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ABSTRACT

A simplified procedure is presented for evaluating liquefaction susceptibility of cohesionless saturated soils based on available technology. In 2001, a Committee of engineers working in the New York City (NYC) area was formed under the direction of the first Author, to review the liquefaction aspects of the 1995 New York City Building Code. The purpose was to gain consensus on a possible revision and augmentation of the exisiting regulations as part of the ongoing Code review by the Structural Engineers Association of New York (SEAoNY). This article summarizes the recommendations of the Committee, as compiled in 2002.

The following topics are reviewed: (a) history of the current code; (b) seismicity and design motions in NYC; (c) updated screening criteria for liquefaction susceptibility. With reference to the topic in (c), recommendations are developed for Code language pertaining to: (1) method of analysis; (2) site classification schemes; (3) design considerations for bearing capacity and displacements of foundations in liquefied soil; (4) maximum depth of liquefaction; (5) field methods to evaluate soil resistance; (6) parameters to be considered in analyses; (7) treatment of sloped strata.

Analytical results for typical NYC profiles subjected to 500-year rock motions are presented. Based on the these results, the Committee proposed a revised liquefaction screening diagram.

INTRODUCTION

In 1995, the Building Code of the City of New York (Code) was amended to consider earthquake loads. The provisions of Section 2312 of the 1990 version of the Uniform Building Code (UBC) were incorporated, with modifications, into the Code by the amendment. Among the modifications was a section relating to soil liquefaction under seismic loading.

In 2001, the Structural Engineers Association of New York (SEAoNY) undertook an internal review of the seismic aspects of the Code. The first author, a member of SEAoNY, assembled an *ad-hoc* committee of geotechnical engineers (Committee) to review the liquefaction section of the Code and suggest changes to SEAoNY, to be considered for inclusion in the recommendations to the New York City Department of Buildings (DOB). Another member of the Committee, Peter Edinger, was directly involved in the preparation of the liquefaction section of the Code, as ammended in 1995.

In 2002, SEAoNY expanded its review to consider all aspects of the Code by comparing it with the model 2000 International Building Code (IBC). The Committee's recommendations became part of the expanded ongoing review. This paper represents the Committee's view which should not be construed to be SEAoNY's policy, since SEAoNY's review is still underway.

HISTORY OF THE PRESENT CODE LIQUEFACTION SCREENING DIAGRAM

The liquefaction screening procedure defined in the present Code including the screening diagram shown in Fig. 1 (Code Figure 4) was developed in 1989 by a geotechnical Subcommittee. The procedure and the screening diagram were based on the simplified procedure by Seed and Idriss (1971). The procedure defines the potential for liquefaction at a given depth in a soil deposit in terms of:

1. The Standard Penetration Resistance (N) as defined by ASTM D-1586;
2. The peak shear stress induced by the design earthquake. This stress is (primarily) a function of Peak Ground Acceleration (PGA) at the soil surface;
3. The duration of shaking. In the simplified procedure, the duration of shaking is implicitly incorporated into the Magnitude factor (M).

Initially, the Subcommittee intended to develop a relationship between Standard Penetration Resistance and depth below ground surface that would define a design boundary between soils that would probably liquefy and soils that probably will not liquefy during a design earthquake. This approach was intended to be similar to the Massachusetts Building Code.

The Subcommittee noted that the Code requires higher seismic design loadings on "Essential Facilities" and "Hazardous Facilities", than on other types of structures. The intent of this requirement is that "Essential" and "Hazardous" facilities will survive as functioning entities, even if an earthquake stronger than the design level for ordinary structures occurs. Hence, if only a single liquefaction / non-liquefaction boundary was defined, there was a potential that during earthquakes stronger than the design level, the foundation of an "Essential" or "Hazardous" facility could be compromised by liquefaction, even if the superstructure was strong enough to survive, as intended.

To obtain compatibility in safety between superstructure and foundation, the Subcommittee elected to define two boundaries, obtaining three category areas for liquefaction screening (Figure 1):

1. *Category A:* N less than the lower boundary, soil shall be considered liquefiable.
2. *Category B:* N between the upper and lower boundaries, liquefaction possible, and soil shall be considered liquefiable for soils underlying "Essential" and "Hazardous" facilities.
3. *Category C:* N above the upper boundary, liquefaction unlikely.

The analyses to quantify the boundaries were made using the simplified procedure, assuming that liquefaction is unlikely to occur below a depth of fifty feet, under any level of earthquake shaking, and for groundwater depths of 0, 20 and 40 feet below ground surface.

