PROPOSED CHANGES TO EARTHQUAKE DESIGN SECTIONS MASSACHUSETTS STATE BUILDING CODE

By Seismic Design Advisory Committee of the Boston Society of Civil Engineers Section/American Society of Civil Engineers

Rene W. Luft, Chairman
Gonzalo Castro, Vice Chairman
Myle J. Holley, Jr. (1)
Edward B. Kinner
Maurice A. Reidy, Jr.
Norton S. Remmer
Kentaro Tsutsumi
Robert V. Whitman
Kenneth B. Wiesner

INTRODUCTION

The Seismic Design Advisory Committee of BSCES/ASCE is the successor committee of the Seismic Advisory Committee to the State Building Code Commission. The Seismic Advisory Committee developed the earthquake design provisions incorporated in the Massachusetts State Building Code.

A purpose of the Seismic Design Advisory Committee is to review the earthquake design provisions of the State Building Code and to propose changes to these code provisions. The committee reviews the earthquake design provisions in response to requests for clarification or modifications submitted by users of the code, and in response to advances in earthquake engineering.

The changes proposed herein are published in accordance with the current BSCES/ASCE procedure to initiate code changes. Written discussion on the committee's proposed changes is invited from all interested parties and such discussions must reach the committee by 1 December 1984. Proposed modifications (1) should be presented in code format and (2) should be accompanied by an explanatory commentary. A joint meeting of the Structural and Geotechnical Groups is planned for the Fall of 1984. Subsequent to that meeting, the Seismic Design Advisory Committee will review all written discussions, incorporate such proposed changes as seem appropriate, and submit a final document to the BSCES/ASCE Board of Government. On recommendation of the Board, the President of BSCES/ASCE will submit the proposed code amendments to the appropriate State agency.

---

(1) Resigned, 13 September 1983

LATERAL FORCE ON PARTS OR PORTIONS OF BUILDINGS OR OTHER STRUCTURES

SECTIONS REQUIRING CHANGE: 716.4.5, 716.6.6, and Table 716.2

1. Section 716.4.5

Editorial correction of Section 716.4.5 as follows:

716.4.5 Lateral force on parts or portions of buildings or other structures: Parts or portions of structures and their anchorage shall be designed for lateral forces in accordance with the following formula:

$$F_p = \frac{1}{3} C_p W_p$$

The values of C_p are set forth in Table 716.2. The ~~distribution of these forces shall be according to the gravity loads pertaining thereto.~~

2. Section 716.6.6, Item 1

DELETE:

1. Connections and panel joints shall allow for a relative movement between stories of not less than two (2) times story drift caused by wind or seismic forces; or one quarter (1/4) inch, whichever is greater.

SUBSTITUTE:

1. Connections and panel joints shall allow for a relative movement between stories of not less than one half (1/2) inch, or 3.0/K times the elastic seismic story drift, whichever is greater. Values of K are set forth in Table 716.1.

3. Table 716.2

Modify Table 716.2 by (i) change of C_p for Connections for exterior panels; (ii) addition of Superscript 3; and (iii) substitution of Note 3 as follows:

Table 716.2

Horizontal Force Factor "C_p" for Parts or Portions of Structures

Part or portion of structures	Direction of force	Value of C_p
Exterior bearing and nonbearing walls, interior bearing walls and partitions, interior nonbearing walls and partitions over ten (10) feet in height, masonry or concrete fences over six (6) feet in height	Normal to flat surface	0.20[3]
Cantilever parapet and other cantilever walls, except retaining walls	Normal to flat surface	1.00
Exterior and interior ornamentations and appendages	Any direction	1.00
When connected to, part of, or housed within a building: towers, tanks towers and tanks plus contents, storage racks over six (6) feet in height plus contents, chimneys, smokestacks, penthouses, equipment and machinery	Any direction	0.20[1,2]
When resting on the ground, tank plus effective mass of its contents	Any direction	0.12[6]
Floors and roofs acting as diaphragms[4]	Any direction	0.10
Connections for exterior panels or for elements complying with Section 716.6.6	Any direction	1.00
Connections for prefabricated structural elements other than walls, with force applied at center of gravity of assembly	Any horizontal direction	0.30[5]

NOTE 1. When located in the upper portion of any building where the "h_n/D" ratio is five-to-one (5/1) or greater the value shall be increased by fifty (50) per cent.

NOTE 2. "W_p" for storage racks shall be the weight of the racks plus contents. The value of "C_p" for racks over two (2) storage support levels in height shall be zero point sixteen (0.16) for the levels below the top two (2) levels.

NOTE 3. Exterior bearing and nonbearing walls, interior bearing walls and partitions, interior nonbearing walls and partitions over ten (10) feet in height shall be designed for a minimum value of C_p of 0.20 unless a greater value of C_p is required by the basic seismic formula V = 1/3 KCSW and the coefficient F_x/W_x at the height h_x where the wall or partition is located.

