Preliminary Guidelines for Liquefaction Assessment using Shear Wave Velocity

by

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ABSTRACT

This paper presents preliminary guidelines for assessing the liquefaction resistance of soils using small-strain shear wave velocity. The guidelines are based on field performance data from 17 earthquakes and in situ shear wave velocity measurements at over 40 different sites in soils ranging from sandy gravel with cobbles to profiles including silty clay layers. Additional data are needed from denser soil sites shaken by stronger ground motions to further validate the proposed liquefaction potential boundaries.

KEYWORDS: Building technology; earthquake engineering; in situ measurements; seismic testing; shear wave velocity; soil liquefaction.

1. INTRODUCTION

Currently there is no widely accepted procedure for assessing the liquefaction resistance of granular soils using small-strain shear wave velocity, V_S. A number of V_S-based procedures have been proposed during the past decade. These procedures were developed from analytical studies (Stokoe et al. 1988; Andrus 1994), laboratory cyclic triaxial test results (Tokimatsu et al. 1992), or a limited amount of field performance data (Robertson et al. 1992; Kayen et al. 1992; Lodge 1994).

The use of V_S as a field index of liquefaction resistance is justified since both are influenced by density, confinement, stress history, and geologic age. The advantages of using V_S include:

- V_S can be accurately measured in situ using a number of techniques such as the seismic crosshole test, the Seismic Cone Penetration Test (SCPT), or the Spectral-Analysis-of-Surface-Wave (SASW) test;
- Measurements are possible in soils that are hard to sample, such as gravelly soils, and at sites where borings or soundings may not be permitted, such as many landfills;
- Measurements can be performed in small laboratory specimens, making direct comparisons between laboratory and field behavior possible;
- V_S is directly related to small-strain shear modulus, G_{max}, a required parameter in analytical procedures evaluating dynamic shearing strain in soils; and
- For large earthquake magnitudes and long durations of shaking the cyclic shear strain needed for liquefaction decreases and approaches the threshold strain in sand ($\approx 0.02\%$), thus making it possible to conduct more analytical evaluations of liquefaction using V_S and G_{max} as basic parameters (Dobry et al. 1981; Seed et al. 1983).

The two main limitations of using V_S to evaluate liquefaction resistance are the lack of a sample for identifying non-liquefiable fine-grained soils, and its high sensitivity to weak interparticle bonding, caused by aging or cementation, which is eliminated at large strains. Thus, a limited amount of drilling and penetration testing should be performed to identify weakly cemented soils and non-liquefiable clayey soils.

This paper presents preliminary guidelines, currently under development (Andrus and Stokoe 1996), for assessing liquefaction resistance using V_S. The guidelines are based on field performance data from 17 earthquakes and in situ V_S measurements at over 40 different sites (112 test arrays), resulting in a total of 177 liquefaction and non-liquefaction case histories. The 17 earthquakes are listed as follows.

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Earthquake	Magnitude
1906 San Francisco, California	7.7
1964 Niigata, Japan	7.5
1975 Haicheng, PRC	7.3
1979 Imperial Valley, California	6.5
1981 Westmorland, California	5.9
1983 Borah Peak, Idaho	6.9
1986 Event LSST4, Taiwan	6.5
1986 Event LSST7, Taiwan	6.5
1986 Event LSST8, Taiwan	6.2
1986 Event LSST12, Taiwan	6.2
1986 Event LSST13, Taiwan	6.2
1986 Event LSST16, Taiwan	6.2
1987 Elmore Ranch, California	5.9
1987 Superstition Hills, California	6.5
1989 Loma Prieta, California	7.0
1993 Hokkaido-nansei-oki, Japan	7.8
1995 Hyogo-ken Nanbu, Japan	6.9

2. ESTIMATING LIQUEFACTION RESISTANCE

The most widely used procedure for evaluating the liquefaction resistance of granular soils is the stress approach by Seed and his colleagues (1971, 1982, and 1985) based on modified *Standard Penetration Test (SPT) blow count.* Following the general format of this approach, Robertson et al. (1992) developed a procedure based on modified V_S.