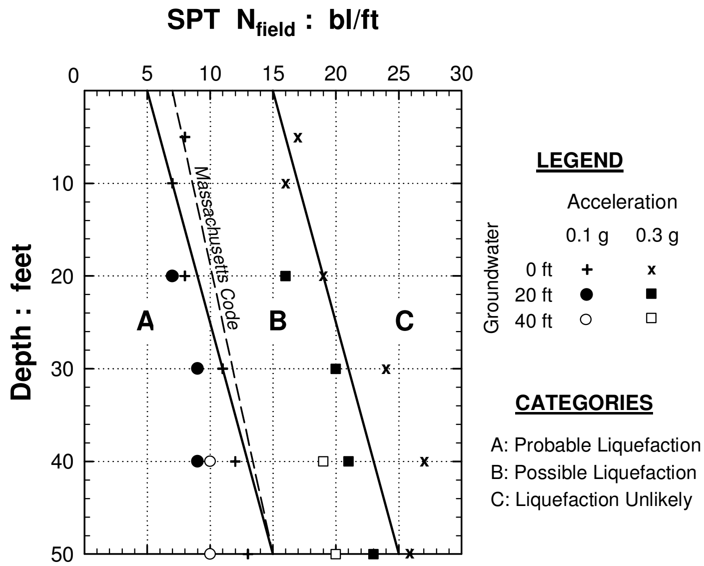

Fig. 1. Liquefaction screening diagram in present Code.

To define the lower boundary, the Subcommittee determined, based on seismic hazard information available at the time that the earthquake most likely to cause liquefaction at the design level selected for NYC would be a distant event of high magnitude and long duration with the following parameters:

- Peak Ground Acceleration at site = 0.1 g
- Magnitude = 7.5
- Median Epicentral Distance from site = 1000 km

The Subcommittee considered that this earthquake had a statistical probability of occurring at an average return period of 500 years – compatible with the design level event – within an area of about 6,000,000 km^2 centered on NYC.

To define the upper boundary of liquefaction screening the Committee assumed an upper-limit earthquake occurring close to or even within New York City having parameters:

- Peak Ground Acceleration at site = 0.3 g
- Magnitude = 6.0
- Median Epicentral Distance from site = 50 km

The Subcommittee considered that this earthquake had a statistical probability of occurring at an average return period of 3,000 years within an area of about 16,000 km^2 centered on New York City.

The computed points as well as the screening limit from the Massachusetts Building Code are shown on the screening diagram (Fig. 1), presently included in the New York City Code (but without the plotted points and Massachusetts screening limit). The actual boundary lines of the screening diagram were drawn on the basis that:

1. Shallow liquefaction (on level ground) is potentially more damaging than deeper liquefaction. In addition, a shallow water table tends to lower the effective stresses at all depths in the soil, reducing resistance to liquefaction. Hence, the results for shallow groundwater should have precedence.
2. There should be reasonable correspondence to the Massachusetts Building Code liquefaction definition.
3. The data on which the simplified Seed-Idriss analysis is based has a large scatter; hence, it was sufficient and convenient to define the boundary lines to the nearest 5 blows per foot (bl/ft) as straight lines.

Although not explicitly stated, Fig. 1 was intended to be a screening tool requiring the actions specified if no further analysis was done. Most geotechnical engineers interpreted the Code as allowing the engineer to further analyze the conditions and demonstrate site safety with regard to liquefaction, as approved by the Commissioner.

HISTORIC SEISMICITY & DESIGN GROUND MOTIONS

A compilation of the historic seismicity since 1534 is depicted in Fig. 2. Recordings of seismic events in the New York metropolitan area are available for the past 50 years. Prior to that, magnitudes are derived using earthquake intensity data. The most severe events occurred at Rockaway beach in 1737 and 1884, with estimated local magnitudes of 4.6 and 5.1, respectively, and in Morris County, New Jersey in 1783 with magnitude 4.8.

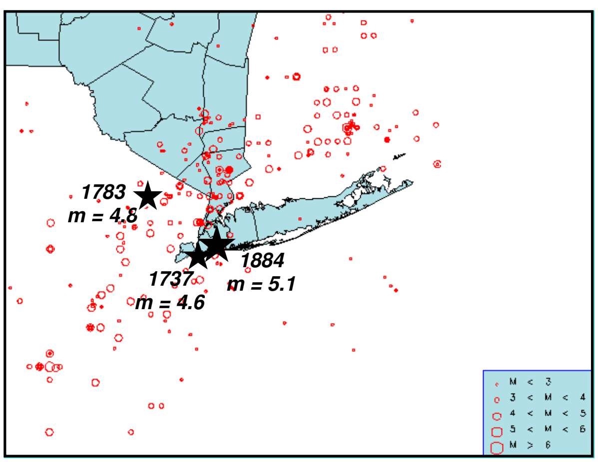

Fig. 2. Spatial distribution of historic seismicity and major events around NYC from 1534 to today (after Nikolaou, 1998).

