JOURNAL OF THE

Boston Society of Civil Engineers Section American Society of Civil Engineers

1848 BOSTON SOCIETY OF CIVIL ENGINEERS

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

VOLUME 66 • NUMBER 2 • SUMMER 1979

JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS SECTION

Contents

AMERICAN SOCIETY OF CIVIL ENGINEERS

PROFESSIONAL PAPERS

PROCEEDINGS

Second-Class postage paid at Boston, Massachusetts

Published four times a year, Spring, Summer, Autumn, Winter, by the Society
80 Boylston Street, Boston, Massacusetts 02116

Subscription price $10.00 a Year (4 Copies $2.50 a copy)

All orders must be accompanied by check. Members' subscriptions at $5.00 are part of their dues.

VOLUME 66 • NUMBER 2 • SUMMER 1979

B.S.C.E. SECTION OFFICERS 1979-1980

PRESIDENT
William S. Zoino

VICE PRESIDENTS

Frank E. Perkins
(Term expires 1980)

Edward B. Kinner
(Term expires 1981)

SECRETARY
Leo F. Peters

TREASURER
Norman W. Bennett

DIRECTORS

Robert L. Meserve
Stanley C. Rossier
(Terms expire 1980)

Peter K. Taylor
Paul J. Trudeau
(Terms expire 1981)

PAST PRESIDENTS - BOARD MEMBERS

Bertram Berger | Charles C. Ladd | Howard Simpson

VICE PRESIDENT, WESTERN MASS. BRANCH, John Collura
NATIONAL VICE PRESIDENT, ZONE I, Cranston R. Rogers
NATIONAL DIRECTOR, DISTRICT 2, S. Russell Stearns

TECHNICAL GROUPS

COMPUTER
Robert A. Wells, Jr., Chairman; Glenn Orenstein, Clerk
CONSTRUCTION
John R. Roma, Chairman; Robert D. Lignowski, Clerk
ENVIRONMENTAL
Benjamin J. Fehan, Chairman; Gerald C. Potamis, Clerk
GEOTECHNICAL
Anthony Barila, Chairman; Asaf A. Qazilbash, Clerk
HYDRAULICS
Russell Peterson, Chairman; Robert H. Taylor, Jr., Clerk
STRUCTURAL
Richard A. Foley, Chairman; Kenneth B. Wiesner, Clerk
TRANSPORTATION
Rocco Mancini, Chairman; Robert A. Snowber, Clerk

(Committee Chairmen are listed on inside back cover)

JOURNAL EDITOR — Edward C. Keane
NEWSLETTER EDITOR — Leo F. Peters
EXECUTIVE DIRECTOR — Susan Albert
ADMINISTRATIVE SECRETARY — Barbara Dickenson

JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS SECTION

Professional Papers

AMERICAN SOCIETY OF CIVIL ENGINEERS

MASSACHUSETTS EARTHQUAKE DESIGN REQUIREMENTS

By Rene W. Luft,[1] M. ASCE, and Howard Simpson,[2] F.ASCE

Introduction

The earthquake design provisions of the Massachusetts State Building Code (subsequently referred to as the Code) are the first seismic criteria developed specifically for a jurisdiction in the eastern United States. These provisions represent the recommendations of an ad hoc committee appointed jointly by the Boston Society of Civil Engineers and the Massachusetts Section of the American Society of Civil Engineers in July 1973. While the provisions are largely based on the 1973 edition of the Uniform Building Code [39] (hereinafter referred to as "UBC-1973") there are a number of significant differences. This paper describes the underlying philosophy and objectives of the seismic provisions, and the design decisions on which they are based. Some of the innovative provisions and certain provisions of special interest are discussed in detail.

The history behind the decision to develop a seismic code for Massachusetts is well documented by Krimgold [11]. The initial task of the ad hoc committee was to determine whether Massachusetts needed a seismic code. To arrive at this decision, seismic risk and cost-benefit analyses previously conducted as part of the Seismic Design Decision Analysis project at M.I.T. were studied. These also served to help define the objectives of the proposed criteria, and to guide policy decisions and design philosophy. The primary finding of these studies was that the probable maximum earthquake intensities for Massachusetts are as great as those for Zone 3 regions in California, but have much longer return periods. Because of the long return periods for destructive earthquakes, it was found that the cost to society of the earthquake-resistant design would be considerably greater than the projected savings in damage and loss of life due to an earthquake. Nevertheless, the committee felt that society would insist on reasonable

[1] Associate, Simpson Gumpertz & Heger Inc., Consulting Engineers, Cambridge, Massachusetts

[2] Senior Principal, Simpson Gumpertz & Heger Inc., Consulting Engineers, Cambridge, Massachusetts

measures to mitigate the number of casualties from a major seismic event. It was therefore considered necessary to develop a set of criteria which would minimize the projected loss of life resulting from such an event, without causing construction costs to increase by an unacceptable amount.

The cost/benefit studies lead to the conclusion that the protection of life safety could be achieved for new construction at minimum cost by comprehensive ductility requirements, rather than by requirements for large lateral resistance. All the ductility requirements in the Code were placed in one section to emphasize their importance.

An innovative, comprehensive set of soil and foundation design requirements was developed for the Code. These include a soil factor for lateral forces which depends on the class of soil, guidelines to check susceptibility to liquefaction of saturated cohesionless soils, guidelines to check the transfer of earthquake forces between foundations and the soil, and a requirement to consider dynamic active soil pressures on free-standing retaining walls and basement walls.

The advisory committee's recommendations were embodied in Article 7 of the Code promulgated by the Massachusetts State Building Code Commission, effective January 1, 1975.

Seismicity of Massachusetts

Historical Seismicity. — Since 1643, 19 earthquakes with intensities of Modified Mercalli V (MM V) or greater and with epicenters in, or offshore, Massachusetts have been recorded historically or instrumentally. In addition, 43 earthquakes of this same intensity range that occurred out-of-state have been felt in Massachusetts; they occurred primarily in the regions of the St. Lawrence Valley and the Laurentian Trough, upstate New York, central New Hampshire, Vermont, the Connecticut River mouth, and Narragansett Bay. The epicentral intensities of the 62 earthquakes ranged up to MM VIII. Most of the assigned intensities are estimates. Extensive information on seismicity in Massachusetts can be found in "Historical Seismicity of New England," [33] "Earthquake History of the United States," [31] "Earthquake History of Massachusetts," [24] and "Our New England Earthquakes" [34].

The strongest known earthquake to affect Massachusetts occurred on November 18, 1755. The location has been estimated at about 10 miles east of Cape Ann. Its epicentral intensity estimate in reference [31] is MM VIII; the corresponding maximum intensity observed in Boston was estimated by Weston Geophysical as MM VII. [33] The earthquake was felt in a region estimated to be between 300,000 sq. mi. and 385,000 sq. mi. in area. Research by Weston Geophysical indicates that most of the MM VII damage in Boston occurred in "an area that had been filled near the Wharves." [34] Little or no damage occurred on Beacon Hill, which is an area of very stiff glacial deposits. To the extent that the estimates of the Cape Ann earthquake epicentral intensity have been extrapolated from its effect in the extensive fill areas in Boston, they may be high.

Seismic Risk Studies. — Statistical seismic risk studies are based on past earthquakes, on empirical attenuation laws, and on geological data such as location of active faults and pertinent subsurface conditions [4]. The first step in the development of a seismic risk model is the identification of the earthquake sources; these may be points, lines, or areas. For each source, a relationship based on historical information is established between number of earthquakes per unit time and earthquake magnitude or intensity. Then attenuation laws for intensity or acceleration are obtained for the subsurface conditions of the region. Finally, the annual probability of earthquake occurrence for a given site is computed as a function of intensity or peak ground acceleration. Tong et al.,[22] at the request of the seismic committee, developed seismic risk maps for the state for return periods of 100 years, 1,000 years, 100,000 years, and 1,000,000 years, using three assumptions for earthquake sources. Table 1 shows the results for a Boston site. Table 2 (from reference [14] shows results obtained by Cornell and Merz, [5] and the extrapolations of results obtained by Algermissen and Perkins [1]. Cornell and Merz, adjusting historic data for soil effects, assumed a maximum firm ground epicentral intensity of 8.2, while Algermissen and Perkins assumed a maximum epicentral intensity of 9.0.

