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Anchorage

DATE: September 7, 2000

TO: Jim Thomson, P.E., HNTB

Boston

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

Chicago

RE: **Methods and Results of Liquefaction Analyses**
Third Runway Embankment
Sea-Tac, Washington
J-4978-28

Denver

This memorandum presents the methods and results of Hart Crowser's analysis of potential liquefaction and post-liquefaction residual strength for the proposed Third Runway embankment and retaining walls. We analyzed a total of 120 borings and their corresponding Standard Penetration Test (SPT) results. Logs of these borings can be found in previous subsurface conditions data reports (references are listed at the end of this memorandum, (see Civil Tech, 1997, 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).

Fairbanks

Jersey City

Potential for liquefaction, and resulting soil behavior is influenced by a number of factors. This memorandum is intended to document 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, (see for instance Hart Crowser, 2000d and 2000f).

Juneau

Long Beach

SUMMARY

The 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 between the NSA Wall and West Wall. 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 more evident when looking at specific areas. A statistical analysis was performed for each area on the SPT samples that

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were potentially liquefiable. 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.

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) were found to exist intermittently within the subgrade support area for all 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, was found to vary by location 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 SPT results, but were 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 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 workshop (NCEER, 1996) using SPT blow counts. The method is empirical, which means it is based on results of case studies of sites where liquefaction occurred. In this method, a calculated cyclic stress ratio (CSR) for the soil profile, induced by the design earthquake, is compared to a calculated cyclic resistance ratio (CRR), provided by the soil to resist liquefaction. Factor of safety (FS = CRR / CSR) against liquefaction is determined for each sample based on its SPT blow count. A factor of safety less than one 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:

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

In this equation, a_{max} is the peak horizontal acceleration expressed in comparison to gravity, g, σ_v is the total vertical stress at the sample depth, r_d is a stress reduction factor, *MSF* is a magnitude scaling factor, and σ_v' is the effective vertical stress at the sample depth. For this analysis we calculated effective stress based on existing groundwater conditions at the time of drilling.

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

Table 1 - Design Criteria for Various Seismic Events

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

These 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 site-specific probabilistic seismic hazard analysis (Hart Crowser, 1999c). We used engineering judgment to select corresponding values of magnitude, because the probability of exceedence does not directly correlate to a specific magnitude of a seismic event.

Cyclic Resistance Ratio (CRR)

The cyclic resistance ratio 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). 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 were corrected to $(N_1)_{60}$ blow counts by multiplying the field blow count by each of the following correction factors.

Overburden Pressure. This correction normalizes blow counts to an overburden pressure of approximately one atmosphere (1 ton/ft^2.) The correction factor C_N was calculated as follows:

$$C_N = \sqrt{2000/\sigma_v'} \le 2$$

where:

σ_v' is the effective vertical stress in pounds per square foot at the sample depth. A maximum value of 2 was used to keep shallow samples from having very large correction factors.

Energy Ratio. This correction normalizes blow counts to account for a variation in energy from drill rig to drill rig. The energy ratio was taken as 1.0 with an assumed hammer efficiency of 60%. (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% for depths greater than 10 feet. Using 55% instead of the assumed 60% efficiency would have reduced the measured blow counts by 8%).

Borehole Diameter. 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%.

Rod Length. 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 2.

Table 2 - 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. 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. I.M. Idriss with assistance from R.B. Seed developed the following recommendations (NCEER, 1996) to correct blow counts for the presence of fines.

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

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

$\alpha = 0$	for fines content ≤ 5%
α = exp[1.76-(190/fines content^2)]	for 5% ≤ fines content ≤ 35%
$\alpha = 5.0$	for fines content ≥ 35%
$\beta = 1.0$	for fines content ≤ 5%
β = [0.99+(fines content^1.5/1000)]	for 5% ≤ fines content ≤ 35%
$\beta = 1.2$	for fines content ≥ 35%

Figure 1 shows an empirical chart for evaluation of 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 the corrected blow count for clean sand $(N_1)_{60CS}$ to obtain the CRR for each SPT sample. A cutoff value of 2.0 was used for samples where an infinite CRR was calculated.