NOTE 4. Floors and roofs acting as diaphragms shall be designed for a minimum value of "C_p" of ten (10) per cent applied to loads tributory from that story unless a greater value of "C_p" is required by the basic seismic formula V = 1/3 KCSW.

NOTE 5. The "W_p" shall be equal to the total load plus twenty-five (25) per cent of the floor live load in storage and warehouse occupancies.

NOTE 6. When the soil factor S is less than or equal to one point two (1.2), "C_p" may be taken as zero point one (0.1) S.

COMMENTARY: Lateral Force on Parts or Portions of buildings or Other Structures

716.4.5 and Note 3 of Table 716.2

The provisions for distribution of the building base shear force can imply ratios of seismic forces to local weights at the top of the building of about twice the ratio of base shear to total building weight, i.e.,

$$2 \text{ times } \frac{V}{W} = 2 \frac{KCS}{3}$$

Since K can be as large as 1.33 and CS can be as large as 0.12, the local ratio of seismic force to weight can reach

$$\frac{2}{3} (1.33) (0.12) = 0.11$$

In contrast the value $C_p = 0.2$, for bearing and nonbearing walls, presently given in Table 716.2, implies for these elements a ratio of seismic force to weight of only

$$\frac{1}{3} (0.20) = 0.07$$

Hence the need for the proposed Note 3 to Table 716.2.

Note 3 in the present code was originally included by an editorial error, and should be deleted.

716.6.6 and Row Seven of Table 716.2

A comparison of the seismic force F on connections for exterior panels required by present provisions of the Code with corresponding requirements of other modern codes disclosed the former to be far more severe than the latter. No justification could be found for the difference, and the proposed revised value of C_p represents, in the Committee's judgment, a realistic revision.

Existing provisions of the Code for accommodating story drift in the connections of exterior panels to the structure are judged to be unconservative. Specifically, they do not reflect the emphasis on structural ductility which characterizes the seismic provisions in general. The revised provisions of Item 1. of Section 716.6.6 will correct this deficiency.

COLUMN SPLICES, COLUMN BASE ANCHORAGES, AND SIMILAR CONNECTIONS

SECTIONS REQUIRING CHANGE: 716.5.7

1. Section 716.5.7, Item 4

DELETE:

4. Column splices, base plate anchors and other types of connections that act primarily in bearing shall be designed to resist the required forces, and also shall be capable of resisting the forces resulting from the full seismic loading combined with two-thirds (2/3) of the dead load forces acting concurrently.

SUBSTITUTE:

4. Column splices, column base anchorages, and similar connections or anchorage elements in which forces induced by seismic loading counteract forces due to dead load shall, in addition to other design requirements, be designed to resist the forces resulting from sixty-seven (67) percent of the dead load combined with the forces of opposite sign resulting from the full seismic loading (0.67 D - E). For this loading combination the splice, anchorage or connection is not permitted the thirty-three (33) percent increase in allowable stress otherwise permitted by the accepted engineering practice standards. The above provisions shall not apply to portions of the splice, anchorage or connection governed by reinforced concrete provisions of this Code based on factored loads and ultimate strength design.

COMMENTARY: Column Splices, Column Base Anchorages, and Similar Connections

* First sentence clarifies intent of present wording. "base plate anchors and other types of connections that act primarily in bearing...." is revised to "column base anchorages, and similar connections or anchorage elements..." The 67% dead load loading case is consistent with the requirement specified in Section 717.2 Counteracting Loads.

* Second sentence is a new requirement which should be imposed to avoid premature failure of these splice and anchorage type connections. Eliminating the usual 33% allowable stress increase will make the strength of such connections more consistent with the strength they would have if designed with a limit states design approach (LRFD) and with the ACI Code strength design approach.

Consider design of a steel tension part of a splice or anchor connection. Let:

	P	= design tension force at service load
	A	= required steel area
	ϕ	= understrength factor
Use	ϕ	= 0.90 for tension
	F.S.	= Factor of Safety = Resistance/Design Force

Present Code	Proposed Revision
$P = .67\ D - E$	$P = .67\ D - E$
$A = \dfrac{.75\ P}{.60\ F_y}$	$A = \dfrac{P}{.60\ F_y}$
$F.S. = \phi F_y\ A/P$	$F.S. = \phi F_y\ A/P$
$F.S. = \dfrac{.9\ \cancel{F_y}\ (.75P)}{.6\ F_y\ P}$	$F.S. = \dfrac{.9\ \cancel{F_y}\ (P)}{.6\ F_y\ P}$
$F.S. = 1.125$	$F.S. = 1.50$

Minimum Desired F.S. = 1.4 to 1.5

Note that present Building Code Requirements for Reinforced Concrete (ACI 318-83) requires U = 0.9 D + 1.43 E, and ANSI A58.1-82 requires, in LRFD section, load combination 0.9 D - 1.5 E.