TABLE 1

RESULTS OF SEISMIC RISK STUDIES FOR BOSTON
Return Period versus Intensity

		Tong et al. [22]	
Return Period (years)	Assumption C-1 MM Intensity	Assumption C-2 MM Intensity	Assumption C-3 MM Intensity
500	6.0	6.4	6.8
1,000	6.4	6.7	7.2
5,000	7.1	7.6	8.0
10,000	7.4	7.9	8.3

A major study of earthquake activity in New England was performed in connection with Boston Edison's Pilgrim Nuclear Power Station at Plymouth, Massachusetts [32]. A result of this study is a theory, verified for only the Cape Ann earthquake of 1755 and the Lake Ossipee, New Hampshire, earthquakes of December 1940, which states that major earthquakes in New England will have epicenters located near the

TABLE 2

RESULTS OF SEISMIC RISK STUDIES FOR BOSTON
Return Period versus Acceleration

	Cornell and Merz [5]		Algermissen and Perkins [1]	
Return Period (years)	Most Likely Risk Curve (% g)	Bayesian Risk Curve (% g)	Estimated from Seismic Map (% g)	Adjusted to max. MMI used by C&M [5] (% g)
500	3.5	3.5	10.0*	5.4***
1,000	4.0	4.0	13.5**	7.4***
5,000	6.2	9.0	26.9**	14.6***
10,000	8.0	14.0	36.3**	19.8***

* Return period is actually 475 years
** Estimated from $(a_1/a_2) = (T_1/T_2)^{0.43}$
*** Estimated from $\log (a_1/a_2) = 0.33*(I_1-I_2) = 0.264$

boundaries of certain plutonic rock intrusions of the White Mountains magma series [32], [34]. Should this theory be further validated, it will affect the results of Tong et al. [22] and of Algermissen and Perkins [1], primarily for the eastern part of Massachusetts. This is because the theory is inconsistent with the assumption made in both studies that an earthquake of magnitude similar to that of the Cape Ann earthquake could occur with equal probability anywhere within certain regions surrounding Cape Ann in Massachusetts and New Hampshire.

Since historical estimates of epicentral intensities are based on extrapolations from reported damage which in turn is influenced by local soil conditions, the epicentral intensity tends to be biased by the more severe damage observed over soft soils. Therefore, to arrive at a nominal design earthquake for firm ground, the seismic committee reduced Modified Mercalli intensities of historical earthquakes by approximately one-half unit. If such an adjustment is not made, local soft soil effects are actually taken into account twice in the formula for base shear: once, by the multiplicative soils factor, and then by the magnitude of the response spectrum which is set for each zone based on historical epicentral intensities. The acceleration map proposed in Applied Technology Council's publication ATC 3-06 [38] is based on the results labeled "maximum acceleration in rock" given by Algermissen and Perkins [1]; with respect to Massachusetts, however, the latter data apparently have not been adjusted for damage observed over soft soil.

Design Earthquake. — The result of the seismic studies was the definition of a nominal design earthquake. This nominal earthquake has a peak ground acceleration on firm soil of 0.12 g; it corresponds approximately to an epicentral intensity of between MM VII and MM VIII. The return period of this nominal earthquake for a Boston site is approximately 5,000 years; because of uncertainty in seismic risk, the bounds of this return period are

2,000 years and 10,000 years (see Tables 1 and 2). Since the estimated variation in seismic risk across the State falls approximately within the error bound associated with the determination of the level of risk, it was decided to use the same nominal design earthquake for the entire State.

Assuming a design life of 100 years, the probability of a structure experiencing an event exceeding the nominal design earthquake during its design life is approximately 2 percent.

Two decisions were based on the design earthquake. First, a zone factor equal to 3/8, later changed to 1/3, was selected for use in a formula for base shear similar to that of UBC-1976 [39]. Second, an elastic design spectrum for firm ground was defined consistent with the Newmark-Hall [16] response spectrum for structures with 5 percent damping; the design parameters for the spectrum are 0.12 g peak ground acceleration and 5.2 in./sec peak ground velocity. A later section describes how this response spectrum must be modified to account for soil amplification.

Design Philosophy of Seismic Code

The stated purpose of the seismic design provisions of the Code is "to protect life safety by limiting structural failure." It is not the purpose of the Code to limit structural damage, except when damage may pose a life hazard. The objectives of the Code should be compared with those of the SEAOC Recommendations [35] which are to:

"(i) resist minor earthquakes without damage;
"(ii) resist moderate earthquakes without structural damage, but with some nonstructural damage; and
"(iii) resist major earthquakes, of the intensity of severity of the strongest experienced in California, without collapse, but with some structural as well as nonstructural damage."

The fundamental reason for the differnce in purposes resides in the expected return periods for strong earthquakes: relatively short for California, say 100 years [23], and relatively long for Massachusetts, say 5000 years [22]. Because of the low annual probability of occurrence of damaging earthquakes in Massachusetts, the additional cost associated with improved seismic resistance in most new construction could not be economically justified. It was considered appropriate, however, to adopt measures to mitigate the loss of life due to a possible earthquake, no matter how infrequent its occurrence, with the understanding that the economic impact of these measures must not, in general, create an unacceptable financial hardship. It is the recognition of this distinction between designing to reduce expected damage and designing solely to protect life safety that leads to seismic provisions conceived especially for Massachusetts. The approach of the model codes (Uniform Building Code, BOCA Basic Building Code, National Building Code, American National Standards Institute A58.1), of adopting the SEAOC Recommendations and incorporating a zone factor to lower the base shear, is not valid for Massachusetts: it considers only the largest magnitude (or intensity or peak ground acceleration) to be expected at a site, and not the recurrence interval of strong earthquakes. The proposed seismic regulations of ATC 3-06 [38] are a step in the right direction: the design requirements are based on the same probability for the entire United States of not exceeding the design earthquake in 50 years.

Experience and numerous studies have shown that a structure will not collapse if it has sufficient ductility to absorb the energy of shaking by inelastic deformation, unless the lateral deformations are so large as to make the structure unstable due to P-delta moments. Therefore, a minimum lateral strength and stiffness are required; this is achieved by specifying minimum design lateral forces and by setting a limit on acceptable drift. The Code uses, from UBC-1973, the wording that drift "shall be considered in accordance with accepted practice." While there is disagreement sometimes as to what constitutes accepted practice, the drift may be considered acceptable if the vertical load-carrying capacity of the structure is not significantly impaired by the anticipated earthquake distortions, and if no element of the building, such as an exterior panel or curtain wall, collapses due to the earthquake distortions.

The Code provides for two optional approaches to obtaining design forces and moments. The first is a set of rules adapted from UBC-1973; they prescribe the minimum base shear, the methods of distributing the total lateral force, and the design requirements for minimum ductility. These rules in their entirety may be used only if the prescribed ductility requirements are met. When they are not met, the designer must use the alternative approach: A structure is considered adequate if studies show that there would be "negligible risk to life safety" if the structure experienced an earthquake with a firm ground peak acceleration of 1.12 g, with a frequency content defined by a specified response spectrum. The studies to be performed for compliance with the alternative rule must show that the structure can safely withstand the displacements and distortions caused by the design earthquake; that is, that the structure will not collapse. The intent of the Code is to provide an alternative for unconventional construction involving members or connections that do not conform to, or cannot be shown to conform to, the ductility requirements. Proof that a structure has adequate seismic resistance may be based on dynamic analysis, on a history of successful performance of a comparable structure with similar foundation conditions in actual earthquakes of an intensity equal to or higher than the design earthquake, on dynamic testing, or on other appropriate procedures.