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 following embankment construction. To account for high overburden pressures, an additional correction factor was applied to the CRR before calculating factor of safety. This correction factor incorporates the nonlinear effect of decreasing cyclic resistance ratio with increasing confining pressure. In general, the calculated CRR decreases (or susceptibility to liquefaction relative to a CSR value increases) as confining pressure increases. This could produce an overly conservative prediction of true liquefaction potential if stress conditions after fill placement were used, because the empirical analysis does not explicitly consider the effect of consolidation that would occur due to the weight of the embankment. A decrease in void ratio due to consolidation under the weight of the embankment would reduce potential for liquefaction. We used the *in situ* stress conditions to calculate the overburden correction. We evaluated the stress conditions for the full embankment height after fill placement, but decided not to use this approach. Use of the full embankment pressure was deemed overly conservative even though sands have a relatively small compression index.

Identification of Nonliquefiable Soils

Samples which 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 next.

Fine-grained Soils. The beneficial resistance to liquefaction by soils with a fines (silt and/or clay) content greater than 35% 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 empirical criteria 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). Fined grained soils that satisfy all of the following criteria were deemed liquefiable. Therefore, if any one of these criteria was not met the soil was deemed nonliquefiable.

- (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.

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. Soil needs to be saturated in order to generate excess pore water pressures, 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 potentially liquefiable soils 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 onsite borings and test pits (see Hart Crowser 2000d and 2000f).

Post-Liquefaction Residual Strength

Results of the analysis described above were used to create a data set of liquefaction susceptible samples that was then used to evaluate post-liquefaction residual shear strength for soils at the site. Hart Crowser used the residual shear strength in stability analyses for embankment and retaining wall design. The post-liquefaction residual strength was calculated according to the empirical procedure developed by Seed and Harder (1990). This method relates the residual strength to an equivalent clean sand SPT blow count $(N_1)_{60\text{-}CS}$, as shown on Figure 2. The corrected clean sand blow count $(N_1)_{60\text{-}CS}$ is the $(N_1)_{60}$ blow count plus the value shown in Table 3.

Table 3 - Recommended Fines Correction for Residual Strength Calculation (Seed & 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.

- We used the middle of the ranges shown in Figure 2 to estimate average residual shear strength.

- We extrapolated Figure 2 to a maximum $(N_1)_{60\text{-}CS}$ value of 24, corresponding to an average residual shear strength of about 1750 psf, because the previous analysis indicated liquefaction susceptible soils with $(N_1)_{60\text{-}CS}$ values up to about 24. Any potentially liquefiable soils with adjusted blow counts higher than 24 were assigned a default maximum residual strength value of 1,750 psf.

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

Estimated Residual Strength

A total of 146 "samples" of potentially liquefiable soil were identified in the 120 borings that were evaluated. The following sections discuss these results for the site as a whole and by area.

As expected, our analysis found that 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, as well as the estimated post-liquefaction residual strength for the site as a whole. Figures 4, 5, 6 and 7 show the same type of information for different areas within the overall site. Comparison of these figures shows some minor variability but overall similar results. The apparent variability is probably at least partially a result of statistical uncertainty due to 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.

Attachments:
References
Figure 1 - Relationship Between Stress Ratio Causing Liquefaction and $(N_1)_{60}$ Values for Silty Sand for M = 7.5
Figure 2 - Relationship Between Corrected "Clean Sand" Blowcount $(N_1)_{60\text{-}CS}$ and Undrained Residual Strength (S_r) from Case Studies
Figure 3 - Comparison of Liquefied Samples for Various Seismic Events (NSA, West, South Walls & 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