* Third sentence is to clarify intent of the subparagraph in reference to reinforced concrete design, for which different factors on dead and earthquake load apply.

STRUCTURAL SYSTEM ANCHORAGE

SECTIONS REQUIRING CHANGE: 716.6.4

1. Section 716.6.4

DELETE:

Section 716.6.4 Deleted

SUBSTITUTE:

716.6.4 Structural System Anchorage: The design of the structural system and its elements for uplift, overturning moment, or horizontal shear, or their combination, shall not depend on more than sixty-seven (67) percent of the available resistance due to dead load effects. When, at joints between parts of the structure or at the foundation bearing level, the uplift, overturning moment, or horizontal shear, or their combination, is in excess of sixty-seven (67) percent of the available resistance due to dead load effects, the additional required capacity shall be provided by suitable connections and anchorage.

COMMENTARY: Structural System Anchorage

This is a new section. It is identical to Section 715.3.2 which covers anchorage of structural system for wind forces. Present Code had no similar requirements for anchorage for earthquake forces. Section 716.6.4 will correct that probably unintentional omission.

RL/ct(58-83)

INTERCONNECTIONS OF FOUNDATIONS

SECTION REQUIRING CHANGE: 716.6.9

1. Section 716.6.9

DELETE:

716.6.9 Interconnections of foundations: Pile, pier and caisson caps shall be interconnected by ties when the caps overlie Class B soil. Each tie shall carry by tension or compression a horizontal force equal to ten (10) per cent of the larger pile, pier or caisson cap loading, unless it can be demonstrated that equivalent restraint can be provided by other means. At sites where footings are underlain at shallow depths by cohesionless granular soils, the blow counts of which only slightly exceed the criteria given in Figure 720.1, adequate consideration shall be given to the lateral and vertical movements of footings that may occur during the design earthquake specified in Section 716.7.

SUBSTITUTE:

716.6.9 Interconnections of foundations: Pile, pier and caisson caps shall be interconnected by ties. Each tie shall carry by tension or compression a horizontal force equal to ten (10) percent of the larger pile, pier or caisson cap loading, unless it can be demonstrated that equivalent restraint can be provided by other means. At sites where footings are used, adequate consideration shall be given to the lateral and vertical movements of footings that may occur during the design earthquake specified in Section 716.7. Particular consideration shall be given to those sites where there are saturated cohesionless granular soils with blowcounts which only slightly exceed the criteria given in Fig. 720.1.

TRANSMISSION OF SEISMIC BASE SHEAR TO FOUNDATION SOIL OR ROCK

SECTION REQUIRING CHANGE: 716.4.6

1. Section 716.4.6

DELETE:

716.4.6 Lateral force on foundations: Provision shall be made for transmission of the base shear, acting in any direction, between structure and soil or rock, by means of one of the following:

1. lateral soil pressure against foundation walls, footings, grade beams and pipe caps;
2. lateral soil pressure against piles, piers, or caissons;
3. batter piles;
4. side or bottom friction on walls or footings; or
5. combinations of the foregoing.

Lateral pressure may not be more than one-third (1/3) the passive pressure. Bottom friction may not be relied upon where a building overlies Class B soil and is supported upon piles, piers or caissons. Even if not relied upon to transmit the base shear, foundation walls shall comply with the provisions of Section 716.6.10.

SUBSTITUTE:

716.4.6 Lateral force on foundations: Consideration shall be given to the manner in which the earthquake lateral force, computed in accordance with Section 716.4.1, will be transmitted from the soil or rock to the structure. Transmission of the lateral force will occur through one or more of the following foundation elements:

1. Lateral soil pressure against foundation walls, footings, grade beams, and pile caps;
2. Lateral soil pressure against piles, piers, or caissons;
3. Side or bottom friction on walls or footings;
4. Batter piles.

Bottom friction under pile caps should be assumed to be ineffective in transmitting horizontal forces.

The horizontal force shall be distributed among the various elements in the foundation in proportion to their estimated rigidities. Any element which will participate in the transfer of horizontal forces from the soil to the structure shall be designed to resist these forces in such a way that its ability to sustain static loads will not be impaired.

COMMENTARY: Transmission of Seismic Base Shear to Foundation Soil or Rock

Section 716.4.6 of the Code contains requirements for designing the foundation elements for transmitting the total seismic lateral force (design base shear) to (or from) the foundation soil. The intent of these provisions was not to cause the designer to modify or enlarge its foundation elements if the design computations indicate the soil not being capable of transmitting the force to the structure. If this were actually the case the design base shear could not be transmitted. Thus the seismic loads on the structure will be reduced, which is not an undesirable outcome. The intent of section 716.4.6 is that the structural foundation elements (foundation walls, pile and pile caps, footings) be designed so that they would not be damaged if the base shear forces are transmitted by the soil to the structure.