Lateral Force Requirements

Design Base Shear. — The formula for base shear in the Code,

$$V = 1/3KCSW, \tag{1}$$

is similar to that contained in UBC-1976, with Z equal to 1/3 and C as given by UBC-1973. Fig. 1 shows the maximum and minimum values of 1/3 KCS according to the Code; for comparison, the corresponding values from UBC-1976 for Zone 2 and from ATC 3-06 for Massachusetts also are shown. The upper curve for ATC 3-06 corresponds to types of construction permitted by the Code as a prerequisite for use of the above base shear. The lateral forces specified by the Code are lower than those of UBC-1976 and ATC 3-06; the largest differences occur for short periods.

The maximum ground accelerations for the zones used in UBC-1973 are, according to Housner [8], 0.08 g for Zone 1 and 0.16 g for Zone 2. Massachusetts, with a design earthquake of 0.12 g peak firm ground acceleration, is therefore in a zone intermediate between 1 and 2. Consistent with this, the zone coefficient was set as 1/3 for the entire state.

The base shear formula incorporates a soil factor S that is, in contrast to that in UBC-1976, independent of the period of the structure. S is 1.0 for a Class A soil site (see section on Foundation Design for definition of classes of soil), and is 1.5 for a Class B soil site. Intermediate values of S may be used; these may be obtained by interpolation between the curves for S equal to 1.0 and 1.5 in Fig. 2, or by appropriate geotechnical studies.

The justification of the use of a soil factor that is independent of building period is based on theoretical studies of soil amplification, on statistical studies of spectral shapes for actual earthquakes, and on studies of inelastic response spectra for site-modified ground motions, combined with a desire for conservatism in the design of tall buildings. From soil amplification studies, Whitman [26] has shown that such a soil factor is reasonable for a shallow soil deposit; he has also shown that a soil factor that is constant for building periods shorter than the soil period, but decreases for longer building periods, is reasonable for other cases, such as a deep deposit of firm soil or a shallow soft soil overlying a deep deposit of firm soil. Seed et al. [21] based on a statistical study, covering 104 past earthquakes, of the elastic response spectra of structures close to the epicenter, concluded that a soil amplification factor constant with respect to building period, but applied only to the region on the response spectrum that depends on peak ground velocity, is an appropriate simplification, for design purposes, of the earthquake data studied. Whitman and Protonotarios [27] have shown that, while elastic response spectra for site-modified ground motions display a

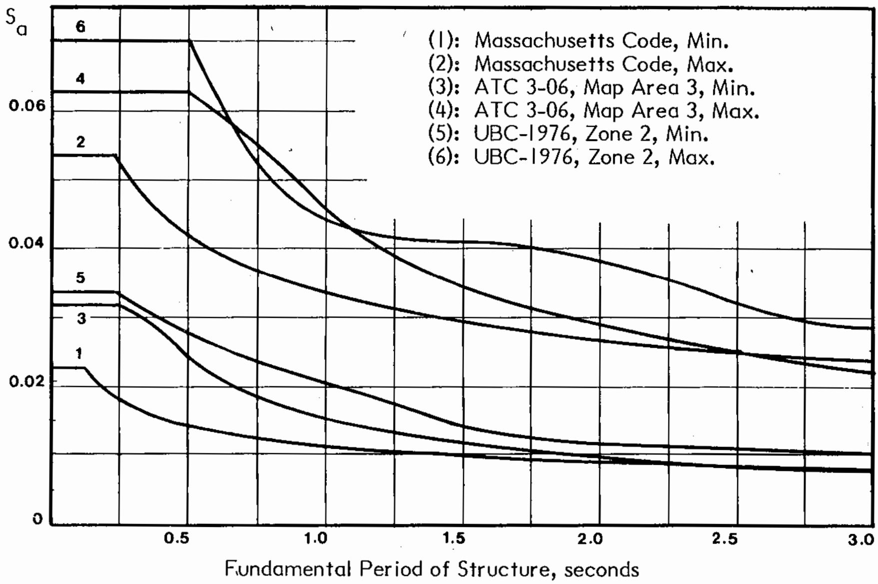

FIG. 1 - Lateral Force Coefficients for Massachusetts in Percent of g

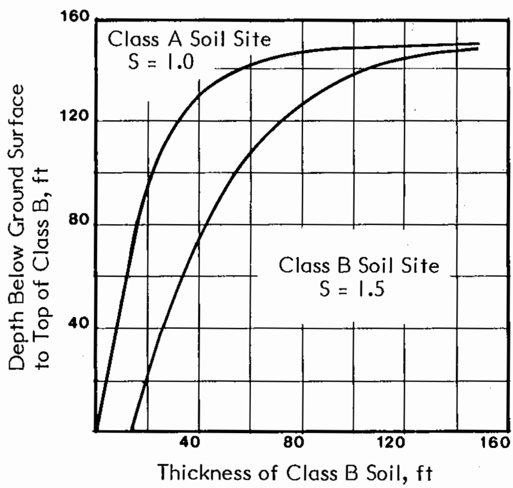

FIG. 2 - Determination of Soil Factor S

resonance effect, the corresponding inelastic response spectra do not, for ductility factors larger than 2, have a pronounced peak at the fundamental period of the site. In summary, a soil factor as defined in the Code, and also as proposed by ATC 3-06 for the velocity-dependent region of the response spectrum [38], accounts adequately for the effect of soil amplification on inelastic structural response. Such a factor is desirably conservative for very tall buildings founded over soft soil (buildings with a period longer than the longest natural period possible for any soil profile) and for stiff buildings over soft soil.

Design Spectra. — The design earthquake is specified by the Code as a family of site-dependent elastic response spectra. The firm ground response spectrum is the Newmark-Hall elastic spectrum for 5-percent damping [16]. A soil is considered firm if the cumulative depth of soft soil, H, below foundation level is less than 25 ft. A soil amplification effect is introduced for H larger than 25 ft, but H need not be taken as larger than 150 ft; the soil effect is introduced by modifying the peak ground velocity from 5.2 in/sec for firm ground to 5.2S(H) in/sec for soft ground where

$$S(H) = \frac{H + 100}{125} \qquad 25 \leqslant H \leqslant 150 \qquad (2)$$

The soil factor S included in the formula for base shear differs from the soil amplification effect defined by S(H). First, the factor S applies to all building periods, while S(H) applies only to that portion of the spectrum that depends on peak ground velocity; second, the factor S is less than or equal to 1.5, while S(H) can reach 2.0 for soft soil layers of 150 ft or deeper; and finally, for shallow surface deposits of soft soil (12 ft$\leqslant$H$\leqslant$25 ft) the factor S is 1.5, while S(H) equals 1.0.

Methods of Analysis. — For a building design which satisfies the ductility requirements, the minimum base shear is given by Eq. 1. This base shear must be distributed among the lateral force-resisting elements by the use of either a static or a dynamic analysis. The dynamic analysis is prescribed for "structures which have highly irregular shapes, large differences in lateral resistance or stiffness between adjacent stories, or other unusual structural features affecting seismic response." The decision to perform a dynamic analysis is a matter of engineering judgment; however, considering the cost of performing a dynamic analysis, it is anticipated that engineers generally will strive to achieve regular structures by interaction with architects and by proper choice of framing.

The static analysis rules are those of UBC-1973, except that new rules were developed for setbacks. These latter are similar to, but simpler than, those given in Appendix C of the SEAOC Recommendations [35].