Evidence exists that some earthquakes may have triggered liquefaction in NYC (Tuttle & Seeber 1989; Budhu et al 1990). An example is the 1884 NYC earthquake, during which beach houses reportedly tilted and subsided, most likely due to liquefaction of the surficial beach sands.

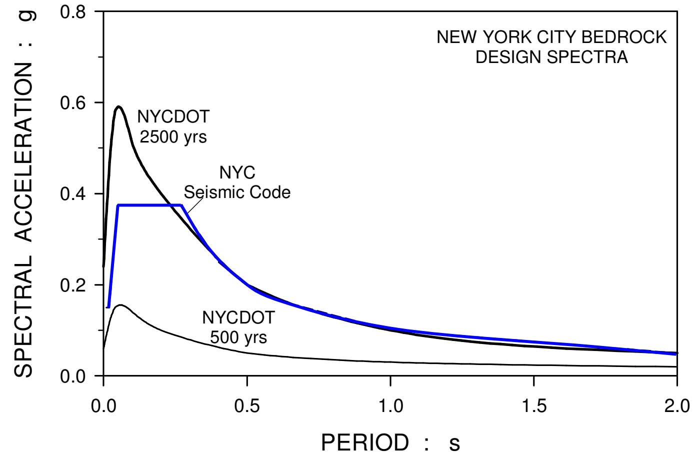

Fig. 3. Bedrock design spectra according to NYC Seismic Code (1995) and NYCDOT Seismic Criteria Guidelines (1998), for 5% structural damping.

In 1998, the NYC Department of Transportation (NYCDOT) released Seismic Criteria Guidelines for bridges and other highway structures. Peak ground accelerations in the City for hard rock conditions were estimated to be 0.06 g for a return period of 500 years and 0.24 g for a return period of 2,500 years.

The design spectra in the NYCDOT Guidelines for hard rock, shown in Fig. 3, have evident high-frequency content with peak spectral acceleration occurring at a period of approximately 0.1 sec. The figure also shows the Code spectrum, whose ordinates lie between the NYCDOT spectra for periods less than 0.25 seconds, and at longer periods they are almost identical to those of the 2,500-year NYCDOT spectrum. However, the two spectra are not strictly comparable, since the amplification factors used to scale their ordinates to a "reference" soft rock base (soil type "S1" in the NYC Code and soil type "B" in the DOT Guidelines) are different in the two codes (i.e., 0.67 in NYC Code and 0.8 in NYCDOT). Discussion of the sources of these differences is beyond the scope of this paper.

With reference to the design event, hazard de-aggregation helps identify magnitude-distance (M-R) pairs that contribute mostly to a given seismic parameter [usually Peak Ground Acceleration (PGA) and Spectral Acceleration (SA) for particular structural periods]. For NYC, the highest contribution to the seismic hazard for a structural period of 1 sec range from a magnitude M = 6.5 and epicentral distance R = 22.5 km for the 2,500-year return period, to M = 6 and R = 22.5 km for the 500-year event (Risk Engineering, 1998). For PGA's, the dominant M-R pairs were (5, 12.5 km) for 2,500 years and (5.1, 18 km) for 500 years. Other de-aggregation studies (Nikolaou, 1998) have provided similar results (Fig. 4) and have shown that the earthquake which can create the worst-case-scenario for the acceleration of the ground can be different from the earthquake that will create the largest response of a structure.

