Liquefaction criteria for New England considering local SPT practice and fines content

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ABSTRACT: Historical seismicity and earthquake source mechanisms in New England were reviewed. A design earthquake of M = 6.5 with peak ground acceleration of 0.12 g was selected for the region. Liquefaction criteria for New England were developed following the methodology suggested by Seed et al (1985), and taking into account the regional SPT practice as well as the effect of fines content. Use of the proposed criteria requires only SPT, groundwater level and representative grain size distribution data to check liquefaction susceptibility of a particular site in New England.

1 REGIONAL SEISMICITY

1.1 General

New England and southeastern Canada are generally considered within the same larger seismo-tectonic region. The first documented earthquake in the region, an event of MMIX intensity in 1534, was located in the lower St. Lawrence Valley. Pulley (1982) reported that since then over 3,000 earthquakes have been documented in the region. Up to 1975 when the installation of the M.I.T. seismic network seismometers was started, intensity of the earthquakes has been estimated based on descriptive reports of the ground shaking and the associated damage.

A review of the historical seismicity (1534-1975) reveals that earthquakes have occurred in almost every subregion in New England. However, seismic activity has been more outstanding in the lower St. Lawrence River Valley, southern New Hampshire to eastern Massachusetts, northern New York to western Quebec and coastal Maine. Interestingly, instrumental data obtained since 1975 suggest a very similar picture for the seismically more active areas in New England.

1.2 Significant earthquakes

The following significant earthquakes, that is those with epicentral intensities greater than MMVII or magnitudes of $M \geq 5$, have been documented in New England (Pulley 1982, Algermissen 1983): November 1727 and November 1757 - Cape Ann, Mass. (MMVII and MMVIII, respectively), May 1791 - East Haddam, Ct. (MMVIII), October 1817 - Woburn, Mass. (MMVII-VIII), 20 and 24 December 1940 - Ossipee, N.H. (both MMVII, M = 5.4), 10 April 1962 - Middlebury, Vt. (MMV, M = 5.0), and 18 January 1982 - Gaza, N.H. (M = 4.8). Also, 9 and 11 January 1982 - New Brunswick earthquakes (M = 5.7 and 5.4, respectively) were strongly felt in eastern Maine with minor damage.

1.3 Source mechanisms

Although nearly 70 earthquakes with epicentral intensities of MMV or greater have been documented in New England and the immediate vicinity since early 1600s, the source mechanisms of these seismic events are very little understood at the present.

None of the documented earthquakes in New England is known to have been accompanied by surface fault movement, and no faults have yet been established to be active in the region (Barosh 1979). Based on available data Sanders et al (1981) concluded that all faults of the southern New England are "dead" and belong to a pre-Wisconsin tectonic regime.

Northeast United States is classified as an intraplate (midplate) region. From

Nuttli's (1983) model for the minimum depth of intraplate earthquakes Acharya (1986) estimated that damaging earthquakes such as M = 6 would occur at a minimum focal depth of about 10 km. Acharya (1986) then suggested that for such earthquakes in the eastern United States there would be no surface faulting.

Mareschal et al (1986) suggested that even though the stress field in the midplate litosphere is rather uniform, local changes due to crustal thickness inhomogeneities and/or density heterogeneities may cause local nonuniformities and thus seismic activity. For example such conditions exist on the borders of Mesozoic volcanic centers in the White Mountains, N.H. (Barosh 1979). As a matter of fact, Simmons (1977) strongly claimed that the larger 1727, 1755 Cape Ann and 1940 twin Ossipee earthquakes were associated with those White Mountain magma series plutons that are relatively younger, more magnetic and have higher than normal gravity. The presence of these cylindrical plutons causes local stress increases and reliefs which lead to seismic activity.