2.1 Cyclic Stress Ratio

In the stress approach, liquefaction resistance is related to the ratio of cyclic shear stress to initial vertical effective stress, called cyclic stress ratio. The cyclic stress ratio, τ_{av}/σ'_v, at a particular depth in a level soil deposit can be expressed as (Seed and Idriss 1971):

$$\tau_{av}/\sigma'_v = 0.65\,(a_{max}/g)\,(\sigma_v/\sigma'_v)\,r_d \qquad (1)$$

where τ_{av} is average cyclic shear stress caused by the earthquake, σ'_v is initial effective vertical (overburden) stress, σ_v is total overburden stress, a_{max} is peak horizontal ground surface acceleration, g is acceleration of gravity, and r_d is a shear stress reduction factor. The factor r_d can be estimated by:

$$r_d = 1 - 0.015z \qquad (2)$$

where z is depth in meters.

2.2 Modified Shear Wave Velocity

Robertson et al. (1992) modified V_S by:

$$V_{S1} = V_S\,(P_a/\sigma'_v)^{0.25} \qquad (3)$$

where P_a is a reference stress, typically 100 kPa, and σ'_v is in kPa. They chose to modify in terms of σ'_v to follow the traditional way SPT and Cone Penetration Test (CPT) data are modified. The exponent of 0.25 was selected based on laboratory studies (Hardin and Drnevich 1972). Equation 3 assumes that the coefficient of earth pressure at rest equals 1.

2.3 Development of V_S-based Charts

Field performance data for magnitude 7 earthquakes are shown in Figure 1. The occurrence of liquefaction is based on the *appearance of sand boils, ground cracks and* fissures, or ground settlement. For each case history, the shear wave velocity shown is the average of values reported for the most vulnerable layer, and modified using Equation 3. Since most attenuation relationships are for the randomly oriented horizontal component of ground motion rather than the larger of two horizontal components, the values of a_{max} used to estimate the cyclic stress ratios are based on the randomly oriented horizontal component that would have occurred at the site in the absence of liquefaction.

The preliminary liquefaction potential boundary shown in Figure 1 is drawn to pass through the origin, include nearly all liquefaction case histories, and become vertical at higher values of V_{S1}. A boundary passing through the origin is supported by the following relationship between cyclic stress ratio and V_{S1} for a line of constant average cyclic shear strain, γ_{av}, (Dobry 1996):

$$\tau_{av}/\sigma'_v = f(\gamma_{av})\,V_{S1}^{\,2} \qquad (4)$$

A *vertical boundary at higher values of* V_{S1} *is* justified since dense granular soils exhibit dilative behavior even at large strains. Based on engineering judgment, a value of about 280 m/s is assumed as an upper bound for most contractive soil types. This potential boundary is similar to the boundaries proposed by Kayen et al. (1992) and Lodge (1994) using field performance data primarily from the 1989 Loma Prieta, California earthquake.

For earthquakes with magnitude between 5.5 and 8.5, liquefaction potential boundaries can be formed using *all available field* performance data, as shown in Figure 2. The preliminary magnitude scaling factors used to construct these boundaries are listed in column 6 of Table 1. These scaling factors compare well with SPT-based factors developed in recent years by other investigators, also listed in Table 1. The boundaries for magnitude 6, 6.5 and 7.5 earthquakes shown in figure 2 correctly predict liquefaction at all sites where surface manifestations of liquefaction were observed. The few liquefaction cases that lie slightly below the liquefaction boundary for magnitude 7 earthquakes are sites where the ground is sloping and/or where liquefaction may have been marginal.

It is interesting to note that similar liquefaction potential boundaries can be constructed using V_S and a_{max} directly, as shown in figure 3. The number of non-liquefaction case histories incorrectly classified in figure 3 is more or less the same as the number of non-liquefaction case histories incorrectly classified in figure 2. Figure 3 suggests that liquefaction will never occur in any earthquake if the shear wave velocity in the upper 12 m of soil exceeds about 240 m/s. A similar but preliminary conclusion was reached by Seed et al. (1983). They concluded that liquefaction will never occur if the shear wave velocity in the upper 15 m of soil exceeds about 366 m/s. The boundaries shown in figure 3 provide a simpler assessment procedure than the stress-based approach. However, their application should be limited to initial screening of sites with characteristics similar to the database (depth of most vulnerable layer less than 12 m, and depth of water table 0.5-7.6 m).

