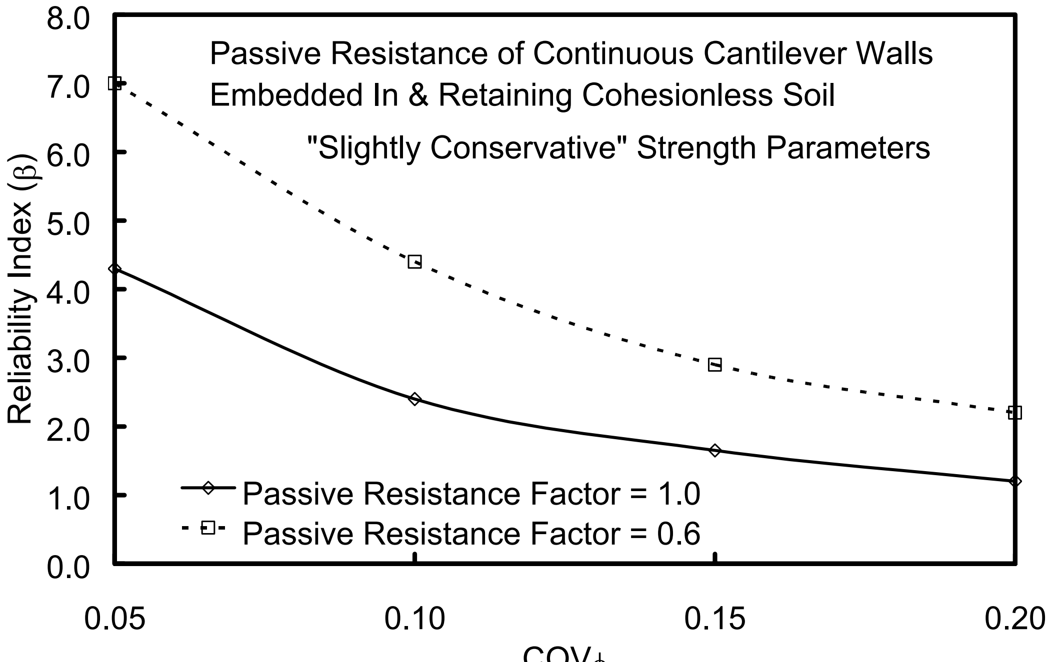

Figure 20. Sensitivity of β to Resistance Factor and COV_{ϕ} for the Passive Resistance Limit State of Continuous Cantilever Walls Embedded In and Retaining Cohesionless Soil
(Using “Slightly Conservative” Strength Parameters)

Table 25. Existing β for Passive Resistance of Continuous Flexible Cantilever Walls Embedded in Cohesionless Soil and Retaining Cohesionless Soil,
Using "More Conservative" Strength Parameters

Variation in $\bar{\phi}$ for Retained and Embedment Soil	$\lambda_\phi = 35°/31° = 1.13$	
	$\gamma_s = 120$ pcf, $\lambda_{\gamma s} = 1.0$, $COV_{\gamma s} = 0.10$	
	Load & Resistance Model Bias: 1.00	
	Earth Load Factor: $\gamma_{EH} = 1.50$	
	Passive Resistance Factor ϕ	
	1.00	0.60
	Reliability Index, β	
$COV_\phi = 0.05$	5.30	8.00
$COV_\phi = 0.10$	3.20	5.40
$COV_\phi = 0.15$	2.15	3.50
$COV_\phi = 0.20$	1.65	2.60