As preliminary results of a six year (1976-1982) research on New England seismotectonics, Barosh (1981) observed that seismic activity in the eastern United States and adjacent Canada appears to be caused by minor adjustments of the earth's crust primarily in low-lying areas along the Cretaceous continental margin. In particular, he noted that in New England the great majority of the earthquakes occurs in areas below an altitude of about 200 m., concentrated along the coast at bays and major river mouths and extending inland along the river valleys. Barosh (1981) added that river valleys and bays are deposited with thick unconsolidated sediments which may further amplify the seismic intensity.

Hays (1986) also pointed out that soil-rock columns in many parts of the eastern United States, including the greater Boston basin have physical properties that will cause local site amplification of earthquake induced ground motions. It should be noted that such areas subject to site amplification in New England may also be often suspect for potential liquefaction.

2 DESIGN EARTHQUAKE FOR NEW ENGLAND

2.1 General

Historically, The National Building Code, NBC, of the American Insurance Association (1976) has been used most extensively in the northeast United States (Berg 1982). NBC's (1976) seismic zone map includes most of New England in Zone 2, having a zone coefficient, Z, of 0.5 in the formulation for the total lateral force or the base shear. Only, northeasterly portion of Maine, and southwesterly corner of Connecticut are included in Zone 1 with Z = 0.25. In the Basic Building Code (1980), BOCA, which is adopted in Connecticut, zone coefficients are identical to those in the NBC (1976).

Both the NBC (1976) and BOCA (1981) traditionally follow the recommendations of the Structural Engineers Association of California, SEAOC, for seismic design provisions. In SEAOC's (1986) recently proposed revised seismic zone map, all New England states are included in Zone 2, having a zone factor of 0.15. Only, approximately northerly half of Maine and most northerly portions of New Hampshire and Vermont are included in Zone 1, having a zone factor of 0.10.

In the nationally more widely adopted Applied Technology Council's report, Tentative Provisions for the Development of Seismic Regulations for Buildings, ATC 3-06 (1978), regional seismicity is prescribed by a seismicity index and two coefficients. The coefficients A_a and A_v characterize the short and long period components of the ground motion, respectively. All New England states are categorized with the same seismicity index and with $A_a = 0.10$ and $A_v = 0.15$.

In parallel to the nationally recognized ATC's recommendations, it is assumed that all New England states may be assigned with the same design earthquake within the scope of this study.

2.2 Design earthquake for Massachusetts

The Massachusetts State Building Code (1980), the Code, is the first building code in the United States that contains specific soil and foundation design provisions for earthquake induced ground motions (Luft & Simpson 1979). Recommendations by the Massachusetts Seismic Advisory Committee, MSAC, including guidelines to assess susceptibility of saturated, clean sands to potential liquefaction were incorporated in the Code in January 1975.

For seismic design analysis of buildings, MSAC selected a nominal design earthquake characterized by a peak ground acceleration of 0.12 g on firm soil, and an epicentral intensity between MMVII and MMVIII (Luft & Simpson 1979). In its more recent discussion of a set of revisions for earthquake resistant design of foundations of buildings MSAC (1983) considered a design earthquake having a maximum ground acceleration of 0.12 g on firm soil and a magnitude of 6.5.

2.3 Design earthquake for New England

Within the framework of this study and consistent with the currently (1986) adopted design earthquake for Massachusetts, a design earthquake having a peak ground acceleration of 0.12 g on firm soil and a magnitude of 6.5 was assumed as applicable to the New England region in general.

3 CURRENT (1986) LIQUEFACTION PROVISIONS FOR MASSACHUSETTS

At the present (1986) the criteria for checking the susceptibility of saturated, clean medium to fine sands to liquefaction with level ground are included in the Code's (1980) Article 720.4 and Figure 720. These criteria are reproduced in Figure 1 and 2 for groundwater levels at ground surface and at a depth of 10 ft., respectively. Apparently, the subject liquefaction curves were developed based on Castro's (1975) relatively conservative upper bound curve (envelope) as discussed by Soydemir & LeCount (1981).