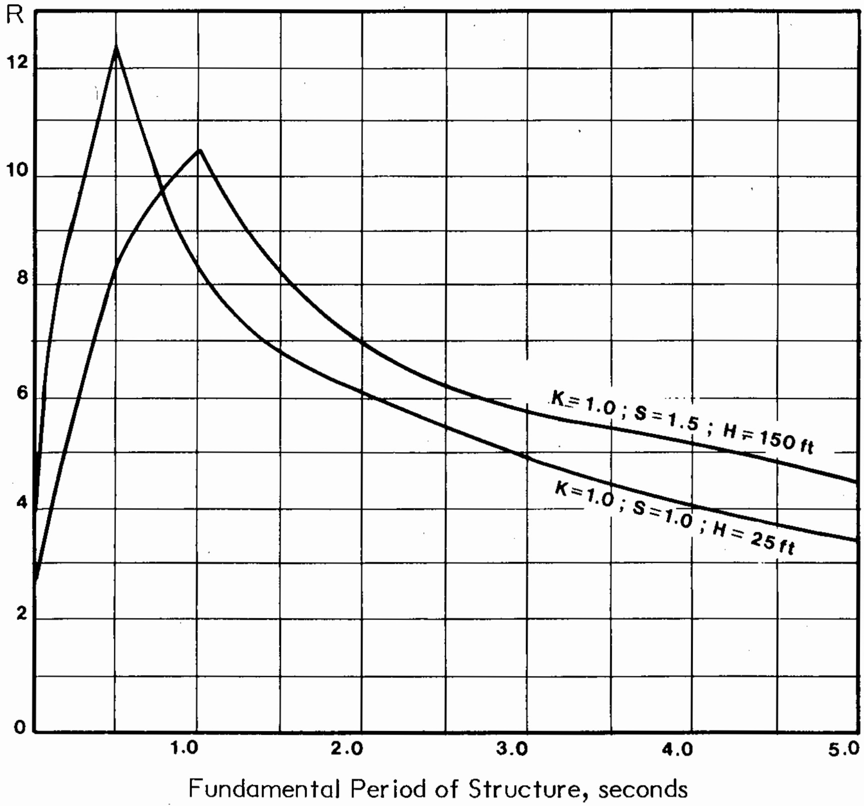

FIG. 3 - Scale Reduction Factor to Obtain Minimum Total Lateral Force from Design Response Spectrum

The dynamic analysis rules for buildings meeting the ductility requirements permit the use of either modal decomposition or of ground motion time-histories consistent with the elastic response spectrum. The combined modal base shear or the peak base shear as a function of time need not exceed that required in Eq. 1 for the period of the fundamental mode; therefore, the elastic response spectrum may be scaled down accordingly.

For a modal analysis, the modal accelerations must be obtained from the elastic spectrum that corresponds to the soil at the site. The Code does not specify a rule for combining modes, nor does it specify a minimum number of modes to be used. The authors recommend that the modes be combined by the square root of the sum of the squares (SRSS) method. The number of modes used should be as follows: for two- and three-story buildings, all modes; for buildings with four or more stories, the greater of (a) the lowest three modes, (b) all modes with periods longer than 0.4 seconds, or (c) a sufficient number of modes to account, by summing the effective modal masses, for 95 percent of the total mass.

To match the minimum base shear the elastic spectrum will always be scaled down. It is of interest to compute selected scale factors, using the SRSS method for combining modes, to assess the degree of conservatism implied by the elastic spectrum. As an example, take a structure where:

- The ratio of the second mode period to the first mode period is 0.25, and the ratio of the third mode period to the first mode period is 0.14.
- The effective modal mass of the first mode is 0.80 M, of the second mode is 0.15 M, and of the third mode is 0.05 M, where M is the total mass.

This hypothetical structure corresponds approximately to that used by SEAOC to arrive at the response spectra in its 1974 recommendations [6]. Fig. 3 shows that scale reduction factor as a function of period for a building with K equal to 1.0, site on firm ground and on soft ground. For periods longer than 1.0 second, the two curves are approximately proportional; the curve for soft ground is higher because S(H) is 2.0, while S is equal to 1.5. The large drop in scale factor below T equal to 0.5 seconds for firm ground and below T equal to 1.0 second for soft ground is due to the difference between the periods at which the factor C drops below its maximum value in Eq. 1 and the elastic spectrum drops below the maximum spectral acceleration.

When scaling is a device used only to achieve a different distribution of lateral forces, the value of the scale factor is irrelevant; however, the Code requires the use of the unscaled elastic spectrum for structures not meeting the special design requirements for ductility. The range of the scaling factor is 2.9 to 12.4 for firm ground, and 2.5 to 10.4 for soft ground, for the hypothetical building with K equal to 1.0 (Fig.3). The significance of this variation of scale factor with period can be assessed in terms of the ductility demand (required ductility ratio), defined as μ in the inelastic spectrum for firm ground derived from the specified elastic spectrum [16]:

$$S_a = \min \begin{cases} \dfrac{0.32}{\sqrt{2\mu-1}} \\ \dfrac{0.16}{\mu T} \end{cases} \tag{3}$$

When this inelastic spectrum is used in lieu of the elastic spectrum to obtain modal accelerations for the example structure defined above, and if the base shear is scaled to the value specified by Eq. 1, the computed ductility demand is larger than 6 for periods shorter than 2.0 seconds; for periods longer than 2.0 seconds, the ductility demand is approximately constant and equal to 6. This means that the required ductility ratio, as defined above, implicit in the base shear equation of the Code is equal to or higher than 6; the larger values correspond to structures with periods shorter than 2.0 seconds. Therefore, designers should pay much attention to detailing for ductility, especially for short-period structures.

It is unlikely, considering the cost involved, that many structures in Massachusetts will be designed using the time-history approach. In any event, there are currently available several programs to obtain time-histories consistent with a given elastic response spectrum, and with a given peak acceleration [7], [10]. Time-histories must be compatible with the response spectrum that corresponds to the local soil conditions; the results may be scaled to obtain the base shear given by Eq. 1 for the fundamental period.

Ductility Requirements. — This section discusses some of the special requirements for structures designed for the minimum base shear and the implications of making these requirements mandatory for all structures so designed.

Since all structures designed for the base shear given by Eq. 1 must meet the special design requirements, a structure that carries all the lateral load by moment frames may be designed for a factor K equal to 0.67. This is different from both UBC-1976 and ATC 3-06. According to UBC-1976, structures which have a steel or concrete moment-resisting frame must meet no special design requirements for ductility if designed with a factor K equal to 1.0; only when K is 0.67 or 0.80 must the moment resisting frames comply with the special requirements for ductility. Moment frames of steel or concrete designed for K equal to 1.0 by UBC-1976 have some ductility, because both ACI-318 [29] and the AISC Specifications [37] lead to relatively ductile structures; the special design requirements provide for additional ductility. The ATC 3-06 recommendations specify that structures in Massachusetts must meet the requirements for seismic performance category B. These requirements allow, for a steel building, ordinary moment frames in which noncompact structural steel sections or cold-formed members may be used; such frames are not allowed by the Massachusetts provisions, which specify that compact sections must be used for moment frames.

The special ductility requirements for steel moment frames are similar to those of UBC-1973. Note that the requirement, that the slenderness ratio for columns be computed without consideration for lateral support provided by bracing or shear walls, has been deleted in UBC-1976.

The special ductility requirements for concrete frames are derived from UBC-1976 and from selected provisions of ACI 318, Appendix A. For flexural members, top and bottom reinforcement is required and the positive moment strength at column connections must be at least 25 percent of the required negative moment strength. The requirement of the body of ACI 318 (10.3.3) that the ratio of reinforcement in flexural members not exceed 75 percent of the ratio that produces balanced conditions was considered sufficient; therefore, the special requirement of Appendix A of ACI 318 that reduces this maximum ratio to 50 percent was not adopted.