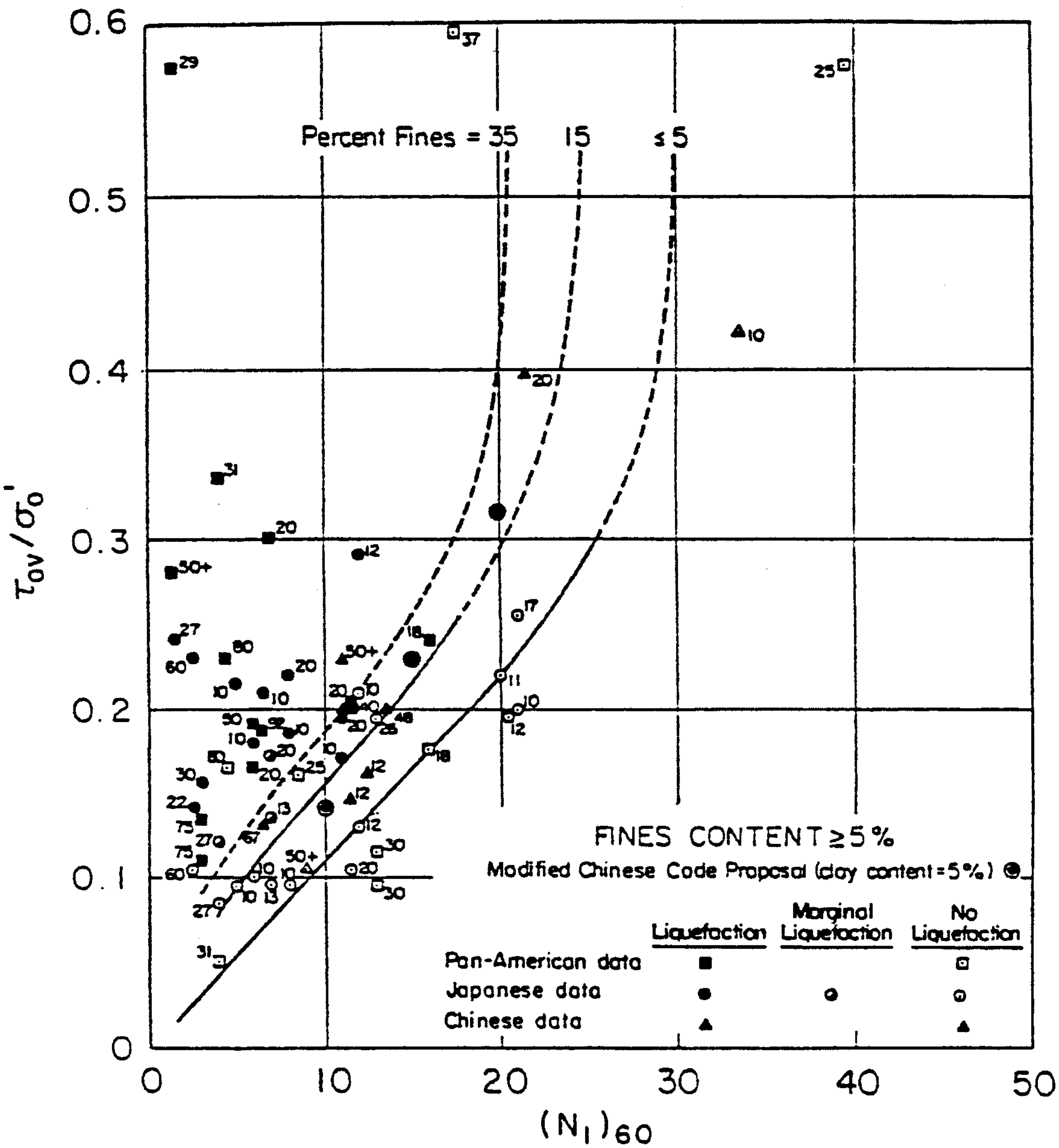

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Figure 1

Relationship Between Corrected "Clean Sand" Blowcount $(N_1)_{60\text{-}cs}$ and Undrained Residual Strength (S_r) from Case Studies