The base shear forces between the foundation elements and the soil will be distributed in proportion to the relative stiffness of all the elements involved. For example, in the case of a building on vertical piles with a deep basement, the base shear will be transmitted mostly through lateral pressure against the basement walls with very little shear being transmitted through the piles. For a building with no basement the base shear will be transmitted either through base friction for spread footings or through shear in the piles for pile foundations. It is suggested that one should assume that the full design base shear develops and that it is transmitted through the structure-soil system with the largest rigidity.

SOIL FACTORS

SECTION REQUIRING CHANGE: 201.0, 716.3, 716.4.1 (including Fig. 716.1), 716.7 (Fig. 716.2), 720.5, 720.6

1. Section 201.0

PRESENT CODE:

Contains definitions of: Class A soil
Class A soil site
Class B soil
Class B soil site

PROPOSED CHANGE:

Delete definition of: Class A soil
Class A soil site
Class B soil
Class B soil site

Add definitions as follows:

Soil Site S1: Bedrock of any type including material Classes 1 through 4 of Table 720.

Stiff soil conditions where the soil depth below foundation level is less than 200 ft and the soil types overlying bedrock consist of glacial till; gravel or well-graded sand and gravel, sands that are not susceptible to liquefaction in accordance with Section 720.4, clay having an undrained shear strength of at least one thousand (1,000) psf, dense silts and compacted granular fill provided that fill soils are compacted throughout as required in Section 720.3.1.

Soil Site S2: Soil sites that cannot be classified as Soil Sites S1 or S3.

Soil Site S3: Soil profiles that contain 30 ft or more of soft clays having an undrained shear strength smaller than 1,000 psf, loose silts, organic soils, loose sands, or miscellaneous fill.

2. Section 716.3, definition for S

DELETE:

S = Numerical coefficient as specified in Section 716.5.1

SUBSTITUTE:

S = Numerical coefficient as specified in Section 716.4.1

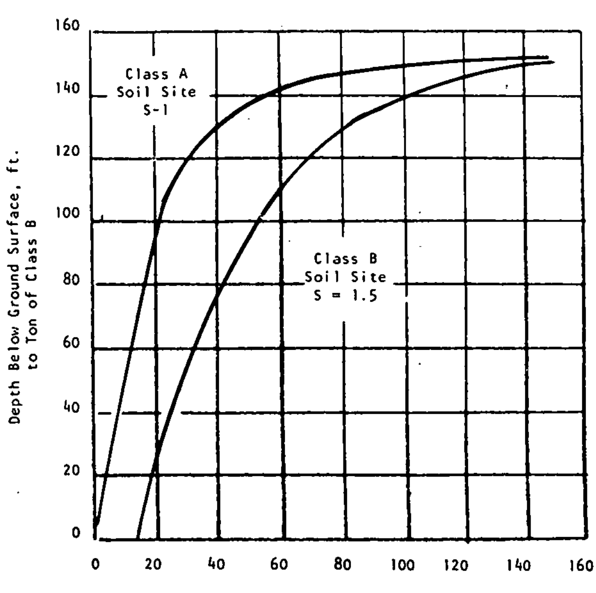
3. Section 716.4.1, Item 3, including Fig. 716.1

DELETE:

3. S factor: For a Class A soil site, S = 1. For a Class B soil site, S = 1.5. Intermediate values of S may be used as justified on the basis of Figure 716.1 or by the results of adequate studies by a registered professional engineer. The value of CS need not exceed zero point twelve (0.12). (See Section 720.5 for definition of Class A soil.)

(Figure 716.1 is deleted.)

Figure 716.1



SUBSTITUTE:

3. S factor. The S factor shall have the following values according to the types of soil sites as defined in Section 720.5:

Soil Site S1, S = 1
Soil Site S2, S = 1.2
Soil Site S3, S = 1.5

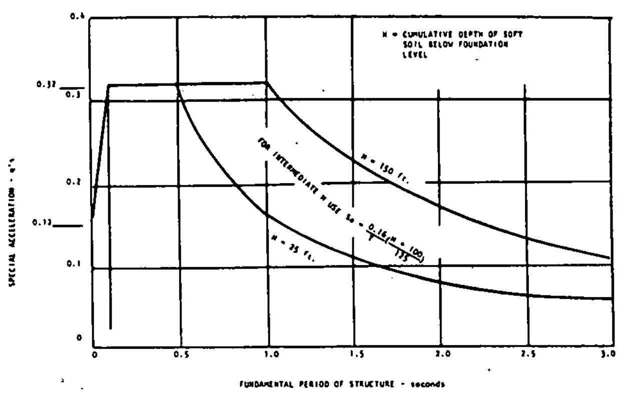
Values other than those tabulated may be used provided they are based on studies by a registered professional engineer and are not less than 1.0.