The ductility requirements for masonry were developed according to the principle that damage but not collapse is acceptable. Masonry bearing walls, shear walls, exterior walls, chimneys and parapets must be reinforced according to the provisions of the BIA [28], the NCMA [36], or ANSI [30] standards. The maximum spacing of principal reinforcement is set at four feet, rather than the two feet required by UBC-1976; spacing of reinforcement in the direction perpendicular to principal reinforcement is set at six feet. A four-foot by six-foot reinforcement mesh is considered adequate to insure the bearing capacity of walls after an earthquake and to preclude partial collapse. Nonstructural masonry must be designed as at least partially reinforced masonry when enclosing stairwells or elevator shafts.

The Code includes the following rules for timber design: lumber and plywood diaphragms may be used; positive connections are required to transfer axial and shear forces produced by earthquakes; toenailing or nails subject to withdrawal are not acceptable; and sheathing may be used as tension ties, provided no cross-grain bending or tension arises in the framing members.

A controversial ductility requirement is that structures of materials for which no special design requirements are given in the Code must "safely withstand lateral distortions eight (8) times that computed for the lateral forces specified." This requirement would apply, for example, to moment frames made of cold-formed sections, since these members do not usually conform to the requirements for "plastic design sections;" however, such a frame may be stable when some of its members are in the post-buckling state. The requirement would also apply to new materials of construction.

Foundation Design Requirements

The Code is the first building code in the United States that contains a comprehensive set of soil and foundation design requirements for earthquake loads. The soil provisions either give specific design guidelines or they point out those areas of design that require special consideration.

The purposes of the foundation requirements are: to insure site stability, including the prevention of potential liquefication; to provide foundations capable of transferring all horizontal and vertical earthquake loads to the soil; and to provide a foundation system that will not, because of relative movement between foundations, damage the superstructure to the extent of jeopardizing life safety. In addition to the foundation requirements, the effect of soil amplification is incorporated into the base shear requirement through the soil factor S.

The soil factor and all the foundation requirements except the guidelines to check the susceptibility of a site to liquefaction are tied to the class of soil or soil site. Two classes of soil are defined by the Code; in essence, firm soil and soft soil. Class A soils are firm or very firm soils: igneous rock and conglomerate; slate; shale; glacial till; dense to very dense gravel, sand and gravel, and sand; clay having an undrained shear strength above 1,000 lbs per sq ft; and well compacted granular fill. All other soils are Class B. Soil site classes are defined based on the thickness and layering of the component soil classes; the site class is either that of the single component soil or, when both classes of soils exist, the soil site is determined in accordance with Fig. 2. It follows from Fig. 2 that a site is Class B when a thickness of more than 12 ft of Class B soil overlies a Class A soil, or when a sufficiently thick layer of Class B soil underlies a Class A soil.

The Code includes a screening test to determine when a site is considered not susceptible to liquefaction. Liquefaction is the phenomenon in which a saturated loose sand under undrained conditions develops a substantial loss of shear strength due to either monotonic or cyclic loading; after the sand liquefies, it has a residual strength so low that the soil actually flows. Liquefaction is defined in the Code to include liquefaction of loose saturated sands and cyclic mobility (susceptibility to large strains under cyclic loading) of medium-dense saturated sands [3]. When susceptibility to liquefaction is indicated, based on the screening test, further soil studies are required. While laboratory tests [2], [13], [19] have been significant in explaining the liquefaction phenomena, to develop a screening test actual experience of soils during past earthquakes has proved more useful. Whitman [25] has shown that during past earthquakes liquefaction has occurred only for certain combinations of the ratio of dynamic shear stress to vertical effective stress (both on the same horizontal plane) and the relative density of the sand; similar results based on combinations of dynamic shear stress normalized with respect to effective overburden pressure and corrected standard penetration resistance were presented by Castro [3]. Considering the wide scatter that exists in correlations between in situ relative density of saturated sands and standard penetration resistance, the standard penetration resistance was considered a more reliable parameter for use in a code screening test. For conditions when lateral sliding cannot occur, the Code includes a figure to determine those saturated sand soil sites not susceptible to liquefaction; the curves in this figure are drawn for a maximum ground acceleration of 0.12 g. The curves are cut off at a depth of 60 ft, because the liquefaction failures on which the empirical criteria are based all have occurred at shallow depths.

Soil sites that do not pass the screening test are not necessarily liquefaction-susceptible, but may be, and therefore must be further investigated. Geotechnical studies are also required for sites where lateral sliding (slope instability) may occur and where the site is underlain by saturated silty sands and inorganic nonplastic silts.

The design of foundations must make provision for the transmission of the base shear between the structure and the soil or rock. The Code specifies five acceptable means by which the lateral forces may be transmitted to the soil. The Code also specifies that lateral pressure may not exceed one-third the passive pressure and that bottom friction may not be relied on where a building overlies a Class B soil and is supported by piles, piers, or caissons. These requirements are spelled out so that the transmission of lateral forces from the foundation to the soil will not be overlooked; there are no similar provisions in UBC-1976.

Individual pile, pier, and caisson caps are required to be interconnected only when the caps overlie a Class B soil; this provision is similar to the corresponding one in UBC-1976, except that the Code specifically exempts caps in Class A soils from the interconnection requirement. The lateral and vertical movement potential of footings is required to be investigated for cases where the footing overlies at shallow depths cohesionless soils that only slightly exceed the criteria for sites not susceptible to liquefaction.

The Code contains a specification for an additional active soil pressure due to earthquakes to be applied to retaining walls and to foundation walls.

This force should not be applied when there is a reasonable expectation for liquefaction of the backfill during seismic loading; rather, further studies must be performed for such cases. The earliest method for determining the dynamic lateral pressure on earth retaining structures was that developed by Okabe in 1926 [17] and by Mononobe in 1929 [15]. The Mononobe-Okabe analysis is basically a computation of active pressure, using Coulomb theory, that includes, in addition to the weight of the triangular sliding wedge, horizontal and vertical inertia forces. The key assumptions of the analysis are that active pressure develops because of wall movement; that the soil behaves as a rigid body, with the result that accelerations throughout the mass of the wedge are uniform; and that the soil wedge behind the wall is at the point of incipient failure. Recent theoretical analyses by Prakash and Basavanna, [18] and experimental analyses by Ishii et al. [9] and Kurata et al. [12] have validated the Mononobe-Okabe theory; the experimental studies also show that the resultant of the earthquake force acts at a height of 0.5 H to 0.67 H above the base of the wall, where H is the height of the wall. Seed and Whitman [20] have shown that a very good approximation to the earthquake active soil pressure coefficient K is 0.75 α, where α is the lateral acceleration as a fraction of gravity; this approximation was derived for the case of a vertical wall, horizontal backfill slope, and angle of friction of soil of 35°. A resultant at 0.67 H above the base of the wall is obtained when the earthquake pressure is distributed as an inverse triangle over the height of the wall. The additional earthquake force due to active soil pressure in the Code is that proposed by Seed and Whitman, [20] evaluated for the design earthquake of 0.12 g, that is,

$$P = \tfrac{1}{2} K \gamma_t H^2 = \tfrac{1}{2} (0.75 \times 0.12) \gamma_t H^2 = 0.045 \gamma_t H^2 \qquad (4)$$

The extension of applicability of this retaining wall formula to foundation walls has been criticized; however, there are currently no known better alternative approaches. Finite element analyses performed for foundation walls of nuclear power plants show lateral forces which are higher than those given by the Mononobe-Okabe analysis.