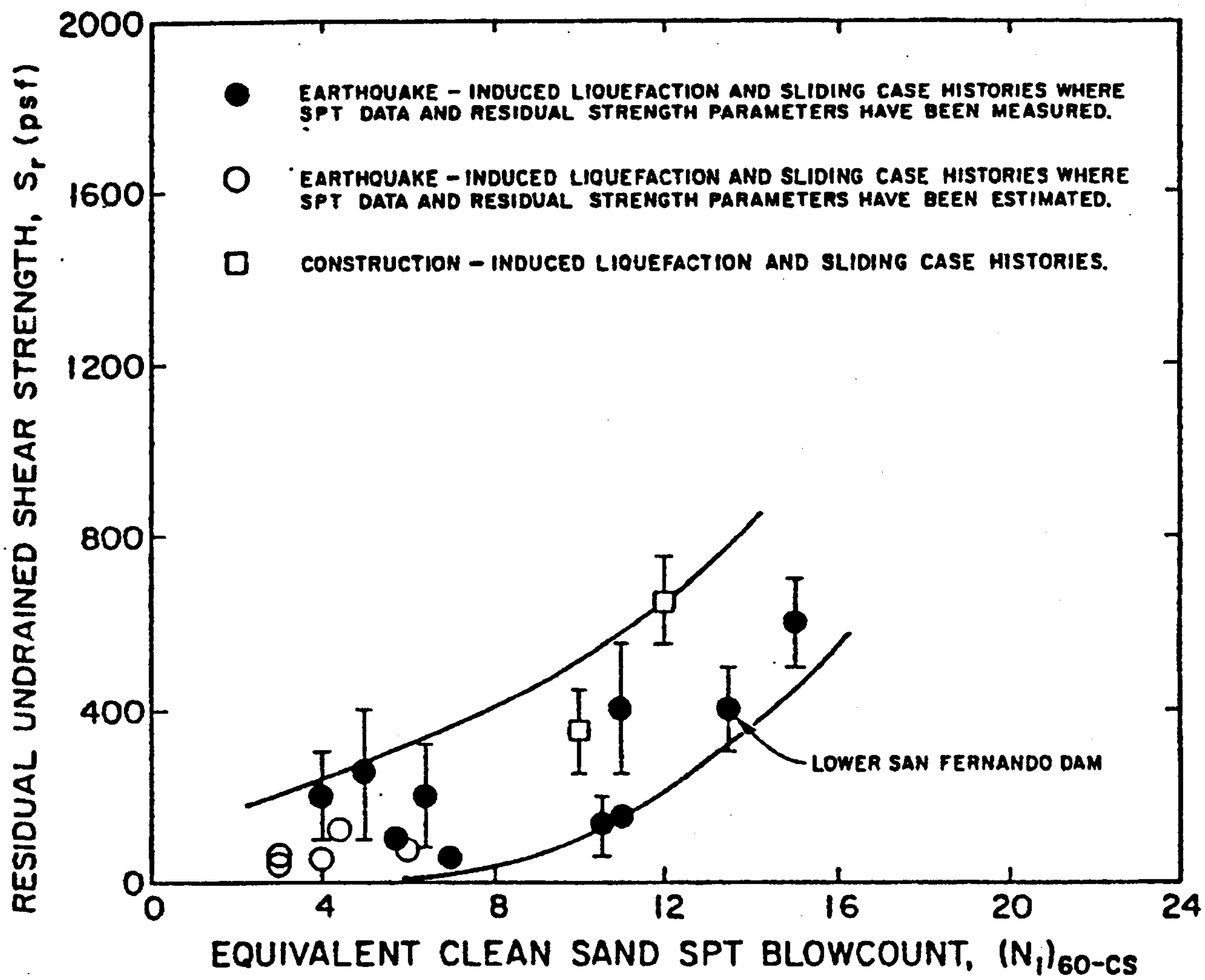

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Figure 2

Comparison of Liquefied Samples for Various Seismic Events

NSA, West, South Walls & 2:1 Slope

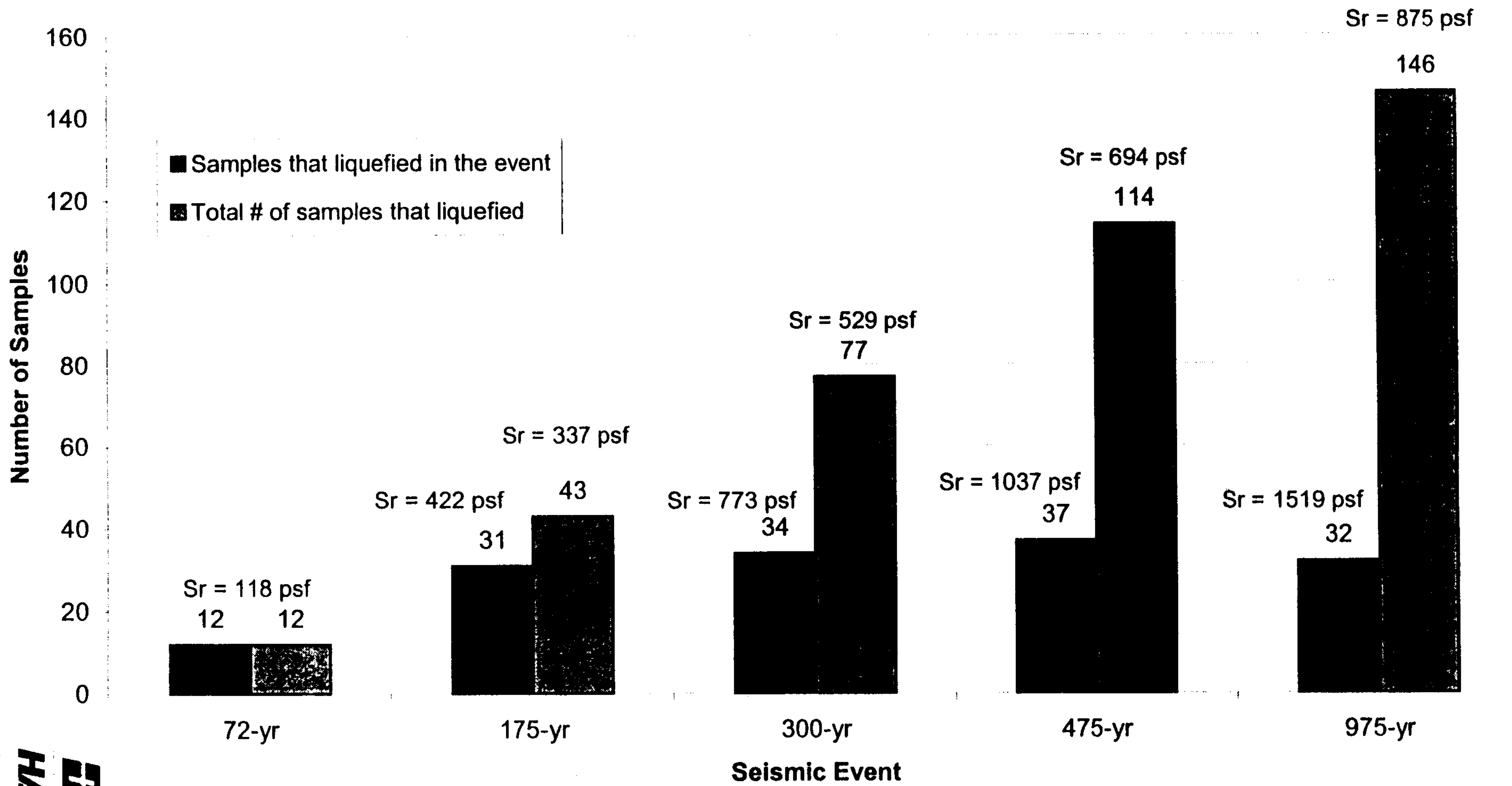

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Figure 3

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Comparison of Liquefied Samples for Various Seismic Events