3. CONCLUSIONS

The case histories presented represent a wide range of soil types, ranging from sandy gravel with cobbles to profiles including silty clay layers. Thus, for current use, the preliminary liquefaction potential boundaries presented in figures 1 and 2 are recommended for all liquefiable soil types. Non-liquefiable soil types include fine-grained soils with clay contents greater than 15%, liquid limits greater than 35%, or moisture contents less than 90% of the liquid limit (Seed and Idriss 1982). The potential boundaries presented in figure 3 are suggested for initial screening of sites only where conditions are similar to the database, which can be viewed from the figure. Additional well-documented case histories of all types of soil that have and have not liquefied during earthquakes should be compiled, particularly denser deposits shaken by stronger ground motions.

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Table 1. Magnitude Scaling Factors Obtained by Various Investigators (modified after Youd 1996)

Moment Magnitude, M_w (1)	Magnitude Scaling Factor				
	Seed and Idriss (1982) (2)	Ambraseys (1988) (3)	Youd (1996) (4)	Arango (1996) (5)	This Paper (6)
5.5	1.43	2.86	3.26	2.25 - 3.00	2.75
6.0	1.32	2.20	2.15	1.75 - 2.00	2.15
6.5	1.19	1.69	1.50	1.50 - 1.57	1.65
7.0	1.08	1.30	1.20	1.25	1.25
7.5	1.00	1.00	1.00	1.00	1.00
8.0	0.94	0.67	0.90	0.75	0.75
8.5	0.89	0.44	0.77	0.62	0.60

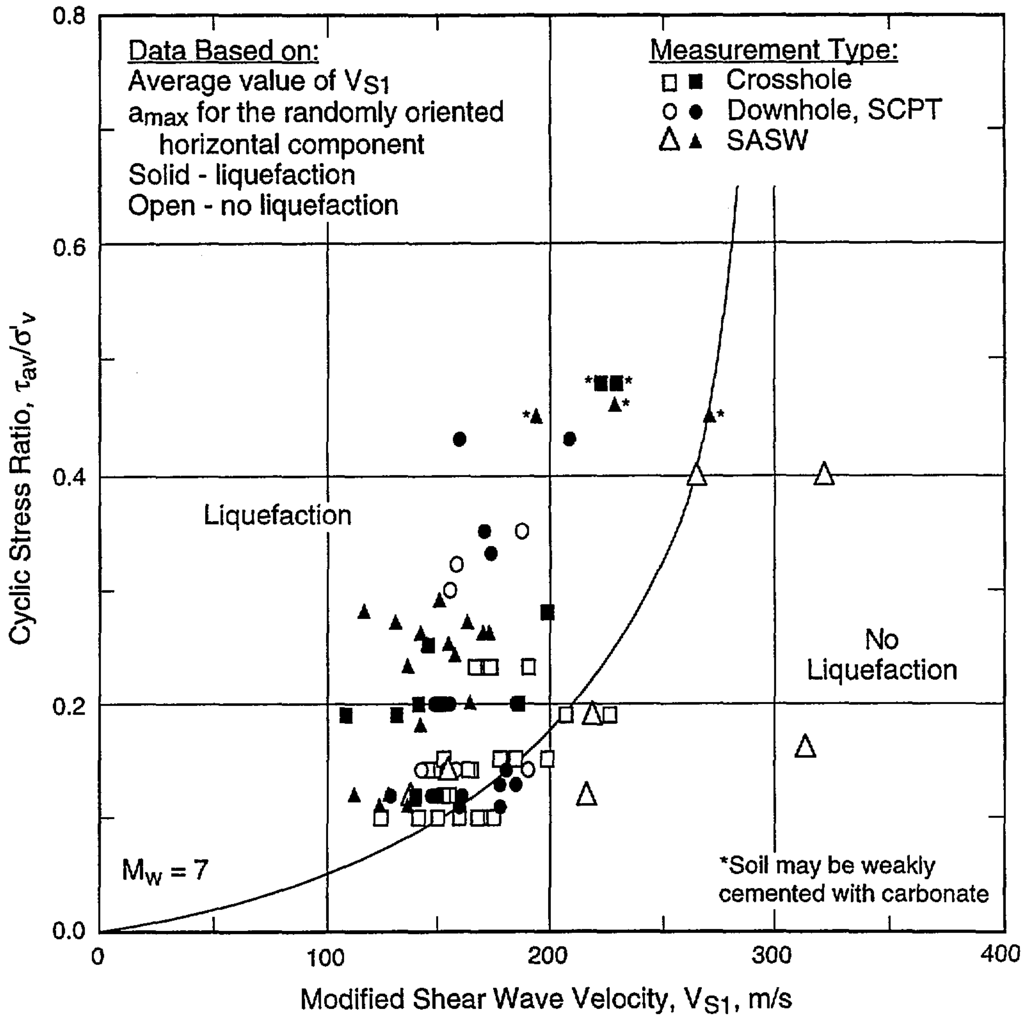

Figure 1. Preliminary Liquefaction Potential Boundary Based on Modified Shear Wave Velocity and Cyclic Stress Ratio with Results from Magnitude 6.9 to 7.0 Earthquakes.