Summary and Future Developments

The recognition of the fact that the earthquake return period in Massachusetts is substantially longer than that for earthquakes in the West led to the formulation of a design philosophy that states that the sole purpose of earthquake regulations here should be the mitigation of danger to life safety. The seismic provisions based on this design philosophy differ conceptually from those contained in most major model codes; to minimize the cost of earthquake protection, the Massachusetts Code emphasizes ductility requirements and prescribes relatively modest lateral forces, while the model codes simply apply a zone factor to the lateral force requirements developed for California by SEAOC (UBC-1976 also includes ductility exemptions for Seismic Zone 1).

Since the ductility requirements of the Code are applicable primarily to new construction, the Seismic Advisory Committee to the State Building Code Commission is in the process of studying the problems associated with strength evaluations and seismic design for the renovation of existing buildings.

References

1. Algermissen, S. T., and Perkins, D. M., "A Probabilistic Estimate of Maximum Acceleration in Rock in the Contiguous United States," *Open File Report 76-416, U. S. Geological Survey,* 1976.
2. Castro, G., "Liquefaction of Sands, " *Harvard Soil Mechanics Series No. 81,* January, 1969.
3. Castro, G., "Liquefaction and Cyclic Mobility of Saturated Sands," *Journal of the Geotechnical Division,* ASCE, Vol. 101, No. GT6, 1975, p. 551.
4. Cornell, C. A., "Engineering Seismic Risk Analysis," *Bulletin Seismological Society of America,* Vol. 58, 1968, pp. 1583-1606.
5. Cornell, C. A., and Merz, H. A., "Seismic Risk Analysis of Boston," *Journal of the Structural Division,* ASCE, Vol. 101, No. ST10, October, 1975, pp. 2027-2043.
6. Freeman, S. A., "Introduction to New Features of the SEAOC Blue Book," *New Earthquake Design Provisions,* Seminar Papers Presented by Professional Development Committee, SEA of Northern California and San Francisco Section ASCE, 1975.
7. Gasparini, D. and Vanmarcke, E. H., "Simulated Earthquake Motions Compatible with Prescribed Response Spectra," *Publication No. R76-4, Order No. 527,* Department of Civil Engineering, Massachusetts Institute of Technology, 1976.
8. Housner, G. W. "Engineering Estimates of Ground Shaking and Maximum Earthquake Magnitude," *Proc. Fourth World Conference on Earthquake Engineering,* Santiago, Chile, 1969, Vol. I, pp. 1-13.
9. Ishii, Y., Arai, H., and Tsuchida, H., "Lateral Earthpressure in an Earthquake," *Proc. Second World Conference on Earthquake Engineering,* Tokyo, 1960, Vol. I, pp. 211-230.
10. Kaul, M. K., "Spectrum-Consistent Time-History Generation," *Journal of the Engineering Mechanics Division,* ASCE, Vol. 104, No. EM4, August, 1978, pp. 781-788.
11. Krimgold, F., "Seismic Design Decisions for the Commonwealth of Massachusetts State Building Code," *Seismic Design Decision Analysis Series, Report No. 32,* Department of Civil Engineering, Massachusetts Institute of Technology, 1977.
12. Kurata, S., Arai, H. and Yokoi, T. "On the Earthquake Resistance of Anchored Sheet-pile Bulkheads," *Proc. Third World Conference on Earthquake Engineering,* Auckland and Wellington, New Zealand, 1965, Vol. II, pp. 11-369.
13. Lee, L. and Seed, H. B., "Cyclic Stress Conditions Causing Liquefaction of Sand," *Journal of the Soil Mechanics and Foundations Division,* ASCE, Vol. 93, No. SM1, January, 1967, pp. 47-70.
14. Luft, R. L., "Considerations for Establishing Seismic Design Criteria for New England Medical Center Hospital," Report Prepared for Perry, Dean, Stahl & Rogers, Inc., Architects, by Simpson Gumpertz & Heger Inc., Consulting Engineers, August, 1978.
15. Mononobe, N., "Earthquake-Proof Construction of Masonry Dams," *Proc. World Engineering Conference,* Vol. 9, p. 176, 1929.
16. Newmark, N. M. and Hall, W. J., "Seismic Design Criteria for Nuclear Reactor Facilities," *Proc. Fourth World Conference on Earthquake Engineering,* Santiago, Chile, 1969, Vol. 2, pp. 37-50.

17. Okabe, S., "General Theory of Earth Pressure," *Journal of the Japanese Society of Civil Engineers,* Vol. 12, No. 1, 1926.
18. Prakash, S. and Basavanna, B. M., "Earth Pressure Distribution Behind Retaining Wall During Earthquake," *Proc. Fourth World Conference on Earthquake Engineering,* Santiago, Chile, 1969, Vol. III.
19. Seed, H. B. and Lee, K. L., "Liquefaction of Saturated Sands During Cyclic Loading," *Journal of the Soil Mechanics and Foundations Division,* ASCE, Vol. 92, No. SM6, November, 1966, pp. 105-134.
20. Seed, H. B. and Whitman, R. V., "Design of Earth Retaining Structures for Dynamic Loads," ASCE Specialty Conference, Lateral Stresses in the Ground and the Design of Earth Retaining Structures, 1970, pp. 103-147.
21. Seed, H. B., Ugas, C. and Lysmer, J., "Site Dependent Spectra for Earthquake Resistant Design," *Bulletin of the Seismological Society of America,* Vol. 66, No. 1, February, 1976, pp. 221-244.
22. Tong, W.-H., Schumacker, B., Cornell, C. A., and Whitman, R. V., "Seismic Hazard Maps for Massachusetts," *Seismic Design Decision Analysis Series,* Internal Study Report No. 52, Dept. of Civil Engineering, Massachusetts Institute of Technology, 1975.
23. Vanmarcke, E. H. and Cornell, C. A., "Seismic Risk and Design Response Spectra," *Proc. Specialty Conference on Safety and Reliability of Metal Structures,* ASCE, Pittsburgh, Pennsylvania, 1972.
24. von Hake, C. A., "Earthquake History of Massachusetts," *Earthquake Information Bulletin,* U. S. Department of the Interior, Vol. 5, No. 5, 1973, pp. 22-24.
25. Whitman, R. V., "Resistance of Soil to Liquefaction and Settlement," *Soils and Foundations,* Japanese Society of Soil Mechanics, Vol. II, No. 4, 1971, pp. 59-68.
26. Whitman, R. V., "Effect of Local Soil Conditions Upon Earthquake Ground Motions," *Fundamentals of Earthquake Engineering for Buildings,* Ch. 12, Department of Civil Engineering, Massachusetts Institute of Technology, 1972.
27. Whitman, R. V. and Protonotarios, J. N., "Inelastic Response to Site Modified Ground Motions," *Seismic Design Decision Analysis Series,* Report No. 31, Department of Civil Engineering, Massachusetts Institute of Technology, 1977.
28. *Building Code Requirements for Engineered Brick Masonry,* Brick Institute of America, McLean, Virginia, 1969.
29. *Building Code Requirements for Reinforced Concrete (ACI 318-77),* American Concrete Institute, Detroit, Michigan.
30. *Building Code Requirements for Reinforced Masonry,* American National Standards Institute, A41.2-1960 (R1970), New York.
31. *Earthquake History of the United States,* U. S. Department of Commerce: 1928, Special Publication 149, Heck, N. H.; 1938, Part 1, Serial No. 609, Heck, N. H.; 1958, Part 1, Publication 41-1, Heck, N. H. and Eppley, R. A.; 1965, Part I, Publication 41-1, Eppley, R. A.; 1970, Revised Edition (Through 1970), Publication 41-1, Coffman, J. L. and von Hake, C. A., Editors.
32. *Field Velocity Measurements and Rock Mechanics Studies,* Prepared by Weston Geophysical Research, Inc. for Boston Edison Company, Pilgrim Unit No. 2, BESG7606, 1976.
33. *Historical Seismicity of New England,* Prepared by Weston Geophysical Research, Inc. for Boston Edison Company, Pilgrim Unit No. 2, BE-SG7601, 1976.
34. *Our New England Earthquakes,* Prepared by Weston Geophysical Research, Inc. for Boston Edison Company, 1977.
35. *Recommended Lateral Force Requirements and Commentary,* Seismology Committee, Structural Engineers Association of California, San Francisco, 1974.
36. *Specification for the Design and Construction of Load-Bearing Concrete Masonry,* National Concrete Masonry Association, Rosslyn Station, Arlington, Virginia.
37. *Specification for the Design, Fabrication & Erection of Structural Steel for Buildings,* American Institute of Steel Construction, New York, 1969.