NSA Wall

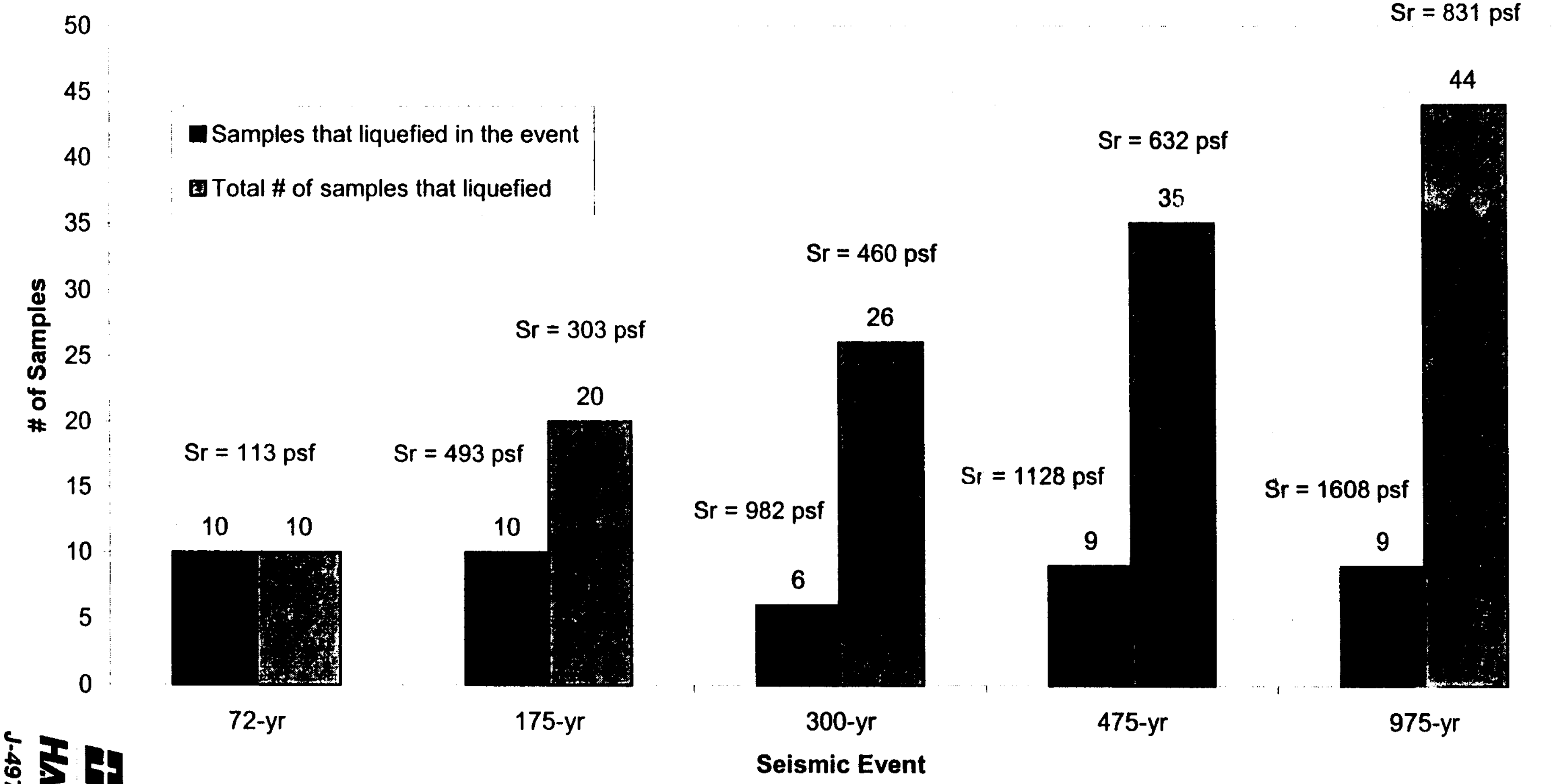

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Figure 4

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Comparison of Liquefied Samples for Various Seismic Events

