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MEMORANDUM

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DATE: March 5, 2001

TO: Jim Thomson, P.E., HNTB

FROM: Douglas Lindquist, E.I.T., Barry Chen, PhD., P.E., and Michael Bailey, P.E., Hart Crowser, Inc.

RE: **Revised Methods and Results of Liquefaction Analyses**
Third Runway Embankment
Sea-Tac, Washington
J-4978-30

This memorandum provides results of liquefaction analyses for the SeaTac Third Runway project. Preliminary liquefaction analyses were presented in our September 7, 2000 draft memorandum. This update is based upon the results from the revised probabilistic seismic hazard analysis (PSHA) and one-dimensional site response analyses. The site response analyses were performed for proposed embankment heights of 50 and 150 feet above the existing ground surface. See Hart Crowser's memorandum entitled "Additional Information on the Seismic Design" (Hart Crowser, 2001) for additional information on these analyses.

This memorandum presents the methods and results of Hart Crowser's analyses of potential liquefaction and post-liquefaction residual strength for the proposed Third Runway embankment and retaining walls. Results of both the preliminary and revised analyses are presented. We analyzed a total of 120 borings and their corresponding Standard Penetration Test (SPT) results. Logs of these borings are presented in previous subsurface conditions data reports (references are listed at the end of this memorandum, (see Civil Tech, 1997 and Hart Crowser, 1999a, 2000a, 2000b, 2000c, and 2000e for information on subsurface conditions). See Hart Crowser's report entitled "Geotechnical Engineering Report, 404 Permit Support" for an overview of the project (Hart Crowser, 1999b).

Potential for liquefaction, and resulting soil behavior, is influenced by a number of factors. This memorandum documents the approach used by Hart Crowser in determining which areas of the Third Runway embankment site are susceptible to liquefaction. Results of the analysis presented in this memorandum were used in separate design analyses that are discussed in companion reports, (e.g., Hart Crowser, 2000d and 2000f).

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SUMMARY

Liquefaction potential was evaluated for the overall site as well as separately at the NSA Wall, West Wall, South Wall, and the 2H:1V slope area along the west side of the proposed embankment. The advantage of looking at the site as a whole is to provide a larger data set for statistical analyses, whereas site-specific conditions are beneficial when looking at specific areas. A statistical analysis was also performed for each area on the SPT samples that were potentially liquefiable. For our preliminary analyses, the seismic event required to trigger liquefaction of a sample and the corresponding post-liquefaction residual strength are presented herein for each area and the site as a whole. For our revised analyses, results are presented for the design level event corresponding to a 475-year return period.

Liquefaction-susceptible soils (loose to medium dense sands and soft to stiff, very sandy silts, below the water table or likely to become saturated over time) exist intermittently within the subgrade support area for the three proposed MSE walls as well as portions of the embankment between wall locations. The extent of potential liquefaction, as well as the post-liquefaction residual shear strength, varies by location, depth, and for different size seismic events.

ANALYSIS METHODS

We used results of SPT tests from 112 borings accomplished by Hart Crowser and eight borings accomplished by Civil Tech as input to the liquefaction analyses. Cone penetrometer tests accomplished by Hart Crowser and others supported interpolation of the standard penetration test (SPT) results, but are not included in the analyses presented in this draft memorandum. SPT tests by AGI also supported interpolation but were not included because of potential significant variations (which in many cases were not documented) pertaining to SPT methods and equipment used. Hart Crowser made special efforts to measure or verify field variables in some of the borings, and we examined the effect of small variations in SPT procedure. We accepted small deviations in some SPT procedures and in the values of various correction factors as discussed below.

The methods used to evaluate triggering of potential liquefaction and post-liquefaction residual strength are presented in the following sections.

Liquefaction Potential

The method of analysis is based on the work of H.B. Seed (Seed et al., 1985) and the most recent update by the National Center for Earthquake Engineering Research (NCEER, 1996)

Workshop. The method is empirical, which means it is based on results of case studies of sites where liquefaction has and has not occurred in actual earthquakes. In this method, a calculated measure of earthquake loading, the cyclic stress ratio (CSR), is compared to a calculated measure of resistance, the cyclic resistance ratio (CRR). The factor of safety against liquefaction is taken as the ratio of resistance to loading, i.e., FS = CRR/CSR. A factor of safety less than 1 indicates that liquefaction would likely occur for a specific design level earthquake, unless certain other criteria are met, (i.e., the characteristics of fine-grained soils).

Cyclic Stress Ratio (CSR)

The average CSR was calculated by the following equation:

$$CSR = 0.65 \frac{a_{max}}{g} \frac{\sigma_v}{\sigma_v'} \frac{r_d}{MSF}$$

Where:

a_{max}	is the peak horizontal acceleration;
g	is the acceleration due to gravity;
σ_v and σ_v'	are the total and effective stresses at the sample depth;
r_d	is a stress reduction factor; and
MSF	is an earthquake magnitude scaling factor.

We calculated total and effective stresses based on existing groundwater conditions at the time of drilling.

To evaluate the effect of the design level ground motion on the extent of liquefaction a number of different seismic events were evaluated. Table 1 shows the peak horizontal acceleration, magnitude, and magnitude scaling factor corresponding to the seismic events we considered.

Table 1 - Design Criteria for Various Seismic Events (Preliminary Analysis)

Probability of Exceedence	Return Period in Years	Peak Horizontal Acceleration	Magnitude	Magnitude Scaling Factor
50% in 50 years	72	0.16 g	6.5	1.44
25% in 50 years	175	0.23 g	6.9	1.24
15% in 50 years	300	0.30 g	7.2	1.11
10% in 50 years	475	0.36 g*	7.5	1.00
5% in 50 years	975	0.47 g	8.0	0.85

* Revised PSHA reduced this acceleration value to 0.31 g (Hart Crowser, 2001).

In the preliminary analyses the seismic events were selected to encompass a broad range of potential earthquakes in the Puget Sound area. The peak horizontal accelerations were obtained from the results of our preliminary site-specific probabilistic seismic hazard analysis (Hart Crowser, 1999c). The magnitudes assigned to the return period seismic events were based on the results of our PSHA and deaggregated hazard analysis. This analysis indicates that the 475-year return period earthquake would have a magnitude of 7.0 to 7.5. We have conservatively used M=7.5 in our liquefaction analyses.