38. *Tentative Provisions for the Development of Seismic Regulations for Buildings, a Cooperative Effort with the Design Professions, Building Code Interests and the Research Community,* Prepared by Applied Technology Council (ATC) associated with the Structural Engineers Association of California, ATC Publication ATC 3-06, *National Bureau of Standards Special Publication 510, National Science Foundation Publication* 78-8, 1978.
39. *Uniform Building Code,* International Conference of Building Officials (formerly Pacific Coast Building Officials Conference), Whittier, California, Editions 1952, 1955, 1958, 1961, 1964, 1967, 1970, 1973, and 1976. Also, *Analysis of Revisions to the Uniform Building Code,* Editions 1970, 1973, and 1976.

Discussion

By Robert V. Whitman,[3] *F. ASCE*

I think it unfortunate to conclude that the probable period for the Massachusetts Code design earthquake motion is 5000 years, with bounds of 2000 and 10,000 years. I know the authors have used MIT results to reach that conclusion, and suppose I am saying that I no longer really believe those results. I agree that Algermissen & Perkins may have based their analysis upon inflated estimates of intensities during past earthquakes. On the other hand, they also made a potentially unconservative assumption: neglecting scatter in the attenuation rule. The net result is that Algermissen & Perkins estimates may be about correct. Our various studies at MIT also neglected uncertainty on the attenuation law for intensity and uncertainty in the relation between intensity and acceleration, and hence would tend to be unconservative. My feeling is that the return period for our design earthquake motion, at least in northeastern Massachusetts, is on the order of 1000 years. I feel that the design philosphy incorporated into the Massachusetts Code is quite consistent with this shorter period.

Closure

By Rene W. Luft and Howard Simpson

The authors thank Professor Whitman for his discussion. The seismic risk studies reviewed by the Seismic Advisory Committee did not, as noted in the discussion, consider scatter in the attenuation rule nor the uncertainty in the relation between intensity and peak ground acceleration. Studies that treat intensity as a random function of the distance from a site to the epicenter and of epicentral peak ground acceleration, obtain a higher seismic risk at a site [40] than studies that neglect scatter. Neglecting the scatter that is introduced by treating intensity as a random function may, however, be reasonable to define a design earthquake for Massachusetts.

[3] Professor of Civil Engineering, Massachusetts Institute of Technology, Cambridge, Massachusetts

The design earthquake selected for the Code was arrived at using a seismic risk analysis with the conservative assumption that a point source of upper bound M M intensity 8.7 could occur in Boston harbor. This point source is an extrapolation from the 1755 Cape Ann earthquake. Therefore, scatter in the attenuation rule of intensity with distance from the epicenter would not significantly affect the design earthquake.

The uncertainty in the correlation between intensity and peak ground motion is of concern primarily for recorded earthquakes. A wide scatter is shown by Trifunac and Brady [41]; for example, for 150 M M intensity VII earthquakes the mean acceleration was 131.29 cm/sec^2 (0.134 g) and the standard deviation was 61.30 cm/sec^2 (0.063 g). Averages obtained from the same data, grouped under soft, intermediate, and hard ground classifications show even wider scatter. The severity of historical earthquakes must be estimated from reported damage, from a knowledge of building practices at the time of the earthquake, and an understanding of local soil conditions. The estimates obtained can be expressed in terms of M M intensity or in terms of the "effective" ground acceleration as defined by Teal [42]; the effective ground acceleration, which differs from peak recorded ground acceleration, is related to base shear. The uncertainty in either of these estimates is inherent in their determination; therefore, judgment is used to assign most probable values to the intensity and acceleration. Because these estimates should be obtained independently, the uncertainty in the correlation between acceleration and intensity is less important for historical earthquakes.

The single most important number, however, that must be estimated is the maximum probable epicentral intensity. Algermissen and Perkins [1] use a maximum M M intensity of IX and arrive at a return period of 475 years for a 0.1 g peak ground acceleration earthquake; reducing the maximum M M intensity to 8.2 results in a two- to ten-fold increase in return period estimate. To the authors' knowledge, the question of maximum probable epicentral intensity in New England is still being debated by local seismologists; Chinnery, of MIT Lincoln Laboratory, states that there is no evidence for the existence of upper bounds to maximum epicentral intensity [43], while Chiburis, of Boston College Weston Observatory, states that earthquakes of epicentral intensities in the M M IX to X range are not consistent with local geology [44]. This latter position is shared by researchers at Weston Geophysical Corporation, and by Gene Simmons of MIT [34].

The Seismic Advisory Committee has defined a design earthquake that is based on the best knowledge to date, and has proposed design regulations which are consistent with the design earthquake. Whether the return period is 500 years or 5,000 years is, in essence, an academic question; undoubtedly, the return period is longer than for earthquakes on the West Coast. The acceptance of too long a return period may, however, affect the credibility of the need for seismic provisions.

Additional References

40. O'Rourke, M. J., Wang, L. R. L. and Pikul, R., "Seismic Wave Effects on Water Systems," *ASCE Convention & Exposition, Reprint 4396,* Boston, April 2-6, 1979.
41. Trifunac, M. D. and Brady, A. G. "On the Correlation of Seismic Intensity Scales with Peaks of Recorded Strong Ground Motion," *Bulletin Seismological Society of America,* Vol. 65, No. 1, pp. 139-162, February, 1975
42. Teal, E. J., "Seismic Drift Control and Building Period," *Engineering Journal, American Institute of Steel Construction,* Vol. 15, No. 2, pp. 30-38, 1978
43. Chinnery, M. A., "A Comparison of the Seismicity of Three Regions of the Eastern U. S." *Bulletin Seismological Society of America,* Vol. 69, No. 3, pp. 757-772, June 1979.
44. Chiburis, E., "Seismicity, Recurrence Rates, and Regionalization of the Northeastern United States and Adjacent Areas," to be published by *Bulletin Seismological Society of America.*

JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS SECTION

Proceedings

AMERICAN SOCIETY OF CIVIL ENGINEERS

MEETINGS HELD — TECHNICAL GROUPS

Construction Group

May 16, 1979. Dinner meeting at the Engineers Club, held jointly with the Geotechnical Group and with the Foundation and Marine Constructors of New England. Panel discussion on "Slurry Wall Test Panel in Quick Clay" with three speakers: Donald T. Goldberg, President, Goldberg, Zoino, Dunnicliff & Associates, Inc.; James J. Reagan, Vice President, Sverdrup & Parcel and Associates; and James Roop, Jr., Ground Support Systems, Franki Foundation Company. Attendance 80.

Geotechnical Group

Joint meeting; see report of Construction Group.

