HARTCROWSER

Earth and Environmental Technologies

Hart Crowser, Inc.
1910 Fairview Avenue East
Seattle, Washington 98102-3699
Fax 206.328.5581
Tel 206.324.9530
www.hartcrowser.com

MEMORANDUM

DATE: August 13, 1998

TO: Michael Bailey, Hart Crowser, Inc.

FROM: Jamie Beaver and Allen Jones, Hart Crowser, Inc.

RE: Base Preparation Stability Analysis (Phase II)
J-4978-01

CC: Pete Douglass, Peter M. Douglass, Inc.

This memo presents the results of our search for soil strength parameters we intend to use in our stability analyses. This memo is focused towards evaluating stability of the Phase II embankment from a subgrade preparation standpoint. Stability analyses for design of the Phase II and III embankments will be performed under separate scopes of work.

Approach to Soil Strength Assignments/Stability Assessment

We will be analyzing the stability of the Phase II embankment using the program UTEXAS3 to provide base preparation recommendations. Our approach to assigning soil strengths will be to use relationships that have been established for soil strength in the geotechnical literature, while considering previous projects at similar sites in the Seattle area as a general guide (Attachment 1). The relationships presented on Figures 1 and 2 are strength parameters that have been correlated with standard penetration resistance (SPT) N-values in the literature, and will be used to establish the baseline values for the stability analysis.

In general, friction angles will be assigned to cohesionless soils (sands, sandy gravel, etc.). Both shear strengths and cohesion/friction angle values will be used for both plastic and non-plastic silt/clays. We will use shear strength (undrained) based on SPT N-value to assign the initial strengths to the fine-grained soil units or embankment fill zones. Friction angle and cohesion will also be used, primarily to test the sensitivity of the model to the strengths of the fine-grained soil units. Our experience has shown that cohesion is a difficult parameter to provide a representative value to without running the risk of being non-conservative. We will vary this parameter to assess the sensitivity of the results (discussed below), rather than attempting to assign a single value. Friction angle and cohesion will also be used to represent strength of silty to very silty materials within the embankment.

Soil Strength and Unit Weight Parameters

Shear Strength. As discussed above, shear strengths based on SPT N-value will be used for silt/clay soils, primarily those underlying the embankment. Figure 1 shows the recommended relationship that we will be using in the stability assessment.

Friction Angle. Figure 2 shows the relationships we will be using to assign strengths to individual cohesionless soil units based on average SPT N-value for the given layer. Information provided in Attachment 1 will be used in conjunction with the results of these relationships.

Cohesion. Cohesion will be varied from 0, 100, 250, 500, and 750 psf to the degree appropriate for both silty to very silty embankment fill and native silt/clay units underlying the embankment. The degree of slickensides of the native glacial soils reported in boring logs will be considered (i.e., the greater degree of slickensides, the lower the cohesion). Silty to very silty soils within the embankment will be assigned cohesion values at the lower end of this range, if appropriate.

Residual Friction Angle. This condition occurs when certain soils undergo strain deformation in excess of a characteristic yield strength which may occur within the embankment. This condition may presently exist within the slickensided finer-grained glacial units underlying the proposed embankment and/or may be mobilized during construction. Native glacial and/or interglacial silt/clay soils often exhibit strain softening behavior during shearing, which supports an approach of residual strength analysis. Some areas within the embankment fill may experience loss of strength due to excessive differential settlement. Residual friction angles will be used to test residual strengths for only the fine-grained silty to very silty and silt/clay soils in the embankment and the native silt/clay glacial soils. This parameter has been related to the Atterberg limits value of plasticity index in the literature and has been used to establish the relationship shown on Figure 3 for local soils. Attachment 2 provides sample results based on laboratory results from samples taken from the site.

Unit Weight. We used AGI Technologies results for dry unit weight reported on boring logs within the Phase II area to determine appropriate values for total unit weight. AGI Technologies used a Dames and Moore sampling device in explorations to obtain relatively undisturbed samples *in situ*.

We averaged reported dry unit weights for given soil types after incorporating the moisture content to obtain the recommended total unit weights shown in Table 1.

Sensitivity Analysis

Sensitivity of the stability model to the soil layer strength values chosen is an important consideration in the stability assessment. We will vary soil strengths to determine the sensitivity of the model to changes in a given strength parameter. Individual layers that exhibit significant changes in Factor of Safety with small changes of soil strength will be given special consideration.