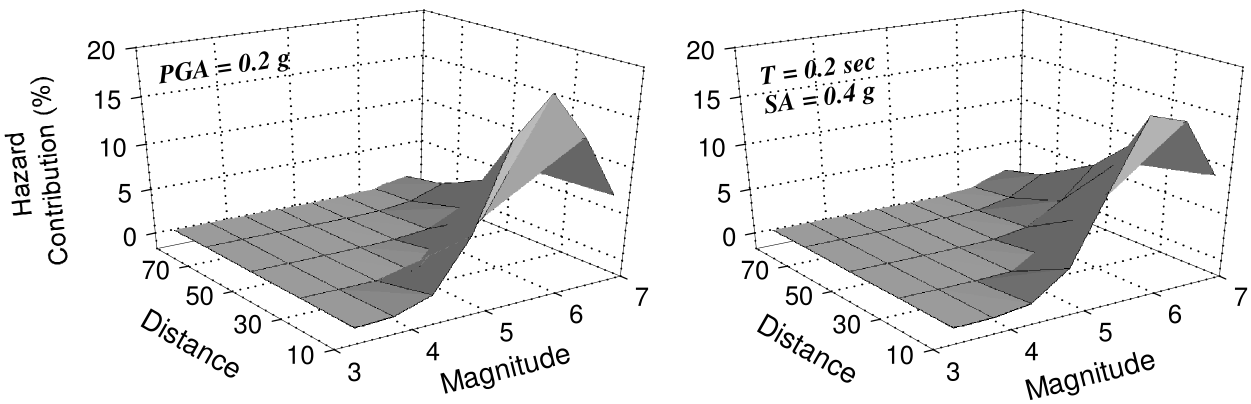

Fig. 4. Seismic hazard contribution of earthquakes with different magnitude, M, and distance, R, on: (a) Peak Ground Acceleration (PGA) and (b) Spectral Acceleration (SA) of an oscillator having period T = 0.2 sec (after Nikolaou, 1998).

For simplicity and as a conservative assumption, an average event with magnitude M = 6 was used in the analyses. On the other hand, no specific epicentral distance was adopted, since this parameter is not directly involved in the simplified procedure.

The hazard studies suggest that the anticipated intensity of seismic shaking in NYC is lower than in more seismically prone areas in the Western United States. However, the unique geological conditions of NYC (Tamaro et al, 2000), such as the hard crystalline bedrock and its large impedance contrast with the overlying soil, the presence of soils of high plasticity, etc., may amplify the surface ground motions much more than in other regions in the United States (Dobry 1998; Nikolaou et al 2001).

DEVELOPMENT OF REVISED SCREENING CRITERIA

After Fig. 1 was adopted, USGS developed new ground motion maps and NYCDOT developed their own ground motions that show lower bedrock accelerations for the NYC area than those used to derive the liquefaction screening diagram. Recent studies regarding magnitude scaling factors used in liquefaction assessments indicate that the original values may be conservative. Currently, there is still much debate regarding this issue (Youd et al, 2001). In addition, site specific response analyses performed by various practitioners in the area indicate that, for certain sites, soil amplification may not be as pronounced as suggested in building codes and agency documents. The following sections describe these issues in more detail, and make recommendations for modifications of the current liquefaction portion of the Code.

Revised screening criteria were developed to reflect a range of soil profiles typically encountered in and around NYC. The profiles for which evaluation of liquefaction potential was considered relevant, according to NYCDOT Guidelines, are:

Soil Class D: Stiff soil with 600 ft/sec < V_s < 1,200 ft/sec or with either $15 \leq N \leq 50$ or 1,000 psf $\leq S_u \leq$ 2,000 psf, within the top 100 ft.

Soil Class E: Softer profile with V_s < 600 ft/sec, or any profile with more than 10 ft of soft clay defined as soil with PI > 20, $w_c \geq 40$ %, and S_u < 500 psf, within the top 100 ft.

where: V_s is the soil shear wave velocity, N is the standard penetration resistance, S_u is the undrained shear strength, PI is the plasticity index, and w_c is the water content. Soils were assumed to consist exclusively of clean sand with insignificant amount of fines.

Three soil profiles were selected for analysis, with thickness ranging from 40 to 100 ft, as shown in Fig. 5.

Parametric Studies

The initial intent was to evaluate the response of selected profiles to rock motion time histories given in the NYCDOT guidelines using the commercial program PROSHAKE. Three rock time histories corresponding to 500-year event (Risk Engineering, 1998) were utilized. PROSHAKE analyses provided a PGA at the soil surface that was considerably lower than the NYCDOT recommended value for soil profile E (approximately 0.09 g vs. 0.19 g). Since a more conservative approach was deemed appropriate, evaluation of liquefaction potential was based on the NYCDOT recommended values of PGA (i.e., 0.12 g for soil profile D and 0.19 g for soil profile E). Further, since the screening criteria are for a building code that applies to a wide range of structures that are not necessarily "Essential" structures, evaluation of liquefaction potential based on a 500-year event was also deemed appropriate. A site-specific analysis using an appropriate earthquake event should be done for critical and essential structures.

According to the simplified approach, the variation of Cyclic Stress Ratio (CSR_{EQ}) with depth is given as:

$$CSR_{EQ} = 0.65\ (PGA / g)\ (\sigma_{vo} / \sigma'_{vo})\ r_d \qquad (1)$$

where σ_{vo} and σ'_{vo} denote the total and effective normal stresses at the given depth; r_d is a "flexibility" factor ranging from 0.9 to 1 (Seed & Idriss 1971, 1982).