Transportation Group

May 14, 1979. Seminar and luncheon 9:00 AM to 2:00 PM at Pier 4 Restaurant, held in cooperation with the Boston Transportation Group and the Traffic Research Forum. Coordinator was Salvatore E. Tollo, Corporate Traffic Manager, Stop and Shop Companies, Inc. Seminar subject was "Efficiency in Transportation, or the Lack Thereof." Panelists: Barry M. Locke, Massachusetts Secretary of Transportation; Dean P. Amidon, Commissioner, Massachusetts Department of Public Works; Robert L. Foster, Chairman, Massachusetts Bay Transportation Authority; E. Kenneth Bowers, Director, Corporate Transportation and Distribution, Combustion Engineering, Inc.; Virginia May Brown, Vice Chairman, Interstate Commerce Commission; Barbara Neuman, Massachusetts Coordinator of Consumer Affairs; and Albert E. Winemiller, President, Quinn Freight Lines, Inc. Paul O. Roberts, MIT Civil Engineering Department, was Moderator. Governor Edward J. King was the luncheon speaker. Attendance, 400.

U.S. POSTAL SERVICE

STATEMENT OF OWNERSHIP, MANAGEMENT AND CIRCULATION

(Required by 39 U.S.C. 3685)

1. TITLE OF PUBLICATION Journal of the Boston Society of Civil Engineers Section/ASCE

A. PUBLICATION NO.

2. DATE OF FILING 10/1/79

3. FREQUENCY OF ISSUE quarterly

A. NO. OF ISSUES PUBLISHED ANNUALLY 4

B. ANNUAL SUBSCRIPTION PRICE $10.00

4. LOCATION OF KNOWN OFFICE OF PUBLICATION *(Street, City, County, State and ZIP Code) (Not printers)*
80 Boylston Street, Suite 1110, Boston, Suffolk, MA, 02116

5. LOCATION OF THE HEADQUARTERS OR GENERAL BUSINESS OFFICES OF THE PUBLISHERS *(Not printers)*
80 Boylston Street, Suite 1110, Boston, Suffolk, MA, 02116

6. NAMES AND COMPLETE ADDRESSES OF PUBLISHER, EDITOR, AND MANAGING EDITOR

PUBLISHER *(Name and Address)*
Boston Society of Civil Engineers Section of the American Society of Civil Engineers

EDITOR *(Name and Address)*
Edward C. Keane, 80 Boylston Street, Suite 1110, Boston, Suffolk, MA, 02116

MANAGING EDITOR *(Name and Address)*

7. OWNER *(If owned by a corporation, its name and address must be stated and also immediately thereunder the names and addresses of stockholders owning or holding 1 percent or more of total amount of stock. If not owned by a corporation, the names and addresses of the individual owners must be given. If owned by a partnership or other unincorporated firm, its name and address, as well as that of each individual must be giving. If the publication is published by a nonprofit organization, its name and address must be stated.)*

NAME	ADDRESS
Boston Society of Civil Engineers Section of the American Society of Civil Engineers	80 Boylston Street, Suite 1110 Boston, Suffolk, MA, 02116

8. KNOWN BONDHOLDERS, MORTGAGEES, AND OTHER SECURITY HOLDERS OWNING OR HOLDING 1 PERCENT OR MORE OF TOTAL AMOUNT OF BONDS, MORTGAGES OR OTHER SECURITIES *(If there are none, so state)*

NAME	ADDRESS
none	none

9. FOR COMPLETION BY NONPROFIT ORGANIZATIONS AUTHORIZED TO MAIL AT SPECIAL RATES *(Section 132.122, PSM)*
The purpose, function, and nonprofit status of this organization and the exempt status for Federal income tax purposes *(Check one)*

[X] HAVE NOT CHANGED DURING PRECEDING 12 MONTHS
[ ] HAVE CHANGED DURING PRECEDING 12 MONTHS
(If changed, publisher must submit explanation of change with this statement.)

10. EXTENT AND NATURE OF CIRCULATION	AVERAGE NO. COPIES EACH ISSUE DURING PRECEDING 12 MONTHS	ACTUAL NO. COPIES OF SINGLE ISSUE PUBLISHED NEAREST TO FILING DATE
A. TOTAL NO. COPIES PRINTED *(Net Press Run)*	3562	7077
B. PAID CIRCULATION 1. SALES THROUGH DEALERS AND CARRIERS, STREET VENDORS AND COUNTER SALES	none	none
2. MAIL SUBSCRIPTIONS	2088	2080
C. TOTAL PAID CIRCULATION *(Sum of 10B1 and 10B2)*	2088	2080
D. FREE DISTRIBUTION BY MAIL, CARRIER OR OTHER MEANS SAMPLES, COMPLIMENTARY, AND OTHER FREE COPIES	1243	4897
E. TOTAL DISTRIBUTION *(Sum of C and D)*	3331	6977
F. COPIES NOT DISTRIBUTED 1. OFFICE USE, LEFT OVER, UNACCOUNTED, SPOILED AFTER PRINTING	231	100
2. RETURNS FROM NEWS AGENTS	none	none
G. TOTAL *(Sum of E, F1 and 2—should equal net press run shown in A)*	3562	7077

11. I certify that the statements made by me above are correct and complete.

SIGNATURE AND TITLE OF EDITOR, PUBLISHER, BUSINESS MANAGER, OR OWNER
X Edward C. Keane

12. FOR COMPLETION BY PUBLISHERS MAILING AT THE REGULAR RATES *(Section 132.121, Postal Service Manual)*

39 U. S. C. 3626 provides in pertinent part: "No person who would have been entitled to mail matter under former section 4359 of this title shall mail such matter at the rates provided under this subsection unless he files annually with the Postal Service a written request for permission to mail matter at such rates."

In accordance with the provisions of this statute, I hereby request permission to mail the publication named in Item 1 at the phased postage rates presently authorized by 39 U. S. C. 3626.

SIGNATURE AND TITLE OF EDITOR, PUBLISHER, BUSINESS MANAGER, OR OWNER
X Edward C. Keane

PS Form 3526 *(Page 1)* Aug. 1978

(See instructions on reverse)

BOSTON SOCIETY OF CIVIL ENGINEERS SECTION, ASCE

Committee Chairmen, 1979-1980

Committee	*Chairman*
ACTION PROGRAM - PROFESSIONAL PRACTICE	Domenic E. D'Eramo
ADVERTISING	John P. Hurney
ANNUAL MEETING	Edward B. Kinner
AUDITING	Robert L. Meserve
AWARDS	Paul J. Trudeau
CONSTITUTION AND BYLAWS	Saul Namyet
THOMAS R. CAMP FUND	Benjamin J. Fehan
CONTINUING EDUCATION	Ronald E. Sharpin
DISADVANTAGED YOUTH	Donald T. Goldberg
EMPLOYMENT CONDITIONS	Kevin Regan
JOHN R. FREEMAN FUND	Lee M.G. Wolman
HISTORY AND HERITAGE	H. Hobart Holly
INVESTMENT	Norman W. Bennett
KEY MAN	Frank J. Cullatti
MEMBERSHIP	Richard M. Simon
MONTHLY LUNCHEON SERIES	Brian R. Hogan
OPERATIONS MANUAL	Edward B. Kinner
PROGRAM	Edward B. Kinner
PUBLIC RELATIONS	Virgil Paulson
PUBLICATIONS	Frank E. Perkins
SOCIAL AFFAIRS	Robert A. Snowber
STUDENT AFFAIRS	Michael Kupferman

Division Chairmen

Division	*Chairman*
ADMINISTRATIVE DIVISION	Stanley C. Rossier
GENERAL TASKS DIVISION	Robert L. Meserve
PROFESSIONAL AND SOCIETY AFFAIRS DIVISION	Paul J. Trudeau
TECHNICAL DIVISION	Peter K. Taylor
LECTURE SERIES	Edward B. Kinner

SUMMER 1979

1848
BOSTON
SOCIETY
OF CIVIL
ENGINEERS

AMERICAN
SOCIETY OF
CIVIL
ENGINEERS
FOUNDED
1852