West Wall

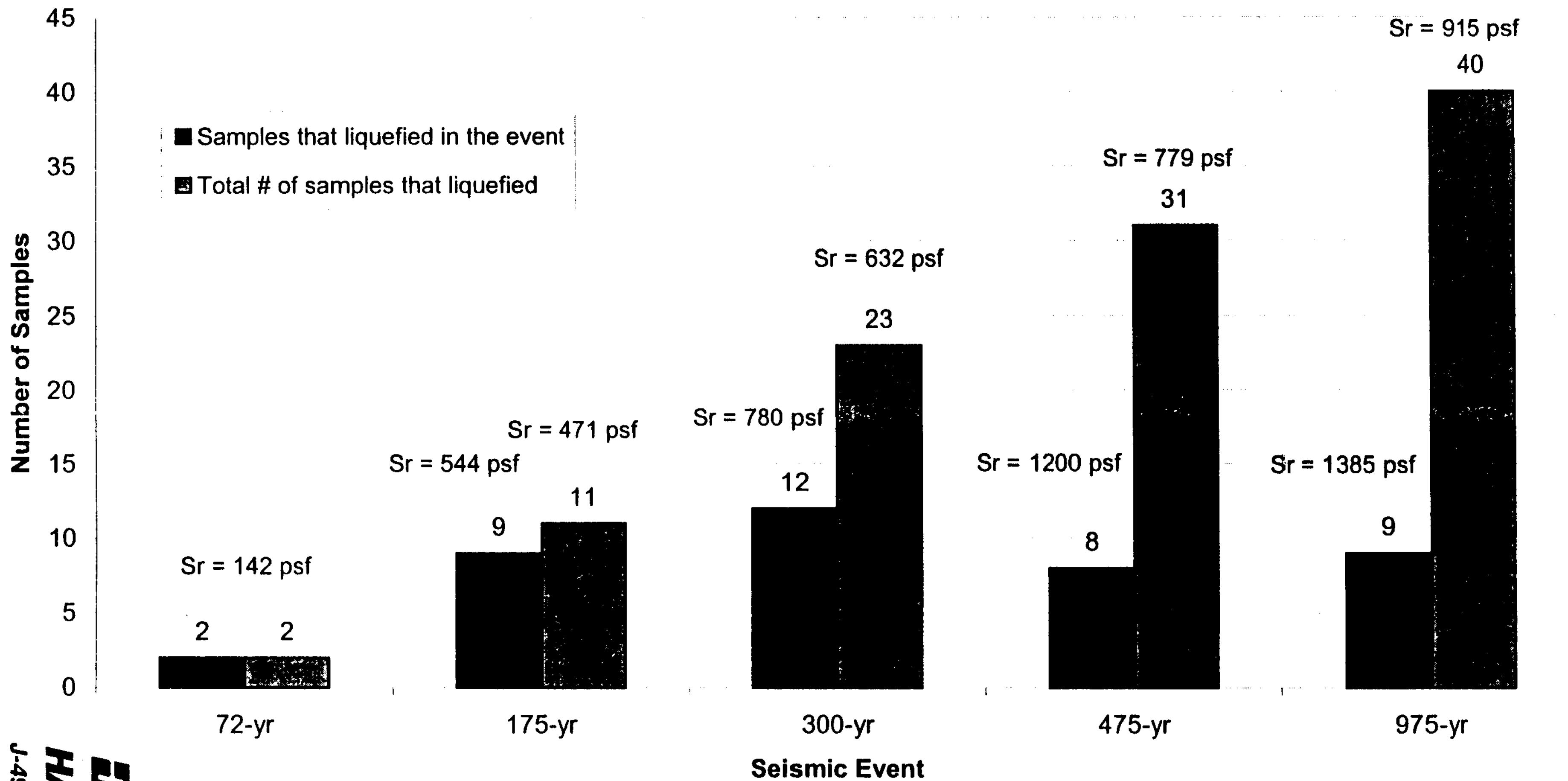

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Figure 5

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Comparison of Liquefied Samples for Various Seismic Events