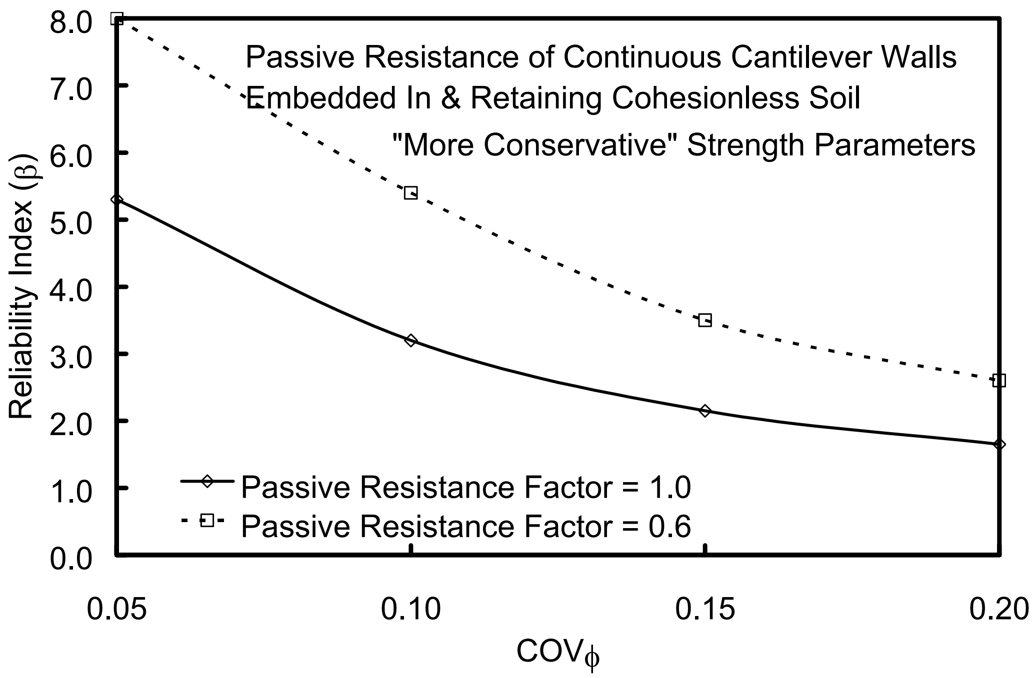

Figure 21. Sensitivity of β to Resistance Factor and COV_ϕ for the Passive Resistance Limit State of Continuous Cantilever Walls Embedded In and Retaining Cohesionless Soil
(Using "More Conservative" Strength Parameters)

7 SUMMARY AND PRELIMINARY RECOMMENDATIONS

This paper has provided a description of a reliability analysis procedure to evaluate the degree of reliability inherent in the existing geotechnical design of anchored and cantilevered flexible retaining structures, as expressed by the reliability index (β), using the LRFD Bridge Design Specifications (AASHTO, 2002). A review of the applicable design models was presented, and the statistical variations of both earth load and resistance were provided.

Trial geotechnical designs were subsequently performed for both anchored and cantilevered flexible retaining structures.

The key findings and preliminary recommendations presented in this paper are as follows:

Anchor Pullout Resistance

Using the existing earth load factor γ_{EH} = 1.50 and current resistance factors (as provided in Table 12), values of β for anchor pullout resistance ranging from 3.2 to 3.7, with an average of approximately 3.5 (which corresponds to the target reliability index for superstructure design in the existing LRFD Bridge Design Specifications), were observed. These values of β for the anchor pullout limit state are valid *only if the minimum values of ultimate soil-grout bond stress (τ_a) reported in Tables 5 and 6 are employed in preliminary design*. Higher values of τ_a are permitted by the AASHTO LRFD Bridge Design Specifications. However, quantification of the engineering judgment and experience involved in such a selection is not possible at this time. A more rigorous analysis of this limit state will require the development of resistance statistics based upon a rational design model, taking into account a number of other soil variables (such as soil plasticity and water content), in addition to installation technique.

Passive Resistance of Discrete Anchor Wall Elements

For discrete wall elements embedded in cohesionless soil, the existing range of reliability indices for passive resistance (with resistance factor ϕ = 1.0), is approximately 2.1 to 3.0 with an average β of approximately 2.5. However, it is expected that the use of a passive resistance factor ϕ = 0.8, in conjunction with the existing γ_{EH} = 1.50, would most likely yield an average β of approximately 3.25 for this limit state, which is rather close to the target reliability index of 3.5 for superstructure design. These findings are based on the assumption that the triaxial compression (TC) effective stress friction angle ($\overline{\phi}_{tc}$) is used to estimate the passive resistance.

For discrete wall elements embedded in cohesive soil, the degree of reliability is highly dependent upon the type of laboratory test used to measure the undrained shear strength (s_u). For instance, if triaxial extension (TE) values of s_u (i.e., CK_oUE triaxial) are employed in a passive resistance prediction, the use of the existing resistance factor ϕ = 1.0 would most likely yield values of β in the range of approximately 3.4 to 4.5, with an average value of approximately 4.0. However, if the same resistance factor of 1.0 is used in conjunction with triaxial compression (TC) values of s_u (i.e., CIUC triaxial), the resulting values of β are in the range of approximately 0.8 to 1.4, with an average value of approximately 1.0.