Additionally, Hart Crowser assigned earthquake magnitude values that increased for longer return periods. This is a conservative way to account for the trend that increasingly larger magnitude earthquakes produce motions of longer duration. It is likely that a lower magnitude, local, shallow source, such as the Seattle Fault, could produce an equally high acceleration at the site as a higher magnitude subduction zone source further away.

Based on the results of the revised PSHA (Hart Crowser, 2001), new time histories of acceleration were developed for the 72-, 475-, 975-, and 2,475-year return period seismic events. However, the additional round of liquefaction analyses was performed only for the design level event corresponding to a 475-year return period. These analyses are based upon a maximum acceleration of 0.31 g for the depth of the potentially liquefiable soils. The extent of liquefaction slightly decreased in our revised analyses compared to the extent in our preliminary analyses.

Cyclic Resistance Ratio (CRR)

The CRR is a measure of the soil resistance to liquefaction. This is calculated based on SPT blow count and the percent of fines (soils that pass the No. 200 U.S. sieve). Additionally, correction factors for high overburden pressures (K_σ) and sloping ground conditions (K_α) have been proposed in some of the literature.

CRR Correction Factors

High Overburden Pressure, K_σ. This correction decreases the CRR by a factor of 1.0 to 0.6 for overburden pressures greater than about 1 ton per square foot (tsf). We followed the recommendations of the NCEER Workshop in using this parameter (NCEER, 1996).

The empirical database for the $(N_1)_{60}$ and CRR relationship was created from results at sites with mostly shallow conditions producing liquefaction. This is typical of the conditions encountered in our borings, but would not necessarily represent conditions that follow the completion of embankment construction. In the preliminary analyses for the need and extent of ground improvement, the high overburden pressures were accounted for by including the weight of fill in calculating the K_σ factor. However, the depth of fill was not included in the r_d factor in the CSR. This was a very conservative approach.

The revised analyses used the current *in situ* stress and depth conditions to calculate the stress (K_σ) and depth (r_d) factors. Table 2 shows that r_d decreases CSR more than K_σ decreases CRR. Because CSR decreases more than CRR, our liquefaction analyses are conservative by neglecting the effects of the fill on K_σ and r_d. Additionally, the ratio of total stress to effective stress will decrease CSR after the fill is placed. Because of significant variation in the depth of fill placed throughout the site, the effect of the reduction in the ratio of stresses was conservatively neglected in our analyses.

Table 2 – Comparison of the Stress Reduction and Overburden Factors with Depth

Depth in Feet	Stress Reduction Factor, r_d used in the CSR	High Overburden Factor, K_σ used in the CRR (based on 135 pcf fill)
10	0.98	1.0
50	0.75	0.76
100	0.50	0.63
150	0.50	0.60

Sloping Ground, K_α. This correction could increase or decrease the CRR depending on soil density and stress conditions. We followed the recommendations of the NCEER Workshop (NCEER, 1996) by neglecting this parameter. Available data indicate that K_α only increases liquefaction potential for loose soils, which would already be deemed liquefiable in our analyses.

SPT Correction Factors

A number of adjustments or corrections to field SPT results are used to verify that the input data correspond to the empirical data used to assess potential for liquefaction. The field SPT blow counts (*Nm*) were corrected to obtain $(N_1)_{60}$ blow counts by using the following equation:

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_S$$

Each of the terms used to modify the measured blow count (N_m) are described below.

Overburden Pressure, C_N. This correction normalizes blow counts to an overburden pressure of one atmosphere. The correction factor C_N was calculated as follows (NCEER, 1996):

$$C_N = \sqrt{P_a / \sigma_v'} \leq 2$$

where:
P_a is the atmospheric pressure; and
σ_v' is the effective vertical stress at the sample depth.

A maximum value of $C_N = 2$ was used to keep shallow samples from having very large correction factors.

Energy Ratio, C_E. This correction normalizes blow counts to account for variation in energy from drill rig to drill rig. The energy ratio correction factor was taken as 1.0, which corresponds to a hammer efficiency of 60 percent. (This value is typical for the type of equipment used, as indicated by prior results obtained by the drilling contractor. Holt Drilling had three of their rigs measured for efficiency in April 1996, and found the measured energy averaged 55 percent for depths greater than 10 feet. This slightly lower efficiency effectively increased the measured blow counts by 9 percent.)

Borehole Diameter, C_B. This correction normalizes blow counts to a typical borehole diameter of 2.5 to 4.5 inches. Our borings typically used a 4-inch ID hollow-stem auger for which the correction factor is 1.0. Occasionally a 6-inch hollow-stem auger was used for which the correction factor is 1.05. Including this adjustment would have increased the blow counts on a small number of borings by 5 percent.

Rod Length, C_R. This correction factor normalizes blow counts to account for dissipation in energy for very short rod lengths due to wave propagation. The rod length was measured from where the hammer strikes the rod to the base of the sampler. The height above ground where the hammer strikes the rod was typically about 10 feet for the drill rigs used in this project. This correction factor was calculated according to Table 3 (NCEER, 1996).

Table 3 - Rod Length Correction Factor

Rod Length in Feet	Rod Length Correction
9.8 to 13.1	0.75
13.1 to 19.7	0.85
19.7 to 32.8	0.95
32.8 to 98.8	1.0
> 98.8	0.9

Sampling Method, C_S. This correction normalizes blow counts to those of a standard split-spoon sampler with liners. The samplers we used did not contain liners. The recommended correction for samplers without liners is 1.1 to 1.3. Loose soils typically are at the low end of this correction and dense soils are at the high end. Because liquefaction most often occurs in loose soils, a correction factor of 1.1 was used throughout the analysis.

Once the corrected $(N_1)_{60}$ blow counts were obtained, they were corrected for the presence of fines to clean sand $(N_1)_{60CS}$ blow counts. It is widely accepted that the presence of fines generally reduces liquefaction potential of granular soils for a given SPT blow count. I.M. Idriss with assistance from R.B. Seed developed the following recommendations (NCEER, 1996) to correct blow counts for the presence of fines content (FC).

$$(N_1)_{60CS} = \alpha + \beta(N_1)_{60}$$

where α and β are coefficients determined from the following equations:

$\alpha = 0$	for FC ≤ 5%
$\alpha = e^{[1.76-(190/FC^2)]}$	for 5% ≤ FC ≤ 35%
$\alpha = 5$	for FC ≥ 35%
$\beta = 1.0$	for FC ≤ 5%
$\beta = 0.99 + (FC^{1.5}/1000)$	for 5% ≤ FC ≤ 35%
$\beta = 1.2$	for FC ≥ 35%

Figure 1 shows an empirical chart for evaluating trigger liquefaction potential using the corrected blow counts $(N_1)_{60}$ and CRR for a magnitude 7.5 earthquake and a range of fines contents (Seed et al., 1985). We used an updated version of this chart (NCEER, 1996) based on corrected clean sand blow count $(N_1)_{60CS}$ to obtain the CRR.