A revised set of liquefaction curves based on Seed et al's (1983) upper bound curve were recently recommended by MASC (1983) and is currently under consideration by the professional community.

4 FACTORS AFFECTING THE SPT RESULTS

4.1 General

Assessment of susceptibility of saturated sand deposits to potential liquefaction relative to the standard penetration test, SPT, data has been recognized as an effective and economical approach in the current (1986) practice of geotechnical earthquake engineering (National Research Council 1985, Seed & Idriss 1982). Accordingly, it is most relevant to establish the factors affecting the SPT blow count, N, data.

During the late 1970's, Schmertmann and his colleagues at the University of Florida (Schmertmann 1978, Schmertmann 1979, Schmertmann & Palacios 1979) carried out a systematic theoretical and experimental study of the mechanics of the SPT. A key conclusion of these studies has been that the measured N values vary inversely with that part of the theoretical free-fall hammer energy which reaches the sampler through the drill rods in the form of an initial compression wave, E_i. That is, factors which affect the SPT N-values do so by actually affecting E_i. By measuring E_i from over 500 SPT blows from a variety of drill rigs and hammers used in current (1986) U.S. practice, Schmertmann and his colleagues documented that E_i could vary from 30 to 85 percent of the free-fall hammer energy.

4.2 Factors affecting the SPT results

Kovacs et al (1981) presented a comprehensive summary of factors affecting E_i, and thus N. Only those factors which are relevant to this study are discussed herein.

1. Hydrostatic head in the borehole: Failure to maintain sufficient hydrostatic head in the borehole may decrease N by as much as 100%.

2. Length of drill rods: Schmertmann & Palacios (1979) explained that the shape of the compression wave that initially travels down the drill rods to the sampler depends primarily on the hammer-rod impedance ratio. Since the drill rods are finite in length, the initial compression wave reflects upward upon reaching the sampler end and returns as a tension wave. When the tension wave reaches the hammer-rod contact, it causes the rods to pull down and away from the hammer. At this moment the returning compression wave stops and the rest of the hammer energy is wasted. The "tension cutoff" time is directly proportional to the length of the rod-sampler assembly.

For a safety hammer system Schmertmann & Palacios (1979) established that at about 40 ft. of rod length all available hammer energy passes to the rod before the tension cutoff occurs. Theoretically, for the relatively short rod length range of 5 to 10 ft., about 70 to 40% of the available hammer energy is wasted, respectively.

Kovacs et al (1981) estimated that for rod lengths under 10 ft., that is for SPT very near the ground surface, N-values would increase by 50% due to wasted hammer energy. They concluded that for rod lengths over 30 ft. no such loss would occur.

Seed et al (1983) suggested that for the "short" rods the measured N-values would be fictitiously high, and they proposed that in the depth range of 0 to 10 ft. the measured N-values be corrected by a factor of 0.75. Seed et al (1985) have incorporated this correction factor in evaluating liquefaction field data in their more recent works.

3. Use of no liner: It is common practice in the U.S. that the 1.5-in. I.D. split-barrel sampler (ASTM D 1586-84) is used without the 1/16 in. thick liner. This, as expected, would decrease the friction in the sampling process and the corresponding N-values.

Seed et al (1985) compiled the test data reported by Kovacs & Salomone (1984) and Schmertmann (1979) to estimate the effect of a 1.5-in. I.D. ASTM sampler with no liner, U.S. practice, as against a constant 1.375-in. I.D. sampler, Japanese practice. Test data in sands disclosed that not using a liner decreases N-values by about 8% in the N = 1 to 10 range, and 10 to 15% in the N = 10-20 range. Seed et al (1985) concluded that the effect of not using a liner is about 10% for looser sands (more relevant to liquefaction) and may be as high as 25 to 30% for denser sands.