The values of CS need not exceed zero point twelve (0.12.)

4. Section 716.7, Fig. 716.2

DELETE:

Figure 716.2



SUBSTITUTE:

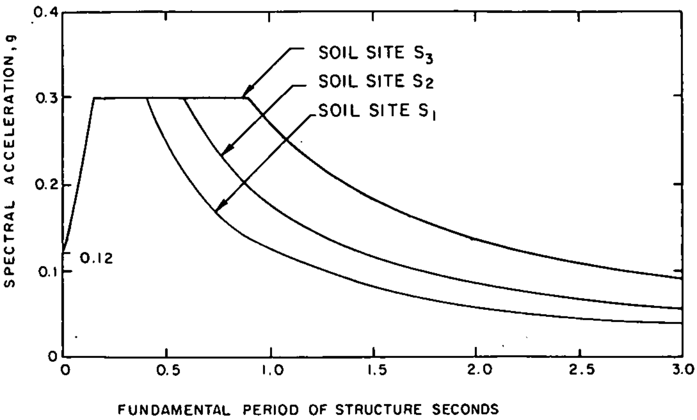


Fig 716.2 DESIGN RESPONSE SPECTRUM

5. Sections 720.5 and 720.6

DELETE:

720.5 Class A soils: For purposes of determining the S factor for earthquake design forces as specified in Sections 716.4 and 716.7, Class A soil includes the following classes from Table 720: massive igneous rocks and conglomerate; slate, shale in sound conditions, glacial till; gravel or well-graded sand and gravel, if dense to very dense; coarse sand, if dense to very dense; medium sand, if dense to very dense; fine sand, if dense to very dense; clay having an undrained shear strength of at least one thousand (1,000) psf; and compacted granular fill provided that fill soils are compacted throughout as required in Section 720.4 under continuous observations by a registered professional engineer or his authorized representative. (See Figure 716.1)

720.6 Class B soils: All other soils shall be considered Class B.

SUBSTITUTE:

720.5 Soil Factor S. For purposes of determining the S-factor for earthquake design forces as specified in Sections 716.4 and 716.7, the following types of soil sites are defined according to the materials encountered below the foundation level:

Soil Site S1: Bedrock of any type including material Classes 1 through 4 of Table 720. Stiff soil conditions where the soil depth below foundation level is less than 200 ft and the soil types overlying bedrock consist of glacial till; gravel or well-graded sand and gravel, sands that are not susceptible to liquefaction in accordance with Section 720.4, clay having an undrained shear strength of at least one thousand (1,000) psf, dense silts and compacted granular fill provided that fill soils are compacted throughout as required in Section 720.3.1.

Soil Site S2: Soil sites that cannot be classified as Soil Sites S1 or S3.

Soil Site S3: Soil profiles that contain 30 ft or more of soft clays having an undrained shear strength smaller than 1,000 psf, loose silts, organic soils, loose sands, or miscellaneous fill.

COMMENTARY: Soil Factors

Soil factors appear in two places within the seismic provisions of the Massachusetts Building Code: in Section 716.4.1 which gives the minimum total lateral seismic forces to be used for design of structures which qualify for the use of an equivalent static analysis, and in Section 716.7 which gives an elastic response spectrum which may be used as input for a dynamic analysis. The appropriate classifications of soils are given in Sections 720.5 and 720.6.

The provisions and definitions originally adopted are relatively cumbersome and have proved to give some difficulties in practice. In addition, they are quite different from those now used in other Codes within the United States. The new proposed definitions are essentially those developed for the ATC-3 provisions and those adopted in the 1982 ANSI standard.

In the proposed revisions, three categories of sites are defined, based upon both the quality and the depth of the soils that are presented. In Section 716.4.1, a numerical soil factor is assigned to each category. In Fig. 716.2 of Section 716.7, there are curves corresponding to each category. The soil factors are intended to provide for the effect of local soil conditions upon the amplitude and frequency content of ground motions. They also cover other possible adverse effects of poorer soils. For example, greater strength is desirable in buildings founded over ground that may experience differential settlement during an earthquake.

For the most part, the three categories are defined using words and phrases to describe different kinds of soils. It is intended that those words and phrases be interpreted in accordance with their common usage in soil mechanics practice. While some vagueness is inevitable in such definitions, with materials such as soils, it is necessary to leave some room for judgment by experienced geotechnical engineers. One exception is the introduction of a specified undrained shear strength to separate soft clays from firmer cohesive soils. Category 2, while also serving as a catch-all category, is specifically intended to include deep deposits of denser granular soils, provided the total thickness of poorer soils below foundation level is less than 30 ft. In the application of these provisions, it make no difference at which depth poorer soils are encountered. Having such soils at depth is as bad as having them just beneath a foundation. Foundation level is defined in Section 201.0.