Identification of Nonliquefiable Soils

Samples that had a calculated factor of safety (CRR/CSR) of less than 1 were further evaluated to assess potential for liquefaction to actually occur. The two criteria used for this further evaluation were characteristics of fine-grained soils within the individual sample, and evaluation of potential saturation for samples apparently located above the water table at the time of drilling. The criteria used for each of these are described below.

Fine-Grained Soils. The beneficial resistance to liquefaction by soils with a fines (silt and/or clay) content greater than 35 percent has traditionally been neglected in liquefaction analyses. However, soils with a high percentage of fines may not be susceptible to liquefaction based on other empirical criteria (e.g., clay fraction, plasticity as indicated by the Atterberg Limits, and water content). On a sample-by-sample basis, we used the following empirical criteria, which were originally developed by the Chinese and later modified by the U.S. Army Corps of Engineers to match index properties used in the United States (Kramer, 1996).

- (Fraction of fines finer than 0.005 mm - 5%) ≤ 15%;
- (Liquid limit + 1%) ≤ 35%;
- (Natural water content + 2%) ≥ 0.9 LL; and
- Liquidity index ≤ 0.75.

These criteria indicate that fine-grained soils that satisfy all four of the preceding conditions are potentially liquefiable. Therefore, if any one of these criteria was not met, the soil was deemed nonliquefiable.

Each sample with a high fines content and CRR/CSR factor of safety less than 1.0 was evaluated based on these criteria. In some cases where Atterberg Limits or grain size distributions were not available, the visual classification was used to eliminate soils that were classified as "clay" or "very clayey" based on comparison to similar samples from elsewhere on site.

Saturation. A soil must be saturated to generate the excess pore water pressure required for liquefaction to occur. The groundwater level observed at the time of drilling (ATD) is typically not a good indicator of the actual extent of saturated soils, because of the disturbance produced by drilling. Water level measurements in observation wells at the Third Runway site typically (but not always) are several feet above ATD, especially in silty soils, but this varies depending on interbedding of the strata and how the well is completed. In addition, seasonal variations in groundwater level on the order of several feet have been observed in some wells at the site, and longer period variations may also exist. Finally, we also considered the effect of the constructed embankment on infiltration and long-term changes in groundwater level at the site (see Appendix C, Hart Crowser, 2000f).

We found there is considerable uncertainty as to whether some of the soil assumed to be liquefiable in the analyses would ever be saturated. For design purposes, Hart Crowser compiled and interpolated groundwater observation data to assess liquefaction potential for different parts of the site using soil conditions from all of the on-site borings and test pits (see Hart Crowser, 2000d and 2000f).

Post-Liquefaction Residual Strength

Results of the analyses described above were used to create a data set of liquefaction-susceptible samples that was then used to estimate the post-liquefaction residual shear strength for site soils. Hart Crowser used the residual shear strength in stability analyses for embankment and retaining wall design. In our preliminary analyses, the post-liquefaction residual strength was calculated according to the empirical procedure developed by Seed and Harder (1990). The revised analyses are based on a curve developed by I.M. Idriss (Idriss, 1998) and shown on Figure 2. These curves relate the residual strength to an equivalent clean sand SPT blow count $(N_1)_{60\,CS}$. The corrected clean sand blow count $(N_1)_{60\,CS}$ is the $(N_1)_{60}$ blow count plus the value shown in Table 4.

Table 4 - Recommended Fines Correction for Residual Strength Calculation (Seed and Harder, 1990)

Percent Fines	Additional SPT Blow Counts
10	1
25	2
50	4
75	5

Assumptions and Interpretations

Several assumptions were made to estimate residual shear strength to overcome limitations in Seed and Harder's data set, and in our own data.

- In the preliminary analyses the Seed and Harder (1990) data was extrapolated to $(N_1)_{60\,cs}$ of 24 with a corresponding residual strength of 1,750 psf.

- In the revised analyses the effects of extrapolating the Seed and Harder (1990) data set using the dashed portion of the curve on Figure 2 was evaluated by using residual strengths with a maximum of 600, 1,200, and 2,000 psf, which correspond to $(N_1)_{60\,cs}$ of 15.5, 20, and 23, respectively.

- Although many engineers suggest that flow liquefaction cannot occur for $(N_1)_{60\,cs}$ greater than 16, we have conservatively assumed that it may and have used the maximum residual strengths just described.

- Samples that were described as "peat" were not included in the analysis because the peat will be excavated and replaced with granular fill or improved *in situ* with stone columns in areas potentially affected by flow sliding or excessive consolidation.

- For the purpose of estimating post-liquefaction residual strength, we considered all potentially liquefiable samples, including some that are not saturated under existing conditions.

Estimated Residual Strength

In our preliminary analyses we evaluated 120 borings for five different seismic events. In our revised analyses, we evaluated the same 120 borings while focusing only on the design level (475-year) seismic event. We discuss these results for the site as a whole and by area below.

As expected, our preliminary analyses indicated the number of liquefaction susceptible samples increased for increasingly larger seismic events. Post-liquefaction residual strength is lowest for the samples which liquefy in smaller, lower return period (more frequent) events, because the more dense soils (which only liquefy at higher levels of shaking) have corresponding higher values of residual strength as indicated by Seed and Harder's empirical studies.

Figure 3 shows a summary of the number of samples that would liquefy for various levels of seismic event based on our preliminary analyses, as well as our estimated post-liquefaction residual strength for the site as a whole. The results for the revised analyses are also shown on this figure for the 475-year event. The results of the revised analyses indicate that fewer samples will liquefy with a corresponding lower residual strength when compared to the preliminary analyses. Table 5 summarizes the revised liquefaction results.

Table 5 – Summary of Revised Liquefaction Results

Location	Residual Strength, Sr in psf based on Sr_{max} = 600 psf	Residual Strength, Sr, in psf based on Sr_{max} = 1,200 psf	Residual Strength, Sr, in psf based on Sr_{max} = 2,000 psf
Cumulative Results	416	566	618
NSA Wall	392	565	607
West Wall	461	623	710
South Wall	362	462	595
2H:1V Slope	406	536	550

Figures 4, 5, 6, and 7 show the same type of information for different areas within the overall site. These figures show the results of our preliminary analyses with revised analyses for the 475-year seismic event only.