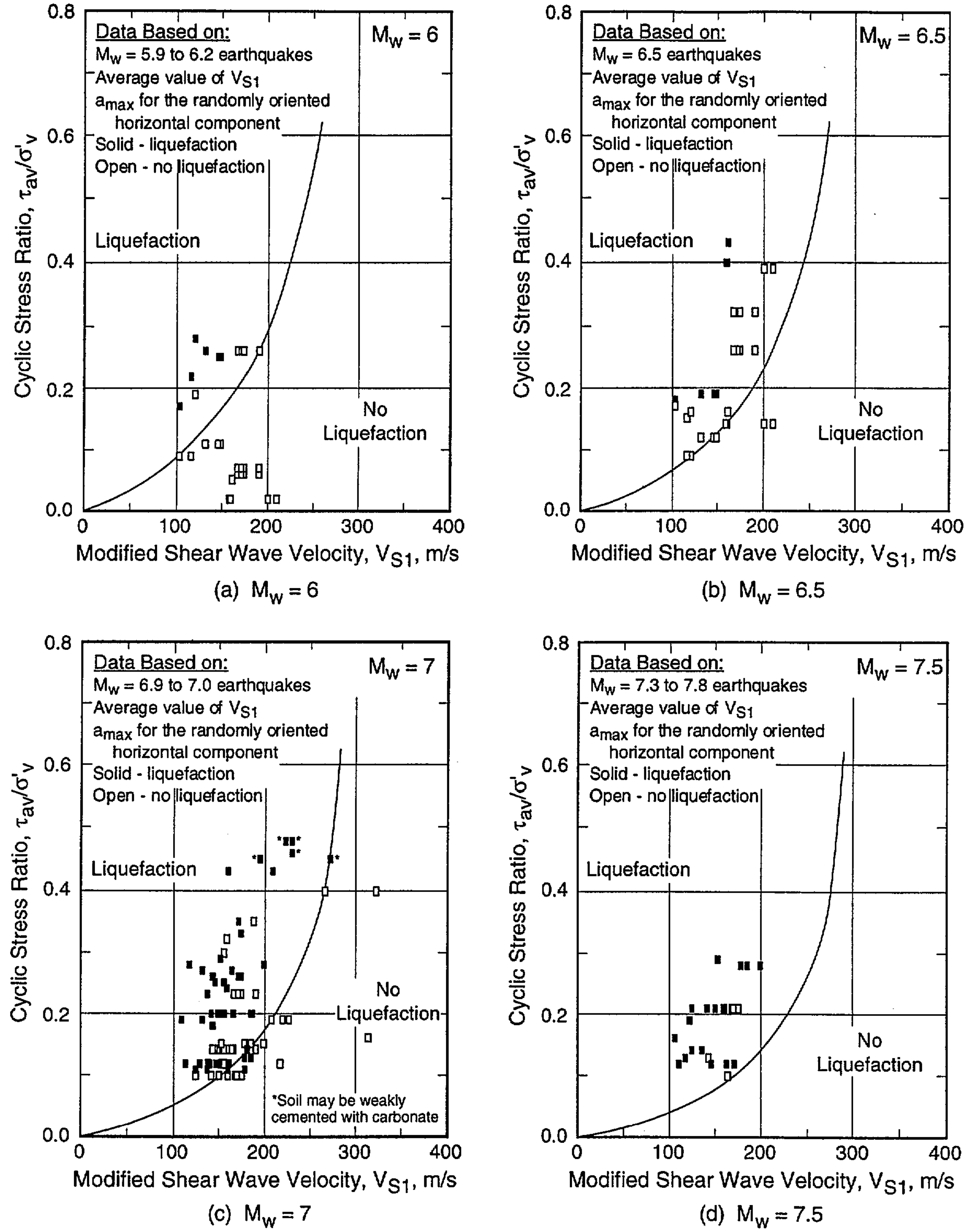

Figure 2. Preliminary Liquefaction Potential Boundaries Based on Modified Shear Wave Velocity and Cyclic Stress Ratio with Results from 17 Earthquakes.