The soil factors range from 1 to 1.5 for the lateral force equation and for 1 to 2 for the response spectrum. Since the response spectrum refers basically to elastic behavior, an increase in lateral forces of up to a factor of 2 appears reasonable, particularly for soft ground and long period buildings. The lateral force equation refers to elasto-plastic behavior of ductile structures for which the increase in lateral force would be smaller than elastic behavior, and thus a maximum factor of 1.5 is appropriate.

The soil factors for the lateral force equation apply over the entire range of building periods. In the response spectrum there is no increase in lateral force for soft ground if the building period is less than 0.5 sec. The elastic-plastic behavior assumed for the lateral force method will result in an increase of the building period during the earthquake and thus even for buildings with a small initial or elastic period, one should increase the lateral force for cases with soft ground. This is not necessary if the analysis is elastic.

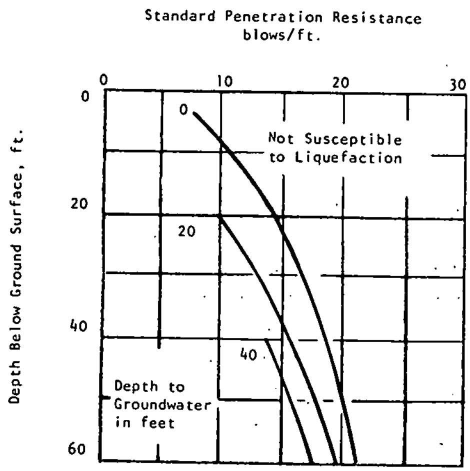
Interpolation between soil factors, or between the curves of Fig. 716.2, is permitted. However, it must be recognized that precision in the choice of soil factors is very seldom warranted and use of interpolation is not encouraged.

LIQUEFACTION

SECTION REQUIRING CHANGE: 720.4, Fig. 720.1

DELETE:

Figure 720



SUBSTITUTE:

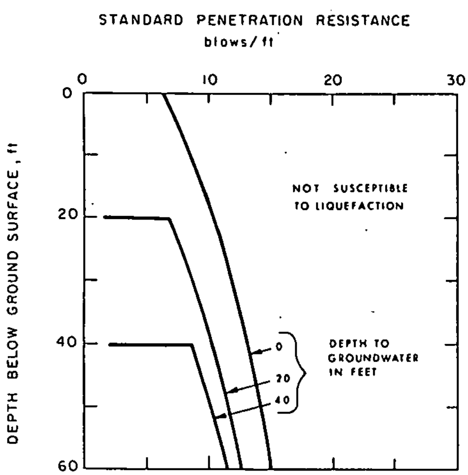


Fig. 720.1 - LIQUEFACTION POTENTIAL OF CLEAN FINE TO MEDIUM SANDS

COMMENTARY: Liquefaction

The Massachusetts Building Code contains provisions for evaluating the liquefaction potential of saturated sands for the design seismic event. These provisions are contained in Section 720.4 and in Fig. 720.1, which are reproduced in the appendix to this commentary. A revision to Fig. 720.1 has been prepared by the Seismic Advisory Committee of BSCES/ASCE. The purpose of this commentary is to present the background for the liquefaction regulations, the reasons behind the proposed change, and comments on its proper use.

Background

Liquefaction of saturated sands during earthquakes has been the cause of numerous failures of building foundations, embankments, and natural slopes (e.g., Dobry and Alvarez, 1967; Duke and Leeds, 1963; Flores and Dawson, 1977; Kishida, 1966; Lee and Albaisa, 1974; Marsal, 1961; Ohsaki, 1966; Ross et al., 1969; Seed, 1968; Seed et al., 1969; Seed et al., 1975; Youd, 1975; Youd and Hoose, 1976). Perhaps the best known examples are the foundation failures in the San Fernando earthquake of 1971 and in the 1964 earthquake in Niigata, Japan; the slope failures in Alaska in 1964; and the slides in the Lower San Fernando Dam in California in 1971. Buildings supported on sands that liquefy may develop settlements from several inches to several feet. Since settlements are rarely uniform, severe tilting or distortion may occur and collapse of the structure is possible.

Currently, there are two basic approaches for evaluating the susceptibility of saturated sand deposits to earthquake-induced liquefaction, namely:

1) analytical procedures based on the results of laboratory tests on high quality "undisturbed" samples.

2) an empirical in situ index test approach based on correlations between the standard penetration test blowcounts (SPT) and the observed occurrence or nonoccurrence of ground failure at sites subjected to past earthquakes.

Analytical procedures based on the results of laboratory tests on "undisturbed" samples are expensive, time consuming, and involve significant uncertainties.

Empirical correlations relating the standard penetration resistance of sands, the cyclic shear stresses induced by earthquakes, and the occurrence or nonoccurrence of ground failure have been developed by several investigators (e.g., Whitman, 1971; Castro, 1975; Seed, 1979; Seed and Idriss, 1981; and Seed et al., 1983).