Hart Crowser's preliminary liquefaction analysis included extrapolation of residual strength values up to 1,750 psf, beyond the empirical data reported by Seed and Harder (maximum 600 psf). The results of the revised residual strength analyses are shown for maximum residual strengths of 600, 1,200, and 2,000 psf as calculated/extrapolated from Figure 2. The design team inferred the 2,000 psf value to be reasonable because extensive laboratory testing has shown that the steady state, or residual, strength of laboratory test specimen increases as soil density increases. Because the SPT resistance of a given soil is also known to increase with increasing soil density (Gibbs and Holtz, 1957; Kulhawy and Mayne, 1990), residual strength will also increase as SPT resistance increases.

We checked the 1,200 and 2,000 psf using the steady state approach to extrapolate the upper and lower bound residual shear strengths of the Seed and Harder data set. The upper/lower bound of residual strength was approximately 1,750/1,050 and 2,600/1,750 psf for blow counts of 20 and 24, respectively. This indicates that Idriss' curve used in our analysis is conservatively near the lower bound of the extrapolated Seed and Harder data set.

Comparison of Figures 4 through 7 shows some minor variability but overall similar results. The apparent variability is probably at least partially a result of statistical uncertainty based on the relatively small number of samples, as well as the result of actual differences in soil conditions; i.e., loose soils at the South Wall typically appear to be fill and colluvium (slope debris) whereas loose soils at the NSA Wall typically appear to be alluvium (stream deposits). Use of the residual strength data and specific results are discussed in other project reports.

USE OF RESULTS

The results of the liquefaction and residual strength analyses were used to assess the stability of the embankment after liquefaction has occurred. These analyses are presented in a number of reports (Hart Crowser, 2000d and 2000f). In these reports the spatial variability of liquefaction was closely examined in looking for loose zones and weak seams. Stability cross sections were analyzed using conservative assumptions on the extent of liquefaction. See each report for specific details.

Attachments:
References
Figure 1 - Relationship Between Stress Ratio Causing Liquefaction and $(N_1)_{60}$ Values for Silty Sand for M = 7.5
Figure 2 - Undrained Residual Strength, Sr, versus Equivalent Clean Sand SPT Corrected Blowcount Based on Field Case Studies Published by Seed (1987) and by Seed and Harder (1990)
Figure 3 - Comparison of Liquefied Samples for Various Seismic Events (NSA, West, and South Walls and 2:1 Slope)
Figure 4 - Comparison of Liquefied Samples for Various Seismic Events (NSA Wall)
Figure 5 - Comparison of Liquefied Samples for Various Seismic Events (West Wall)
Figure 6 - Comparison of Liquefied Samples for Various Seismic Events (South Wall)
Figure 7 - Comparison of Liquefied Samples for Various Seismic Events (2H:1V Slope)

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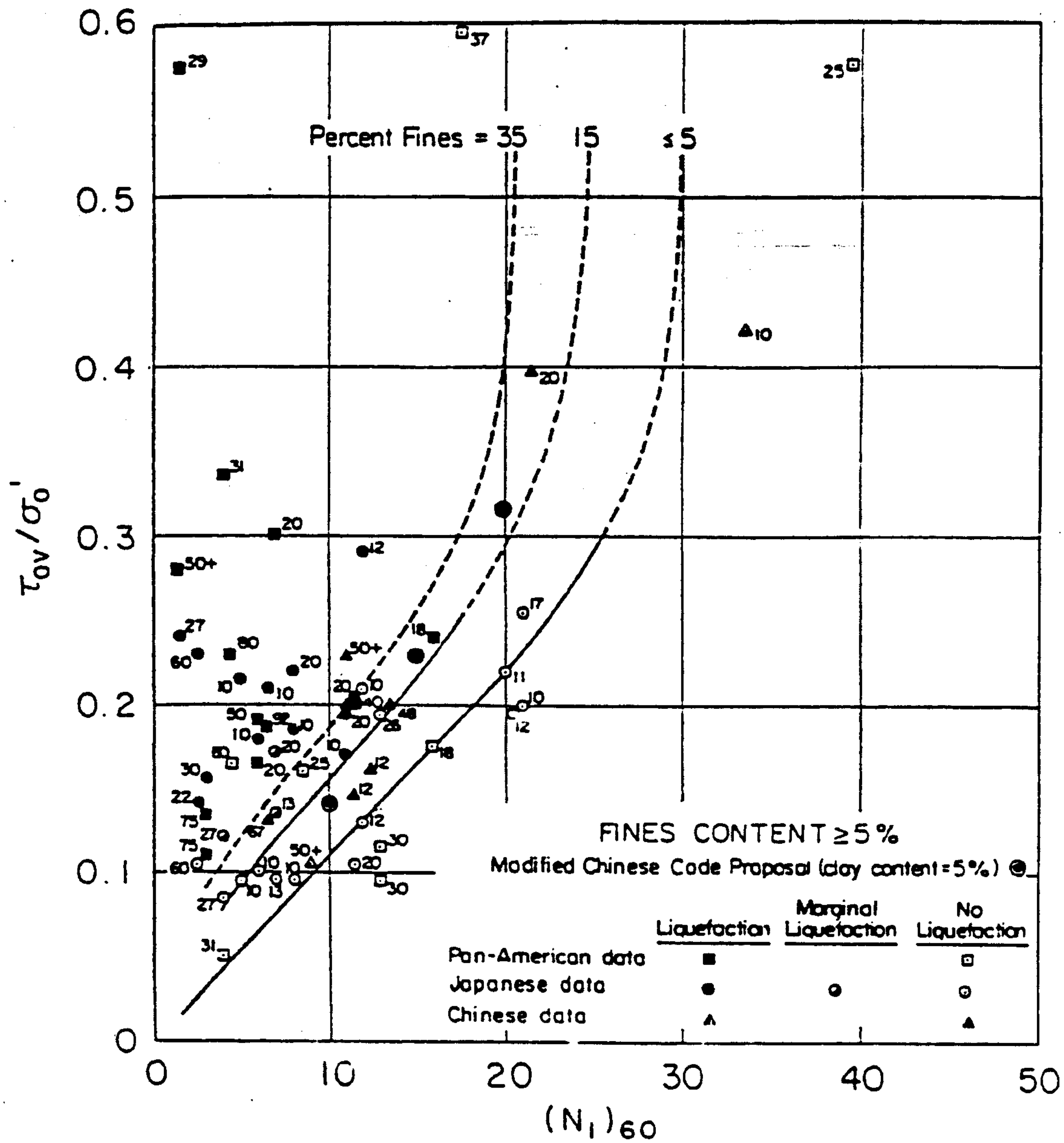
Relationship Between Stress Ratio Causing Liquefaction and $(N_1)_{60}$ Values for Silty Sand for M=7.5