Test Reasonable "Catastrophic" Modes of Failure

In addition to both circular and wedge-type failure surfaces through the embankment into the base preparation zone, the following conditions would be analyzed to qualitatively ascertain their impacts to embankment stability:

- "Drainage blanket failure" by adding groundwater mounding conditions to the stability cross section. Pore pressure coefficients will also be used to represent a condition in which groundwater flow through springs/seeps into the toe drain, or the drainage blanket below the embankment are cut off, causing pressurization.

- Fine-grained soil strengths to residual values within and underlying the embankment fill (see above).

Pore Pressure Considerations

Pore pressures may develop both within the native glacial soils underlying the embankment and the embankment fill itself. For significant pore pressures to develop in native glacial soil deposits underlying the embankment fill, the given soil unit must be fully saturated or the soil unit must compress/consolidate to the point of saturation. As this occurs, the load from the overlying soil will be transferred to the pore water and begin to represent a condition of instability. The groundwater conditions within the upper Recessional Soils and the Advance Outwash soils underlying the Glacial Till cap are likely directly related to recharge from precipitation. However, we will test the scenario discussed above in which groundwater becomes pressurized due to drainage blanket failure. Conditions such as static pore pressures within the embankment during construction, and dynamic pore pressures due to ground shaking from an earthquake will be analyzed in the embankment design phase.

Pore pressures can be modeled in UTEXAS3 using the r_u coefficient by Bishop and Morgenstern (1960) or by using a piezometric line to represent static and/or mounded groundwater levels. We will vary the pore pressure parameter to assess the sensitivity of the stability of the model to changes in pore pressure. Under dynamic conditions in the embankment fill, we will test the condition in which the cyclic pore pressures increase and/or surpass the soil weight to ascertain the implications to global stability. This could be a consideration for the base preparation, but will likely be a topic of discussion among the geotechnical team for interpretation, and ultimately a base preparation cost versus degree of risk decision for the owner.

497801\prepstability.doc

Attachments:
Table 1 - Recommended Total and Submerged Unit Weights for Stability Analyses
Figure 1 - Relationships for Undrained Shear Strength
Figure 2 - Relationships for Total Friction Angle
Figure 3 - Residual Friction Angle for Seattle Silts Based on Direct Shear Tests
Attachment 1 - Supporting Information for Strength Parameters
Attachment 2 - Sample Results for Residual Friction Angle and Supporting Laboratory Results

Third Runway Stability Analysis, J-4978-01

Table 1 -- Recommended Total and Submerged Unit Weights for Stability Analyses

SOIL	Average Total Unit Weight in pcf	Recommended Total Unit Weight in pcf	Recommended Unit Weight Below Groundwater Surface in pcf
SILT	119	**115**	**53**
Silty **SAND**	120	**120**	**58**
SAND	125	**125**	**63**
Sandy **SILT**	127	**120**	**58**
Gravelly **SAND**	133	**135**	**73**
Silty, Gravelly **SAND**	134	**135**	**73**
Fine to Medium **SAND**		**120**	**58**
Gravelly **SILT**		**125**	**63**

REFERENCE: In-place Density Values taken from January 22, 1998 report, prepared by AGI Technologies entitled "Geotechnical Design Recommendations - Phase 1 Embankment Construction"

Third Runway Stability Analysis, J-4978-01
Relationships for Undrained Shear Strength (Su)