The existing empirical correlations have been based on the standard penetration test because it was the most readily available index for the sites for which there was information on the actual behavior of sands during earthquakes. Furthermore, standard penetration is obtained routinely during most subsoil investigations in Massachusetts. The standard penetration test is rather crude and probably does not represent fully those soil properties which determine the susceptibility of a sand to liquefaction. The empirical correlations indicate numerous cases for which sands with similar blowcounts and subjected to similar earthquakes have behaved very differently. Nevertheless, there is an upper bound of blowcounts above which earthquake-induced liquefaction has never been observed for a given intensity of shaking. It is this upper bound that was used to develop the blowcount criteria in the Massachusetts Code.

The blowcount criteria presently in the Code were based on the upper bound curve in Castro, 1975, Fig. 1, for a peak ground surface acceleration of 0.12g, in agreement with the design response spectra in Fig. 716.2 of the Code. The criteria in Castro 1975 does not discriminate among earthquakes of different magnitudes; however, the position of the upper bound curve is strongly influenced by data from the magnitude 7.5 Niigata earthquake. More recent studies, e.g., Seed, 1983, have led to upper bound curves which are a function of earthquake magnitude. The proposed revision to the present blowcount criteria is based on the data in Fig. 2 taken from Seed et al., 1983 where the earthquake data is plotted using the following coordinates:

a) The modified penetration resistance is the actual blowcount normalized to a confining pressure of 1 tsf by a factor C_N so that

$$N_1 = C_N \cdot N$$

where C_N is a function of the overburden pressure at which N was measured, Fig. 3

b) The cyclic stress ratio which is equal to:

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

where a_{max} = maximum ground surface acceleration

σ_v = total overburden pressure at the depth being investigated

σ_v' = effective overburden pressure at the depth being investigated

r_d = a stress reduction factor (equal to 1 at the ground surface and about 0.9 at a depth of 30 ft)

The upper bound line in Fig. 2 was used to develop the proposed revised criteria assuming a maximum ground surface acceleration of 0.12g and an earthquake magnitude of 6.5. The proposed revised criteria are compared with those presently in the Code in Fig. 4.

Discussion of Assumptions

It is recognized that the design acceleration of 0.12g is intended to correspond to "firm ground" and that for loose sands the peak ground surface acceleration could be larger or smaller. There are many uncertainties in the seismicity of Massachusetts and thus it was felt that to make an attempt to estimate potential amplification or deamplification in a loose sand deposit was an unwarranted refinement.

The upper bound criteria in Figs. 1, 2, and 3 represent a conditional probability of failure of about 10% (Christian and Swiger, 1975). This probability applies to the case in which the design earthquake is assumed to occur and the site has conditions that place it at the upper bound in Fig. 1 or on the proposed criteria line in Fig. 2. It is recognized that the 10% conditional probability is higher than the corresponding probability of failure for the design of the structural elements. The higher probability for liquefaction is justified because of the large cost generally associated with measures to avoid the problems associated with liquefaction.

Application of the Criteria

If all blowcounts for sands below the groundwater level lie above the boundary lines in Fig. 2, no further consideration of liquefaction is indicated. Often, however, one finds sites for which some of the blowcounts fall below the line. Whether in such a case further consideration of liquefaction is required would depend on the judgment of a geotechnical engineer based on the following factors:

a) Depth at which the low blowcounts are found. A substantial thickness of nonliquefiable soils overlying soils suspect of being liquefiable will greatly decrease the effects at the ground surface of liquefaction at depth.

b) If the low blowcounts occur randomly through an otherwise dense sand, their effect on the overall behavior of the foundation soils will be small. On the other hand, liquefaction of a continuous loose layer at shallow depths can result in severe effects at the ground surface.

c) A structure that is particularly sensitive to settlement will suggest applying the liquefaction criteria more rigorously. A structure with a rigid foundation and basement box-like structure will be able to sustain large settlements without collapse, and thus, in such cases, one can allow a substantial number of blowcounts below the criteria.

d) If the subsoils were to liquefy, structures that apply to the subsoils large net bearing pressures would settle more than lighter structures.

The criteria in Fig. 2 are applicable to saturated clean (less than 12% fines) sands. Generally for silty sands the criteria in Fig. 2 are conservative; however, in our opinion, there are not, as yet, sufficient data on these soils to develop blowcount criteria applicable to silty sands and suitable for a Code regulation. Clayey sands or any soils exhibiting some plasticity should be considered not susceptible to liquefaction unless they were to have a very high sensitivity (e.g., similar to the Norwegian quick clays).

Measurements of blowcounts are sensitive to the techniques used. It is beyond the scope of this commentary to discuss these techniques in detail. A common error in performing the test is not to maintain the water level at the top of the borehole. If the water level in the borehole drops below the water level in the surrounding soil, for example, as a result of withdrawal of drilling tools, then an upward flow of water at the bottom of the borehole develops, resulting in loosening of the soil to be sampled and thus in erroneously low blowcounts.