4. Number of turns and integrity of rope: Kovacs et al (1981) experimentally observed that the kinetic energy in the hammer just before impact is slightly smaller for two turns of rope around the rotating cathead than for the case of one turn. However, the energy decrease becomes quite significant with three turns.

Kovacs et al (1981) also studied the effect of new rope as against old rope. They measured that for two turns of the rope the energy, as a ratio of the free-fall hammer energy, just before impact is about 5 to 10% smaller for the old rope.

5. Type of hammer: Seed et al (1984, 1985) studied the data reported by Kovacs et al (1983) for the energy transmitted to the drill rods as a ratio of the free-fall hammer energy (i.e., rod energy ratio) both for safety and donut hammers. They have adjusted the data for two turns of rope around the rotating cathead, and for a 40 ft. drill rod length. Seed et al (1984) estimated that the mean rod energy ratio was 0.61 for the eighteen rigs using a safety hammer as compared to 0.48 for the eight rigs using a donut hammer. In addition, they studied the data on safety and donut hammers reported by six other researchers.

Seed et al (1985) suggested the use of safety hammer as a "standard" for the U.S. since it is the prevalent method in the U.S. at the present. They estimated a 60% efficiency (i.e., as a ratio of the available hammer energy) for the safety hammer, and 45% efficiency for the donut hammer. Accordingly, Seed et al (1985) recommended to use a "correction factor" of 0.75 for N-values measured by utilizing a donut hammer to obtain equivalent N-values for the safety hammer.

5 SPT PRACTICE IN NEW ENGLAND

5.1 General

The present (1986) SPT practice in New England utilizes a donut hammer and two turns of old to new rope around a rotating cathead. The ASTM D 1586-84 split-barrel sampler is almost always used without a liner since it allows a better sample recovery. In drilling through sand and silty sand deposits with relatively high groundwater levels, a 3.5-in. or 4-in. I.D. casing is utilized to prevent caving. Since test borings are usually monitored by trained personnel, generally sufficient hydrostatic head is maintained in the casing to prevent "wash-in" or "blow-up".

5.2 Recommended SPT correction factors for New England

In accordance with the SPT practice in New England described above, the following "correction factors" for field measured N-values have been adopted in the study to make appropriate use of the "master" liquefaction curves developed by Seed et al (1984, 1985) for a safety hammer.

1. For the use of donut hammer multiply by 0.75.
2. For the use of old to new rope multiply by 0.95.
3. For the use of split-barrel sampler without a liner multiply by 1.08 for the depth range of 5 ft. to 15 ft., 1.10 for 15 ft. to 30 ft., 1.15 for 30 ft. to 40 ft., and 1.20 for 40 ft. to 60 ft.
4. For the use of short length of drill rods multiply by 0.75 at 0 to 10 ft. depth range, 0.90 at 10 ft. to 15 ft., and 1.0 for greater than 20 ft.

6 PROPOSED LIQUEFACTION CRITERIA FOR NEW ENGLAND

6.1 General

The proposed criteria for potential liquefaction assessment of saturated clean sands, that is percent finer than No. 200 sieve by weight being equal or less than