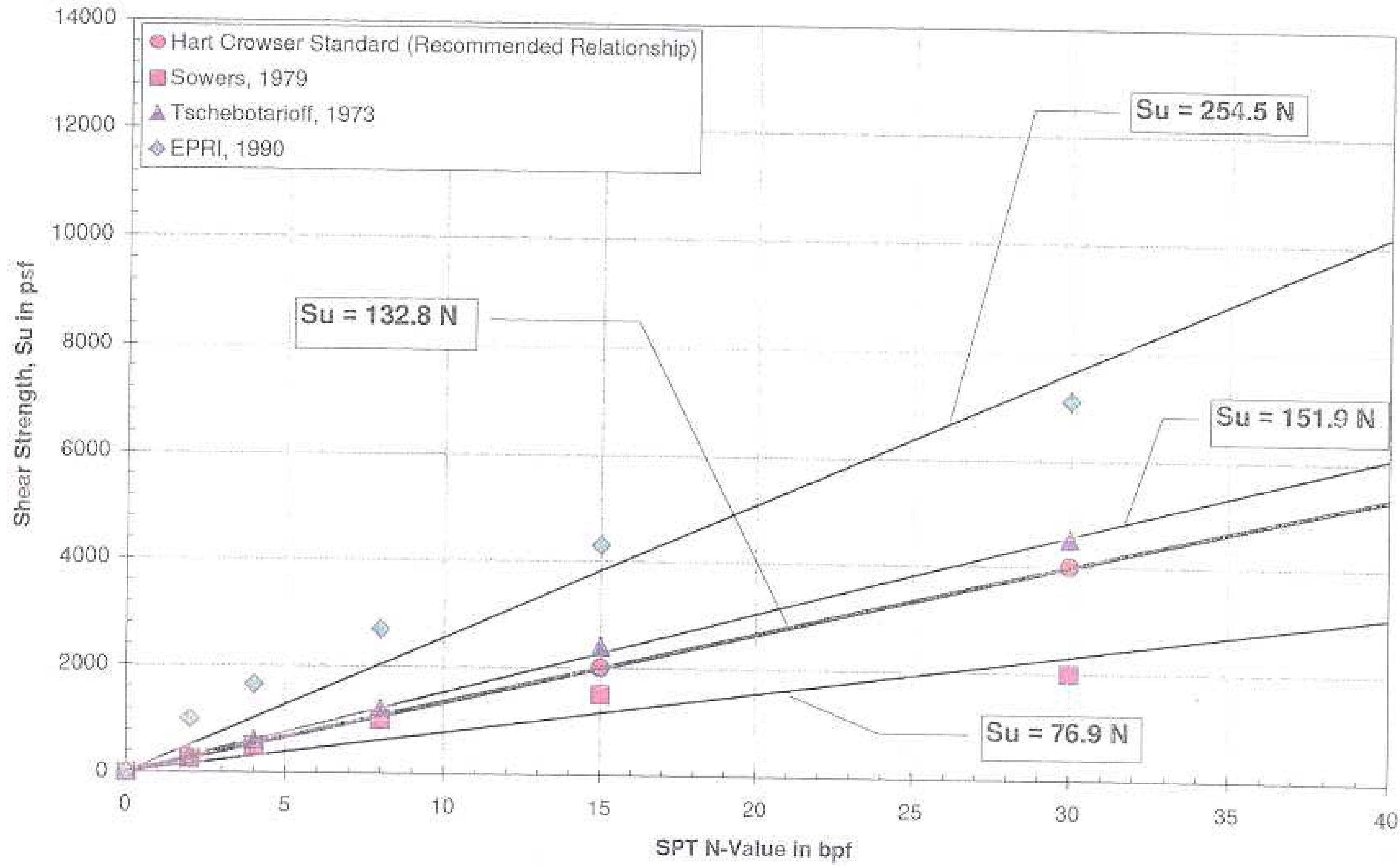


FIGURE 1

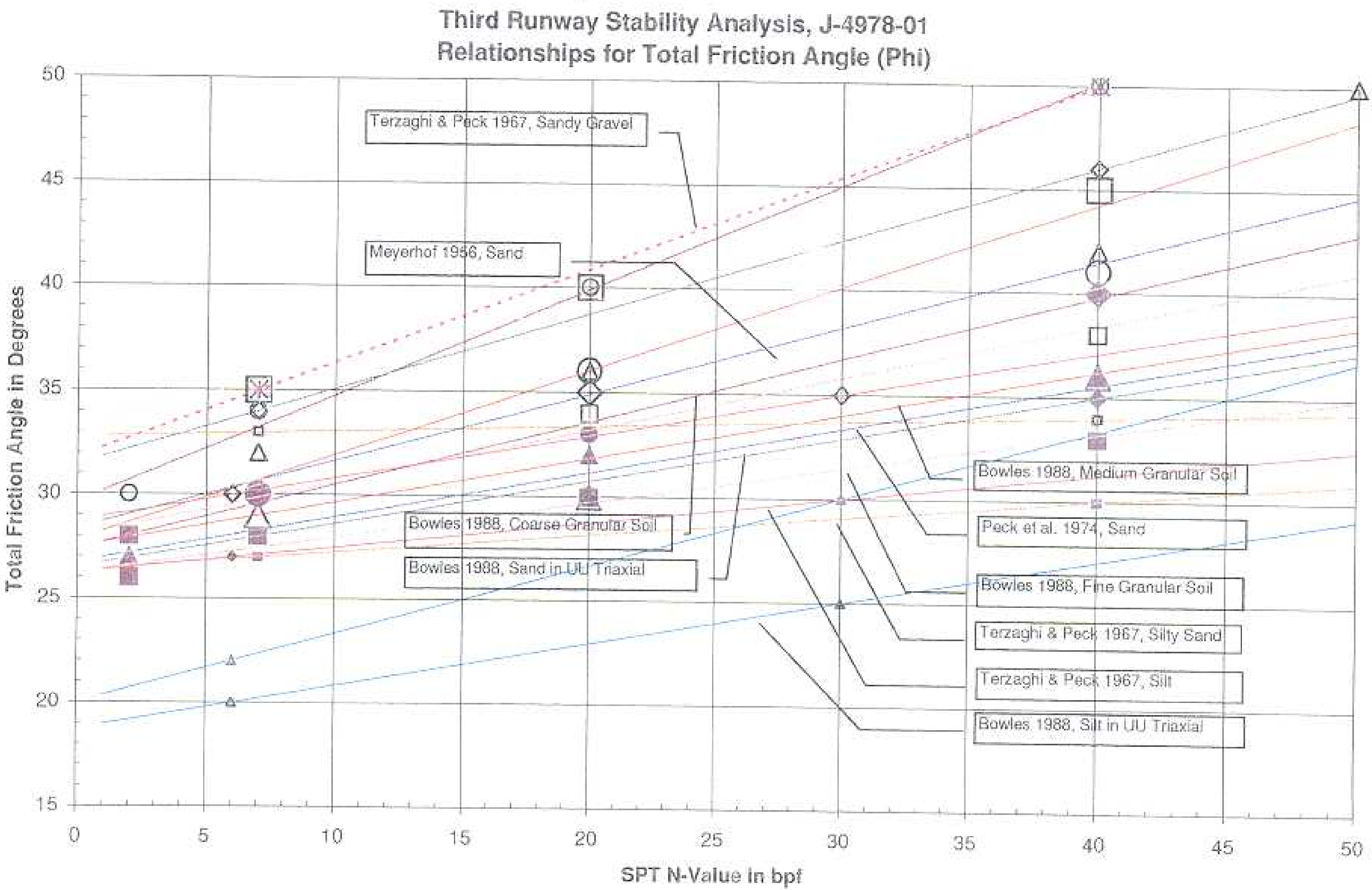

Third Runway Stability Analysis, J-4978-01
Relationships for Total Friction Angle (Phi)
Total Friction Angle in Degrees
SPT N-Value in bpf
50
45
40
35
30
25
20
15
0
5
10
15
20
25
30
35
40
45
50
Terzaghi & Peck 1967, Sandy Gravel
Meyerhof 1956, Sand
Bowles 1988, Coarse Granular Soil
Bowles 1988, Sand in UU Triaxial
Bowles 1988, Medium Granular Soil
Peck et al. 1974, Sand
Bowles 1988, Fine Granular Soil
Terzaghi & Peck 1967, Silty Sand
Terzaghi & Peck 1967, Silt
Bowles 1988, Silt in UU Triaxial