STIA 00565

Source: From Seed et al., 1985

DTN 3/7/01 497830G.cdr

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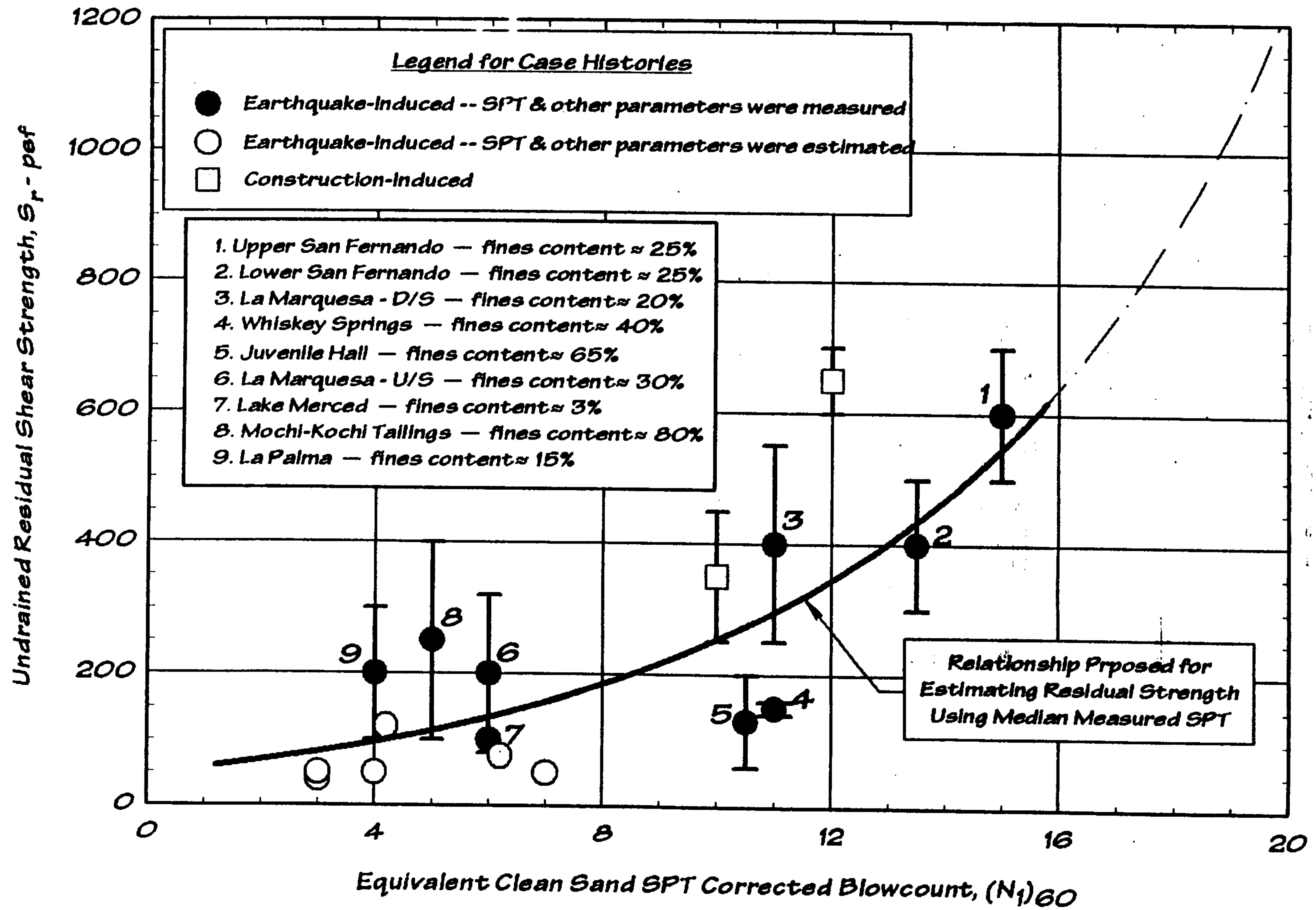
Figure 1

AR 049960

Undrained Residual Strength, Sr, versus Equivalent Clean Sand SPT Corrected Blowcount Based on Field Case Studies Published by Seed (1987) and by Seed and Harder (1990)