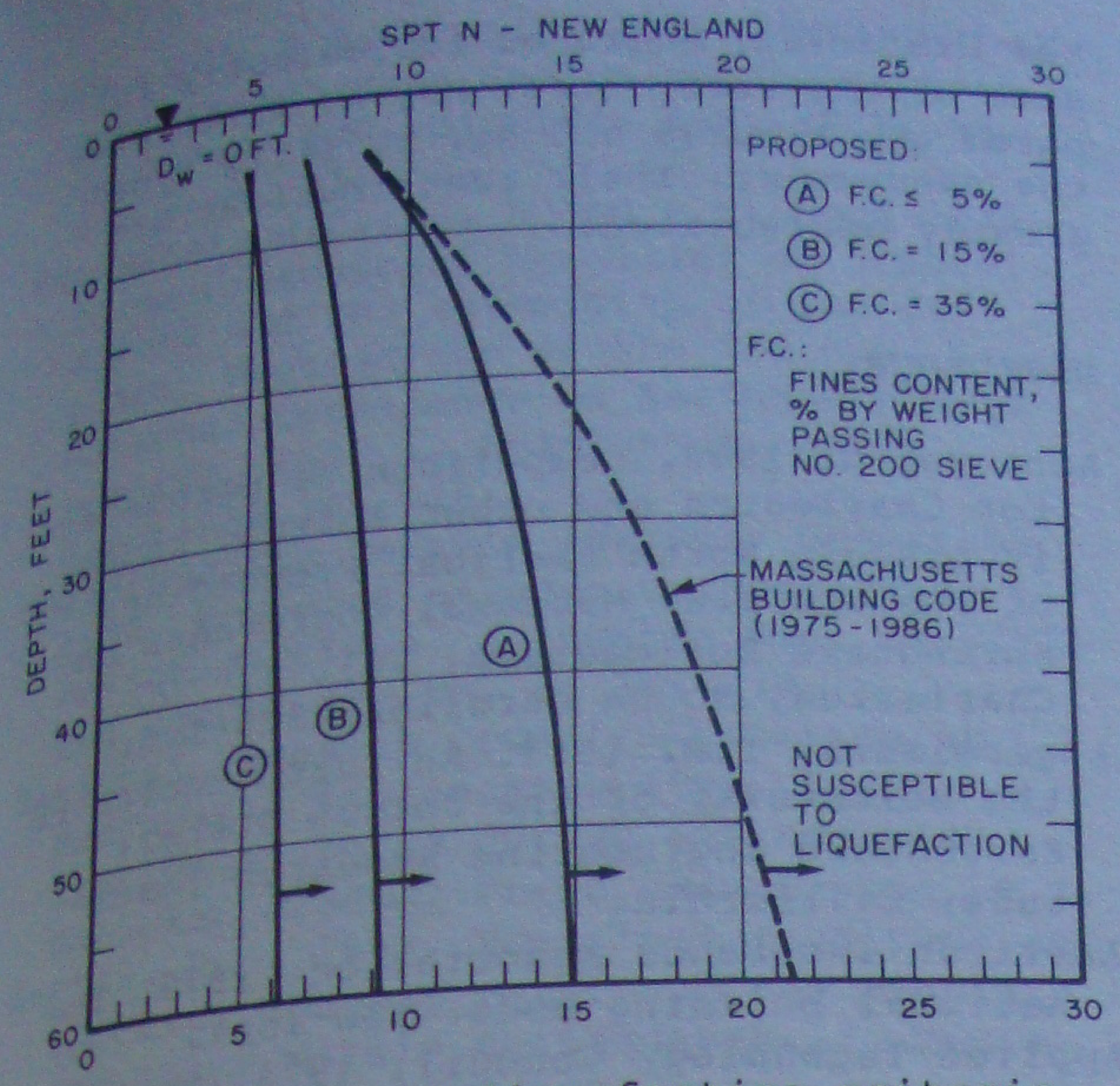

Figure 1. Proposed liquefaction criteria for New England for level ground and groundwater level at ground surface.