FIGURE 2

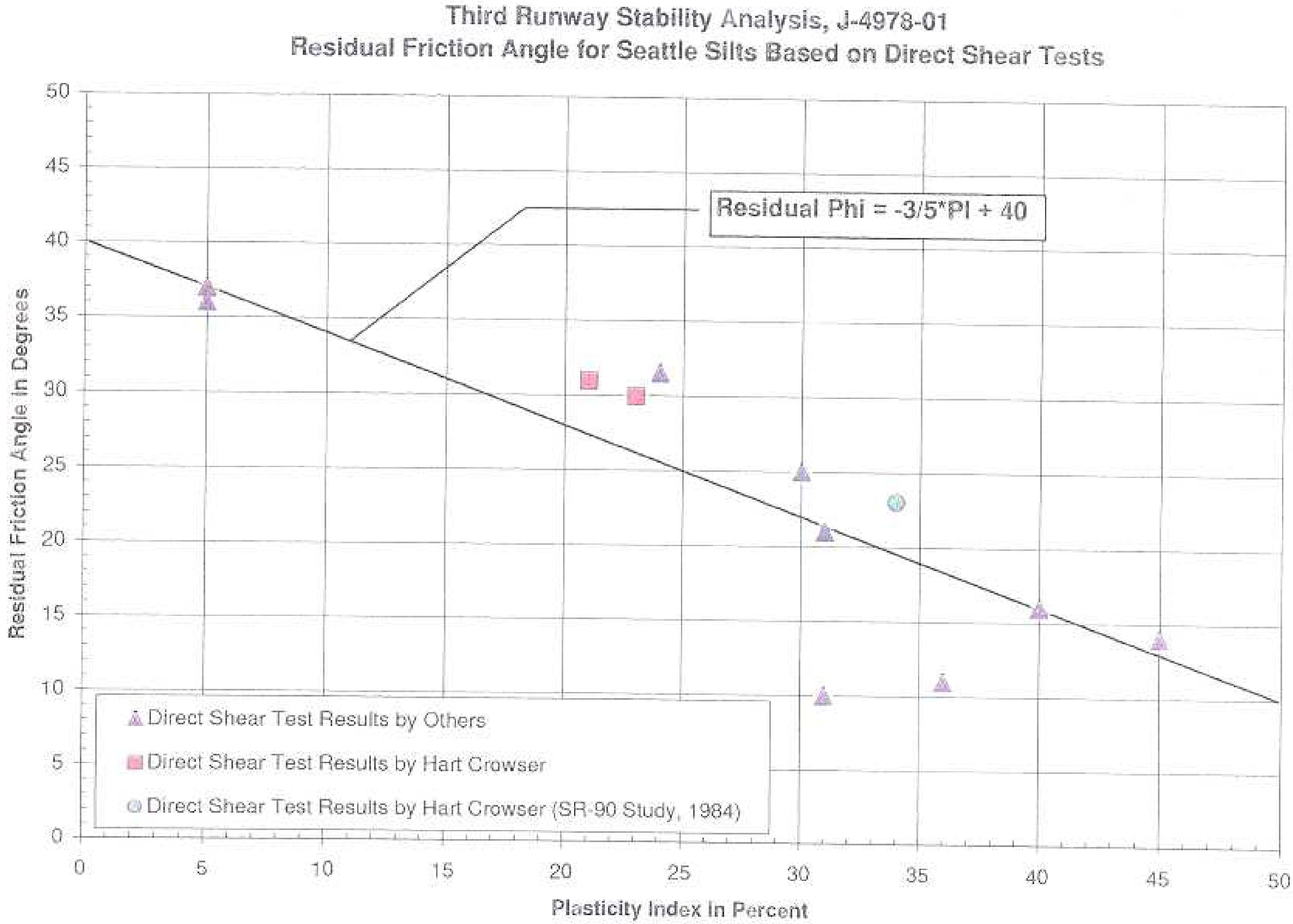

Third Runway Stability Analysis, J-4978-01
Residual Friction Angle for Seattle Silts Based on Direct Shear Tests
Residual Phi = -3/5*PI + 40
Residual Friction Angle in Degrees
0
5
10
15
20
25
30
35
40
45
50
Plasticity Index in Percent
Direct Shear Test Results by Others
Direct Shear Test Results by Hart Crowser
Direct Shear Test Results by Hart Crowser (SR-90 Study, 1984)