South Wall

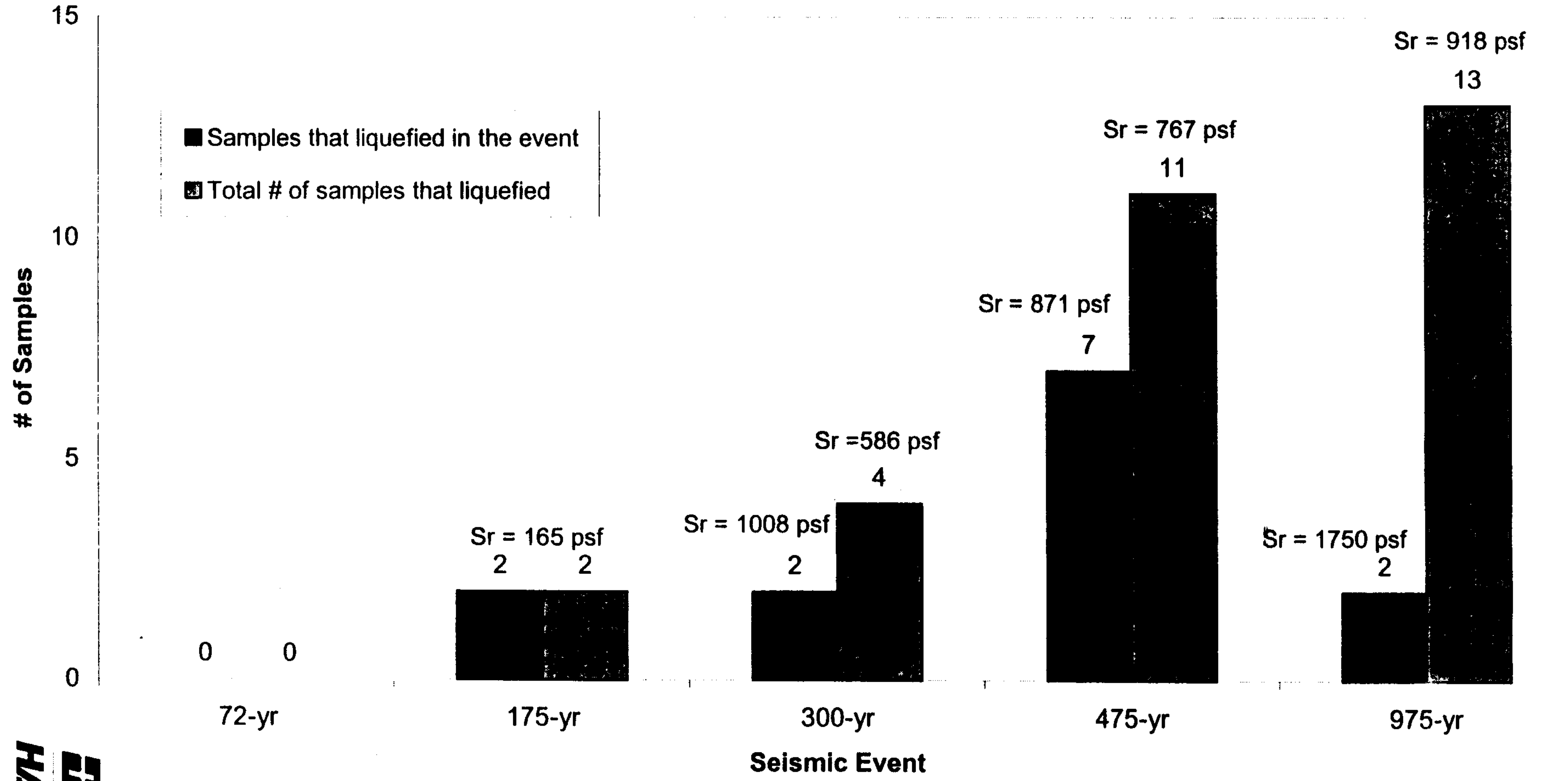

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Figure 6

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Comparison of Liquefied Samples for Various Seismic Events