If the judgment is made that there are too many blowcounts below the criteria, the following courses of action may be considered:

a) Verify the blowcount data by performing additional standard penetration tests with careful control of the procedures.

b) Perform a more detailed investigation with undisturbed sampling and testing to determine whether there is actually a liquefaction problem.

c) Design a type of foundation that will minimize the effects of liquefaction, e.g., a pile foundation or a rigid mat floating foundation.

d) Densify the sands in situ.

e) Excavate and replace the loose sands.

References

Castro, G. (1975) "Liquefaction and Cyclic Mobility of Saturated Sands," Journal of the Geotechnical Engineering Division, ASCE, Vol. 101, No. GT6, pp. 551-569.

Christian, J. T. and Swiger, W. F. (1975), "Statistics of Lique faction and SPT Results," Journal of the Geotechnical Engineering Division, ASCE, Vol. 101, No. GT11, pp. 1135-1150.

Dobry, R. and Alvarez, L. (1967) "Seismic Failures of Chilean Tailings Dams," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 93, No. SM6, pp. 237-260.

Duke, C. M., and Leeds, D. J. (1963) "Response of Soils, Foundations and Earth Structures to the Chilean Earthquakes of 1960," Bulletin, Seismological Society of America, Vol 53, No. 2, pp. 309-357, Feb. 1963.

Flores, R. J. and Dawson, A. (1977) "A Liquefaction Case History, Chiapas, Mexico ," Proceedings, Ninth Internatinal Conference on Soil Mechanics and Foundation Engineering, Tokyo, Vol 2, pp. 237-240.

Kishida, H. (1966) "Damage to Reinforced Concrete Buildings in Niigata City with Special Reference to Foundation Engineering," Soils and Foundation, Tokyo, Japan, Vol. 7, No. 1.

Lee, K. L. and Albaisa, A. (1974) "Earthquake Induced Settlements in Saturated Sands," Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, No. GT4, pp. 387-406.

Marsal, R. J. (1961) "Behavior of a Sandy Uniform Soil During the Jaltipan Earthquake, Mexico," Proceedings, Fifth International Conference on Soil Mechanics and Foundation Engineering, Paris, Vol. 1., pp. 224-233.

Ohsaki, Y. (1966) "Niigata Earthquakes, 1964, Building Damage and Soil Conditions," Soils and Foundation, Tokyo, Japan, Vol. 6, No. 2, pp. 14-37.

Ross, G. A.; Seed, H. B and Migliaccio, R. R. (1969) "Bridge Foundation Behavior in Alaska Earthquake," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 95, No. SM4, pp. 1007-1036.

Seed, H. B. (1968) "Landslides During Earthquakes Due to Soil Liquefaction," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 94, No. SM5, pp. 1055-1122.

Seed, H. B.; Lee, K. L. and Idriss, I. M (1969) "Analysis of Sheffield Dam Failure," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 95, No. SM6, pp. 1453-1490.

Seed, H. B.; Lee, K. L.; Idriss, I. M. and Makdisi, F. I. (1975) "The Slides in the San Fernando Dams During the Earthquake of February 9, 1971," Journal of the Geotechnical Engineering Division, ASCE, Vol. 101, No. SM7, pp. 651-688.

Seed, H. B. (1979) "Soil Liquefaction and Cyclic Mobility Evaluation for Level Ground During Earthquakes," Journal of the Geotechnical Engineering Division, ASCE, Vol. 105, No. GT2, pp. 201-255.

Seed, H. B. and Idriss, I. M. (1981) "Evaluation of Liquefaction Potential of Sand Deposits Based on Observations of Performanc in Previous Earthquakes," Insitu Testing to Evaluate Liquefaction Susceptibility, Proc. of Session No. 24, ASCE National Convention, St. Louis, Missouri.

Seed, H. B. and Idriss, I. M. (1983) "Evaluation of Liquefaction Potential Using Field Performance Data," Journal of the Geotechnical Engineering Division, ASCE, Vol. 109, No. 3, pp. 458-482.

Whitman, R. V. (1971) "Resistance to Soil Liquefaction and Settlement," Soils and Foundations, Tokyo, Japan, Vol. 11, No. 4 , pp. 59-67.

Youd, T. L. (1975) "Liquefaction, Flow and Associated Ground Failure," Proceedings of U.S. National Conference on Earthquake Engineering, Earthquake Engineering Research Institute, Ann Arbor, Michigan, 1975, pp. 146-155.

Youd, T. L. and Hoose, S. N. (1976) "Liquefaction During 1906 San Francisco Earthquake," Journal of the Geotechnical Engineering Division, ASCE, Vol. 102, No. GT5, May 1976, pp. 425-439.