FIGURE 3

ATTACHMENT 1
SUPPORTING INFORMATION FOR STRENGTH PARAMETERS

STRENGTHS BASED ON N-VALUE

	EPRI, p. 2-7 Sowers, 1979	Bowles, 1988	Hart Crowser, Figure A-1	Tschebotarioff, 1973	From Unconsolidated Undrained Triaxial Bowles, 1988	Terzaghi & Peck, 1967	Fang, 1991 After Peck et al., 1974	Fang, 1991 After Meyerhof, 1956
COHESIVE	***Su=1/2qu (psf)***		***Su (psf)***	***Su (psf)***				
Very Soft (N=0 to 2)	0-250		<125	<300				
Soft (N=2 to 4)	250-500		125-250	300-600				
Firm (N=4 to 8)	500-1000		250-500	600-1200				
Stiff (N=8 to 15)	1000-1500		500-1000	1200-2400				
Very Stiff (N=15 to 30)	1500-2000		1000-2000	2400				
Hard (N>30)	>2000		>2000	>4500				
GRANULAR (Fine)								
Very Loose (N=0 to 4)		26-28						
Loose (N=4 to 10)		28-30						
Medium dense (N=10 to 30)		30-34						
Dense (N=30 to 50)		33-38						
Very Dense (N>50)								
GRANULAR (Medium)								
Very Loose (N=0 to 4)		27-28						
Loose (N=4 to 10)		30-32						
Medium dense (N=10 to 30)		32-36						
Dense (N=30 to 50)		36-42						
Very Dense (N>50)		<50						
GRANULAR (Coarse)								
Very Loose (N=0 to 4)		28-30						
Loose (N=4 to 10)		30-34						
Medium dense (N=10 to 30)		33-40						
Dense (N=30 to 50)		40-50						
Very Dense (N>50)								
SANDS								
Very Loose (N=0 to 4)							<29	<30
Loose (N=4 to 10)					28-34		29-30	30-35
Medium dense (N=10 to 30)							30-36	35-40
Dense (N=30 to 50)					35-46		36-41	40-45
Very Dense (N>50)							>41	>45
Sandy GRAVEL								
Very Loose (N=0 to 4)								
Loose (N=4 to 10)						35		
Medium dense (N=10 to 30)								
Dense (N=30 to 50)						50		
Very Dense (N>50)								
Silty SAND								
Very Loose (N=0 to 4)								
Loose (N=4 to 10)						27-33		
Medium dense (N=10 to 30)								
Dense (N=30 to 50)						30-34		
Very Dense (N>50)								
SILT								
Very Soft (N=0 to 2)								
Soft (N=2 to 4)								
Firm (N=4 to 8)					20-22	27-30		
Stiff (N=8 to 15)								
Very Stiff (N=15 to 30)								
Hard (N>30)					25-30	30-35		

STRENGTHS BY MATERIAL TYPE

	From Unconsolidated Undrained Triaxial Bowles, 1988	Terzaghi & Peck, 1967
GRAVEL/ Sandy GRAVEL		
Medium size	40-55	
Sandy	35-50	
SANDS		
Loose, dry	28-34	
Loose, saturated	28-34	
Dense, dry	35-46	
Dense, saturated	32-42	
Round grains, uniform; (loose)		27.5
Round grains, uniform; (dense)		34
Angular grains, well graded (loose)		33
Angular grains, well graded (dense)		45
SILTY SANDs		
SILTS		

STRENGTHS PROVIDED IN CASE HISTORIES

	c, psf	*c(residual), psf*	*c(dynamic), psf*	*Phi*	*Phi(residual)*	*Phi(dynamic)*	*Gam(DRY), pcf*	*Gam(TOT), pcf*	*Su, psf*
(1) Seattle **CLAYS**, Alki Transfer System	0		0-750	16-32		20-32			
Perrone and Byers, 1998 (Earthquake Conference)									
(2) Seattle **SILT/CLAYs**, Fauntleroy District		50-350; Mean 150			11-29; Mean 23			115	
Esperance **SAND**	0			32				120	
Engineering in Washington, p. 687									
(3) **GLACIAL TILL**	1000-4000			35-45			120-140		
GLACIAL OUTWASH	0-1000			30-40			115-130		
GLACIOLACUSTRINE	0-3000			15-35			100-120		
Engineering in Washington, p. 20									
(4) Loose to Med. Dense **SAND**, Mt. Baker Ridge Tunnel									1700
Upper Dense **SAND**									
Upper Laminated **SILT/CLAY**									2300
Non-Laminated **SILT/CLAY**									3200
Lower Laminated **SILT/CLAY**									3400
Hard SILT									
Very Dense **SAND**									2000
Engineering in Washington, p. 833									
(5) Some silt, medium **SAND**, Construction of Valley Freeway	280			34				120	
Engineering in Washington, p. 803									
(6) Recessional **SILT**, Seattle Access	600-4100								
OUTWASH SILT	4500-5500								
Engineering in Washington, p. 852									

GRAPHS FOR STRENGTH

(1) Angles of Internal Friction vs. Unit Weight (g/cm^3) - Provided for various sands; (Bowles, 1988), p. 76
(2) Effective Friction Angle vs. Plasticity Index - Provided for various states of clay (e.g., residual, remolded, undisturbed); (Bowles, 1988), p. 85.
(3) Residual Cohesion (kg/cm^3) vs. Liquid Limit - Provided for various Liquidity Indices; (Fang, 1991), p. 137.
(4) Angle of Internal Friction (Effective) vs. Dry Density (Mg/m3) - Provided for various material types; (Holtz & Kovacs), p. 518
(5) Residual Friction Angle vs. Plasticity Index, (Hart Crowser,1984), J-712-31, Figure B-4