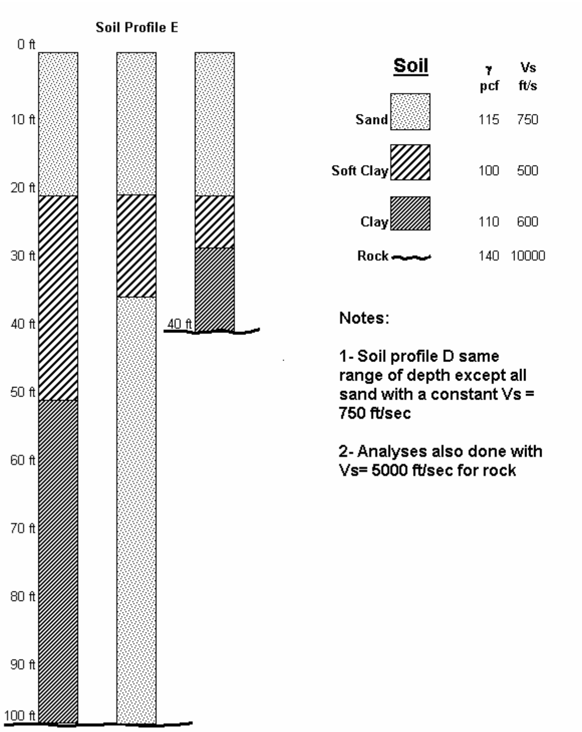

Fig. 5. The soil profiles considered in the analyses.

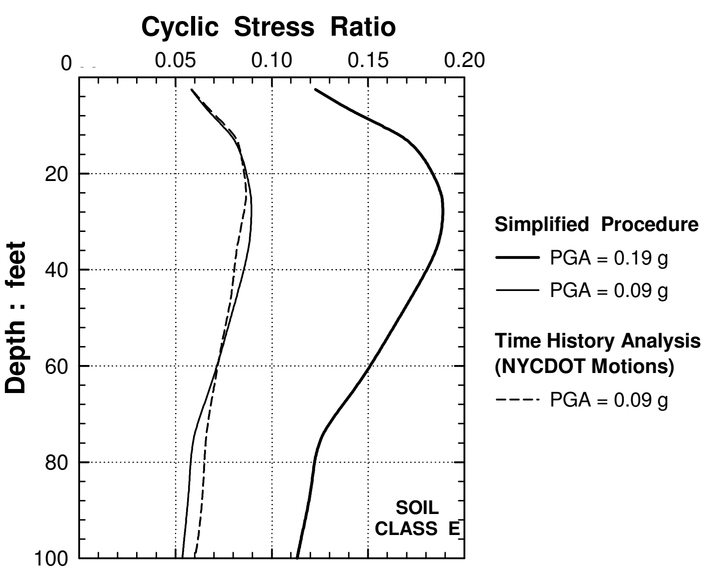

Fig. 6. Comparison of results with time history analysis and simplified procedure, for different surface PGA levels.

Comparison of the variation of CSR_{EQ} vs depth based on the PROSHAKE analyses and the Seed and Idriss simplified approach for a PGA of 0.19g shows excellent agreement. A typical comparison for soil profile E is shown on Fig. 6. Similar analyses for soil profile D showed the simplified approach to be somewhat more conservative than the PROSHAKE analyses. This is the justification for using the simplified approach rather than time history analysis to determine the variation of CSR_{EQ} with depth.

Determination of Lower and Upper Limits for Zone B

A demand ("safety") factor of 1 for soil profile D was used to determine the lower limit of Zone B and a demand factor of 1.3 for soil profile E was used to determine the upper limit of Zone B. Demand Factor, DF, is defined as follows:

$$DF = CSR_L / CSR_{EQ} \qquad (2)$$

where: CSR_L is the corrected critical stress ratio resisting liquefaction:

$$CSR_L = K_M\ K_\sigma\ K_\alpha\ CSR_{M = 7.5} \qquad (3)$$

K_M = correction factor for earthquake magnitudes other than 7.5. A value of 2 for an assumed magnitude of 6 was used following Youd & Nobel (1997)

K_σ = correction factor for stress level larger than 1 tsf (Youd & Idriss, 1997)

K_α = correction factor for the initial driving static shear stress, assumed 1.0

Therefore, the equivalent critical stress ratio is:

$$CSR_{M=7.5} = DF\ CSR_{EQ}\ /\ (K_M\ K_\sigma\ K_\alpha) \qquad (4)$$

To relate corrected and uncorrected blow counts, $(N_1)_{60}$ was first interpolated from the chart developed by Seed et al (1985) using $CSR_{M=7.5}$, determined as described above. This was then converted to the corresponding N_{field} with applicable correction factors, as follows:

$$(N_1)_{60} = C_n\ \Sigma C_d\ N_{field} \qquad (5)$$

where: C_n = correction factor to a reference stress of 1 tsf (Liao & Whitman 1986) and ΣC_d = correction factors for drilling operation (e.g., method, size of rod and hammer energy). In our calculations, ΣC_d varied with depth from 0.68 to 0.9.