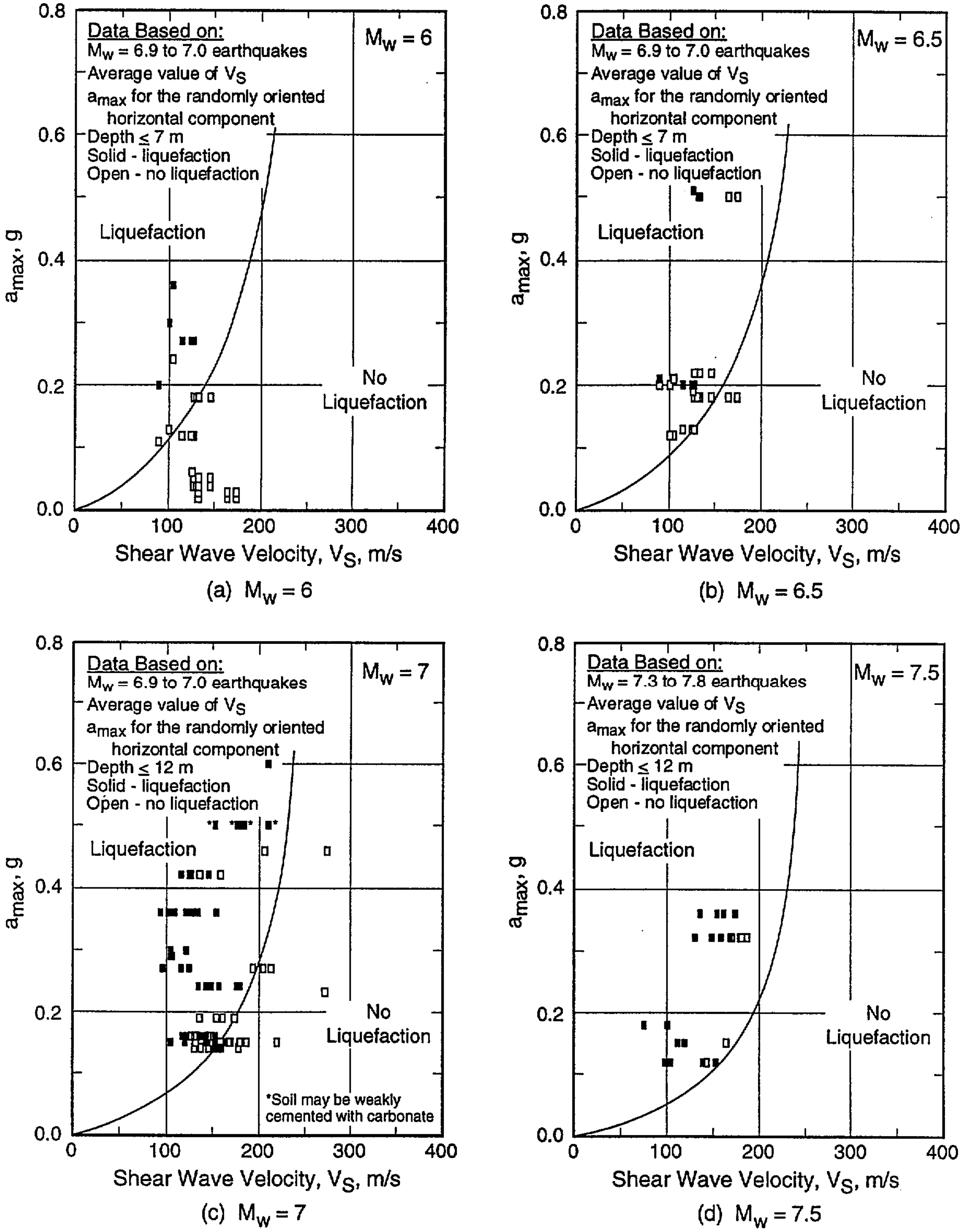

Figure 3. Preliminary Liquefaction Potential Boundaries Based on Shear Wave Velocity and Peak Horizontal Ground Surface Acceleration with Results from 17 Earthquakes.

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