ATTACHMENT 2
SAMPLE RESULTS FOR RESIDUAL FRICTION ANGLE AND SUPPORTING LABORATORY RESULTS

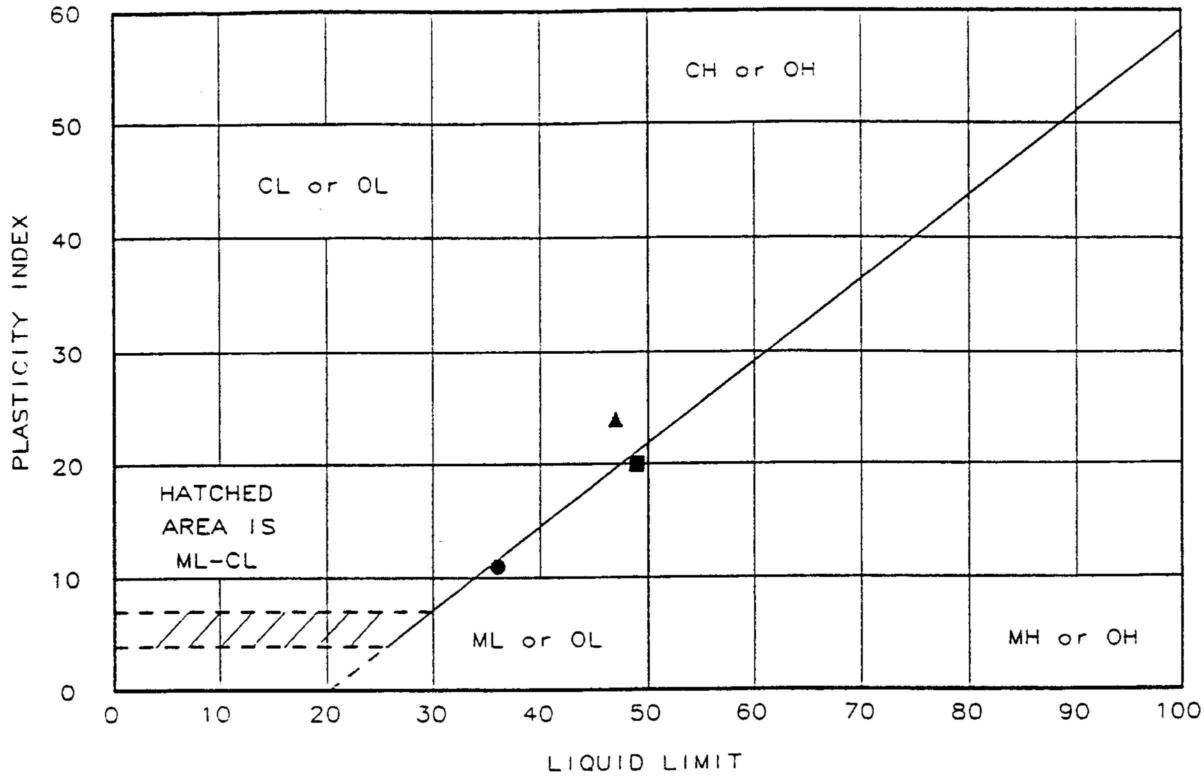
STRENGTHS BASED ON SITE-SPECIFIC LABORATORY TESTING

SOIL	Consistency	PI	Residual Friction Angle[1]
Clayey SILT	(Stiff to very stiff)	11	33
Clayey SILT	(Stiff)	20	28
Clayey SILT	(Stiff)	24	26

(1) Based on Residual Friction Angle vs. Plasticity Index (PI) relationships developed in the SR-90 Study, Hart Crowser J-712-31, Figure B-4

J. BEAVER

LIQUID AND PLASTIC LIMITS TEST REPORT



	Location + Description	LL	PL	PI	-200	ASTM D 2487-90
●	TP-2, S-3 Depth 3.5 to 4.5 feet	36	25	11		
▲	TP-9, S-7 Depth 15 to 16 feet	47	23	24		
■	TP-12, S-4 Depth 4 to 4.5 feet	49	29	20		

Remarks:

Project: 3rd Runway

Client:

Location: Seattle, Washington

HARTCROWSER

J-4978-01 8/3/98

Figure B-4

AR 041714