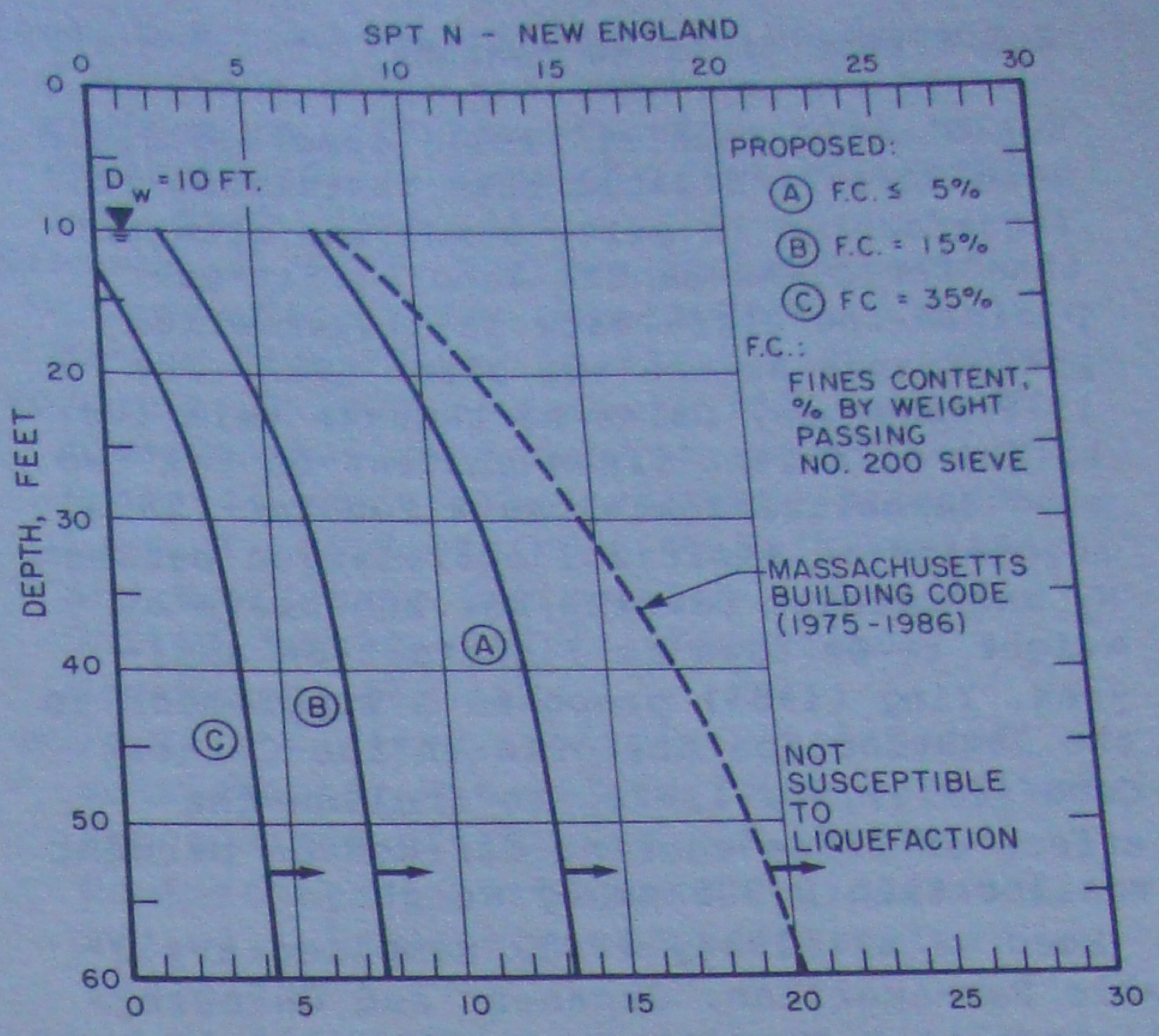

Figure 2. Proposed liquefaction criteria for New England for level ground and groundwater level at 10 ft. below ground surface.

5%, were developed following the methodology suggested by Seed et al (1983, 1984, 1985). Only level ground condition was considered. A design earthquake of M = 6.5 with a peak ground acceleration of 0.12 g on firm soil was considered. Regional SPT practice with recommended "correction factors" as stated in Section 5.2 was taken into account.

6.2 Procedure for the proposed criteria

The following steps were undertaken in developing the proposed liquefaction criteria (i.e., liquefaction curves) as presented in Figure 1 and 2.