The results of (uncorrected) N_{field} vs depth for the boundaries between Zones A, B, and C are presented in Fig. 7 and the proposed revision to the screening criteria for potential liquefaction is presented in Fig. 8. Note that depth is limited to 50 ft, since liquefaction at deeper elevations is not considered. Figure 8 is proposed to replace the existing Figure 4 in the Code (Fig. 1 in this paper).

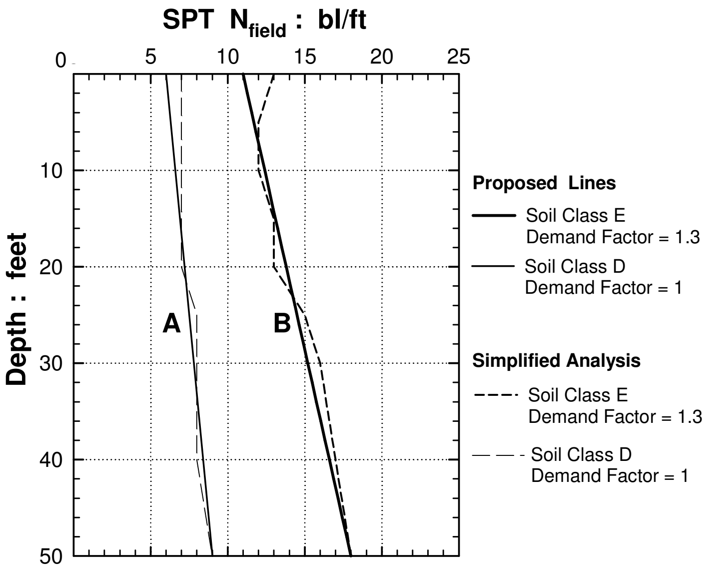

Fig 7. Analysis results for upper and lower limits of soil category B.

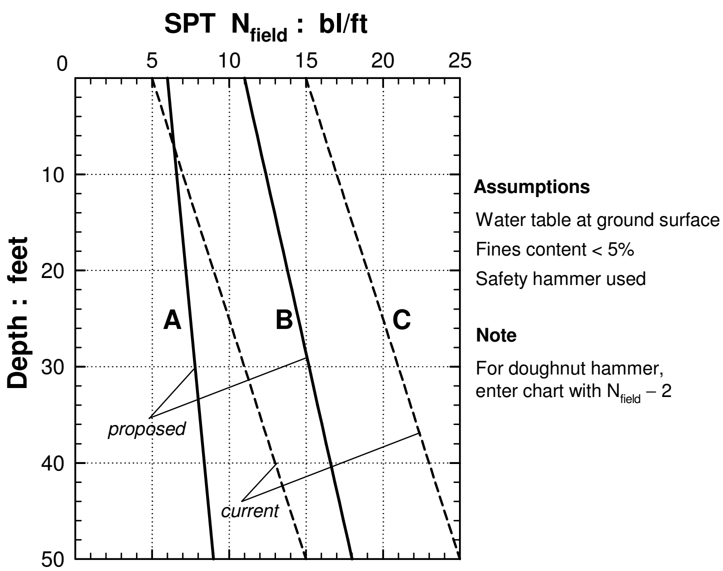

Fig 8. Proposed liquefaction screening diagram.

PROPOSED CODE LANGUAGE REGARDING LIQUEFACTION IN CURRENT NYC BUILDING CODE

The proposed Code language regarding liquefaction was developed to clarify certain ambiguous aspects of the present language, especially when analyses may supersede the screening diagram and the distinction between “zone” and “category”. The proposed language provides guidance regarding the parameters required for the analyses and specifies acceptable risk levels for various occupancy category structures.

Recognizing that SEAoNY is considering IBC 2000 in its overall review of the present NYC Code, favorable language relating to liquefaction was integrated into the proposed text. Criteria for shear demand (inverse of demand factor) are included in the proposed code language based on the authors' judgment regarding levels likely to assure minimal problems due to liquefaction. The following paragraphs in this section comprise the Code language proposed by the authors.

The evaluation of liquefaction potential shall include the following considerations:

1. Non-cohesive soils below ground water table and less than fifty feet below the ground surface shall be considered to have potential for liquefaction.
2. The potential for liquefaction on level ground shall be determined on the basis of the zones associated with the uncorrected Standard Penetration Resistance (N) at the site, as defined in Figure No. 4. The liquefaction potential in each zone at any depth within the upper fifty feet is defined as:

   Zone A: Liquefaction is probable. Soil in this zone shall be considered liquefiable for all occupancy categories.