To make the design of discrete wall elements embedded in cohesive soil somewhat consistent with superstructure design, it is recommended that a passive resistance factor ϕ = 1.0 be used in conjunction with values of s_u obtained from a triaxial extension (TE) laboratory test. If such testing is not available, equivalent s_u(TE) data can be developed from the relationships among different laboratory test types proposed by Kulhawy and Mayne (1990) and Chen and Kulhawy (1994). Since the SPT is highly unreliable in cohesive soils, its use in developing values of s_u for the purpose of estimating passive resistance should be discouraged.

Passive Resistance of Continuous Cantilever Wall Elements in Cohesionless Soil

Using mean strength parameters, the range of β in existing design practice (γ_{EH} = 1.50, ϕ = 1.0) for the passive resistance of continuous cantilever wall elements ranges from 0.95 to 3.25 (from Table 23), and is highly dependent upon COV_ϕ. To achieve target values of β in the range of 3.0 to 3.5, the following recommendations are proposed:

Method of $\bar{\phi}_{tc}$ determination	Value of $\bar{\phi}$ adopted in design to evaluate K_a & K_p	Active Load Factor, γ_{EH}	Recommended Passive Resistance Factor, ϕ	Resulting Mean β
Laboratory Measurement	Mean $\bar{\phi}_{tc}$ Value	1.50	0.8	3.5
Correlation with CPT q_c	Mean $\bar{\phi}_{tc}$ Value	1.50	0.6	3.3
Correlation with SPT N Value	$\bar{\phi}_{no\,min\,al} = 0.9\ \bar{\phi}_{tc}$	1.50	0.6	3.0

8 DISCLAIMER

Any opinions, findings, conclusions, or recommendations expressed in this paper are strictly those of the authors and do not necessarily represent the views of the Transportation Research Board, the National Research Council, the American Association of State Highway and Transportation Officials, the Federal Highway Administration, or other participants in or sponsors of this work.

9 REFERENCES

AASHTO, (2002) *LRFD Bridge Design Specifications*. American Association of State Highway and Transportation Officials, Washington, D.C.

Armento, W.J. (1972) *Criteria for Lateral Pressures for Braced Cuts*. Proc. ASCE Specialty Conference on the Performance of Earth and Earth-Supported Structures, Purdue University. Vol. 1, Part 2, p1341-1366.

Barley, A.D., and McBarron, P.L. (1997) *Field Trials on Four High Capacity Removable Multiple Anchors Founded in Marine Sand Fill and in Completely Decomposed Granite.* Ground Anchorages and Anchored Structures, Proc. International Conference of the Institution of Civil Engineers, London, UK, p148-157.

Briske, R. and Pirlet, E. (1968) *Messungen uber die Beanspruchung des Baugrubenverbauces der Kolner U-Bahn.* Die Bautecknik, Vol. 45, p290-299.

Broms, B.B. (1964a) *Lateral Resistance of Piles in Cohesive Soils*. Journal of the Soil Mechanics and Foundations Division. Vol. 90, No. 2, p27-63, ASCE.

Broms, B.B. (1964b) *Lateral Resistance of Piles in Cohesionless Soils*. Journal of the Soil Mechanics and Foundations Division. Vol. 90, No. 3, p123-156, ASCE.

Bruce, D.A. (1998) *Test Results of Anchors in Clay Soil.* Personal Communication.

Chapman, K.R. (1970) *Performance of Braced Excavations in Washington, D.C.* M.S. Thesis, University of Illinois, Urbana.

Chen, Y.-J., and Kulhawy, F.H. (1994) *Case History Evaluation of the Behavior of Drilled Shafts Under Axial and Lateral Loading.* Report TR-104601. Electric Power Research Institute, Palo Alto, 392 p.

Duncan, J.M. (2000) *Factors of Safety and Reliability in Geotechnical Engineering*, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 126, No. 4, p307-316.

Flaate, K.S. (1966) *Stresses and Movements in Connection With Braced Cuts in Sand and Clay*. Ph.D Thesis, University of Illinois, Urbana.

GeoSyntec (1999) *Ground Anchors and Anchored Systems*. Geotechnical Engineering Circular No. 4, FHWA-SA-99-015, 281 p.

Hegazy, Y.A. (2003) *Reliability of Estimated Anchor Pullout Resistance*, Proc. of the Third International Conference on Grouting and Ground Treatment, New Orleans, p772-779, ASCE.