2H:1V Slope

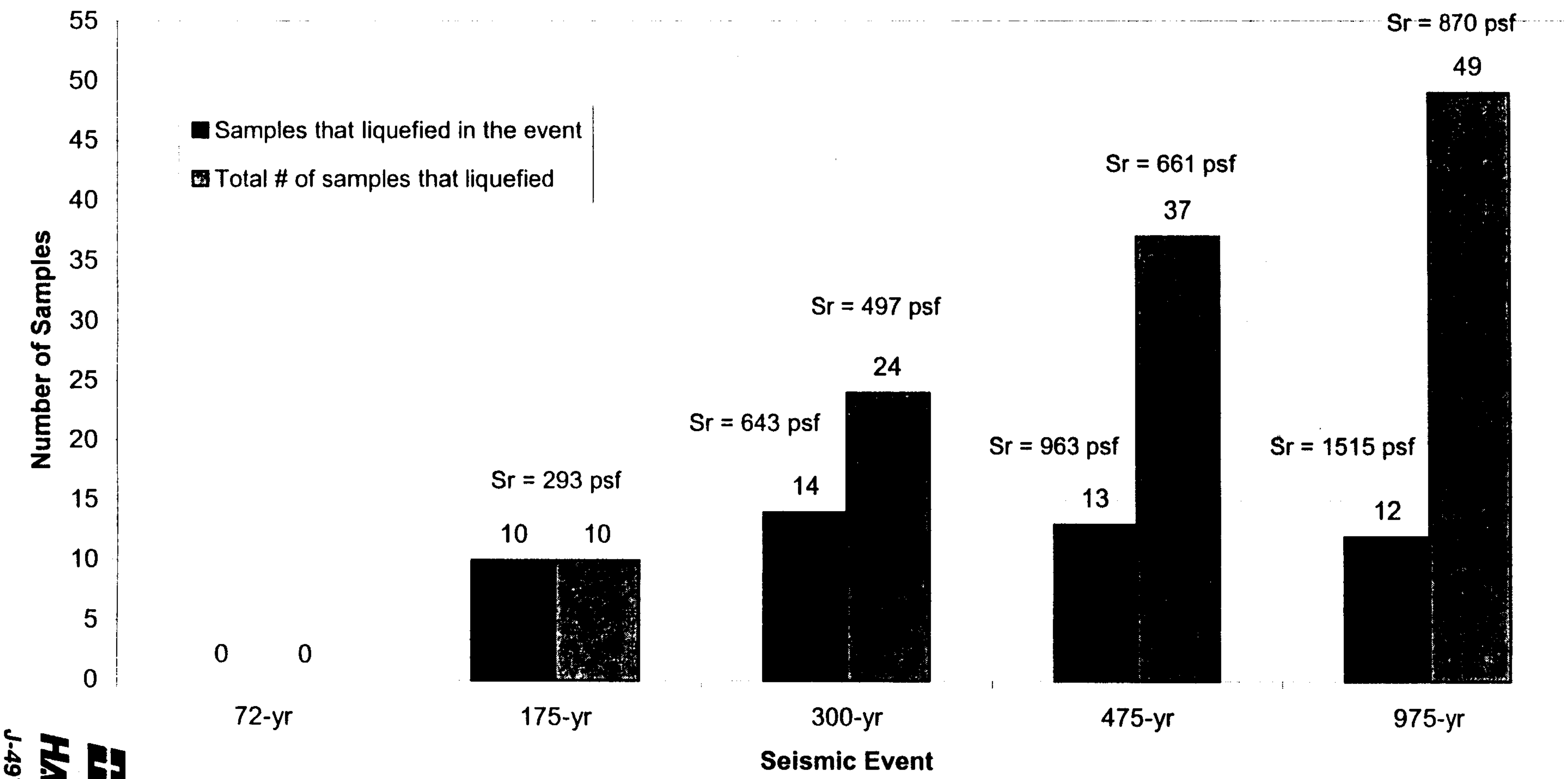

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Figure 7

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