1. Total, σ_o, and effective, σ'_o, overburden stress profiles for the depth range of 5 ft. to 60 ft. were computed at 5-ft. intervals using a saturated unit weight of 115 lbs. per cu. ft.

2. Average cyclic shear stress, τ_{av}, values were computed at 5-ft. intervals for the depth range of 5 ft. to 60 ft. using the following formulation proposed by Seed et al (1983):

$$\frac{\tau_{av}}{\sigma'_o} = 0.65 \; \frac{a_{max}}{g} \; \frac{\sigma_o}{\sigma'_o} \; r_d \qquad (1)$$

Values for r_d, stress reduction factor, were estimated as suggested by Seed & Idriss (1971).

3. Cyclic stress ratios, τ_{av}/σ'_o, computed from equation (1) were divided by 1.2 for consistent use of Seed et al's (1984, 1985) master liquefaction curve developed for an M = 7.5 earthquake. The value of 1.2 relative to an M = 6.5 earthquake was estimated by interpolation from the correction factors relative to the influence of earthquake magnitude as provided by Seed et al (1984, 1985).

4. Seed et al's (1984, 1985) most recent master liquefaction curve, τ_{av}/σ'_o vs $(N_1)60$ relationship for percent of fines passing No. 200 sieve by weight $\leq$ 5% was used to obtain $(N_1)60$ values corresponding to the respective adjusted cyclic stress ratio values. The $(N_1)60$ values obtained correspond to the SPT N-values associated with the safety hammer with 60 percent efficiency, at an effective overburden pressure of 1 ton per sq. ft.

5. Obtained $(N_1)60$ values were converted to N_1 values for New England multiplying by the correction factors related to SPT practice in New England as stated in Section 5.2.

6. Obtained N_1 values were converted to N-values by adjusting for the respective effective overburden stress as a function of depth. This was done by using the $N_1 = C_N N$ relationship and the C_N values suggested by Seed et al (1984, 1985).

7. Obtained N-values were plotted at 5-ft. intervals for groundwater levels at ground surface and at a depth of 10 ft. in Figure 1 and 2, respectively. The plotted points were joined by smooth curves.

6.3 Effect of fines content

Tokimatsu & Yoshimi (1981) reported field data which indicated that resistance to liquefaction in silty sands was greater than for clean sands. Zhou (1981) explained the difference in liquefaction performance at the two sites after the 1976 Tangshan, China earthquake relative to the different fines content of the two sand deposits. Tokimatsu & Yoshimi (1983) suggested an empirical correlation between N_1 and percent passing No. 200 sieve by weight to be used in liquefaction analyses. Ying (1983) proposed a supplement to the liquefaction criteria in the Chinese Code (1978), TJ 11-78, to include the effect of fines content defined as percent smaller than 0.005 mm by weight.

Seed et al (1984, 1985) compiled available Pan-American, Japanese and Chinese liquefaction field data and studied the observed performance relative to the amount of fines present in the sand deposits at the respective sites. Based on this study they developed master liquefaction curves for silty sand deposits with 15% and 35% of fines passing the No. 200 sieve by weight for an M = 7.5 earthquake.

Master liquefaction curves proposed by Seed et al (1984, 1985) for 15% and 35% of fines content were utilized in developing respective liquefaction curves for New England. The procedure to develop these curves followed the same steps described in Section 6.2. The proposed 15% and 35% fines content liquefaction curves for New England are presented in Figure 1 and 2 for groundwater levels at ground surface and at a depth of 10 ft., respectively.

7 MANIFESTATION OF LIQUEFACTION

It is expected that the manifestation of liquefaction at the New England sites underlain by loose, saturated, clean medium to fine sands may be primarily in the form of settlements and associated differential settlements of buildings supported by shallow foundations. An approximate simplified approach to estimate such seismically induced settlements was proposed by Soydemir (1986) relative to the design earthquake adopted for New England in this study.

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