   Zone B: Liquefaction is possible. Soil in this zone shall be considered liquefiable for all structures, unless shown otherwise by a recognized method of analysis.

   Zone C: Liquefaction is unlikely during a 500 year event, and need not be considered in design of Occupancy Category IV structures. Occupancy Categories I, II, and III will require evaluation using a recognized method of analysis.

In evaluating liquefaction potential, the analysis shall consider the following parameters: ground surface acceleration, earthquake magnitude, magnitude scaling factor, effective overburden pressure, hammer energy, cone penetration resistance (where applicable), and fines content. If a site response analysis is conducted, bedrock acceleration time histories and a shear wave velocity profile based on in situ measurements may be utilized. These analyses may consider the results of laboratory cyclic shear tests.

The evaluation shall consider an assessment of potential consequences of any liquefaction and soil strength loss including estimation of differential settlement, lateral movement or reduction in foundation soil bearing capacity, and may incorporate the potential benefits of any proposed mitigation measures. Such measures may be given consideration in the design of the structure and can include, but are not limited to, ground improvement, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements, or any combination of these measures.

In evaluating the potential for liquefaction, the effect of settlements induced by seismic motions and loss of soil strength, shall be considered. The analysis performed shall incorporate the effects of peak ground acceleration, appropriate earthquake magnitudes and duration consistent with the design earthquake ground motions as well as uncertainty and variability of soil properties across the site. Peak ground acceleration, seismically induced cyclic stress ratios and pore pressure development may be determined from a site-specific study taking into account soil amplification effects and ground motions appropriate for the seismic hazard. Recognized methods of analysis, including so-called "simplified procedure" (Youd et al 2001), can be used in the evaluation process.

Effects of pore water pressure buildup shall be considered in the design except for the following conditions:

1. The calculated cyclic shear demand is equal or less than 75% of the calculated cyclic shear strength for Category I, II, and III structures.
2. The calculated cyclic shear demand is equal to or less than 85% of the calculated cyclic shear strength for Category IV structures.

At sites where liquefaction is determined to be probable, the following considerations shall be included in the design.

1. Liquefiable soils shall be considered to have no passive (lateral) resistance or bearing capacity value during an earthquake, unless shown otherwise by accepted methods of analysis. An analysis shall be submitted by a geotechnical engineer (Professional Engineer), which demonstrates, subject to the approval of the Commissioner, that the proposed construction is safe against the effects of soil liquefaction.
2. Where liquefiable soils are present in sloped ground or over sloped non-liquefiable substrata and where lateral displacement is possible, a stability analysis shall be submitted by a geotechnical engineer (Professional Engineer) which demonstrates, subject to the approval of the Commissioner, that the proposed construction is safe against failure of the soil and that the effect of potential lateral displacements are acceptable.

ALTERNATE SCREENING CRITERIA

Because NYC is in the process of reviewing its Building Code for conforming with IBC 2000, an alternate screening diagram has been developed. The alternate criteria are based on similar analyses to the ones described previously.

The alternate screening diagram, shown in Fig. 9, is based on Soil Class E and return periods consistent with the IBC Categories (or Seismic Use Groups) and a constant demand factor of 1. Data points falling to the left of each line representing different Occupancy Categories must be analyzed for possible liquefaction. Data points falling to the right of each line may be assumed to be unlikely to liquefy.

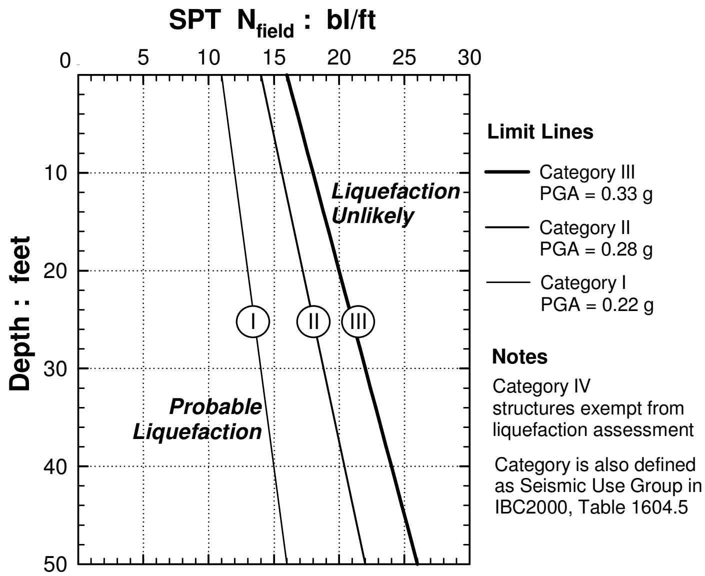

Fig 9. Alternate liquefaction screening diagram based on IBC.