Source From Idriss, 1998

DTN 3/07/01 497830A cdr

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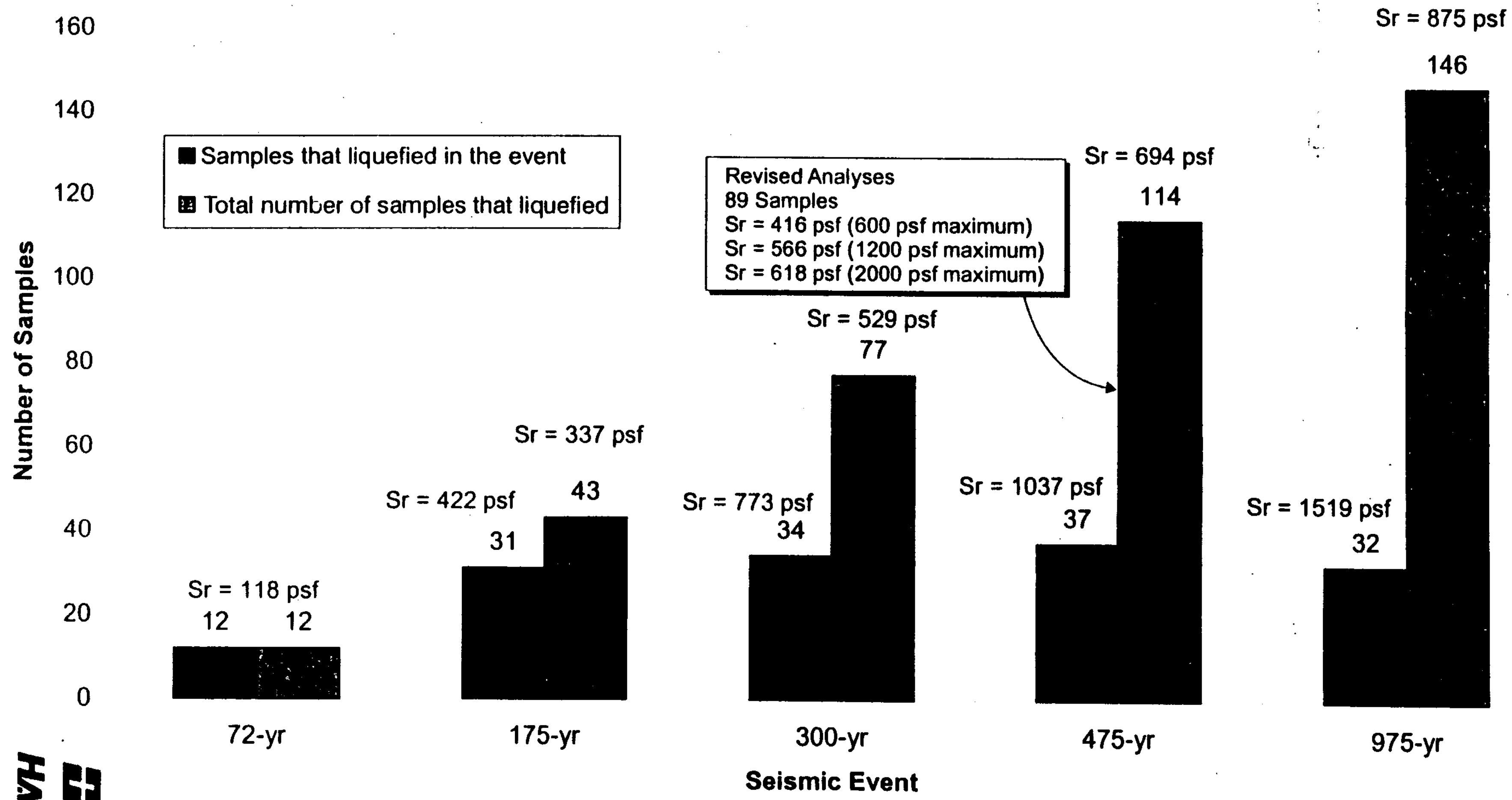
Figure 2

AR 049961

STIA 00566

Comparison of Liquefied Samples for Various Seismic Events

NSA, West, South Walls, and 2:1 Slope



Results are based on preliminary liquefaction and residual strength analyses except for the cross hatched area that shows revised results for the 475-yr seismic event.

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Figure 3

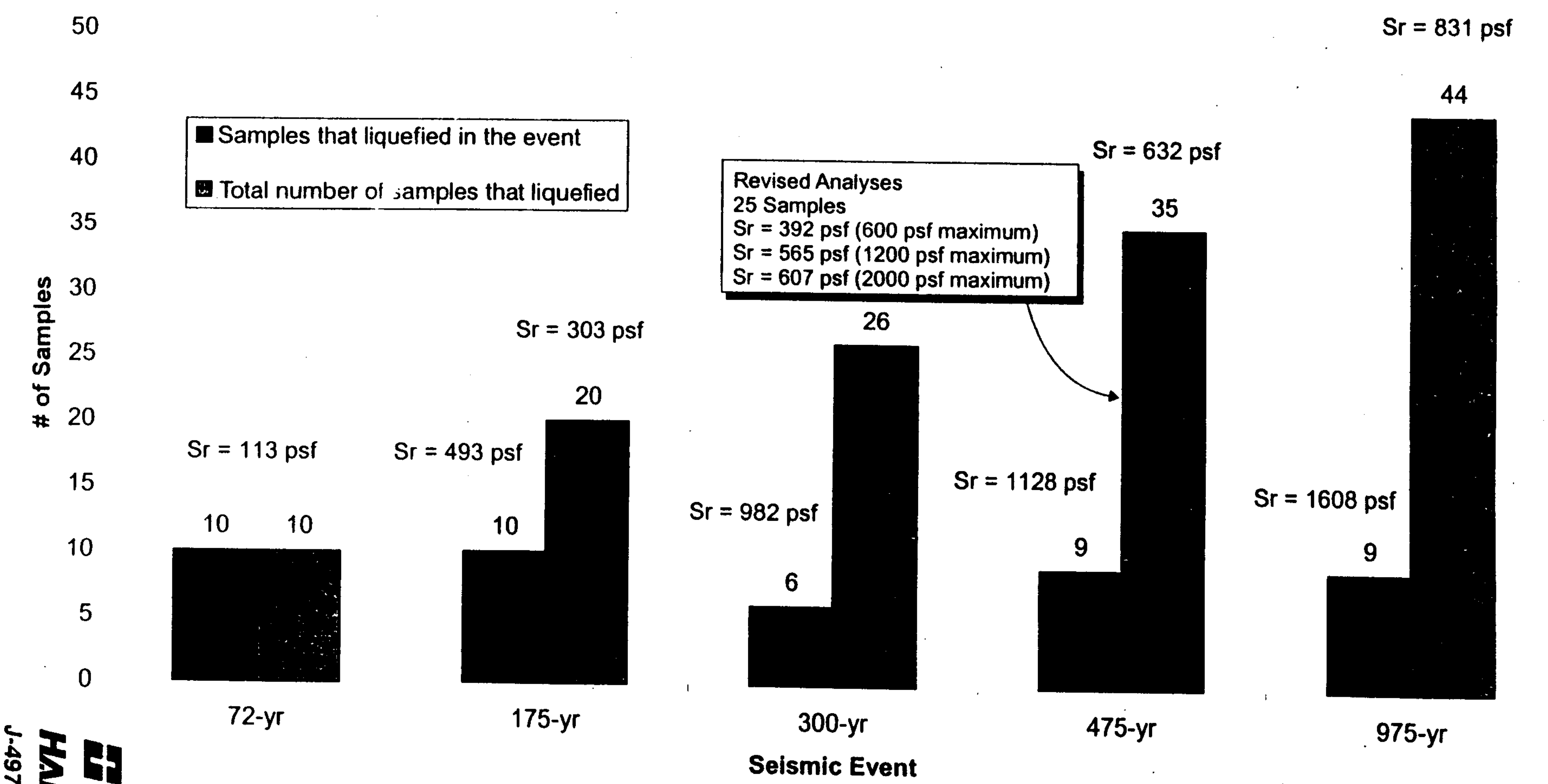
STIA 00567

DTN 3/7/01 497830c.cdr

AR 049962

Comparison of Liquefied Samples for Various Seismic Events

NSA Wall



Results are based on preliminary liquefaction and residual strength analyses except for the cross hatched area that shows revised results for the 475-yr seismic event.