Jones, D.L. (1997) *Full Scale Trials for Straight Shafted and Underreamed Anchorages in Cemented Sands.* Ground Anchorages and Anchored Structures. Proc. International Conference of the Institution of Civil Engineers, London, UK, p140-147.

Klenner, C. (1941) *Versuche über die Verteilung des Erddruckes über die Wände ausgesteifter Baugruben. (Experiments with the Distribution of Earth Pressure on the Walls of Braced Cuts.)* Die Bautechnik, Vol. 19, p316-319.

Kulhawy, F. H. and Mayne, P. W. (1990) *Manual on Estimating Soil Properties for Foundation Design*. Report EL-6800. Electric Power Research Institute, Palo Alto, 360 p.

Lacasse, S. and Nadim, F. (1996) *Uncertainties in Characterizing Soil Properties.* Uncertainty in the Geologic Environment: From Theory to Practice (GSP No. 58). Vol. 1, p49-75, ASCE.

Larson, M.L., Willette, W.R, Hall, H.C, and Gnaedinger, J.P. (1972) *A Case Study of a Soil Anchor Tied-Back System.* Proc. ASCE Specialty Conference on the Performance of Earth and Earth-Supported Structures, Purdue University, Vol. 1, Part 2, p1341-1366.

Liao, H.J., Wu, K.W., and Shu, S.C. (1997) *Uplift Behavior of a Cone-Shape Anchor in Sand.* Ground Anchorages and Anchored Structures. Proc. International Conference of the Institution of Civil Engineers, London, UK, p401-410.

Manoliu, I., Dimitriu, D.V., Radulescu, N., and Dobrescu, G.H. (1985) *Load-Deformation Characteristics of Drilled Piers*. Proc. Eleventh International Conference on Soil Mechanics and Foundation Engineering. Vol. 3, San Francisco, p1553-1558.

Meyerhof, G.G. (1995) *Development of Geotechnical Limit State Design.* Canadian Geotechnical Journal, Vol. 32, No. 1, p128-136.

Nowak, A.N. (1999) *Calibration of LRFD Bridge Design Code*. NCHRP Report 368, Transportation Research Board, Washington, D.C.

Nowak, A.S. and Collins, K.R. (2000) *Reliability of Structures.* McGraw-Hill, New York.

O'Rourke, T.D. (1975) *A Study of Two Braced Excavations in Sand and Interbedded Stiff Clay.* Ph.D. Thesis, University of Illinois, Urbana, 255 p.

Ostermayer, H. (1975) *Construction, Carrying Behavior and Creep Characteristics of Ground Anchors*. Diaphragm Walls and Anchorages. Proc. Institution of Civil Engineers, London, UK, p141-151.

Phoon, K.-K., and Kulhawy, F.H. (1999) *Characterization of Geotechnical Variability.* Canadian Journal of Geotechnical Engineering, Vol. 36, No. 4, p612-624.

Phoon, K.-K., Kulhawy, F.H., and Grigoriu, M.D. (1995) *Reliability-Based Design of Foundations for Transmission Line Structures.* Report TR-105000. Electric Power Research Institute, Palo Alto.

PTI (1996) *Recommendations for Prestressed Rock and Soil Anchors*, Post-Tensioning Institute, Phoenix, 70 p.

Spilker, A. (1937) *Mitteilung uber die Messung der Kräfte in einer Baugrubenaussteifung. (Description of the Measurement Forces in a Strutted Excavation.)* Die Bautechnik, Vol. 15, p16-18.

Teng, W.C. (1962) *Foundation Design.* Prentice Hall, Englewood Cliffs, NJ, 466 p.

Terzaghi, K. and Peck, R.B. (1967) *Soil Mechanics in Engineering Practice*, *Second Edition*. Wiley, New York, 729 p.

Terzaghi, K., Peck, R.B, and Mesri, G. (1996) *Soil Mechanics in Engineering Practice, Third Edition.* Wiley, New York, 549 p.

TTC (1967) *Lateral Earth Pressure Studies on Strutted Excavations of the Toronto Subway Systems*, Toronto Transit Commission.

White, L. and Prentis, E.A. (1940) *Cofferdams*, Columbia University Press, New York, 273 p.

Woodland, A., Lomax, C., and Barley, A.D. (1997) *The Design, Construction and Performance of an Anchored Retaining Wall, Newcastle upon Tyne.* Ground Anchorages and Anchored Structures. Proc. International Conference of the Institution of Civil Engineers, London, UK, p308-317.