CONCLUSION

The intent of this paper is to provide input for the code revision process and not a substitute for the present Code. Nevertheless, the writers believe that the proposed Code revisions provide for public safety, reduce excessive conservatism, are consistent with the current engineering practice and clarify the intent of the Code.

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APPENDIX I: CURRENT CODE

Table 23-K. Occupancy Categories, 1995 NYC Building Code.

Occupancy Categories	***Occupancy Type or Function of Structure***
I. Essential Facilities	▪ Hospitals and other medical facilities having surgery and emergency treatment areas. ▪ Fire and Police stations. ▪ Buildings for schools through secondary or day-care centers with capacity >250 students. ▪ Tanks or other structures containing, housing or supporting water or other fire-suppression materials or equipment required for the protection of essential or hazardous facilities, or special occupancy structures. ▪ Emergency vehicle shelters and garages. ▪ Structures and equipment in emergency-preparedness centers. ▪ Stand-by-power generating equipment for essential facilities. ▪ Structures and equipment in government communication centers and other facilities required for emergency response.
II. Hazardous Facilities	▪ Structures housing, supporting or containing sufficient quantities of toxic or explosive substances to be dangerous to the safety of the general public if released.
III. Special Occupancy Structure	▪ Covered structures whose primary occupancy is public assembly with capacity > 300 persons. ▪ Buildings for colleges or adult education schools with capacity > 500 students. ▪ Medical facilities with > 50 resident incapacitated patients, but not included above.
IV. Standard Occupancy Structure	▪ All structures having occupancies or functions not listed above.

The Code reads as follows:

(i) Soils of classes 7-65, 8-65, 10-65 [essentially sands, fine sands, silts, respectively] and non-cohesive class 11-65 [uncontrolled fill] below the groundwater table and less than fifty feet below the ground surface shall be considered to have potential for liquefaction.

(ii) The potential for liquefaction for level ground shall be determined on the basis of the Standard Penetration Resistance (N) in accordance with Figure No. 4 (Fig. 1 in this paper);

Category A: Soil shall be considered liquefiable.

Category B: Liquefaction is possible. Soil shall be considered liquefiable for structures of Occupancy Categories I, II and III of Table No. 23-K.

Category C: Liquefaction is unlikely and need not be considered in design.

At any site the highest category of liquefaction potential shall apply to the most critical strata or substrata.

(iii) Liquefiable soils shall be considered to have no passive (lateral) resistance or bearing capacity value during an earthquake. An analysis shall be submitted by an engineer who demonstrates, subject to the approval of the Commissioner, that the proposed construction is safe against liquefaction effects on the soil.

(iv) Where liquefiable soils are present in sloped ground or over sloped nonliquefiable substrata and where lateral displacement is possible, a stability analysis shall be submitted by an engineer who demonstrates, subject to the approval of the Commissioner, that the proposed construction is safe against failure of the soil.

APPENDIX II: IBC 2000

Classification of Buildings & Other Structures according to IBC Table 1604.5.

Category	***Nature of Occupancy***
I	Buildings and other structures except those listed in Categories II, III, IV
II	Buildings and other structures that represent a substantial hazard to human life in the event of failure including, but not limited to: ▪ Buildings and other structures where > 300 people congregate in one area ▪ Buildings and other structures with elementary school, secondary school or ▪ Day-care facilities with capacity > 250 ▪ Buildings and other structures with a capacity > 500 for colleges or adult education facilities ▪ Health care facilities with a capacity of 50 or more resident patients but not having surgery or emergency treatment facilities ▪ Jails and detention facilities ▪ Any other occupancy with an occupant load greater that 5,000 ▪ Power-generating stations, water treatment for potable water, wastewater ▪ Treatment facilities and other public utility facilities not included in Category III ▪ Buildings and other structures not included in Category III containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released
III	Buildings and other structures designated as essential facilities including, but not limited to: ▪ Hospitals and other health care facilities having surgery or emergency treatment facilities ▪ Fire, rescue and police stations and emergency vehicle garages ▪ Designated earthquake, hurricane or other emergency shelters ▪ Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response ▪ Power-generating stations and other public utility facilities required as emergency back-up facilities for Category III structures ▪ Structures containing highly toxic materials as defined by Section 307 where the quantity of the material exceeds the exempt amounts of Table 307.7(2) ▪ Aviation control towers, air traffic control centers and emergency aircraft hangars ▪ Buildings and other structures having critical national defense functions ▪ Water treatment facilities required to maintain water pressure for fire suppression
IV	Buildings and other structures that represent a low hazard to human life in the event of failure including, but not limited to: ▪ Agricultural facilities ▪ Certain temporary facilities ▪ Minor storage facilities