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Figure 4

AR 049963

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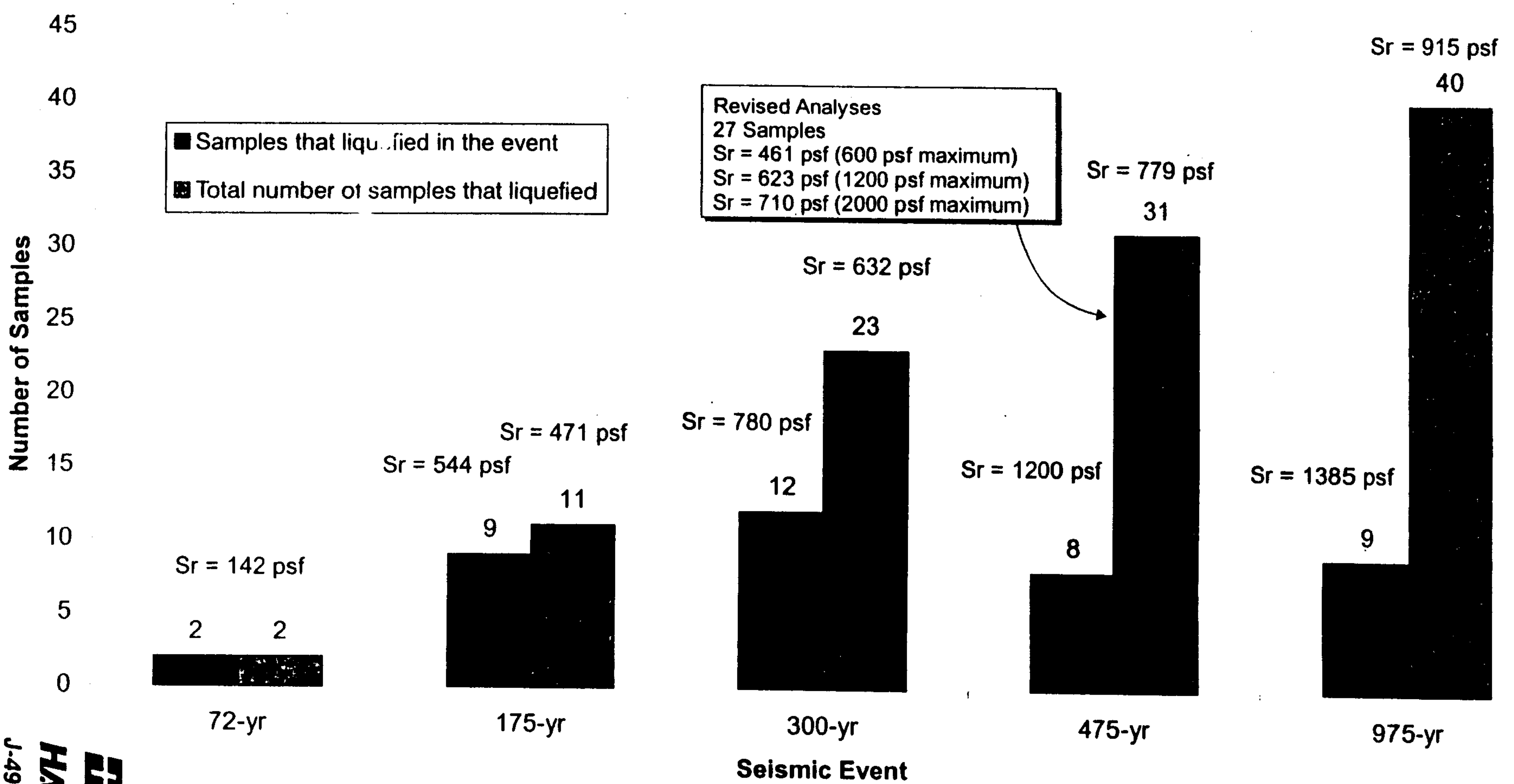
DTN 3/7/01 497830b cdr

Comparison of Liquefied Samples for Various Seismic Events

West Wall



Results are based on preliminary liquefaction and residual strength analyses except for the cross hatched area that shows revised results for the 475-yr seismic event.

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Figure 5

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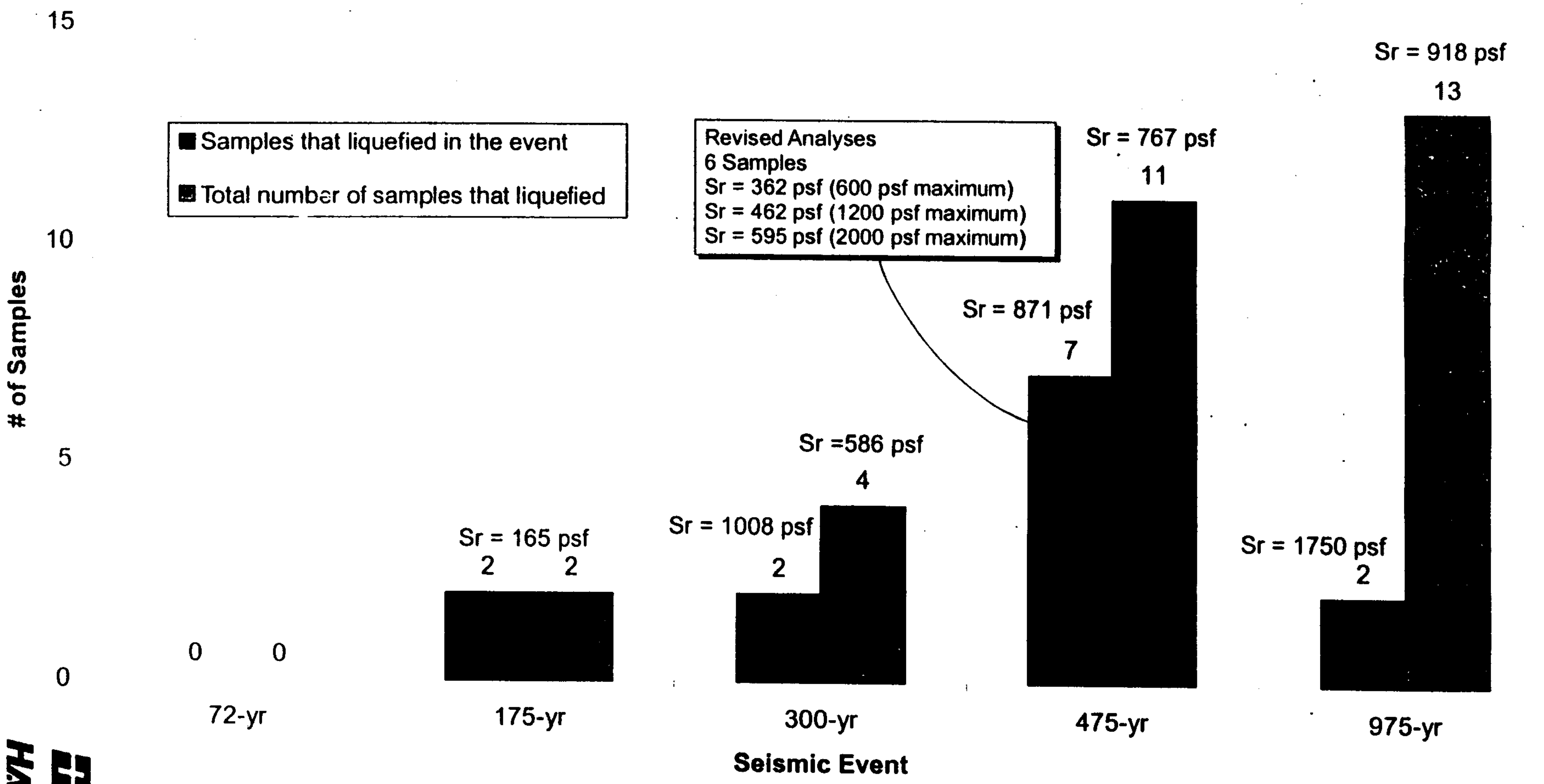
Comparison of Liquefied Samples for Various Seismic Events

South Wall



Results are based on preliminary liquefaction and residual strength analyses except for the cross hatched area that shows revised results for the 475-yr seismic event.

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Figure 6

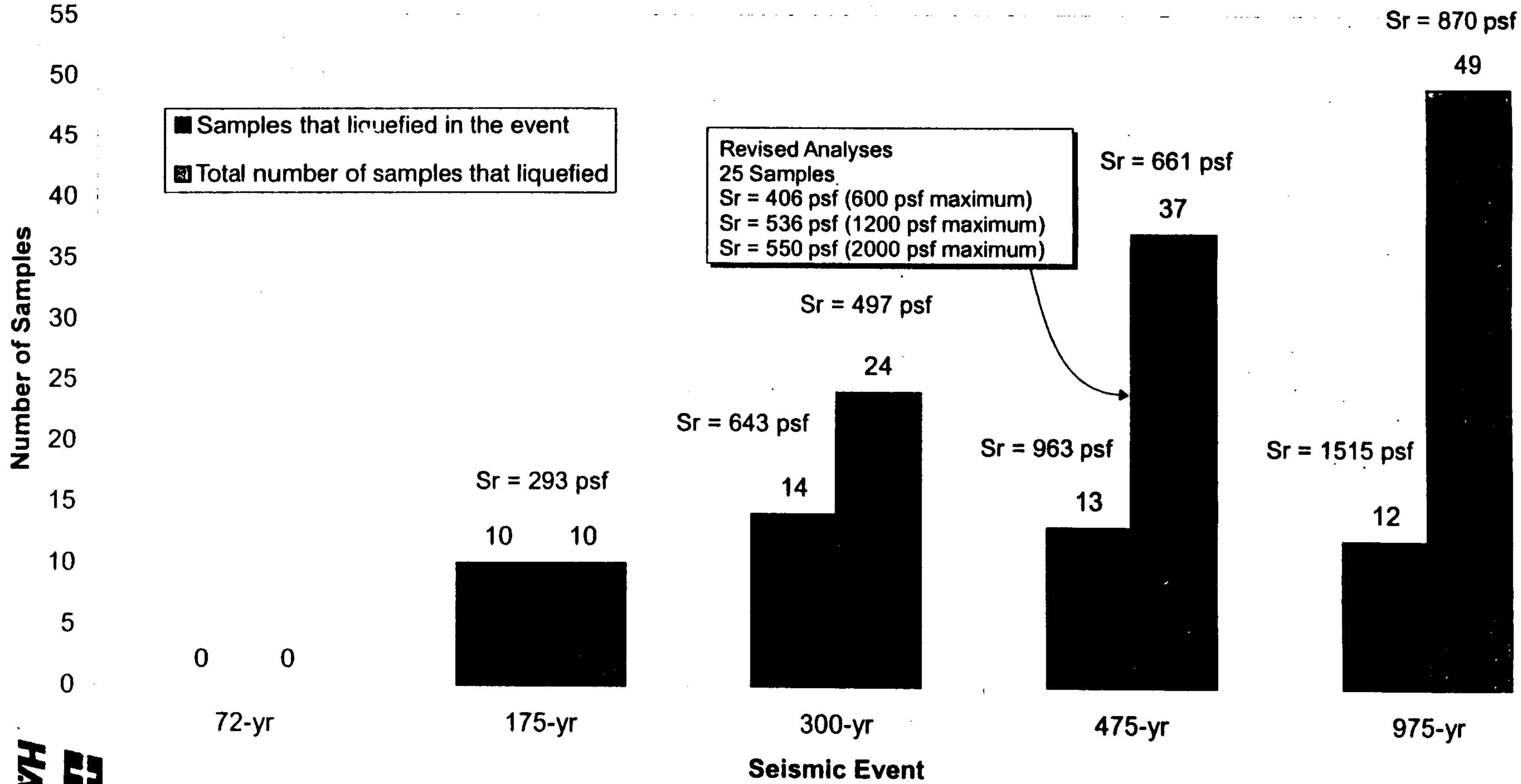
DTN 3/7/01 497830e cdr

AR 049965

STIA 00570

Comparison of Liquefied Samples for Various Seismic Events

2H:1V Slope



Results are based on preliminary liquefaction and residual strength analyses except for the cross hatched area that shows revised results for the 475-yr seismic event.

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J-4978-30 3/01

Figure 7

DTN 3/7/01 497830l cdr

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